Chapter 1 - Introduction

TABLE OF CONTENTS

CHAPTER 1 - INTRODUCTION .................................................................................. 1-1
1.1 Background........................................................................................................... 1-1
1.2 Overview............................................................................................................... 1-2
  1.2.1 Purpose........................................................................................................... 1-2
  1.2.2 Manual Layout/Chapter Templates ............................................................... 1-2
1.3 Drainage Design Memoranda ............................................................................. 1-4
1.4 References ......................................................................................................... 1-5
1.5 User Instructions ............................................................................................... 1-6
1.6 Revisions and Updates ..................................................................................... 1-7
1.7 Acknowledgements ............................................................................................ 1-8

List of Tables

Table 1-1. Chapter Template and Contents ............................................................... 1-3

List of Appendices

Appendix 1A-1 Definitions and Abbreviations
Appendix 1A-2 Symbols
Chapter 1 - Introduction

1.1 Background

The Virginia Department of Transportation (VDOT) has developed the 2002 VDOT Drainage Manual to provide designers a valuable reference and tool for the drainage design of Virginia’s roadways and to document VDOT’s policies and procedures for standard roadway drainage design.

This edition of the VDOT Drainage Manual constitutes a major technical update and compilation of the existing VDOT Drainage Manual, the AASHTO Model Drainage Manual, and other resources and has been prepared in electronic format to be made available on the Internet at the VDOT website. VDOT’s Hydraulics Section prepared this edition of the manual.

The objectives of the manual are to:

- Provide concise technical information for drainage designers
- Establish VDOT’s policies and procedures for drainage design
- Provide an educational tool for aspiring drainage designers and instructors
- Provide in electronic format, available on the World Wide Web for viewing and downloading
- Provide guidelines to enhance the quality of drainage design submittals to VDOT

* Rev. 1/17
1.2 Overview

1.2.1 Purpose

This manual is intended as an operational handbook for use in hydrologic and hydraulic analysis. Design concepts, policies and procedures, criteria, and examples are condensed and written for use by the designer. Where appropriate, relevant hydraulic design publications are referenced. While it is essential that the user of this manual is familiar with the methods of analysis and design of highway drainage for VDOT, the text provides detailed instructions and criteria for the development of analysis and design in most cases. An exception to this rule is the case where another source document expounds upon the method in great detail. In this case, the manual directs the user to the source document or provides a brief synopsis of the subject.

This manual is intended for use in the development of VDOT highway drainage and stormwater management design projects by Department staff, consultants, and Virginia’s municipalities. Educational organizations may use the manual as instructional text in design application. The manual gives the designer a basic working knowledge of hydrology and hydraulics, illustrated with example problems. Basic design elements are included so that the designer can design highway drainage with minimal assistance. However, this manual cannot provide guidance on complex hydrologic or hydraulic problems and is no substitute for experience, formal training, or engineering judgment.

The Department recognizes the difficulty in accurately defining or predicting the dynamic properties of nature. There are numerous methods of analysis available and it is recommended that as many method(s) as may be appropriate be employed in the solution of a problem. Further, all hydraulic designs must give consideration to economic, aesthetic, and environmental aspects of the given design.

Complete documentation of all analyses is essential and must be perpetually maintained. The rapid development of technology in the fields of hydrology and hydraulics necessitates a periodic review and, if necessary, update of all analyses prior to construction of the facility. All analysis completed more than three years before construction must be reviewed prior to construction.

1.2.2 Manual Layout/Chapter Templates

Typical section headings for the main hydraulic chapters are identified in Table 1-1, which indicates the typical contents of the chapter sections.

The Design Concepts section for each technical chapter is generally based on the AASHTO Model Drainage Manual. As such, the material is included for theoretical background and may not conform exactly to VDOT methodology, terminology, or nomenclature. When practical, the text is revised to be consistent with VDOT methodology and policy.

* Rev. 3/19
<table>
<thead>
<tr>
<th>Sections</th>
<th>Contents</th>
</tr>
</thead>
<tbody>
<tr>
<td>Introduction</td>
<td>Objectives</td>
</tr>
<tr>
<td>Policy</td>
<td>Define Course of Action for VDOT, State, Federal and Local Policy</td>
</tr>
<tr>
<td>Design Criteria</td>
<td>Specify Standards by which Policy is Carried Out</td>
</tr>
<tr>
<td>Design Concepts</td>
<td>Design Considerations/ Guidelines</td>
</tr>
<tr>
<td></td>
<td>Theory and Equations</td>
</tr>
<tr>
<td></td>
<td>Requirements</td>
</tr>
<tr>
<td></td>
<td>Figures <strong>Necessary</strong> to Support Procedures or Examples</td>
</tr>
<tr>
<td>Design Procedures &amp; Examples</td>
<td>Step-by-Step Procedures</td>
</tr>
<tr>
<td></td>
<td>Specific Design Considerations</td>
</tr>
<tr>
<td></td>
<td>Specific Software Solutions</td>
</tr>
<tr>
<td></td>
<td>Figures <strong>Necessary</strong> to Support Procedures or Examples</td>
</tr>
<tr>
<td>References</td>
<td>Sources of Information / Bibliography</td>
</tr>
<tr>
<td>Appendices</td>
<td>All Figures, Forms and Design Aids <strong>Not Necessary</strong> to be used in Concepts or Procedures</td>
</tr>
<tr>
<td></td>
<td>Drainage Design Memoranda</td>
</tr>
<tr>
<td></td>
<td>Definitions</td>
</tr>
<tr>
<td></td>
<td>Checklists</td>
</tr>
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<td></td>
<td>Symbology and Nomenclature</td>
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</tbody>
</table>

**Table 1-1 Chapter Template and Contents**
1.3 Drainage Design Memoranda

Drainage Design Memoranda contain instructional and informational guidance related specifically to drainage design. The instructions and information contained in these memoranda are subject to relatively frequent changes and have therefore intentionally been excluded from the main body of text in this manual. These memoranda are all contained in Chapter 15. All Technical Supplements, and Drainage Manual Errata Sheets published prior to issuance of this manual are now included in the manual and are hereby voided.

* Rev. 7/14
1.4 References

The manual provides references at points where the designer may need more detailed source material. The reference section at the end of each chapter includes these source documents, as well as a listing of those documents, which are recommended additions to the designer’s library of references.

The following documents are an integral part of VDOT roadway and drainage design:

VDOT Reference Documents (all latest editions)

- VDOT Road and Bridge Standards, Volume I and II
- VDOT Road and Bridge Specifications
- VDOT Instructional and Informational Memoranda
- VDOT Road Design Manual
- Virginia Stormwater Management Program Regulation (9VAC25-870)
- Virginia Erosion and Sediment Control Regulations (9VAC25-840)
- Virginia Erosion and Sediment Control Handbook
- Virginia Stormwater Management Handbook, Volumes I and II
- VDOT Survey Instruction Manual
- VDOT Stormwater BMP Design Manuals of Practice
- VDOT Annual Standards & Specifications for Erosion and Sediment Control and Stormwater Management

Compliance with the following applicable laws and agencies’ regulations and policies are required:

- Virginia Department of Transportation
- Virginia Stormwater Management Program Regulation
- Virginia Erosion and Sediment Control Regulations
- State Drainage Law
- Code of Federal Regulations
- FHWA Federal Aid Policy Guide
- Federal Emergency Management Agency
- Environmental Protection Agency Regulation
- National Pollution Discharge Elimination System (NPDES)
- Department of Environmental Quality

\* Rev. 3/19
1.5 User Instructions

This manual is divided into 17 chapters, each dealing with a major category of hydrologic or hydraulic analysis. Each chapter is further divided according to specific elements of the subject. Departmental policy and design criteria are presented in each chapter as they relate to the specific subject matter.

The downloaded electronic version of the Drainage Manual and its revisions will be considered the official reference document in agreements with consultants. The manual can be downloaded from VDOT's website at the following location: http://www.virginiadot.org/business/locdes/hydra-drainage-manual.asp. The authors of this manual have strived to maintain the accuracy and reliability of the information and procedures presented herein. However, the execution of engineering design involves the judgment of the designer, and only he or she can ascertain whether a technique or item of information can be applied to a given situation. Therefore, neither the Department nor any contributor accepts responsibility for any real or alleged error, loss, damage, or injury resulting from use of the material contained herein.

References to specific computer programs, AASHTO guidelines, manual, and regulations are included in this manual. It is expected that the designer will be knowledgeable in the use of the referenced items. This manual cannot incorporate computer program user manuals or remain current with these programs and the latest drainage-related Federal regulations. The designers should keep themselves up-to-date by contacting either their local, State, or Federal hydrology/hydraulic departments.

This manual is published in U.S. Customary (English) units. In most cases all units, equations, tables, and figures are given in English units. In a few instances, some existing metric information was not converted to English units. The metric units are given so that the material could still be included in the manual. In most cases, computer software is available that allows the use of English units that can be used to obtain the required information.

Information in this Manual may be supplemented and/or revised by drainage related Location and Design Instructional and Informational Memorandums (IIM’s). Information in these IIM’s shall supersede that noted in this Manual unless otherwise approved by the State Hydraulics and Utilities Engineer. Where language in various sections of this Manual conflict, the more stringent language shall dictate unless otherwise approved by the State Hydraulics and Utilities Engineer.

*Rev. 3/19
1.6 Revisions and Updates

VDOT plans to issue updates and revisions to this manual which will be found at the VDOT website. Updates and revisions would normally be anticipated no more than twice a year. Users of the manual should review the VDOT website periodically and prior to beginning design or preparing a plan submittal, to determine the date of the most recent updates. Users that cannot access the information on the Internet may phone the VDOT Hydraulics Section in the nearest district office or Central Office in Richmond, Virginia. When revisions are available, the user will be notified via an Errata Sheet file on the VDOT website at the location where the manual may be viewed and/or downloaded. These files also briefly describe each revision. All revised material (where possible) will be shaded so the user will be able to recognize it as having been changed. The shaded material within any given chapter will remain shaded until the next revision, at which time all previous shading in that chapter will be removed.
1.7 Acknowledgements

The Department gratefully acknowledges the following for their contribution towards the preparation of this Manual:

- American Association of State Highway and Transportation Officials (AASHTO Drainage Manual, Highway Drainage Guidelines, and other publications)
  - Executive Committee
  - Hydrology and Hydraulics Technical Committee
- Federal Highway Administration
- Federal Emergency Management Agency
- United States Geological Survey
- United States Army Corps of Engineers
- Virginia Department of Transportation –
  - Ms. Victoria “Tory” Bains, P.E., State Hydraulics and Utilities Engineer
  - Mr. Chris Swanson, P.E., State MS4/Stormwater Management Engineer
  - Mr. Roy Mills, Former State Hydraulics Engineer and Member AASHTO Hydrology and Hydraulics Technical Committee
- Virginia Department of Conservation and Recreation – Division of Soil & Water Conservation
- Virginia Department of Environmental Quality
- Materials furnished by other state and federal agencies
- Research publications and materials furnished by the private sector
- Photo on cover courtesy of Virginia Department of Game and Inland Fisheries. Photographed by Mr. Dwight Dyke. Big Tumbling Creek at the Clinch Mountain Wildlife Management Area.

The Department’s Hydraulic Section wishes to express its appreciation to all contributors who assisted in the development of this manual.

^Rev. 3/19
## Abbreviations and Definitions

### Abbreviations:
- **AASHTO**  American Association of State Highway and Transportation Officials
- **BDF**  Basin Development Factor
- **BLM**  Bureau of Land Management
- **BMP**  Best Management Practice
- **BRI-STARS**  Bridge Stream Tube Model for Sediment Routing Alluvial River Simulation
- **BSD**  Better Site Design
- **CBPA**  Chesapeake Bay Preservation Area
- **CEM**  Coastal Engineering Manual
- **CF**  Channel Flow
- **CFR**  Code of Federal Regulations
- **DCR**  Department of Conservation and Recreation
- **DDM**  Drainage Design Memorandum
- **DEQ**  Department of Environmental Quality
- **EO**  Executive Orders
- **EPA**  Environmental Protection Agency
- **ESC**  Erosion and Sediment Control
- **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
- **HIRE**  Highways in the River Environment
- **HUC**  Hydrologic Unit Code
- **HW**  Headwater
- **HYG**  Hydrograph
- **I&IM or IIM**  Instructional and Informational Memorandum
- **IDF**  Intensity Duration Frequency
- **LDP**  Land Development Project
- **LID**  Low Impact Development
- **LTEC**  Least Total Expected Cost
- **MHW**  Mean High Water
- **MHHW**  Mean Higher High Water
- **MLW**  Mean Low Water
- **MLLW**  Mean Lower Low Water
- **MS**  Minimum Standard
- **MS4**  Municipal Separate Storm Sewer System
- **MSL**  Mean Sea Level
- **MTL**  Mean Tide Level
- **MTR**  Mean Tide Range
- **NAS**  National Academy of Sciences
<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Full Form</th>
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</thead>
<tbody>
<tr>
<td>NAVD</td>
<td>North America Vertical Datum</td>
</tr>
<tr>
<td>NEH</td>
<td>National Engineering Handbook</td>
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<tr>
<td>NEPA</td>
<td>National Environmental Protection Agency</td>
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<tr>
<td>NFIA</td>
<td>National Flood Insurance Act</td>
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<td>NFIP</td>
<td>National Flood Insurance Program</td>
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<td>NGS</td>
<td>National Geodetic Survey</td>
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<tr>
<td>NGVD</td>
<td>National Geodetic Vertical Datum</td>
</tr>
<tr>
<td>NHS</td>
<td>National Highway System</td>
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<tr>
<td>NOAA</td>
<td>National Oceanic and Atmospheric Administration</td>
</tr>
<tr>
<td>NPDES</td>
<td>National Pollutant Discharge Elimination System</td>
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<tr>
<td>NRCS</td>
<td>National Resource Conservation Service (formally known as the Soil Conservation Service or SCS)</td>
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<tr>
<td>NTIS</td>
<td>National Technical Information Service</td>
</tr>
<tr>
<td>OLF</td>
<td>Overland Flow</td>
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<tr>
<td>PAC</td>
<td>Pre-Advertisement Conference</td>
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<td>P2</td>
<td>Pollution Prevention</td>
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<tr>
<td>R&amp;B</td>
<td>Road and Bridge</td>
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<tr>
<td>RDM</td>
<td>Road Design Manual</td>
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<td>RFP</td>
<td>Request for Proposal</td>
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<tr>
<td>R/W</td>
<td>Right-of-Way</td>
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<tr>
<td>SCS</td>
<td>Soil Conservation Service (former name of the National Resource Conservation Service)</td>
</tr>
<tr>
<td>SPM</td>
<td>Shore Protection Manual</td>
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<tr>
<td>SWCB</td>
<td>Soil and Water Conservation Board</td>
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<tr>
<td>SWL</td>
<td>Still Water Level</td>
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<tr>
<td>SWM</td>
<td>Stormwater Management</td>
</tr>
<tr>
<td>SWMR</td>
<td>Stormwater Management Regulation</td>
</tr>
<tr>
<td>SWPPP</td>
<td>Stormwater Pollution Prevention Plan</td>
</tr>
<tr>
<td>SYIP</td>
<td>Six-Year Improvement Plan</td>
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<tr>
<td>TMDL</td>
<td>Total Maximum Daily Load</td>
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<td>TR</td>
<td>Technical Release</td>
</tr>
<tr>
<td>TVA</td>
<td>Tennessee Valley Authority</td>
</tr>
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<td>TW</td>
<td>Tailwater</td>
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<tr>
<td>USBR</td>
<td>United States Bureau of Reclamation</td>
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<tr>
<td>USCOE/USACE</td>
<td>United States Corps of Engineers</td>
</tr>
<tr>
<td>USGS</td>
<td>United States Geological Survey</td>
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<tr>
<td>VAC</td>
<td>Virginia Administrative Code</td>
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<tr>
<td>VDOT</td>
<td>Virginia Department of Transportation or the “Department”</td>
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<tr>
<td>VESC</td>
<td>Virginia Erosion and Sediment Control</td>
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<tr>
<td>VESCH</td>
<td>Virginia Erosion and Sediment Control Handbook</td>
</tr>
<tr>
<td>VESCR</td>
<td>Virginia Erosion and Sediment Control Regulations</td>
</tr>
<tr>
<td>VPDES</td>
<td>Virginia Pollutant Discharge Elimination System</td>
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<tr>
<td>VRRM</td>
<td>Virginia Runoff Reduction Methodology</td>
</tr>
<tr>
<td>VSMP</td>
<td>Virginia Stormwater Management Program</td>
</tr>
<tr>
<td>VSMR</td>
<td>Virginia Stormwater Management Regulation</td>
</tr>
<tr>
<td>VSWCB</td>
<td>Virginia Soil and Water Conservation Board</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>Description</td>
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<tr>
<td>--------------</td>
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<tr>
<td>WES</td>
<td>Waterways Experiment Station</td>
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<tr>
<td>WQV</td>
<td>Water Quality Volume</td>
</tr>
<tr>
<td>WRC</td>
<td>Water Resources Council</td>
</tr>
</tbody>
</table>
**Definitions:**

<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Land Cover Condition</td>
<td>A measure (in percent) of the average amount of impervious area within a watershed.</td>
</tr>
<tr>
<td>Backshore</td>
<td>The backshore is the area of the coastal zone that extends from the limit of high tides and storm waves to the base of a cliff or beach ridge.</td>
</tr>
<tr>
<td>Barrier Beach</td>
<td>A bar essentially parallel to the shore, which has been built up so that its crest rises above the normal high water level.</td>
</tr>
<tr>
<td>Base Flood</td>
<td>The 100 year flood. This term is used to be consistent with the language in FHWA and FEMA guidance.</td>
</tr>
<tr>
<td>Base Flow</td>
<td>The typical flow that will be encountered in a stream that will not be exceeded for 25% of the year.</td>
</tr>
<tr>
<td>Bridges</td>
<td>(1) Structures that transport traffic over waterways or other obstructions, (2) Part of a stream crossing system that includes the approach roadway over the floodplain, relief openings, and the bridge structure, (3) Structures with a centerline span of 20 feet or more; however, structures designed hydraulically as bridges are considered bridges, regardless of length.</td>
</tr>
<tr>
<td>CBR Tests</td>
<td>The California Bearing Ratio (CBR) test consists of measuring the relative load required to cause a standard (3 square inches) plunger to penetrate a water-saturated soil specimen at a specific depth.</td>
</tr>
<tr>
<td>Channel</td>
<td>A natural or manmade waterway (includes culverts and storm sewer systems).</td>
</tr>
<tr>
<td>Code of Federal Regulations</td>
<td>Codifies and publishes at least annually Federal regulations currently in force. The CFR is kept up to date by individual issues of the Federal Register. The two publications must be used together to determine the latest version of any given rule.</td>
</tr>
<tr>
<td><strong>Common Enemy Doctrine</strong></td>
<td>Common law rule recognized by some states, pertaining to the disposal of surplus or excess surface waters, which holds that such waters are a “common enemy”; therefore, the land owner has the right to protect his lands from such waters coming from higher lands. Under this rule, surface waters are regarded as a common enemy, which each landowner may fight as he deems best and without regard to the harm that may be caused to others.</td>
</tr>
<tr>
<td><strong>Common Laws</strong></td>
<td>The body of principles which developed from immemorial usage and custom and which receives judicial recognition and sanction through repeated application.</td>
</tr>
<tr>
<td><strong>Critical Depth</strong></td>
<td>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.</td>
</tr>
<tr>
<td><strong>Culvert</strong></td>
<td>(1) A structure which is usually designed hydraulically to take advantage of submergence to increase hydraulic capacity, (2) A structure used to convey surface runoff through embankments, (3) A structure, as distinguished from bridges, which is usually covered with embankment and is composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert, (4) A structure which is traditionally 20 feet or less in centerline length between extreme ends of openings for multiple boxes; however, a structure designed hydraulically as a culvert is treated as a culvert, regardless of length.</td>
</tr>
<tr>
<td><strong>Deep-water Wave</strong></td>
<td>A wave in which the depth of water is greater than one-half the wavelength.</td>
</tr>
<tr>
<td><strong>Department</strong></td>
<td>The Virginia Department of Transportation.</td>
</tr>
<tr>
<td>Term</td>
<td>Description</td>
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<tr>
<td>Detention Basins</td>
<td>A basin or reservoir incorporated into the watershed, whereby runoff is temporarily stored, thus attenuating the peak of the runoff hydrograph. A stormwater management facility that impounds runoff and temporarily impounds runoff and discharges it through a hydraulic outlet structure to a downstream conveyance structure.</td>
</tr>
<tr>
<td>Discharge Point</td>
<td>The location at which stormwater and/or a pollutant leaves the project area. Also “point of discharge”.</td>
</tr>
<tr>
<td>Downcast</td>
<td>The direction of predominant movement of littoral currents and transport.</td>
</tr>
<tr>
<td>Embayment</td>
<td>An indentation in a shoreline forming an open bay.</td>
</tr>
<tr>
<td>English Rule</td>
<td>Based on the doctrine of absolute ownership of water beneath the property by the landowner. The English Rule is analogous to the common enemy rule in surface water law.</td>
</tr>
<tr>
<td>Eutrophication</td>
<td>The process of over-enrichment of water bodies by nutrients often typified by the presence of algal blooms.</td>
</tr>
<tr>
<td>Executive Orders</td>
<td>Federal laws that are issued by the President of the United States.</td>
</tr>
<tr>
<td>Federal Register</td>
<td>A daily publication of the Federal Government making federal regulations, legal notices, Presidential Proclamations, Executive Orders, etc., known to the public as they are proposed and subsequently issued.</td>
</tr>
<tr>
<td>Fetch</td>
<td>The length of unobstructed open sea surface across which the wind can generate waves.</td>
</tr>
<tr>
<td>Flood Insurance Study</td>
<td>Established the regulatory floodplain and floodway.</td>
</tr>
<tr>
<td>Flow Type</td>
<td>The USGS has established seven culvert flow types which assist in determining the flow conditions at a particular culvert site.</td>
</tr>
<tr>
<td>Foreshore</td>
<td>The foreshore is the area of the coastal zone that extends from the low-tide level to the limit of high tides and storm-wave effects.</td>
</tr>
<tr>
<td>Term</td>
<td>Definition</td>
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</tr>
<tr>
<td>Free Outlet</td>
<td>A free outlet has a tailwater 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.</td>
</tr>
<tr>
<td>Headland</td>
<td>A headland is the seaward most projection of land from the shoreline. Because headlands project out into waves and currents, headlands are usually subjected to greater erosion forces. Headlands may be the remnants of submerged ridgelines or be composed of erosion resistant materials.</td>
</tr>
<tr>
<td>HUC6</td>
<td>A watershed unit established in the most recent version of Virginia's 6th Order National Watershed Boundary Dataset.</td>
</tr>
<tr>
<td>Hydraulic Grade Line</td>
<td>The elevation to which the water can be expected to rise within a storm drain (pressure head + elevation head)</td>
</tr>
<tr>
<td>Impervious Surface or Cover</td>
<td>A surface composed of any material that significantly impedes or prevents natural infiltration of water into soil.</td>
</tr>
<tr>
<td>Impervious Area</td>
<td>The area (square feet or acres) of the site composed of an impervious surface.</td>
</tr>
<tr>
<td>Improved Inlet</td>
<td>An improved inlet has an entrance geometry, which contracts the flow as it enters the barrel thus increasing the capacity of culvert. These inlets are referred to as either side- or slope-tapered (walls or walls and bottom tapered).</td>
</tr>
<tr>
<td>Karst Topography</td>
<td>Irregular topography characterized by sinkholes, stream-less valleys and streams that disappear into the underground, all developed by the action of surface and underground water in soluble rock such as limestone.</td>
</tr>
<tr>
<td><strong>Term</strong></td>
<td><strong>Definition</strong></td>
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</tr>
<tr>
<td><strong>Lakeshore</strong></td>
<td>The strip of land from a lake shoreline inland to the first major change in terrain features. Except for tidal effects, large lakes and reservoirs of 100 mi² or more in area, have shores that require many of the same type considerations as ocean bays and estuaries. Analagous to coastal zone tides, some lakeshores are subject to significant changes in water surface elevation due to operation practices.</td>
</tr>
<tr>
<td><strong>Land-Disturbing Activity or Land Disturbance</strong></td>
<td>A manmade change to the land surface that potentially changes its runoff characteristics including any associated clearing, grading or excavation.</td>
</tr>
<tr>
<td><strong>Linear Development Projects</strong></td>
<td>Those land-disturbing activities linear in nature such as, but not limited to, highway construction/maintenance projects/activities, construction/maintenance of stormwater channels and stream restoration projects.</td>
</tr>
<tr>
<td><strong>Longshore</strong></td>
<td>Currents or sediment transport that move parallel to the shoreline.</td>
</tr>
<tr>
<td><strong>Managed Turf</strong></td>
<td>Means portions of roadway rights-of-way that are pervious and do not meet the definition of open space.</td>
</tr>
<tr>
<td><strong>Manmade Channel</strong></td>
<td>A machine/hand-defined landform (ie. roadside ditch, EC-2/EC-3/paved/concrete/ riprap lining, and outfall important) or manufactured conveyance structure (ie. storm sewer pipe, culvert, inlet) located in VDOT R/W or municipal drainage easement, as part of a given roadway improvement project, subdivision or drainage improvement project, with requirements of access and maintenance to remain clear of obstructions (ie. debris, sedimentation, rigid structure), for the passing of design storm events applicable to the respective improvement and does not employ natural stream design principals.</td>
</tr>
<tr>
<td><strong>Mean Higher High Water (MHHW)</strong></td>
<td>MHHW is the average tidal elevation of the highest tidal elevation in a tidal day experienced over the 19-year metonic cycle.</td>
</tr>
<tr>
<td><strong>Mean High Water (MHW)</strong></td>
<td>MHW is the average high water elevation (both Higher High Water and Lower High Water) experienced over the 19-year metonic cycle.</td>
</tr>
<tr>
<td>--------------------------</td>
<td>-------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td><strong>Mean Lower Low Water (MLLW)</strong></td>
<td>MLLW is the average tidal elevation of the lowest tidal elevation in a tidal day experienced over the 19-year metonic cycle.</td>
</tr>
<tr>
<td><strong>Mean Low Water (MLW)</strong></td>
<td>Tidal elevations and the vertical datums of coastal bathymetric maps are often referenced to Mean Low Water (MLW). MLW is the average low water elevation (both Lower Low Water and Higher Low Water) experienced over the 19-year metonic cycle.</td>
</tr>
<tr>
<td><strong>Metonic Cycle</strong></td>
<td>The Metonic cycle is a period of 19 years in which there are 235 lunations, or synodic months, after which the Moon's phases recur on the same days of the solar year, or year of the seasons.</td>
</tr>
<tr>
<td><strong>Natural Channel</strong></td>
<td>A landform defined by nature which conveys water, most commonly known as a tributary, stream, river, river delta or strait. A stream that has been modified using stream restoration principals may be considered a natural channel.</td>
</tr>
<tr>
<td><strong>Non-Linear Projects</strong></td>
<td>Those land-disturbing activities not considered linear in nature such as, but not limited to, parking lots, rest areas and District/Residency/Area Headquarter complexes.</td>
</tr>
<tr>
<td><strong>Normal Flow</strong></td>
<td>Normal flow occurs in a channel reach when the discharge, velocity and depth of flow do not change throughout the reach. The water surface and channel bottom will be parallel. This type of flow will exist in a culvert operating on a constant slope provided the culvert is sufficiently long.</td>
</tr>
<tr>
<td><strong>Offsite</strong></td>
<td>Areas located outside of the VDOT right of way, easement or property boundary.</td>
</tr>
<tr>
<td><strong>Onsite</strong></td>
<td>Areas located inside of VDOT right of way, easement or property boundary.</td>
</tr>
<tr>
<td><strong>Open Space</strong></td>
<td>Means portions of roadway rights-of-way and permanent easements associated with a land-disturbing activity that, following construction, have either not been compacted by the activity; have had soil restoration; or have placed engineered soil mix, and will not be actively maintained. Typically, actively maintained means mowed more than four (4) times a year or fertilized. Surfaced area of stormwater facilities, with the exception of wet ponds, shall qualify as open space.</td>
</tr>
<tr>
<td><strong>Outfall</strong></td>
<td>The location where concentrated stormwater leaves the project area.</td>
</tr>
<tr>
<td><strong>Pre-development</strong></td>
<td>Those conditions that exist prior to commencement of the proposed land-disturbing activity/project.</td>
</tr>
<tr>
<td><strong>Pre-development Impervious Area</strong></td>
<td>The amount of impervious area within the site prior to commencement of the proposed land-disturbing activity/project.</td>
</tr>
<tr>
<td><strong>Pre-development Percent Impervious</strong></td>
<td>The amount of pre-development impervious area within the site divided by the total area of the site times 100.</td>
</tr>
<tr>
<td><strong>Post-development</strong></td>
<td>Those conditions that will, or are expected to, exist after completion of the proposed land-disturbing activity/project.</td>
</tr>
<tr>
<td><strong>Post-development Impervious Area</strong></td>
<td>The amount of impervious area within the site that will or is expected to exist after completion of the proposed land-disturbing activity/project.</td>
</tr>
<tr>
<td><strong>Post-development Percent Impervious</strong></td>
<td>The amount of post-development impervious area within the site divided by the total area of the site times 100.</td>
</tr>
<tr>
<td>Term</td>
<td>Definition</td>
</tr>
<tr>
<td>-------------------------------------------</td>
<td>-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td><strong>Q₁ Control</strong></td>
<td>This stormwater management measure is applied to channels with known or anticipated erosion problems as a quantity control measure. In design, the entire contributing drainage area to the proposed basin is captured and used to develop the detention volume for a 1-year storm.</td>
</tr>
<tr>
<td><strong>Reasonable Use Rule</strong></td>
<td>States in essence that each landowner is restricted to a reasonable exercise of his own right and a reasonable use of his property in view of the similar right of his neighbors.</td>
</tr>
<tr>
<td><strong>Receiving Channel</strong></td>
<td>The drainage facility that receives the stormwater runoff from the proposed land-disturbing activity.</td>
</tr>
<tr>
<td><strong>Redevelopment</strong></td>
<td>Means development on prior developed lands that have been previously utilized for residential, commercial, industrial, institutional, recreation, transportation or utility facilities or structures and that will have the impervious areas associated with those uses altered during a land-disturbing activity.</td>
</tr>
<tr>
<td><strong>Regulated Land Disturbance Activities</strong></td>
<td>Those activities that disturb one (1) acre or greater except in those areas designated as a Chesapeake Bay Preservation Area in which case the land disturbance threshold is 2,500 square feet or greater (unless the activity is specifically exempted by the VSMP Law and/or Regulations).</td>
</tr>
<tr>
<td><strong>Regulated Site</strong></td>
<td>The area within the project limits used to determine water quality requirements.</td>
</tr>
<tr>
<td><strong>Retention Basins</strong></td>
<td>A basin or reservoir wherein water is stored for regulating a flood. It does not have an uncontrolled outlet. The stored water is disposed by such means as infiltration, injection (or dry) wells, or by release to the downstream drainage system after the storm event. The release may be through a gate controlled gravity system or by pumping.</td>
</tr>
<tr>
<td><strong>Riparian Doctrine</strong></td>
<td>A doctrine that holds that the property owner adjacent to a surface water body has first right to withdraw and use the water. This doctrine may be set aside by a state’s statutory law that holds that all surface waters are the property of the state.</td>
</tr>
<tr>
<td><strong>Roadway Section</strong></td>
<td>The traveled way and associated shoulders, ditches, sidewalks, multi-use/shared use paths, back (cut) slopes and fore (fill) slopes</td>
</tr>
<tr>
<td>--------------------</td>
<td>----------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td><strong>Seiche</strong></td>
<td>A seiche is an oscillatory wave generated by an impulse that disturbs the local water level equilibrium. The impulse may be a heavy rainfall, vessel passage, tsunami, flood discharge from a river, or a storm surge. Much like dropping a stone in to a tank of water, seiche waves oscillate back and forth, gradually diminishing in magnitude.</td>
</tr>
<tr>
<td><strong>Site</strong></td>
<td>The area of proposed land disturbance (e.g., the construction limits) plus any R/W acquired in support of the proposed land disturbance activity/project. Any support areas within existing or proposed VDOT R/W associated with the proposed land disturbance activity/project and identified in the pre-construction SWPPP for the proposed land disturbance activity/project shall also be considered a part of the site. Permanent easements and/or other property acquired through the R/W acquisition process in conjunction with the proposed land disturbance activity/project may be considered a part of the site and utilized in the determination of the post-development water quality requirements provided such property will remain under the ownership/control of the VDOT and providing such property is so identified/designated on the proposed land disturbance activity/project plans and is legally encumbered for the purpose of stormwater management.</td>
</tr>
<tr>
<td><strong>Spread</strong></td>
<td>The width of flow measured laterally from the flowline. With a curbed only section of roadway, the flowline is formed by the intersection of the pavement to the curb. With a curb and gutter section, it is the intersection of the gutter pan and the curb.</td>
</tr>
<tr>
<td><strong>Statutory Laws</strong></td>
<td>Enacted by legislatures to enlarge, modify, clarify, or change the common law applicable to particular drainage conditions. This type of law is derived from constitutions, statutes, ordinances, and codes.</td>
</tr>
<tr>
<td>Term</td>
<td>Definition</td>
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<tr>
<td>-------------------------------------------</td>
<td>----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td><strong>Still-water Level (SWL)</strong></td>
<td>SWL is used to refer to the imaginary elevation of water if all wave and wind action were to cease. SWL is used to define limits of coastal inundation during storm surges. Actual water levels are higher due to waves.</td>
</tr>
<tr>
<td><strong>Storm Drain</strong></td>
<td>A storm drain system is a drainage system installed to carry stormwater runoff, consisting of two or more pipes in a series connected by one or more drop inlets. An exception to this general rule is: one or more cross drain pipes connected by one or more drop inlets, “hydraulically designed” to function as a culvert(s) and not connected to a storm drain system.</td>
</tr>
<tr>
<td><strong>Stormwater Conveyance System</strong></td>
<td>A combination of drainage components that are used to convey stormwater discharge, either within or downstream of the land-disturbing activity.</td>
</tr>
<tr>
<td><strong>Submerged</strong></td>
<td>A submerged outlet occurs when the tailwater elevation is higher than the crown of the culvert. A submerged inlet occurs when the headwater is greater than 1.2D where D is the culvert diameter or barrel height.</td>
</tr>
<tr>
<td><strong>Surf Zone</strong></td>
<td>The area where deep-water waves break (collapse) forming breakers. Note that on shallow sloped shorelines, waves may reform and more than one surf zone may be present.</td>
</tr>
<tr>
<td><strong>Tort</strong></td>
<td>A violation of a personal right guaranteed to the individual by law.</td>
</tr>
<tr>
<td><strong>Traveled Way or Travel Lane</strong></td>
<td>That portion of the roadway section, exclusive of shoulders, designated for vehicular use.</td>
</tr>
<tr>
<td><strong>Velocity Head</strong></td>
<td>A quantity of energy head proportional to kinetic energy of flowing water.</td>
</tr>
<tr>
<td><strong>Watershed</strong></td>
<td>The surface area, measured in a horizontal plane, draining to a specific point in a channel, stream, river or other such watercourse. Also often referred to as “Drainage Area” or “Drainage Basin”.</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>--------</td>
<td>----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>A</td>
<td>Drainage Area Cross-sectional area of flow; Clear opening area of curb</td>
</tr>
<tr>
<td></td>
<td>inlet or grate; Cross-sectional or surface area</td>
</tr>
<tr>
<td>a</td>
<td>Rainfall regression constant</td>
</tr>
<tr>
<td>a</td>
<td>Depth of depression</td>
</tr>
<tr>
<td>B</td>
<td>Barrel or box width</td>
</tr>
<tr>
<td>b</td>
<td>Manhole diameter or width</td>
</tr>
<tr>
<td>b</td>
<td>Rainfall regression constant</td>
</tr>
<tr>
<td>b₁</td>
<td>Urban Regression Method exponent</td>
</tr>
<tr>
<td>b₂</td>
<td>Urban Regression Method exponent</td>
</tr>
<tr>
<td>b₃</td>
<td>Urban Regression Method exponent</td>
</tr>
<tr>
<td>b₄</td>
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<tr>
<td>b₅</td>
<td>Urban Regression Method exponent</td>
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<tr>
<td>b₆</td>
<td>Urban Regression Method exponent</td>
</tr>
<tr>
<td>b₇</td>
<td>Urban Regression Method exponent</td>
</tr>
<tr>
<td>BDF</td>
<td>Basin Development Factor</td>
</tr>
</tbody>
</table>
### Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>Runoff coefficient; Urban Regression Method constant; Stone size correction factor; Broad-crested weir coefficient or orifice coefficient</td>
</tr>
<tr>
<td>C_d</td>
<td>Overtopping coefficient (Weir coefficient)</td>
</tr>
<tr>
<td>C_f</td>
<td>Frequency factor</td>
</tr>
<tr>
<td>C_N</td>
<td>SCS-runoff curve number</td>
</tr>
<tr>
<td>C_r</td>
<td>Discharge coefficient</td>
</tr>
<tr>
<td>C_SF</td>
<td>Adjustment to the stability factor</td>
</tr>
<tr>
<td>C_sg</td>
<td>Adjustment to the specific gravity of stone</td>
</tr>
<tr>
<td>C_t, C_p</td>
<td>Physiographic coefficients</td>
</tr>
<tr>
<td>C_W</td>
<td>Weir coefficient</td>
</tr>
<tr>
<td>D</td>
<td>Culvert diameter or barrel height; Diameter of pipe</td>
</tr>
<tr>
<td>d</td>
<td>Depth of flow; Depth of gutter flow at the curb line; Change in elevation; Orifice diameter</td>
</tr>
<tr>
<td>D_50</td>
<td>Mean spherical diameter of the 50% size stone</td>
</tr>
<tr>
<td>D_50_s</td>
<td>Required D_50 for side slopes</td>
</tr>
<tr>
<td>d_50</td>
<td>Mean stone size diameter</td>
</tr>
<tr>
<td>d_ave</td>
<td>Average flow depth in the main channel</td>
</tr>
<tr>
<td>d_b</td>
<td>Critical depth at riprap basin overflow</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$d_c$</td>
<td>Critical depth</td>
</tr>
<tr>
<td>$D_o$</td>
<td>Storm duration</td>
</tr>
<tr>
<td>$d_E$</td>
<td>Equivalent brink depth</td>
</tr>
<tr>
<td>$d_l$</td>
<td>Depth at lip of curb opening</td>
</tr>
<tr>
<td>$D_m$</td>
<td>Mean depth of lake or reservoir</td>
</tr>
<tr>
<td>$d_n$ or $d_o$</td>
<td>Normal depth</td>
</tr>
<tr>
<td>$d_s$</td>
<td>Maximum water depth at toe of rock slope protection or bar</td>
</tr>
<tr>
<td>$E$</td>
<td>Specific energy</td>
</tr>
<tr>
<td>$E$</td>
<td>Curb opening efficiency</td>
</tr>
<tr>
<td>$E_o$</td>
<td>Ratio of depression flow to total gutter flow</td>
</tr>
<tr>
<td>$F_r$</td>
<td>Froude Number</td>
</tr>
<tr>
<td>$G$</td>
<td>Coefficient of Skew</td>
</tr>
<tr>
<td>$g$</td>
<td>Acceleration due to gravity</td>
</tr>
<tr>
<td>$H$</td>
<td>Total head loss; Head loss; Depth of water; Wave height</td>
</tr>
<tr>
<td>$h$</td>
<td>Stage or head; Height of curb opening inlet; Head</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$h_{\Delta}$</td>
<td>Bend head loss</td>
</tr>
<tr>
<td>$H_1$</td>
<td>Average design wave height for highest 1%</td>
</tr>
<tr>
<td>$H_{10}$</td>
<td>Average design wave height for highest 10%</td>
</tr>
<tr>
<td>$h_{\text{avg}}$</td>
<td>Average head</td>
</tr>
<tr>
<td>$H_b$</td>
<td>Bend head loss; Design wave height</td>
</tr>
<tr>
<td>$H_D$</td>
<td>Average hydraulic depth</td>
</tr>
<tr>
<td>$h_e$ or $H_E$</td>
<td>Entrance head loss</td>
</tr>
<tr>
<td>$h_f$ or $H_f$</td>
<td>Friction loss; Friction head loss</td>
</tr>
<tr>
<td>$H_g$</td>
<td>Grate losses</td>
</tr>
<tr>
<td>$H_{\text{GL}_{ds}}$</td>
<td>Elevation of the hydraulic grade line at downstream node</td>
</tr>
<tr>
<td>$H_{\text{GL}_{us}}$</td>
<td>Elevation of the hydraulic grade line at upstream node</td>
</tr>
<tr>
<td>$H_j$</td>
<td>Junction losses</td>
</tr>
<tr>
<td>$h_L$</td>
<td>Total head loss due to local minor and friction losses</td>
</tr>
<tr>
<td>$H_L$</td>
<td>Total energy losses</td>
</tr>
<tr>
<td>$h_m$</td>
<td>Minor head loss</td>
</tr>
<tr>
<td>$h_{\text{max}}$</td>
<td>Maximum head</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$H_o$</td>
<td>Outlet or exit losses</td>
</tr>
<tr>
<td>$h_o$</td>
<td>Hydraulic grade line height above outlet invert; Exit head loss; Summation of minor losses</td>
</tr>
<tr>
<td>$h_s$</td>
<td>Depth of riprap basin</td>
</tr>
<tr>
<td>$H_s$</td>
<td>Significant (design) wave height</td>
</tr>
<tr>
<td>$H_v$</td>
<td>Velocity head</td>
</tr>
<tr>
<td>$HW$</td>
<td>Headwater depth (subscript indicates section)</td>
</tr>
<tr>
<td>$HW_i$</td>
<td>Headwater depth as a function of inlet control</td>
</tr>
<tr>
<td>$HW_o$</td>
<td>Headwater depth above outlet invert</td>
</tr>
<tr>
<td>$HW_{oi}$</td>
<td>Headwater depth as a function of outlet control</td>
</tr>
<tr>
<td>$HW_r$</td>
<td>Headwater depth above roadway</td>
</tr>
<tr>
<td>$I$</td>
<td>Inflow rate</td>
</tr>
<tr>
<td>$i$</td>
<td>Average Rainfall intensity; Rainfall intensity</td>
</tr>
<tr>
<td>$I_1$</td>
<td>Inflow rate at time 1</td>
</tr>
<tr>
<td>$I_2$</td>
<td>Inflow rate at time 2</td>
</tr>
<tr>
<td>$IA$</td>
<td>Percentage of impervious area</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$I_a$</td>
<td>Initial abstraction from total rainfall</td>
</tr>
<tr>
<td>$K$</td>
<td>Channel conveyance; Statistical Method Frequency Factor; Anderson Method Coefficient of Imperviousness; Bend loss coefficient; Entrance loss coefficient; Exit loss coefficient</td>
</tr>
<tr>
<td>$K$</td>
<td>Conveyance of cross section</td>
</tr>
<tr>
<td>$K_e$</td>
<td>Entrance loss coefficient</td>
</tr>
<tr>
<td>$K_o$</td>
<td>Initial head loss coefficient</td>
</tr>
<tr>
<td>$K_o$</td>
<td>Conveyance of the gutter section beyond depression</td>
</tr>
<tr>
<td>$k_l$</td>
<td>Submergence coefficient</td>
</tr>
<tr>
<td>$K_w$</td>
<td>Conveyance of the depressed gutter section</td>
</tr>
<tr>
<td>$L$</td>
<td>Discharge-weighted or conveyance reach length; Length of culvert; Length of roadway crest; Flow Length or Length of Strip; Anderson Method Basin Length or Snyder Method Channel Length; Length of grate inlet; Length of curb opening; Length of pipe; Broad-crested weir length</td>
</tr>
<tr>
<td>$L'$</td>
<td>Equivalent length of channel</td>
</tr>
<tr>
<td>$L_B$</td>
<td>Length of riprap basin</td>
</tr>
<tr>
<td>$L_{ca}$</td>
<td>Length along main channel to a point opposite the watershed centroid</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>--------------------------------------------------</td>
</tr>
<tr>
<td>$L_R$</td>
<td>Require length of inlet</td>
</tr>
<tr>
<td>$L_s$</td>
<td>Length of dissipating pool</td>
</tr>
<tr>
<td>$L_T$</td>
<td>Curb opening length for 100% interception</td>
</tr>
<tr>
<td>$M$</td>
<td>Rank of a flood within a long record</td>
</tr>
<tr>
<td>$m$</td>
<td>Number of flow segments</td>
</tr>
<tr>
<td>$N$</td>
<td>Number of years of flood record</td>
</tr>
<tr>
<td>$n$</td>
<td>Manning's roughness coefficient</td>
</tr>
<tr>
<td>$O_1$</td>
<td>Outflow rate at time 1</td>
</tr>
<tr>
<td>$O_2$</td>
<td>Outflow rate at time 2</td>
</tr>
<tr>
<td>$P$</td>
<td>Precipitation</td>
</tr>
<tr>
<td>$P$ and $P_w$</td>
<td>Wetted perimeter; Perimeter of grate opening</td>
</tr>
<tr>
<td>$Q$</td>
<td>Statistical Method Mean of Logs</td>
</tr>
<tr>
<td>$Q$</td>
<td>Discharge; Total flow to inlet or flow in gutter</td>
</tr>
<tr>
<td>$Q$</td>
<td>SCS Direct Runoff</td>
</tr>
<tr>
<td>$q$</td>
<td>Storm runoff during a time interval</td>
</tr>
<tr>
<td>$Q$, $Q_p$</td>
<td>Maximum rate of runoff or Peak Discharge;</td>
</tr>
<tr>
<td></td>
<td>Discharge or flow rate</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>--------------------------------------</td>
</tr>
<tr>
<td>Q_{avg}</td>
<td>Average flow rate</td>
</tr>
<tr>
<td>Q_{b}</td>
<td>Bypass flow</td>
</tr>
<tr>
<td>Q_{d}</td>
<td>Discharge through the culvert</td>
</tr>
<tr>
<td>Q_{i}</td>
<td>Peak inflow rate</td>
</tr>
<tr>
<td>Q_{L}</td>
<td>Mean of the logarithms of the peak annual floods</td>
</tr>
<tr>
<td>Q_{o}</td>
<td>Peak outflow rate</td>
</tr>
<tr>
<td>q_{o}</td>
<td>Allowable outflow rate</td>
</tr>
<tr>
<td>Q_{s}</td>
<td>Gutter capacity above the depressed section</td>
</tr>
<tr>
<td>Q_{t}</td>
<td>Design or check discharge at culvert</td>
</tr>
<tr>
<td>Q_{T}</td>
<td>Total flow</td>
</tr>
<tr>
<td>Q_{t}</td>
<td>Maximum allowable flow</td>
</tr>
<tr>
<td>Q_{w}</td>
<td>Flow in width W</td>
</tr>
<tr>
<td>R</td>
<td>Flood frequency ratio</td>
</tr>
<tr>
<td>R</td>
<td>Hydraulic radius (A/P)</td>
</tr>
<tr>
<td>RC</td>
<td>Regression constant</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>R&lt;sub&gt;f&lt;/sub&gt;</td>
<td>Ratio of frontal flow intercepted to total flow</td>
</tr>
<tr>
<td>RI2</td>
<td>Rainfall intensity for the 2-hr, 2-yr occurrence</td>
</tr>
<tr>
<td>RQ</td>
<td>Equivalent rural peak runoff rate</td>
</tr>
<tr>
<td>RQT</td>
<td>Rural Regression for Return Period T Peak Discharge</td>
</tr>
<tr>
<td>R&lt;sub&gt;s&lt;/sub&gt;</td>
<td>Ratio of side flow intercepted to total flow</td>
</tr>
<tr>
<td>S</td>
<td>Anderson Method Index of Basin Slope</td>
</tr>
<tr>
<td>S</td>
<td>SCS Method Potential maximum retention storage</td>
</tr>
<tr>
<td>S</td>
<td>Slope of the energy grade line; Longitudinal slope of pavement or gutter slope</td>
</tr>
<tr>
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<td>Storage volume</td>
</tr>
<tr>
<td>S or Y</td>
<td>Ground slope</td>
</tr>
<tr>
<td>S&lt;sub&gt;1&lt;/sub&gt;</td>
<td>Storage volume at time 1</td>
</tr>
<tr>
<td>S&lt;sub&gt;2&lt;/sub&gt;</td>
<td>Storage volume at time 2</td>
</tr>
<tr>
<td>S&lt;sub&gt;A&lt;/sub&gt;</td>
<td>Average slope of the energy grade line</td>
</tr>
<tr>
<td>S&lt;sub&gt;e&lt;/sub&gt;</td>
<td>Equivalent cross slope</td>
</tr>
<tr>
<td>SF</td>
<td>Stability factor applied</td>
</tr>
<tr>
<td>S&lt;sub&gt;f&lt;/sub&gt;</td>
<td>Friction slope</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$S_g$</td>
<td>Specific gravity of rock riprap</td>
</tr>
<tr>
<td>$S_{gr}$</td>
<td>Specific gravity of stone</td>
</tr>
<tr>
<td>$S_{gw}$</td>
<td>Specific gravity of water</td>
</tr>
<tr>
<td>$SL$</td>
<td>Urban Regression Method Main Channel Slope</td>
</tr>
<tr>
<td>$S_L$</td>
<td>Standard Deviation</td>
</tr>
<tr>
<td>$S_o$</td>
<td>Channel slope; Slope of Culvert</td>
</tr>
<tr>
<td>$SP$</td>
<td>Storage parameter (Pagan Method)</td>
</tr>
<tr>
<td>$ST$</td>
<td>Basin storage factor</td>
</tr>
<tr>
<td>$STO$</td>
<td>Maximum storage volume (Pagan Method)</td>
</tr>
<tr>
<td>$S_w$</td>
<td>Depression section slope or gutter cross slope; Gutter cross slope including local depression</td>
</tr>
<tr>
<td>$S_x$</td>
<td>Cross Slope</td>
</tr>
<tr>
<td>$\Delta t$</td>
<td>Routing time period (timestep)</td>
</tr>
<tr>
<td>$T$</td>
<td>Anderson Method Lag Time</td>
</tr>
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<td>$T$</td>
<td>Top width at the water surface; Spread</td>
</tr>
<tr>
<td>$T_b$, $t_b$</td>
<td>Time base on hydrograph</td>
</tr>
</tbody>
</table>
### Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_c, t_c )</td>
<td>Time of concentration; Modified Critical Storm Duration</td>
<td>minutes</td>
</tr>
<tr>
<td>( T_d )</td>
<td>Critical storm duration</td>
<td>minutes</td>
</tr>
<tr>
<td>( T_i )</td>
<td>Duration of basin inflow</td>
<td>hours, minutes</td>
</tr>
<tr>
<td>( \tau_{\text{max}} )</td>
<td>Maximum tractive force</td>
<td>lbs/ft(^2)</td>
</tr>
<tr>
<td>( \tau_o )</td>
<td>Average tractive force</td>
<td>lbs/ft(^2)</td>
</tr>
<tr>
<td>( T_p )</td>
<td>Time to Peak</td>
<td>minutes</td>
</tr>
<tr>
<td>( \tau_p )</td>
<td>Permissible shear stress</td>
<td>lbs/ft(^2)</td>
</tr>
<tr>
<td>( T_r )</td>
<td>Time to Recede</td>
<td>minutes</td>
</tr>
<tr>
<td>( \tau_s )</td>
<td>Side slope shear stress</td>
<td>lbs/ft(^2)</td>
</tr>
<tr>
<td>( T_s )</td>
<td>Spread over depressed section</td>
<td>feet</td>
</tr>
<tr>
<td>( T_t )</td>
<td>Travel time</td>
<td>hours</td>
</tr>
<tr>
<td>( TW )</td>
<td>Tailwater depth above invert of culvert</td>
<td>feet</td>
</tr>
<tr>
<td>( UQ )</td>
<td>Urban Regression Method peak runoff rate</td>
<td>cfs</td>
</tr>
<tr>
<td>( UQ_T )</td>
<td>Peak runoff rate for Urban Watershed for Return Period ( T )</td>
<td>cfs</td>
</tr>
<tr>
<td>( V )</td>
<td>Mean velocity; Average velocity of flow; Velocity of flow in gutter</td>
<td>fps</td>
</tr>
<tr>
<td>( V )</td>
<td>Storage volume</td>
<td></td>
</tr>
</tbody>
</table>

---

**Chapter 1 – Introduction**

**Appendix 1A-2**

**Symbols**
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_a$</td>
<td>Average velocity in main channel</td>
<td>fps</td>
</tr>
<tr>
<td>$V_B$</td>
<td>Average velocity at riprap basin overflow</td>
<td>fps</td>
</tr>
<tr>
<td>$V_{ce}$</td>
<td>Channel erosion control volume</td>
<td>ft$^3$, ac-ft.</td>
</tr>
<tr>
<td>$V_d$</td>
<td>Average velocity in downstream channel</td>
<td>fps</td>
</tr>
<tr>
<td>$V_L$</td>
<td>Average velocity at length (L) downstream from brink</td>
<td>fps</td>
</tr>
<tr>
<td>$V_o$</td>
<td>Average velocity of flow at culvert outlet; Gutter velocity where splash-over first occurs</td>
<td>fps</td>
</tr>
<tr>
<td>$V_s$</td>
<td>Storage volume estimate</td>
<td>ft$^3$, ac-ft.</td>
</tr>
<tr>
<td>$V_u$</td>
<td>Average velocity in upstream channel</td>
<td>fps</td>
</tr>
<tr>
<td>$V_w$</td>
<td>Wave velocity</td>
<td>fps</td>
</tr>
<tr>
<td>$W$</td>
<td>Drainage area width; Width of depression; Width of gutter pan; Width of grate</td>
<td>feet</td>
</tr>
<tr>
<td>$W$</td>
<td>Minimum weight of outside stones for no damage</td>
<td>tons</td>
</tr>
<tr>
<td>$W_{50}$</td>
<td>Weight of the 50% size stone</td>
<td>lbs.</td>
</tr>
<tr>
<td>$W_B$</td>
<td>Width of riprap basin at overflow</td>
<td>feet</td>
</tr>
<tr>
<td>$W_o$</td>
<td>Width dimension of culvert shape</td>
<td>feet</td>
</tr>
<tr>
<td>$WQV$</td>
<td>Water quality volume</td>
<td>ft$^3$</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
<td>Unit</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
<td>------</td>
</tr>
<tr>
<td>$X$</td>
<td>Logarithm of the annual peak</td>
<td>-</td>
</tr>
<tr>
<td>$y$</td>
<td>Depth of flow in approach gutter</td>
<td>feet</td>
</tr>
<tr>
<td>$Z$</td>
<td>$T/d$, reciprocal of the cross slope</td>
<td>-</td>
</tr>
<tr>
<td>$z$</td>
<td>Elevation head</td>
<td>feet</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Velocity distribution coefficient</td>
<td>-</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Angle of face slope from the horizontal</td>
<td>degrees</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Side slope angle; Angle with respect to centerline of outlet pipe; Angle of embankment with respect to the horizontal</td>
<td>degrees</td>
</tr>
<tr>
<td>$\rho$</td>
<td>$70^\circ$ for randomly placed rubble</td>
<td>degrees</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Angle of repose of material</td>
<td>degrees</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Unit weight of water</td>
<td>lbs/ft$^3$</td>
</tr>
</tbody>
</table>
# Chapter 2 - Policy

## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CHAPTER 2 - POLICY</strong></td>
<td>2-1</td>
</tr>
<tr>
<td><strong>2.1 Overview</strong></td>
<td>2-1</td>
</tr>
<tr>
<td>2.1.1 Introduction</td>
<td>2-1</td>
</tr>
<tr>
<td>2.1.2 Policy vs. Criteria</td>
<td>2-1</td>
</tr>
<tr>
<td>2.1.3 Location of Policy and Criteria</td>
<td>2-2</td>
</tr>
<tr>
<td><strong>2.2 General Hydraulic Design Policies</strong></td>
<td>2-3</td>
</tr>
<tr>
<td>2.2.1 Introduction</td>
<td>2-3</td>
</tr>
<tr>
<td>2.2.2 Hydrologic Analysis</td>
<td>2-3</td>
</tr>
<tr>
<td>2.2.3 Hydraulic Analysis</td>
<td>2-3</td>
</tr>
<tr>
<td>2.2.4 Engineering Evaluation</td>
<td>2-4</td>
</tr>
<tr>
<td>2.2.5 General Policies</td>
<td>2-4</td>
</tr>
<tr>
<td><strong>2.3 References</strong></td>
<td>2-6</td>
</tr>
</tbody>
</table>

List of Appendices
Chapter 2 - Policy

2.1 Overview

2.1.1 Introduction

Drainage concerns are one of the most important aspects of highway design and construction. The purpose of this chapter and manual is to outline policies which, when carried out, will provide an appropriate level of consideration for the multitude of variables which influence drainage design.

The drainage policies of the Department have been established to provide continuity in the design and operation of the state highway system, to enhance traffic safety, to ensure the use of technically accepted materials and procedures, to provide the most cost-effective highway facilities, and to ensure the fulfillment of all legal obligations.

Compliance with all policies is essential to ensure the uniformity of the highway system and the timely preparation of plans; however, it is recognized that site specific circumstances may not always be best served by the written policy. In those situations where a waiver from policy is desired, a request for waiver along with proper justification must be submitted to the VDOT Location and Design Division.

Generally, the Department does not waive the basic policies that require hydraulic studies for all projects involving drainage facilities or floodplain encroachments. Typically, it is VDOT's criteria, such as freeboard, that are considered for a waiver, and then only on rehabilitation or replacement projects, not on new construction.

2.1.2 Policy vs. Criteria

Policy and criteria statements are frequently closely related - criteria being the numerical or specific guidance, which is founded in broad policy statements. For this manual, the following definitions of policy and criteria will be used.

- **Policy** - a definite course of action or method of action, selected to guide and determine present and future decisions.

- **Design Criteria** – the standards by which a policy is carried out or placed in action. Therefore, design criteria are needed for design while policy statements are not.

Following is an example of a policy statement:

“The designer will size the drainage structure to accommodate a flood compatible with the projected traffic volumes.”

The design criteria for designing the structure might be:
“For projected traffic volumes less than or equal to 750 vehicles per day, drainage structures shall be designed for a 10-year flood (exceedance probability: 10%). For projected traffic volumes greater than 750 vehicles per day, a drainage structure shall be designed for a 25-year flood (exceedance probability: 4%).”

2.1.3 Location of Policy and Criteria

This chapter presents VDOT general policy. The policy and criteria for specific types of drainage facilities are located in the appropriate chapter for each type of facility (i.e.: culverts, storm sewers, etc.).
2.2 General Hydraulic Design Policies

2.2.1 Introduction

An adequate drainage structure may be defined as one which meets the following policies:

- The design of the structure meets or exceeds standard engineering practice
- The design is consistent with what a reasonably competent and prudent designer would do under similar circumstances

The studies listed below are commonly conducted as a part of the design of most highway drainage structures and serve as a means of achieving an adequate drainage design:

- Hydrologic analysis
- Hydraulic analysis
- Engineering evaluation of alternatives

These studies are discussed further in the following sections.

2.2.2 Hydrologic Analysis

Present state-of-practice formulas and models for estimating flood flows are based on statistical analyses of rainfall and runoff records and therefore provide statistical estimates of flood flows with varying degrees of error. The recommended practice is for the designer to select appropriate hydrologic estimating procedures, and obtain runoff data where available for purposes of evaluation, calibration, and determination of the predicted value of the desired flood frequencies. Since the predicted value of the flood flows represents the designer's best estimate, there is a chance that the true value of the flow for any flood event will be greater or smaller than the predicted value. The expected magnitude of this variation can be determined for some formulas or models as a part of the hydrologic design procedure.

2.2.3 Hydraulic Analysis

The next step in the design process involves development of preliminary alternative designs that are judged to meet the site conditions and to accommodate the flood flows selected for analysis. The hydraulic analysis is made utilizing appropriate formulas, physical models or computer programs for purposes of defining, calibrating, and checking the performance of the preliminary designs over a range of flows.
2.2.4 Engineering Evaluation

The final step in the design process is the engineering evaluation of the trial designs and approval of the selected final design. This process involves consideration and balancing of a number of factors including:

- Legal considerations
- Flood hazards to highway users and neighboring property owners
- Costs
- Environmental and social concerns
- Operations and maintenance
- Other site concerns

2.2.5 General Policies

The hydrologic and hydraulic analyses described above set forth the design process that represents the present “standard engineering practice.” Engineering evaluation outlines the approach to be followed by a “reasonably competent and prudent designer” in evaluating, selecting, and approving a final design. The following policies are made in regard to this process:

- It is the designer’s responsibility to provide an adequate drainage structure. The designer is not required to provide a structure that will handle all conceivable flood flows under all possible site conditions.

- The detail of design studies should be commensurate with the risk associated with the encroachment and with other economic, engineering, social, or environmental concerns.

- The overtopping and/or design flood may serve as criteria for evaluating the adequacy of a proposed design. The “overtopping flood” is the smallest recurrence interval flood, which will result in flow over the highway or other watershed boundary. The “overtopping flood” flow is the flow that overtops the highway or other watershed boundary limit. The “design flood” is the recurrence interval of the flood for which the drainage structure is sized to assure that no traffic interruption or significant damage will result. The overtopping flood and the design flood may vary widely depending on the grade, alignment, and classification of the road and the characteristics of the watercourse and floodplain.
The predicted value of the 100-year or base flood serves as the present engineering standard for evaluating flood hazards and as the basis for regulating flood plains under the National Flood Insurance Program. The designer must make a professional judgment as to the degree of risk that is tolerable for the base flood on a case-by-case basis.

The developed hydraulic performance curve of a drainage structure depicts the relationship between floodwater stage (or elevation) and flood flow magnitudes and frequencies. The performance curve should include the 100-year flood. With the performance curve, the designer can evaluate the adequacy of the design for a range of flows and take into consideration errors of estimate in the hydrologic estimating procedure. It is standard engineering practice to use the predicated value of the 100-year flood as the basis for evaluating flood hazards; however, flows larger than this value may be considered for complex, high risk, or unusual cases that require special studies or risk analyses.

See IIIM-146* for state participation in the cost of storm sewers in counties, towns and cities.

* Rev. 7/14
2.3 References


* Rev. 1/17
# Chapter 3 - Documentation

## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>CHAPTER 3 - DOCUMENTATION</td>
<td>3-1</td>
</tr>
<tr>
<td>3.1 Overview</td>
<td>3-1</td>
</tr>
<tr>
<td>3.1.1 Introduction</td>
<td>3-1</td>
</tr>
<tr>
<td>3.1.2 Definition</td>
<td>3-1</td>
</tr>
<tr>
<td>3.1.3 Purpose</td>
<td>3-1</td>
</tr>
<tr>
<td>3.1.4 Types</td>
<td>3-2</td>
</tr>
<tr>
<td>3.1.5 Scheduling</td>
<td>3-3</td>
</tr>
<tr>
<td>3.1.6 Responsibility</td>
<td>3-4</td>
</tr>
<tr>
<td>3.2 Procedure</td>
<td>3-5</td>
</tr>
<tr>
<td>3.2.1 Introduction</td>
<td>3-5</td>
</tr>
<tr>
<td>3.2.2 Practices</td>
<td>3-5</td>
</tr>
<tr>
<td>3.2.3 Storage</td>
<td>3-6</td>
</tr>
<tr>
<td>3.3 Documentation Procedures</td>
<td>3-7</td>
</tr>
<tr>
<td>3.3.1 Introduction</td>
<td>3-7</td>
</tr>
<tr>
<td>3.3.2 Computer Files</td>
<td>3-7</td>
</tr>
<tr>
<td>3.3.3 Schedule</td>
<td>3-7</td>
</tr>
<tr>
<td>3.3.4 Guidelines</td>
<td>3-8</td>
</tr>
<tr>
<td>3.3.4.1 Plan Measurements</td>
<td>3-8</td>
</tr>
<tr>
<td>3.3.4.2 Pipe Lengths</td>
<td>3-9</td>
</tr>
<tr>
<td>3.3.4.3 Skew Angle of Culverts</td>
<td>3-10</td>
</tr>
<tr>
<td>3.3.4.4 Structure Numbers</td>
<td>3-10</td>
</tr>
<tr>
<td>3.3.4.5 Protective Coatings</td>
<td>3-10</td>
</tr>
<tr>
<td>3.3.4.6 Pipe Descriptions</td>
<td>3-11</td>
</tr>
<tr>
<td>3.3.4.6.1 Typical Culvert Descriptions</td>
<td>3-13</td>
</tr>
<tr>
<td>3.3.4.6.2 Concrete Pipe on Radius</td>
<td>3-13</td>
</tr>
<tr>
<td>3.3.4.6.3 Trenchless Applications</td>
<td>3-14</td>
</tr>
<tr>
<td>3.3.4.6.4 Multiple Pipe Installation</td>
<td>3-14</td>
</tr>
<tr>
<td>3.3.4.6.5 Existing Pipe Extension</td>
<td>3-14</td>
</tr>
<tr>
<td>3.3.4.7 Box Culvert Descriptions</td>
<td>3-15</td>
</tr>
<tr>
<td>3.3.4.7.1 Standard (Cast-In-Place)</td>
<td>3-15</td>
</tr>
<tr>
<td>3.3.4.7.2 Precast</td>
<td>3-15</td>
</tr>
<tr>
<td>3.3.4.8 Structures</td>
<td>3-15</td>
</tr>
<tr>
<td>3.3.4.8.1 Curb Drop Inlets</td>
<td>3-16</td>
</tr>
<tr>
<td>3.3.4.8.2 Grate Drop Inlets</td>
<td>3-16</td>
</tr>
<tr>
<td>3.3.4.8.3 Manholes</td>
<td>3-17</td>
</tr>
<tr>
<td>3.3.4.8.4 Junction Boxes</td>
<td>3-17</td>
</tr>
<tr>
<td>3.3.4.8.5 Stormwater Management Structures</td>
<td>3-17</td>
</tr>
<tr>
<td>3.3.4.8.6 Existing Structures</td>
<td>3-18</td>
</tr>
<tr>
<td>3.3.4.9 Drainage Summaries and Type of Pipe Selection</td>
<td>3-19</td>
</tr>
<tr>
<td>3.3.4.10 Post Installation Pipe Inspection</td>
<td>3-22</td>
</tr>
<tr>
<td>3.4 References</td>
<td>3-24</td>
</tr>
</tbody>
</table>
List of Appendices

Appendix 3A-1  PFI Milestone Deliverable
Appendix 3A-2  PH Milestone Deliverable
Appendix 3A-3  FI Milestone Deliverable
Appendix 3A-4  PAC Milestone Deliverable
Appendix 3A-5  Drainage Summary
Appendix 3B-1  Documentation Data Sheet for Hydrologic & Hydraulic Computations
Appendix 3B-2  Suggested Outline for VDOT H&H Analysis Report
Appendix 3B-3  Field Engineer's Hydraulic Report
Chapter 3 - Documentation

3.1 Overview

3.1.1 Introduction

An important part of the design or analysis of any hydraulic facility is the documentation. Appropriate documentation of the design of any hydraulic facility is essential because of:

- The importance of public safety
- Justification of expenditure of public funds
- Future reference by engineers (when improvements, changes, or rehabilitations are made to the highway facilities or adjacent property)
- Information leading to the development of defense in matters of litigation
- Information is available to public

Frequently, it is necessary to refer to plans, specifications, and analysis long after the actual construction has been completed. Documentation permits evaluation of the performance of structures after flood events to determine if the structures performed as anticipated or to establish the cause of unexpected behavior, if such is the case. In the event of a failure, it is essential that contributing factors be identified in order that recurring damage can be avoided.

3.1.2 Definition

The definition of hydrologic and hydraulic documentation as used in this chapter is the compilation and preservation of the design and related details, as well as all pertinent information related to the basis of design and decisions. This should include drainage area and other maps, field survey information, source references, photographs, engineering calculations and analyses, measured and other data, flood history including narratives from newspapers, individuals such as highway maintenance personnel, and local residents who witnessed or had knowledge of an unusual event.

3.1.3 Purpose

This chapter describes the documentation that should be included in the design files and on the construction plans. While the documentation requirements for existing and proposed drainage facilities are similar, the data retained for existing facilities are often slightly different from that for proposed facilities, and these differences are discussed. This chapter identifies a system for organizing the documentation of hydraulic designs and reviews to provide as complete a history of the design process as is practical.
The major purpose of providing good documentation is to define the design procedure that was used and to show how the final design and decisions were made. There is a myth that avoiding documentation will prevent or limit litigation losses as it supposedly precludes providing the plaintiff with incriminating evidence. This is seldom if ever the case and documentation should be viewed as the record of reasonable and prudent design analysis based on the best available technology. Thus, good documentation can provide the following:

- Protection for the Department by proving that reasonable and prudent actions, were in fact, taken (such proof should certainly not increase the potential court award, and may decrease it by disproving any claims of negligence by the plaintiff)
- Identifying the situation at the time of design which might be very important if legal action occurs in the future
- Documenting that rationally accepted procedures and analysis were used at the time of the design which were commensurate with the perceived site importance and flood hazard (this should further disprove any negligence claims)
- A continuous site history to facilitate future reconstruction
- The file data necessary to quickly evaluate any future site problems that might occur during the facilities service life
- Expediting plan development by clearly providing the reasons and rationale for specific design decisions

3.1.4 Types

Three basic types of documentation should be considered: preconstruction, design, and construction or operation.

1. Preconstruction documentation should include the following if available or within the budgetary constraints of the project.

- Aerial photographs
- Topographic mapping with contours
- Watershed map or plan including
  - Flow directions
  - Watershed boundaries
  - Watershed areas quantified
  - Natural storage areas
  - Existing and proposed contours
- Surveyed data reduced to include
  - Existing hydraulic facilities
  - Existing controls
  - Profiles - roadway, channel, driveways
  - Cross sections - roadway, channels, faces of structures

Chapter 3-2 of 24
• Flood insurance studies (including any available hydraulic model data), and maps by FEMA
• Soil Conservation Service soil maps
• Field trip report(s) which may include:
  - Video cassette recordings
  - Audio tape recordings
  - Still camera photographs
  - Written analysis of findings with sketches
• Reports from other agencies (local, State or Federal), VDOT personnel, newspapers, and abutting property owners

2. Design documentation should include all the information used to justify the design, including:

• Reports from other agencies
• Hydrological report
• Hydraulic report

3. Construction and operation documentation should include:

• Plans
• Revisions
• As-built plans and subsurface borings
• Photographs
• Record of operation: during flooding events, complaints, and resolutions

It is very important to prepare and maintain, in a permanent file, any available as-built plans and plan revisions for every drainage structure to document subsurface foundation elements; such as, footing types and elevations, pile types, and (driven) tip elevations, etc. There may be other information which should be included or may become evident as the design or investigation develops. This additional information should be incorporated at the discretion of the designer.

3.1.5 Scheduling

Documentation should not be considered as occurring at specific times during the design or as the final step in the process, which could be long after the final design is completed. Documentation should be an ongoing process and part of each step in the hydrologic and hydraulic analyses and the design process. This will increase the accuracy of the documentation, provide data for future steps in the plan development process, and provide consistency and continuity in the design even when different designers are involved at different times of the plan development process.
3.1.6 Responsibility

The designer should be responsible for determining what hydrologic analyses, hydraulic design, and related information should be documented during the plan development process. This designer should make a determination that complete documentation has been achieved during the plan development process which will include the final design. To assist in this determination, refer to Appendix 3B for the following:

- Project Documentation Checklist (Appendix 3B-1)
- Suggested outline for a VDOT Hydrologic and Hydraulic Analysis Report (Appendix 3B-2)
- Field Engineer’s Hydraulic Report (Appendix 3B-3)
3.2 Procedure

3.2.1 Introduction

The designer should maintain a complete hydrologic and hydraulic design and analysis documentation file for each waterway encroachment or crossing. Where practicable this file should include such items as:

- Identification and location of the facility
- Roadway functional classification data
- Photographs (ground and aerial)
- Engineering cost estimates
- Actual construction costs
- Hydrologic investigations
- Drainage area maps
- Vicinity maps and topographic maps
- Contour maps
- Interviews (local residents, adjacent property owners, and maintenance forces)
- Newspaper clippings
- Design notes and correspondence relating to design decisions
- History of performance of existing structure(s)
- Assumptions

The documentation file should contain design/analysis data and information that influenced the facility design and which may not appear in other project documentation.

3.2.1 Practices

Following are the practices related to documentation of hydrologic and hydraulic designs and analyses:

- Hydrologic and hydraulic data, preliminary calculations, analyses, and all related information used in developing conclusions and recommendations related to drainage requirements, including estimates of structure size and location should be compiled in a documentation file
- The designer should document all design assumptions and selected criteria including the decisions related thereto
- The amount of detail of documentation for each design or analysis should be commensurate with the risk and the importance of the facility. Typically, culverts would normally require less documentation, whereas bridges and other major drainage structures would require more
- Documentation should be organized to be concise and complete, so that knowledgeable designers can understand years hence what predecessors did
• Circumvent incriminating statements wherever possible by stating uncertainties in less than specific terms - (e.g., "the culvert may cause back water" rather than the "culvert will cause back water"). Be objective in your statements, and opinions
• Provide all related references in the documentation file to include such things as published data and reports, memos and letters, and interviews. Include dates and signatures where appropriate
• Documentation should include data and information from the conceptual stage of project development through service life to provide successors with all information
• Documentation should be organized to logically lead the reader from past history through the problem background, into the findings, and through the performance
• In the case of lengthy documentation assemblies, a summary and table of contents at the beginning of the documentation will provide an outline of the documentation file to assist users in finding detailed information

3.2.2 Storage

Where and how to store and preserve records is an important consideration. Ease of access, durability, legibility, storage space required, and cost are the prime factors to consider when evaluating alternative methods of storage and preservation.

The designer should maintain the documentation files including: microfilm, microfiche, digital media, magnetic media, etc. where it will be readily available for use during construction, for defense of litigation, and future replacement or extension. The designer should retain only documentation that is not retained elsewhere. Original plans, project correspondence files, construction modifications, and inspection reports are the types of documentation that usually do not need to be duplicated. Hydrologic and hydraulic documentation should be retained with the project plans or other permanent location at least until the drainage facility is totally replaced or modified as a result of a new drainage study or a minimum of 10 years after construction.
3.3 Documentation Procedures

3.3.1 Introduction

Documentation procedures for the major hydrologic and hydraulic chapters are in the Procedure section for the respective chapters. The items described should be in the documentation file. The intent is not to limit the data to only those items listed, but rather to establish a suggested minimum requirement consistent with the hydraulic design procedures as outlined in this manual. If circumstances are such that the drainage facility is sized by other than normal procedures or if the size of the facility is governed by factors other than hydrologic or hydraulic factors, a narrative summary detailing the design basis should appear in the documentation file. Additionally, the designer should include in the documentation file items not listed below but which are useful in understanding the analyses, design, findings, and final recommendations.

3.3.2 Computer Files

The following items should be included in the documentation file, and be clearly labeled:

- Input data listing
- Output results of alternatives
- Version of software
- Limitations and capabilities of software
- File names and dates
- Verification of methodology and solution/results
- Quality control practices
- Derivation of formulas for desktop applications (spreadsheets)

3.3.3 Schedule

The hydraulics designer shall refer to the following requirements for design milestone deliverables, unless otherwise waived by the respective District L&D Engineer:

Preliminary Field Inspection (“PFI”) Milestone*: Drainage Narrative – A preliminary drainage study should be conducted to the extent necessary to identify watersheds, drainage areas and determine SWM requirements based on the required Technical Criteria (per LD-IIM-195). A Drainage Narrative shall be submitted with Plans that states how Water Quality and Water Quantity will be met for this project including a determination to the purchasing of nutrient credits for water quality. Also any potential stream relocations, restorations, or enhancements should be identified, if applicable. Any potential issues should be identified at this milestone and included in the narrative for documentation purposes (see appendix 3A-1 for example). Plans shall be completed to the minimum LD-436 requirements.

* Rev. 7/16
Public Hearing (“PH”) (or Public Involvement) Milestone: Drainage Study – A preliminary drainage study should be conducted to the extent necessary to identify constructability issues and determine appropriate R/W limits to accommodate drainage features, including SWM facilities. Potential stream relocation, restoration, or enhancements should be identified, if applicable. All existing drainage structures shall be inspected for structural and functional adequacy. Any remediation measures should be identified and included in the Drainage Study for documentation purposes (see appendix 3A-2 for example). Plans shall be completed to the minimum LD-436 requirements.

Field Inspection (“FI”) Milestone: Drainage, SWM, and ESC Design/Calculations shall be completed and submitted with the Plans. Design/Calculations shall meet VDOT requirements, unless a design waiver is requested and approved by VDOT (See Appendix 3A-3 for example). Plans shall be completed to the minimum LD-436 requirements.

Pre-Advertisement Conference (“PAC”) Milestone: Drainage, SWM, and ESC Design/Calculations and Report shall be completed and submitted with the Plans. The Report shall be signed and sealed in accordance with IIM-LD-243, and all design waivers shall be completed, approved, and included in the Report (See Appendix 3A-4 for example). Plans shall be completed to the minimum LD-436 requirements.

All above deliverables shall be uploaded to ProjectWise as PDF files at the proper milestone and archived with that milestone. See VDOT CADD Manual Section E.2.2 Archiving Files for instructions on how to archive.

3.3.4 Guidelines

Descriptions for hydraulic items shall be written in accordance with these instructional guidelines. General examples of basic drainage descriptions are shown for illustrative purposes. These examples are intended to assist the Drainage Designer in the consistent application of VDOT procedures and practices. The numerical values utilized in the descriptions are for illustration only. These examples are reflective of the VDOT Road and Bridge Standards.

3.3.4.1 Plan Measurements

The length of culverts and storm sewer pipe shall be shown to the nearest 1’.

Invert elevations for culverts and appurtenances shall be shown to the nearest 0.1’.

Invert elevations for storm sewer pipe and appurtenances shall be shown to the nearest 0.01’.

* Rev. 7/16
Linear footage of manholes and heights of junction boxes and drop inlets shall be shown to the nearest 0.1’.

The design height of cover for culverts and storm sewer pipe shall be shown to the nearest 1’.

The skew angle for culverts shall be shown to the nearest 5 degree increment.

### 3.3.4.2 Pipe Lengths

The actual scaled/measured value should be shown.

Pipe lengths are typically determined based on the horizontal plan view distance between the ends of the pipe segment. Where pipes are specified to be laid on steep slopes, such as the outlet pipe from a shoulder slot inlet, the length of the pipe should be determined based on the length measured along the incline.

The location of the ends of a segment of drainage pipe will vary depending on the type of terminal structure specified. The ends of the pipe should be established based on the following:

- For terminal structures such as drop inlets, manholes, junction boxes, etc., the end of the pipe should be established based on the point at which the exterior walls of the pipe intersect the interior wall of the terminal structure. An exception to this would be where a terminal structure would have a base unit with an internal dimension less than the external dimension of the pipe. In this case the end of the pipe should be established based on that point at which the interior walls of the pipe intersect the interior wall of the terminal structure.

- Where endwalls are specified as terminal structures, the end of the pipe and the location of the face of the endwall should be established based on that point at which the embankment slope intersects the interior wall at the crown (top) of the pipe.

- Where end-sections are specified as terminal structures, the point at which the embankment slope intersects the exterior wall at the top of the end-section (at its full height) should be determined. Dimension “C” noted in the appropriate table on the Standard Drawings for ES-1, ES-1A or ES-2 (as applicable) should be subtracted from this point to establish the location (and pay line) for the end of pipe.

- Where the pipe projects beyond the embankment with no type of terminal treatment specified, the end of the pipe should be established based on that point at which the embankment slope intersects the flow line (invert) of the pipe.

* Rev. 7/14
3.3.4.3 Skew Angle of Culverts

The angle of skew shown on the plans for a drainage culvert is the acute angle formed by the centerline of the structure and a line drawn perpendicular to the roadway baseline that the culvert crosses. Where the culvert crosses more than one roadway baseline and where the baselines at the opposite ends of the structure are not parallel, an angle of skew for each end of the structure shall be shown in the description and in the summaries.

3.3.4.4 Structure Numbers

A numbering system is to be used to identify all proposed drainage items on the plans and those existing items to be modified or adjusted with the proposed construction (Exception – Projects with minimal drainage items that will use a Streamline Summary). A two number designation is to be used. The first number will identify the number of the plan sheet that contains the item and the second number will designate the assigned item number (e.g., Structure 4-20 is item number 20 on plan sheet 4; Structure 11B-2 is item number 2 on sheet 11B).

Culverts shall be identified by a single designation (e.g., 15-9).

For storm drain systems, the structures (inlets, manholes, junction boxes, etc.) shall be individually numbered. The pipe connecting two such structures shall be identified as from point to point (e.g., 4-6 to 4-7 is the pipe between structures 4-6 and 4-7).

The structure designation numbers are to be shown within ellipses. The descriptions are to be shown, space permitting, on the corresponding plan sheet. If all of the descriptions cannot be shown on the plan sheet, a separate drainage description sheet should be provided.

3.3.4.5 Protective Coatings

Where a protective coating is required for culverts, storm sewers and concrete structures exposed to the normal ebb and flow of tidal water or a corrosive environment, the Drainage Designer should include the following notation in the drainage description for the specified structures:

- Pipe or structure is to have protective coating applied in accordance with Section 404 of the VDOT Road and Bridge Specifications

Rev. 7/14
3.3.4.6 Pipe Descriptions

Each description should list the categories of information, as may be appropriate in the following order:

- All data pertaining to the pipe or culvert barrel (material, length, size, skew, cover, inverts)
- The type of end treatment (including erosion control protection)
- The recommended foundation data and minor structure excavation quantities

The “Design Height of Cover” must be shown for each pipe description on the plans (including pipes under entrances) and on the Drainage Summary, see Appendix 3A-5. This allows the Engineer to determine the proper strength, sheet thickness, or class of pipe from VDOT’s Road and Bridge Standard PC-1 drawings applicable to a particular location.

When specifying less than the standard minimum cover on concrete pipe, a reference to Drainage General Note D-14 should be included in the description for the structure.

When specifying the height of cover for the pipe culverts or the fill height for box culverts, normal practice is to specify the maximum cover that occurs along the entire length (run) of the culvert. Generally, as the height of cover (dead load) increases so does the required strength of the culvert. Therefore, specifying the maximum height of cover should ensure that the selected culvert is of sufficient strength to withstand the anticipated maximum dead load.

This, however, does not always hold true for culverts in low cover situations due to the greater influence of the live load. Two examples of this would be:

- Pipes or culverts, subject to traffic loading, with less than the standard Minimum cover called for in Standard PC-1 (2016 VDOT Road & Bridge Standards). In such cases, each type of pipe or culvert material will have its own specific requirements for absolute minimum heights of cover both during construction and in the finished condition. Refer to Standard PC-1 for requirements specific to material and size. For situations where the cover is less than the standard minimum, the designer is advised to contact the pipe manufacturer directly and discussing the anticipated loading with their Engineering Manager.

- Concrete box culverts, of like size, with 0’ to 2’ of cover generally require more concrete and steel (greater strength) than those with 2’ to 5’ of cover

Therefore, if a portion of the culvert falls into one of the above categories, the plans should specify the minimum height of cover or fill height to ensure that we get the proper strength pipe or box culvert for the particular site conditions.

* Rev. 1/17
In those cases where the Materials Division’s Subsurface Investigation Report indicates a soft, yielding or otherwise unsuitable foundation material, the description would include the recommended excavation and backfill information and be noted as follows:

*Excavate 20” below bottom of culvert and backfill with Bedding Material Aggregate #25 or 26
200 CY Minor Structure Excavation
100 Tons Bedding Material Aggregate #25 or 26*

- The specified bedding material quantity should be that required for backfilling the unsuitable material excavation below the normal 4” of bedding material and within the vertical limits shown in the Road and Bridge Standard PC-1 drawings.

- The specified minor structure excavation quantity should be measured from the top of the existing ground surface or bottom of the normal roadway excavation limit, whichever is lower, to the bottom of the foundation trench and within the vertical limits shown in the Road and Bridge Standard PC-1 drawings.

- The quantities specified for minor structure excavation and bedding material should include that required for endwalls, wingwalls, or other appurtenances. This quantity is based on the ratio of the plan area of the endwalls, wingwalls, or other appurtenances to the plan area of the culvert or pipe barrel. (See Section 8.4.4.4)

The strength, thickness, gage, class of pipe or method of bedding will be noted on the plans.

Pipe fittings such as tees, wyes, reducers, etc. are paid for as linear feet of pipe based on the largest dimension. Therefore, such items should be included in the description of the larger size pipe and their length included in the total length of that pipe segment.

In instances where a culvert must be countersunk to comply with environmental requirements a notation should also be included in the drainage description indicating that the invert elevations reflect countersinking, e.g., “The invert elevations noted reflect a minimum of “*” countersinking.” (where “*” is either 3” or 6” as required for the culvert’s size.) This will clearly communicates to the field personnel that the proposed invert elevations are intentionally set lower than the streambed. The fact that the culvert is to be countersunk should also be included in the remarks column of the Drainage Summary, see Appendix 3A-5.

* Rev 7/14
3.3.4.6.1 Typical Culvert Descriptions

These descriptions allow the Engineer the option of selecting the pipe material and joint type for a particular project location. Pipe material shall be specified in the description Req'd.

(2-3) 100’-48” Conc. Pipe Class III Req. (6’ Cover) (20°Skew)
- Leak Resistant Joint Type
- Inv(In) = 435.0’, Inv(Out) = 434.0’
- 2 Std. EW-2 Req.
- 21 CY Std. EC-1 Class 1 Req. Lt. Type B Installation
- 378 CY Minor Structure Excavation

(2-5) 100’-24” Aluminum Coated Type 2, 14 Gauge Pipe Req. (3’Cover)
- Silt Tight Joint Type
- Inv(In) = 435.0’, Inv(Out) = 434.0’
- 1 Std. ES-1 or 2 Req. Lt.
- 1 Std. EW-11 Req. Rt. 4:1 Slope

3.3.4.6.2 Concrete Pipe on Radius

Concrete pipe may be installed on a radius using the open joint method or using the bevel pipe method with or without open joints. Concrete pipe that is installed on a radius using the open joint method is standard pipe and should not be specified as concrete radial pipe. See Section 9.4.8.8 for the minimum radius for each method for various pipe sizes.

- OPEN JOINT METHOD
  (2-3) 100’-48” Conc. Pipe Class III Req. (6’ Cover)
  - (530’ Radius with open joints – using 8’ pipe joint lengths)
  - Joints are to be opened a maximum of 25% of the spigot or tongue length.
  - Inv(In) = 435.0’, Inv(Out) = 434.0’
  - 2 Std. EW-2 Req.
  - 21 CY Std. EC-1 Class 1 Req. Type B Installation
  - 378 CY Minor Structure Excavation

- BEVEL PIPE METHOD
  (3-1) 100’-48” Conc. Radial Pipe Class III Req. (6’ Cover)
  - (120’ Radius – using 8’ pipe joint lengths with full bevel)
  - Inv(In) = 435.0’, Inv(Out) = 434.0’
  - 2 Std. EW-2 Req. Lt.
  - 21 CY Std. EC-1 Class 1 Req. Type B Installation
  - 378 CY Minor Structure Excavation

* Rev. 7/16
• **BEVEL PIPE WITH OPEN JOINT METHOD**
  
  (6-7) 100'-48" Conc. Radial Pipe Class III Req. (6' Cover)
  (95’ Radius with open joints – using 8’ pipe joint lengths with full bevel)
  Joints are to be opened a maximum of 25% of the spigot or tongue length.
  Inv(In) = 435.0', Inv(Out) = 434.0'
  2 Std. EW-2 Req.
  21 CY Std. EC-1 Class 1 Req. Type B Installation
  378 CY Minor Structure Excavation

  **3.3.4.6.3 Trenchless Applications**

  (5-6) 80'-48” Jacked Conc. Pipe Class IV Req. (25’ Cover)
  Leak Resistant Joint Type
  Inv(In) = 197.6’, Inv(Out) = 197.0’
  2 Std. EW-2 Req.
  21 CY Std. EC-1 Class 1 Req. Type B Installation

  (5-8) 160'-36” Microtunnel Smooth Steel, 10 Gauge Pipe Req. (15’ Cover)
  Leak Resistant Joint Type
  Inv(In) = 200.6’, Inv(Out) = 198.0’
  2 Std. EW-1 Req.
  17 CY Std. EC-1 Class 1 Req. Type B Installation

  **3.3.4.6.4 Multiple Pipe Installation**

  (8-9) 300'-48” Galvanized Steel, 12 Gauge Pipe Req. (7’ Cover)
  (Triple Line – 100’ each line)
  Silt Tight Joint Type
  Inv(In) = 164.8’, Inv(Out) = 164.1’
  2 Std. EW-7 Req.
  41 CY Std. EC-1 Class 1 Req. Type B Installation
  1,134 CY Minor Structure Excavation

  **3.3.4.6.5 Existing Pipe Extension**

  The vertical and horizontal alignment of the pipe extension should duplicate that of the existing pipe. The type of pipe specified for the extension should be the same as the existing pipe. The cover specified should be the maximum that occurs along the entire run of pipe, including the existing section.
(2-3) Existing Pipe To Be Extended with 50’-36" Corrugated Steel Pipe,

12 Gauge * Req. (7’ Cover)
Inv(In) = 435.0’, Inv(Out) = 434.0’
1 Std. EW-1 Req.

3.3.4.7 Box Culvert Descriptions

3.3.4.7.1 Standard (Cast-In-Place)
The standard description should be used where a cast in place structure can be used. However, the specifications allow the Contractor the option of substituting a precast structure with approval of the Engineer.

(4-3) 150’- 6’ X 8’ Box Culvert Req. (25’ Cover)(15° Skew)
Inv(In) = 60.0’, Inv(Out) = 57.0’
Std. BCS-DT, BCS-30, & BCW-21
4 Std. Type I Wings Req.
75 CY Std. EC-1 Class 1 Req. Rt. Type B Installation
527 CY Minor Structure Excavation

3.3.4.7.2 Precast
The precast description should be used where a precast structure only is desired.

(4-8) 150’- 6’ X 8’ Precast Box Culvert Req. (25’ Cover)(15° Skew)
Inv(In) = 60.0’, Inv(Out) = 57.0’
2 Headwalls Req. (Cost to be included in price bid for linear feet of box culvert) Reference Stds. BCS-DT & BCS-30
4 Wings Req. Reference Std. BCW-21, Type 1(K)
75 CY Std. EC-1 Class 1 Req. Rt. Type B Installation
527 CY Minor Structure Excavation

3.3.4.8 Structures
When specifying precast structures, it is not necessary to identify, in the description, the applicable precast standard base, riser, and top units, unless a particular type of component is desired. The Contractor should, wherever possible, be allowed the option of determining the most economical units to utilize to assemble the desired structure.

In addition to the standard information, the drainage description should include all information required to properly construct the structure. The description should be clear to the extent that there is no doubt as what is to be done at the location. Some examples of additional information to be included in a description would be:

- Connect To Existing 18” Conc. Pipe
- Connect UD-4 TO DI

* Rev. 7/16
Standard IS-1 Inlet Shaping should be specified for manholes, drop inlets, or junction boxes where the main trunk line of a storm sewer changes direction or pipes of approximately the same size intersect and are carried forward in a single pipe.

Standard SL-1 safety slabs shall be specified for manholes, drop inlets, or junction boxes in accordance with the guidance outlined in Section 9.4.5.2.2 and the standard drawing.

All drop inlets (both curb and median), catch basins, junction boxes and other such structures that require a frame and cover or grate at finished ground elevation, shall show the height dimension “H” on the plans and on the Drainage Summary, see Appendix 3A-5. This dimension is to be measured from the invert elevation to the top of the concrete or masonry structure and is to be shown to the nearest 0.1’.

Manholes should be shown as the number of linear feet required, measured from the invert to the top of the concrete or masonry structure. The linear feet of manhole specified should not include the height of the frame and cover.

### 3.3.4.8.1 Curb Drop Inlets

The standard description assumes cast in place; however, the Contractor is allowed the option to substitute a precast structure.

(3-1) 1 Std. DI-4D Req.
L=8’, H=5.2’, Inv = 197.6’
Std. IS-1 Req.

When the required structure height is greater than the maximum allowed for a cast in place structure, or a precast structure is desired, the description would be:

(9-7) 1 Std. DI-4DD (Precast) Req.
L=8’, H=25.0’, Inv = 197.6’
2 Std. SL-1 Req.

### 3.3.4.8.2 Grate Drop Inlets

Descriptions for Standard DI-5, DI-7, and DI-12 series grate drop inlets should specify the type of grate required, i.e., a Type I grate for areas where pedestrian access is unlikely or a Type III (DI-5 & 7) or Type II (DI-12) for pedestrian accessible areas. When a DI-7 inlet is to be located in areas subject to occasional traffic (e.g., shoulders, parking areas, etc.), a load carrying Grate B should be specified.

* Rev. 7/14
Descriptions for Standard DI-5 inlets should include the type of cover. The Standard PG-2A cover type most closely matching the ditch configuration should be specified. The height of the structure is measured from the invert to the top of the concrete cover.

(4-5) 1 Std. DI-5 Req. Type I Grate Req.
Std. PG-2A Type E Cover
H=4.8’, Inv = 13.6’

3.3.4.8.3 Manholes

If a cast in place structure only is to be allowed, show only the MH-1 designation. Show only the MH-2 designation if a precast unit only is to be allowed. The option of utilizing cast in place as well as precast manholes should be allowed at all locations except for those where placement is limited due to existing pipelines, utilities, the size of pipe, etc. Most locations should permit the Contractor the option to utilize either and the descriptions should specify both the cast in place and precast standard.

(3-1) 14.6 LF Std. MH-1 or 2 Req.
1 Std. MH-1 Frame & Cover Req.
Inv = 83.4’
1 Std. SL-1 Req.

3.3.4.8.4 Junction Boxes

(8-3) 1 Std. JB-1 Req.
H=12.8’, W=4’, D=5’
Type A Tower Req.
1 Std. MH-1 Frame & Cover Req.
Inv = 121.4’
1 Std. SL-1 Req.

3.3.4.8.5 Stormwater Management Structures

In those instances where the stormwater management basin is to be utilized as a temporary sediment basin, the description should be so noted with a reference to Standard SWM-DR for details.

• SWM DRAINAGE STRUCTURE
  (14-7) 6.7’ Std. SWM-1 Req.
  Bottom Elev = 23.8’
  3” Diameter Water Quality Orifice Req., Inv = 26.8’
  10” Diameter Orifice Req., Inv = 28.8’
  See Sheet 2G For Details

Rev. 7/14
3.3.4.8.6 Existing Structures

The Drainage Designer will determine if existing pipe and box culverts and storm sewer pipe will remain and be utilized in the proposed design or removed or abandoned.

Pipes to be removed, abandoned or cleaned out are to be indicated on the plans for bidding purposes and labeled "To Be Removed", "To Be Abandoned", or "To Be Cleaned Out".

Any large amount of pipe and appurtenances to be removed, such as an existing storm sewer system*, should be set up as a separate bid item and summarized in a separate column in the Incidental Summary.

When not set up as a separate pay item, small amounts of pipe and appurtenances to be removed are included in the cost of Clearing and Grubbing (See Section 105.15 of the Road & Bridge Specifications) or may be included in the cost of Regular Excavation. (See latest IIM-LD-110 & General Note G-4)

Any drainage pipe that is abandoned and left in place shall be backfilled and plugged in accordance with VDOT’s Road and Bridge Standard PP-1. These pipes are to be labeled on the plans "To Be Abandoned". The pay item for abandoning existing structures is “Flowable Backfill, CY” and includes furnishing and placing backfill material and plugging both ends of the drainage pipe.

The quantity for Flowable Backfill (includes flowable backfill or fine aggregate) is to be estimated in accordance with Standard PP-1. This estimated quantity is to be summarized in the Drainage Summary, see Appendix 3A-5. The pipe location/structure number should be shown in the Drainage Summary, see Appendix 3A-5 and the pipe size should be noted in the remarks column.

General Note D-12 (See latest IIM LD-110) is to be included on the General Note Sheet in all applicable project assemblies.

* Rev. 7/14
“Modify” should be used when a major work effort is required (e.g., connecting or removing pipes, adjusting height more than 1’, etc.).

(4-11) Modify Existing Drop Inlet
    Adjust To Grade, Raise 2.3’
    Add DI-3B, L=6’
    Proposed Top Elev = 153.6’
    See Sheet 2K For Details.

“Adjust” should be used when a minor work effort is required (e.g., adjusting height 1’ or less).

(5-18) Adjust Existing MH
    Adjust To Grade, Raise 0.5’
    1 Std. MH-1 Frame & Cover Req.
    Proposed Top Elev = 234.3’

All work to be performed to modify the structure should be clearly stated in the drainage description. Other such information would be:

- Modify To (Accept/Remove) 15” Conc. Pipe
- Connect UD-4 To Structure
- Convert Existing DI to Manhole
- To Be Cleaned Out

The necessary standard items for completing the work should be specified (e.g., precast units, manhole frame and cover, etc.). The structural condition of an existing structure should be field evaluated to determine the suitability for modification. Those structures found to be structurally deficient or in poor condition should be replaced in lieu of being modified. The cost of total replacement versus modification should also be evaluated to make sure the most economical solution is being proposed.

3.3.4.9 Drainage Summaries and Type of Pipe Selection

A Standard (Detailed) Summary is to be used on normal construction (C) projects.

A Streamlined Summary may be used on Minimum Plan (M), No Plan (N) and Safety projects.

When the Drainage Summary sheets are compiled, the drainage items in the Drainage Summary are to be referenced by their assigned structure numbers with no further reference to sheet number, station, or location needed.

The total linear feet of all like size pipe shall be summarized by material.

* Rev. 7/16
The methods of listing pipe in the Standard Summary, see Appendix 3A-5 and the Streamlined Summary are to be used to eliminate a possible contractor’s error when ordering the pipe.

- Streamline Summary Example:

  800 LF  15” Aluminum Coated, Type 2, Pipe
  40 LF  15” Galvanized Corrugated Steel Pipe
  200 LF  15” Conc. Pipe
  100 LF  24” Polyethylene (PE) Corrugated, Type S Pipe
  200 LF  72” Special Design Conc. Pipe

The total linear feet of all like size pipe, are generally combined by material for the purposes of the estimate.

Projects on which the new pipe installations require end sections, the Drainage Summary, see Appendix 3A-5 shall have a column indicating the optional standard, “Std. ES-1 or Std. ES-2”, for the end sections. A separate column on the Drainage Summary, see Appendix 3A-5 is required when specifying only a Std. ES-1 or Std. ES-2 end section for pipes of a particular material.

Example tabulations for a Route 64 project in York County are as follows:
(The template for the following tables can be found in the CADD Cell Library)
**PIPE CULVERT EXAMPLE**

Allowable Pipe Types Standard PC-1 as shown below will still be included in the Plan Sets.

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>CONCRETE</th>
<th>ALUMINUM COATED TYPE 2 CORRUGATED STEEL</th>
<th>POLYMER COATED CORRUGATED STEEL</th>
<th>UNCOATED GALVANIZED CORRUGATED STEEL</th>
<th>GALVANIZED STEEL STRUCTURAL PLATE</th>
<th>GALVANIZED STEEL STRUCTURAL PLATE WITH CONCRETE INVERT</th>
<th>CORRUGATED ALUMINUM ALLOY</th>
<th>CORRUGATED ALUMINUM STRUCTURAL PLATE</th>
<th>POLYVINYLCHLORIDE (PVC) CORRUGATED RIBBED PIPE (SMOOTH INTERIOR)</th>
<th>POLYETHYLENE (PE) CORRUGATED TYPE C</th>
<th>POLYETHYLENE (PE) CORRUGATED TYPE S</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rte. 64 &amp; Ramps</td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Route 635 (Rural Local Road)</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Entrances</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Shoulder Slot Inlet</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Rev. 7/16
### STORM SEWER PIPE EXAMPLE

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>CONCRETE</th>
<th>CORRUGATED STEEL 10/10</th>
<th>ALUMINUM COATED TYPE 2</th>
<th>FULLY CONCRETE LINED</th>
<th>ALUMINUM COATED TYPE 2 SPIRAL RIB PIPE</th>
<th>POLYMER COATED (10/10) CORRUGATED STEEL SPIRAL RIB PIPE</th>
<th>POLYMER COATED (10/10) CORRUGATED STEEL DOUBLE WALL (SMOOTH INTERIOR)</th>
<th>ALUMINUM SPIRAL RIB PIPE</th>
<th>POLYVINYLCHLORIDE (PVC) RIBBED PIPE (SMOOTH INTERIOR)</th>
<th>POLYETHYLENE (PE) CORRUGATED TYPE S</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rte. 64 &amp; Ramps</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Route 635 (Rural Local Road)</td>
<td>X</td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

3.3.4.10 Post Installation Pipe Inspection

A post installation visual/video camera inspection shall be conducted by the Contractor on all pipes identified on the plans as storm sewer pipe and a select number of pipe culverts.

For pipe culverts, a minimum of one pipe installation for each size of each material type will be inspected or ten percent of the total amount for each size and material type summarized. All pipe installations on the plans not identified as storm sewer pipe shall be considered as culvert pipe for inspection purposes.

For multiple-line pipe installations, each line of pipe should be counted and quantified individually when determining the overall post installation pipe inspection quantity.

The drainage summary, see Appendix 3A-5 is to include a quantity for the total linear feet of Post Installation Inspection (to include both pipe culverts and storm sewer pipe).

*Rev. 7/14*
These requirements shall not be applicable to pipes that are being rehabilitated.
See sample Post Installation Summary Table in Appendix 3A-5 for No Plans(N): ⃰

⃰

Example for “No Plan”:

Rev. 7/14
Chapter 3-23 of 24


3.4 References


*Rev. 1/17*
Appendix 3A-1 PFI Milestone Deliverable

Hydraulics Narrative

UPC 123456 – Rte. 111 Safety Improvements

Stormwater Management

This project utilizes the technical criteria of Part IIB (9VAC25-870-62) for determining its post-development stormwater management design. Nutrient credits shall be purchased to meet the water quality requirements of the technical criteria. The total pollutant removal requirement for this project is 0.37 lb/year. Calculations for the quantity of nutrient credits required can be found at the end of this report.

The water quantity requirements for this project are also governed by the technical criteria of Part IIB (9VAC25-870-66). The limits of analysis for analyzing the downstream channel for channel and flood protection will be to the point where the contributing drainage area from the project is less than 1% of the total drainage area to that point. Therefore, channel and flood protection requirements will not be governed by the energy balance equation, but rather by showing that the channel is adequate for the 10-year storm for capacity and for the 2-year storm for resisting erosion. In order to meet the requirements of the 1% rule, the downstream channel will be analyzed to a point approximately 500 linear feet downstream of the project site. A map demonstrating compliance with the 1% rule can be found at the end of this narrative.

The survey information provided for the downstream channel does not appear to match observations from the field visit. Photos and observations from the field show the downstream channel as a stream with a wide trapezoidal channel, while the survey data shows a v-shaped channel. Additional survey data or a field visit may be necessary to confirm the characteristics of the downstream channel; however, based on photos and field observations, the channel is adequate for channel and flood protection.
Chapter 3 – Documentation

Appendix 3A-1 PFI Milestone Deliverable

---

### Virginia Runoff Reduction Method ReDevelopment Worksheet - v2.0 - June 2014

**Site Data**

- Project Name: [Project Name]
- BMP Standards: [BMP Standards]
- Date: [Date]
- Site Size: [Site Size]
- Soil Type: [Soil Type]
- Vegetative Cover: [Vegetative Cover]
- Impervious Cover: [Impervious Cover]
- Post-ReDevelopment Project & Land Cover Information

<table>
<thead>
<tr>
<th>Total Disturbed Acreage</th>
<th>1.85</th>
</tr>
</thead>
</table>

### Contents

- Forest/Wooded Area
  - [Area]
- Managed Turf
  - [Area]
- Impervious Cover
  - [Area]
- [Total Area]

### Post-ReDevelopment Project & Land Cover Information

<table>
<thead>
<tr>
<th>Acre</th>
<th>B Acre</th>
<th>C Acre</th>
<th>D Acre</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Acre]</td>
<td>[Acre]</td>
<td>[Acre]</td>
<td>[Acre]</td>
<td>[Acre]</td>
</tr>
</tbody>
</table>

### Pre-ReDevelopment Project & Land Cover Information

<table>
<thead>
<tr>
<th>Acre</th>
<th>B Acre</th>
<th>C Acre</th>
<th>D Acre</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Acre]</td>
<td>[Acre]</td>
<td>[Acre]</td>
<td>[Acre]</td>
<td>[Acre]</td>
</tr>
</tbody>
</table>

### Pre-ReDevelopment Summary

<table>
<thead>
<tr>
<th>Adjusted Pre-ReDevelopment</th>
<th>Adjusted Post-ReDevelopment</th>
<th>Adjusted New Impervious</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Adjusted Pre-ReDevelopment]</td>
<td>[Adjusted Post-ReDevelopment]</td>
<td>[Adjusted New Impervious]</td>
</tr>
</tbody>
</table>

### Post-ReDevelopment Summary

<table>
<thead>
<tr>
<th>Adjusted Pre-ReDevelopment</th>
<th>Adjusted Post-ReDevelopment</th>
<th>Adjusted New Impervious</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Adjusted Pre-ReDevelopment]</td>
<td>[Adjusted Post-ReDevelopment]</td>
<td>[Adjusted New Impervious]</td>
</tr>
</tbody>
</table>

### Adjusted Land Cover Summary

<table>
<thead>
<tr>
<th>Adjusted Pre-ReDevelopment</th>
<th>Adjusted Post-ReDevelopment</th>
<th>Adjusted New Impervious</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Adjusted Pre-ReDevelopment]</td>
<td>[Adjusted Post-ReDevelopment]</td>
<td>[Adjusted New Impervious]</td>
</tr>
</tbody>
</table>

---
Appendix 3A-1 PFI Milestone Deliverable

Total contributing drainage area = 99.62 acres

Project area = 0.87 acres
Appendix 3A-2 PH Milestone Deliverable

Hydraulics Design Study

UPC 123456 – Rte. 111 Safety Improvements

Project Description:

VDOT Central Office Hydraulic Section has prepared a drainage design and associated computations for the Route 611 (Spring Creek Road) project located in Washington County. The purpose of this project is to replace the bridge over Spring Creek, widen the roadway and make safety improvements to the horizontal and vertical alignments.

Existing Conditions:

The project is small urban (5,000-49,999) in nature. Route 611 is a two lane roadway with graded shoulders. Roadside ditches and pipe culverts direct stormwater runoff to Spring Creek. There are no wetlands or water quality structures in the project area.

Proposed Drainage System:

The project will maintain the same general flow patterns. Stormwater runoff will sheet flow or drain to a roadside ditch and ultimately discharge into Spring Creek. A stream channel relocation will be required at two separate locations on the project.

Hydrology:

The Rational Method of calculating discharge is used exclusively on this project because all of the drainage areas are less than 200 acres in size.

Culvert Design:

All proposed culverts on the project were designed to accommodate the 10-year storm event and maintain a minimum 18” headwater freeboard to the roadway shoulder point. However, several existing culverts in good condition that are to remain within project limits do not meet the minimum freeboard and will require a design waiver.

Regulations:

The project is grandfathered under the provisions of Section 4VAC50-60-48 of the VSMP Regulations adopted September 13, 2011, and utilizes the technical criteria of Part IIC (4VAC50-60-93.1 et. seq.) for determining its post-development stormwater management design.
Appendix 3A-2 PH Milestone Deliverable

Compliance:

The project is in compliance with DEQ by acquiring the required removal lbs/year from basins at the I-81 Exit 14 Modifications Project. Attached are the performance based water quality calculations for both projects. The performance based calculations for this project (UPC#60792) show a required removal of 1.63 lbs/year of phosphorous and the performance based calculations for the I-81 Exit 14 Project (UPC#97856) show removing an additional 1.65 lbs/year of phosphorous. A letter from DEQ agreeing that the required removal for this project can be done with the I-81 Exit 14 Project will be submitted with the next milestone submittal.
PERFORMANCE-BASED WATER QUALITY CALCULATIONS

WORKSHEET 1

STEP 1: Determine the applicable area (A) and the post-developed impervious cover (I_{post}).

Applicable area (A) = 7.41 acres

Post-development impervious cover:

- structures = 0.00 acres
- parking lot = 0.00 acres
- roadway = 1.98 acres

other:

(\text{input description}) = 0.00 acres
(\text{input description}) = 0.00 acres

Total = 1.98 acres

I_{post} = (\text{total post-development impervious cover} ÷ A) \times 100 = 26.72\%

STEP 2: Determine the avg. land cover condition (I_{watershed}) or the exist. impervious cover (I_{exist})

1. Average land cover condition (I_{watershed}):
   If the locality has determined land cover conditions for individual watersheds within its jurisdiction, use the watershed specific value determined by the locality as I_{watershed}.

   \text{i_{watershed}} = \underline{\text{\%}} \quad \text{(input locality value or leave blank if one does not apply)}

   Otherwise, use the Chesapeake Bay default value:

   \text{i_{watershed}} = 16.00 \%

Page 1 of 3
Appendix 3A-2 PH Milestone Deliverable

Performance Based Water Quality
60792 Route 611

PERFORMANCE-BASED WATER QUALITY CALCULATIONS

WORKSHEET 1

2. Existing impervious cover (Iexisting):

Determine the existing impervious cover of the development site if present.

Existing impervious cover:

<table>
<thead>
<tr>
<th>Type</th>
<th>Area (acres)</th>
</tr>
</thead>
<tbody>
<tr>
<td>structures</td>
<td>0.00</td>
</tr>
<tr>
<td>parking lot</td>
<td>0.00</td>
</tr>
<tr>
<td>roadway</td>
<td>1.13</td>
</tr>
</tbody>
</table>

other:

<table>
<thead>
<tr>
<th>(input description)</th>
<th>Area (acres)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>0.00</td>
</tr>
</tbody>
</table>

Total = 1.13 acres

Iexisting = (total existing impervious cover + A) x 100 = 15.25 %

* The applicable area (A) should be the same as used in STEP 1.

STEP 3: Determine the appropriate development situation.

The site information determined in STEP 1 and STEP 2 provide enough information to determine the appropriate development situation under which the performance criteria will apply. The appropriate development situation will be marked by an "X" except situation 4 that will require user input if it applies.

Situation 1: This consists of land development where the existing percent impervious cover (Iexisting) is less than or equal to the average land cover condition (Iwatershed) and the proposed improvements will create a total percent impervious cover (Ipost) which is less than or equal to the average land cover condition (Iwatershed).

Ipost 28.72 % <= Iwatershed 16.00 %
Appendix 3A-2 PH Milestone Deliverable

**Performance Based Water Quality**

60792 Route 611

**Worksheet 1**

**Situation 2:**

This consists of land development where the existing percent impervious cover (\( I_{\text{existing}} \)) is less than or equal to the average land cover condition (\( I_{\text{watershed}} \)) and the proposed improvements will create a total percent impervious cover (\( I_{\text{post}} \)) which is greater than the average land cover condition (\( I_{\text{watershed}} \)).

\[
I_{\text{existing}} \leq I_{\text{watershed}} \leq I_{\text{post}}
\]

**Situation 3:**

This consists of land development where the existing percent impervious cover (\( I_{\text{existing}} \)) is greater than the average land cover condition (\( I_{\text{watershed}} \)).

\[
I_{\text{existing}} > I_{\text{watershed}}
\]

**Situation 4:**

This consists of land development where the existing percent impervious cover (\( I_{\text{existing}} \)) is served by an existing stormwater management BMP (s) that addresses water quality.

If the proposed development meets the criteria for development situation 1, then the low density development is considered to be the BMP and no pollutant removal is required. The calculation procedure for situation 1 stops here. If the proposed development meets the criteria for development situations 2, 3, or 4, then proceed to STEP 4 on the appropriate worksheet.
PERFORMANCE-BASED WATER QUALITY CALCULATIONS

WORKSHEET 2 : SITUATION 2

SUMMARY OF SITUATION 2 CRITERIA: FROM CALCULATION PROCEDURE STEP 1 THRU STEP 3, WORKSHEET 1:

Applicable area (A)* = 7.41 acres

I_post = \frac{\text{total post-development impervious cover}}{A} \times 100 = 26.72\% \\
I_{\text{watershed}} = 16.00\%

I_{\text{existing}} 15.25\% <= I_{\text{watershed}} 16.00\% ; and \\
I_{\text{post}} 26.72\% > I_{\text{watershed}} 16.00\%

STEP 4: Determine the relative pre-development pollutant load (L_{\text{pre}}).

L_{\text{pre(watershed)}} = \left[ 0.05 + (0.009 \times I_{\text{watershed}}) \right] \times A \times 2.28 \text{ (Equation 5-16)}

L_{\text{pre(watershed)}} = \text{relative pre-development total phosphorous load (pounds per year)}

I_{\text{watershed}} = \text{average land cover condition for specific watershed or locality or the Chesapeake Bay default value of 16\% (percent expressed in whole numbers)}

A = \text{applicable area (acres)}

I_{\text{watershed}} = 16.00\%

A = 7.41 \text{ acres}

L_{\text{pre(watershed)}} = 3.28 \text{ lbs/year}
Performance Based Water Quality
60792 Route 611

Appendix 3A-2 PH Milestone Deliverable

Worksheet 2: Situation 2

STEP 5: Determine the relative post-development pollutant load (L_{post}).

\[ L_{post} = [0.05 + (0.009 \times L_{pre})] \times A \times 2.28 \] (Equation 5-21)

\[ L_{post} = \text{relative post-development total phosphorous load (pounds per year)} \]

\[ L_{pre} = \text{post-development percent impervious cover (percent expressed in whole numbers)} \]

\[ A = \text{applicable area (acres)} \]

\[ L_{post} = 4.91 \text{ pounds per year} \]

STEP 6: Determine the relative pollutant removal requirement (RR).

\[ RR = L_{post} - L_{pre(watershed)} \]

\[ RR = 1.63 \text{ pounds per year} \]

STEP 7: Identify best management practice (BMP) for site.

1. Determine the required pollutant removal efficiency for site:

\[ EFF = \frac{RR}{L_{post}} \times 100 \] (Equation 5-22)

\[ L_{post} = \text{relative post-development total phosphorous load (pounds per year)} \]

\[ EFF = \text{required pollutant removal efficiency (percent in whole numbers)} \]

\[ RR = \text{pollutant removal requirement (pounds per year)} \]

\[ EFF = 33.20 \% \]
### PERFORMANCE-BASED WATER QUALITY CALCULATIONS

#### WORKSHEET 2: SITUATION 2

2. Select BMP(s) from Table 5-14 and locate on the site:

<table>
<thead>
<tr>
<th>BMP 1:</th>
<th>Sta / offset</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>BMP 2:</td>
<td>Sta / offset</td>
<td></td>
</tr>
<tr>
<td>BMP 3:</td>
<td>Sta / offset</td>
<td></td>
</tr>
<tr>
<td>BMP 4:</td>
<td>Sta / offset</td>
<td></td>
</tr>
<tr>
<td>BMP 5:</td>
<td>Sta / offset</td>
<td></td>
</tr>
</tbody>
</table>

\[
L_{BMP} = [0.05 + (0.009 \times I_{BMP})] \times A \times 2.28 \quad \text{(Equation 5-23)}
\]

\[
L_{removed} = EFF_{BMP} \times L_{BMP} \quad \text{(Equation 5-24)}
\]

- \( L_{BMP} \) = relative post-development total phosphorous load entering proposed BMP (pounds per year)
- \( I_{BMP} \) = Post-development percent impervious cover of BMP drainage area (percent expressed in whole numbers)
- \( A \) = drainage area of proposed BMP (acres)
- \( L_{removed} \) = Post-development pollutant removed by proposed BMP (pounds per year)
- \( EFF_{BMP} \) = pollutant removal efficiency of BMP (expressed in decimal form)

3. and 4. Determine the pollutant load entering the proposed BMP(s) and Calculate the pollutant load removed by the proposed BMP(s):

<table>
<thead>
<tr>
<th>BMP Str.#</th>
<th>BMP &quot;A&quot; (acres)</th>
<th>BMP Imp. Area (ac.)</th>
<th>I_{BMP} (%)</th>
<th>L_{BMP}</th>
<th>EFF_{BMP}</th>
<th>L_{removed} (Lbs/yr)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
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</tr>
</tbody>
</table>
Appendix 3A-2 PH Milestone Deliverable

PERFORMANCE-BASED WATER QUALITY CALCULATIONS

WORKSHEET 2: SITUATION 2

5. Calculate the total pollutant load removed by the BMP(s):

\[ L_{\text{removed/total}} = L_{\text{removed/BMP1}} + L_{\text{removed/BMP2}} + \text{Etc.} \ldots \text{ (equation 5-25)} \]

where:
- \( L_{\text{removed/total}} \): Total pollutant load removed by proposed BMP's
- \( L_{\text{removed/BMP1}} \): Pollutant load removed by BMP1
- \( L_{\text{removed/BMP2}} \): Pollutant load removed by BMP2

See chart on sheet 3 of 4 for individual BMP removal

\[ L_{\text{removed/total}} = 0.00 \text{ Pounds/year} \]

6. Verify compliance:

\[ L_{\text{removed/total}} \geq RR \]

\[ 0.00 \geq 1.63 \]

ADD MEASURES
Hydraulic Design Study

UPC 987654 – Rte. 643 Safety Improvements

Hydraulic Design Narrative

General Description:

VDOT Central Office Hydraulic Section has prepared a drainage design and associated computations for the Route 643 (Back Hampden Sydney Road) project located in Prince Edward County. The purpose of this project is to make improvements to the horizontal and vertical alignments and widen shoulders for safety purposes. The drainage plans and computations address the hydraulic changes to the roadway and comply with all regulatory requirements.

Existing Conditions:

The project is rural in nature. Route 643 is a two lane roadway with narrow graded shoulders. Roadside ditches and pipe culverts direct stormwater runoff to an unnamed live stream, which crosses the existing roadway through a box culvert. There are no wetlands or water quality structures in the project area.

Proposed Drainage System:

The project will maintain the same general flow patterns. Stormwater runoff will sheet flow or drain to a roadside ditch and ultimately discharge into the unnamed tributary to Wilck’s Lake. Stream channel relocations will be required at three locations on the project.

Erosion and Sediment Control (ESC):

This ESC Plan has been designed, prepared, reviewed, and approved in accordance with the VDOT’s approved ESC & Stormwater Management (SWM) Program Standards and Specifications. A copy of the Certification Form is part of this document.

Hydrology:

TR-55 Method of calculating discharge is used for Culvert 4-5, due to the size of the drainage area. The Rational Method of calculating discharge is used for all other aspects of this project because the drainage areas are less than 200 acres in size. Drainage areas were calculated with the use of electronic survey and design data.
Appendix 3A-3 FI Milestone Deliverable

**Culvert Design:**

All culverts on the project were designed and sized based on criteria outlined in Chapter 8 of the *VDOT Drainage Manual*. The culverts were designed to accommodate the 10-year storm event and maintain a minimum 18” headwater freeboard to the roadway shoulder point. Culvert computations are part of this document in Appendix __.

**Inlet & Pipe Design:**

There are no proposed inlets or proposed pipe systems on the project.

**Ditch Design:**

All ditches on the project were designed and sized based on criteria outlined in Chapter 7 of the *VDOT Drainage Manual*. The ditches on the project are designed to convey the 10-year discharge, and to resist erosion from the 2-year discharge. Computations for the ditches are part of this document in Appendix __.

**Minimum Standard-19 (MS-19):**

The Virginia ESC Regulation MS-19 for an adequate receiving channel governs requirements for stream channel erosion. The natural outfall channel has been analyzed for adequacy for conveying the 2-year storm while resisting erosion of the bed and banks. Computations for channel adequacy are part of this document in Appendix__.
Appendix 3A-3 FI Milestone Deliverable

**Regulations:**

The project is grandfathered under the provisions of Section 4VAC50-60-48 of the VSMP Regulations adopted September 13, 2011, and utilizes the technical criteria of Part IIC (4VAC50-60-93.1 et. seq.) for determining its post-development stormwater management design. In accordance with the performance-based criteria, this project is considered a Situation 3 because the existing percent impervious cover is greater than the average land cover condition. However, when initially developed, water quality requirements were based on the net increase of impervious cover and did not require treatment. The requirements changed to require treatment of the total post-development impervious area after the project had completed the public involvement stage.

The applicable percent impervious cover of the site is less than the statewide average land cover condition of 16% and therefore a water quality BMP is not required. At the time of public hearing, the applicable post construction impervious cover was defined as the net increase in impervious area of the site divided by the total post-development area of the site.

- Post-development impervious area = 2.20 acres
- Pre-development impervious area = 1.43 acres
- Net increase impervious area = $2.20 - 1.43 = 0.77$ acres
- Total post-development area of site = 5.51 acres
- Applicable percent impervious cover = $0.77 \div 5.51 = 13.97\% < 16\%
**HYDROLOGICAL DATA**

**DESIGN FLOWS**

<table>
<thead>
<tr>
<th>R.L (feet)</th>
<th>FLOW (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>Design: 10.88</td>
</tr>
<tr>
<td>2</td>
<td>Check: 8.44</td>
</tr>
<tr>
<td>15</td>
<td>Max: 100.00</td>
</tr>
</tbody>
</table>

**CULVERT DESCRIPTIONS**

<table>
<thead>
<tr>
<th>Inlet/Outlet: Single/Multiple Conforming:</th>
</tr>
</thead>
</table>

**FLOW PER:**

**HEADWATER CALCULATIONS**

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>SHAPE</th>
<th>Size (m)</th>
<th>N</th>
<th>Manning n</th>
</tr>
</thead>
<tbody>
<tr>
<td>CMPStadC</td>
<td>Circular</td>
<td>54</td>
<td>1</td>
<td>0.92</td>
</tr>
</tbody>
</table>

**BROKEN BACK CULVERT**

**TAILWATER DATA:**

**TAILWATER RESULTS:**

**ROADWAY DATA:**

**ROADWAY OVERTAKING:**

**TECHNICAL FOOTNOTES:**

1) USE Q'NR FOR BOX CULVERTS
2) HW/D = HW/D OR HW/E FROM DESIGN CHARTS
3) FALL = HW1 - (ELHW1 - ELx) (FALL IS ZERO FOR CULVERTS ON GRADE)
4) ELH = HW + ELI (INVERT OF INLET CONTROL SECTION)
5) TW BASED ON DOWNSTREAM CONTROL OR FLOW
6) (4) = INVERT OF INLET CONTROL SECTION
7) (6) = TW OR (4 + D/2) (WHICHEVER IS GREATER)

**SUBSCRIPT DEFINITIONS:**

**COMMENTS / DISCUSSION:**

**CULVERT BARRIER SELECTED**

**PAGE 5 OF 18**
### CULVERT & TAILWATER DATA

**Culvert No.:** 3-4  
**2-Year Outlet Velocity:** 13.97 ft/sec  
**Tailwater Channel Flow Depth:** 2.03 ft  
**Design Discharge:** 80.88 CFS  
**Depth of Flow @ Outlet:** 1.37 ft  
**Brink Velocity:** 15.36 ft/sec  
**Mean Particle size of Sed Material:** 0.00082 - 0.00041  
**Proude No.:** 2.31  
**Non-scour Velocity for Soil Type:** 1.00 ft/sec  
**OUTLET PROTECTION REQUIRED**  
**Depth:** 1.21 ft  
**Width:** 4.08 ft  
**Length:** 9.00 ft  
**Plasticity Index:** 5.00  
**Location of Max. Scour:** 1.60 ft

### SCOUR HOLE SIZE

<table>
<thead>
<tr>
<th>Term</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>D_{L1} Stone Size</td>
<td>0.01 ft</td>
</tr>
<tr>
<td>D_{L2} Stone Size</td>
<td>0.10 ft</td>
</tr>
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</table>

### VDOT METHOD

<table>
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<tr>
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<th>Value</th>
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<tr>
<td>Outlet Protection Type</td>
<td>Class II</td>
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<tr>
<td>Length of Apron</td>
<td>22.50 ft</td>
</tr>
<tr>
<td>Width of Apron</td>
<td>13.50 ft</td>
</tr>
<tr>
<td>Thickness of Apron</td>
<td>3.00 ft</td>
</tr>
</tbody>
</table>

### RECOMMENDATIONS:

- [ ]
- [ ]
### Chapter 3 – Documentation

**PROJECT**: Back Hampden Sydney Road  
**ROAD**: Back Hampden Sydney  
**COUNTY**: Prince Edward  
**CULVERT**: VA  
**DESIGNER**: SSW/RW  
**DATE**: 8/21/2014  
**REVIEWER**: VAR  
**DATE**: 8/21/2014

#### HYDROLOGICAL DATA

- **Drainage Area**: 15,656
- **Time of Concentration**: 29.672

#### DESIGN FLOWS

- **R.I. (years)**: 10  
- **FLOW (cfs)**: 13  
- **Check**: 14.86  
- **Max**: 24.72

#### CULVERT DESCRIPTION

- **Type/Size**: Single/Multi-Conf./Conforming
- **Inlet Edge Description**: Single/Multi-Conforming/Slotted/Back Culverts

#### TAILWATER DATA:

- **Channel Shape**: Parabolic  
- **Discharge**: 4.50  
- **Elevation**: 7.58  
- **Velocity**: 3.10  
- **Shore force**: 0.86

#### TAILWATER RESULTS

- **Roadway Length**: 28
- **Discharge Guttering Design Overtopping Erosion**:
  - **Outlet**: 14.50
  - **Length of Road**: 1000

#### TECHNICAL FOOTNOTES:

1. USE QNS FOR BOX CULVERTS
2. **HWD** = **HW**/D OR **HW**/D FROM DESIGN CHARTS
3. **FALL** = **EL**1 - **EL**2 (INLET CONTROL SECTION)
4. **HWI** = **HW**/D (OUTLET CONTROL SECTION)
5. **TW**: BASED ON DOWNSTREAM CONTROL OR FLOW
6. **H** = (1 + **L**/200)**R**1/2

#### SUBSCRIPT DEFINITIONS:

- **HWI**: Design Headwater  
- **HW**: Inlet Control  
- **HWI**: Outlet Control  

---

**COMMENTS / DISCUSSION**:  
**CULVERT BARREL SELECTED**:  
**SIZE**:  
**SHAPE**:  
**MATERIAL**:  
**ENTRANCE**:  

---

**Page 7 of 18**
### Chapter 3 – Documentation

**PROJECT**
- Bank Hampden Sydney Road

**ROAD**
- Bank Hampden Sydney

**COUNTY**
- Prince Edward

**CULVERT**
- 4-2 115-10

**SHEET OF**
- ENGLISH

**DESIGNER:** SS/RW
**DATE:** 8/21/2014
**REVIEWER:** VA
**DATE:** 8/21/2014

### HYDROLOGICAL DATA
- **Method:** INPUT
- **Drainage Area:** 15.806
- **Type of Concentration:** 20.472

### DESIGN FLOWS

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<th>FLOW (cfs)</th>
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</thead>
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<tr>
<td>19</td>
<td>Design</td>
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<td>2</td>
<td>Check</td>
</tr>
<tr>
<td>25</td>
<td>Max</td>
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</table>

### CULVERT DESCRIPTION:
- **TYPE:** Single/Multiple Conforming
- **Inlet Edge Description:**
  - **Column:**
  - **Material:** SHAPE
  - **Size:** 4 in
  - **N:**

### TAILWATER DESCRIPTION:
- **Length:**
- **Elev.:**
- **SKEW:**

### TAILWATER RESULTS:

<table>
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<th>Discharge</th>
<th>Elev.</th>
<th>Flow depth</th>
<th>Velocity</th>
<th>Shear stress</th>
<th>Roadway Width, B</th>
<th>Discharge</th>
<th>Overlapping Discharge</th>
<th>Overlapping Elevation</th>
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</thead>
<tbody>
<tr>
<td>336.14</td>
<td>0.06</td>
<td>3.10</td>
<td>0.34</td>
<td></td>
<td>5.00</td>
<td>5.00</td>
<td>6.00</td>
<td>0.00</td>
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</table>

### TECHNICAL FOOTNOTES:
1. USE QNB FOR BOX CULVERTS
2. HWI = HWI OR HWI FROM DESIGN CHARTS
3. FALL = HWI - (ELI - ELs) = FALL FOR BOX CULVERTS ON GRADE
4. ELs = HWI + ESL
5. TW BASED ON DOWNSTREAM CONTROL OR FLOW
6. $H = [1 + ke + (20lw)^{1/2}]^{1/2}$

### SUBSCRIPT DEFINITIONS:
- **HWd** = DESIGN HEADWATER
- **I** = INLET
- **HWI** = HW IN INLET CONTROL
- **O** = OUTLET
- **HWo** = HW IN OUTLET CONTROL
- **Streambed**

### COMMENTS / DISCUSSION:

---

**Page 8 of 18**
### CULVERT & TAILWATER DATA

<table>
<thead>
<tr>
<th>Culvert No.</th>
<th>Culvert Dia. / Rise</th>
<th>Design Discharge</th>
<th>Depth of Flow @ Outlet</th>
<th>Brink Velocity</th>
<th>Tailwater Channel Flow Depth</th>
<th>Mean Particle size of Bed Material:</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-2</td>
<td>2.00 Ft.</td>
<td>20.17 CFS.</td>
<td>1.08 Ft.</td>
<td>11.64 Ft/Sec.</td>
<td>0.86 Ft.</td>
<td>0.00042 - 0.00041</td>
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</tbody>
</table>

**Non-scarb Velocity for Soil Type**: 0.96 Ft/Sec.

### OUTLET PROTECTION

**OUTLET PROTECTION REQUIRED!**

<table>
<thead>
<tr>
<th>SCOUR HOLE SIZE</th>
<th>D&lt;sub&gt;16&lt;/sub&gt; Stone Size</th>
<th>D&lt;sub&gt;50&lt;/sub&gt; Stone Size</th>
<th>Plasticity Index</th>
<th>Depth</th>
<th>Width</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.01 Ft.</td>
<td>0.10 Ft.</td>
<td>5.00</td>
<td>0.72 Ft.</td>
<td>2.42 Ft.</td>
<td>5.33 Ft.</td>
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</table>

**LOCATION OF MAX. SCOUR**: 2.13 Ft.

### VDOT METHOD

- Outlet Protection Type: Class A1
- Length of Apron: 6.00 Ft.
- Width of Apron: 6.00 Ft.
- Thickness of Apron: 2.00 Ft.

### RECOMMENDATIONS:
### Chapter 3 – Documentation

#### HYDROLOGICAL DATA
- **Method:** TR-35
- **Drainage Area:** 0.743 Sq. M.
- **Time of Concentration:** 1.221 Hours

#### DESIGN FLOWS
- **R.L.(gauge):**
  - 19: Design
  - 25: Max
- **F.O.W.(gauge):**
  - "n": 9.57

#### CULVERT DESCRIPTION:
- **TYPE:** Single/Multiple Confining

#### CULVERT DESIGN FORM LD-249

#### TAILWATER DATA:
<table>
<thead>
<tr>
<th>LENGTH (ft)</th>
<th>Discharge (cfs)</th>
<th>Elevation (ft)</th>
<th>Depth (ft)</th>
<th>Velocity (fps)</th>
<th>Shear Stress (psf)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### ROADWAY DATA:
- **CULVERT BARREL SELECTED:** 
  - **SIZE:** 
  - **SHAPE:** 
  - **MATERIAL:** 

#### TECHNICAL FOOTNOTES:
1. USE Q/NB FOR BOX CULVERTS
2. HW/D = HW/D OR LW/D FROM DESIGN CULVERTS
3. FALL = HW - EL4 or EL1/EH1, FALL IS ZERO FOR CULVERTS ON GRADE
4. EL4 = HW + EL (INVERT OF INLET CONTROL SECTION)
5. TW = TW OR (D + D/2) (WHEREVER IS GREATER)
6. H = [1 + k_e [(N-L)/(R+e)]^2]^{1/2}

#### SUBSCRIPT DEFINITIONS:
- HW/D = DESIGN HEADWATER
- HW/e = HW IN EMBANKMENT
- HW/o = HW IN OUTLET CONTROL
- EL4 = INVERT OF EMBANKMENT
- EL1/EH1 = INVERT OF OUTLET CONTROL
- N = Channel Slope
- L = Channel Length
- R = Embankment Slope
- k_e = Embankment Coefficient
- S = Streambed Slope
- e = Embankment Erosion Coefficient

#### COMMENTS / DISCUSSION:

---

**Page 10 of 18**
### OUTLET PROTECTION

<table>
<thead>
<tr>
<th>CULVERT &amp; TAILWATER DATA</th>
<th>DESIGN SS/RJW</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Culvert No:</strong></td>
<td>4-S</td>
</tr>
<tr>
<td><strong>Culvert Dia. / Rise:</strong></td>
<td>8.00 Ft.</td>
</tr>
<tr>
<td><strong>Design Discharge:</strong></td>
<td>295.17 CFS.</td>
</tr>
<tr>
<td><strong>Depth of Flow @ Outlet:</strong></td>
<td>2.99 Ft.</td>
</tr>
<tr>
<td><strong>Drain Velocity:</strong></td>
<td>12.35 Ft/Sec.</td>
</tr>
<tr>
<td><strong>Froude No.:</strong></td>
<td>1.26</td>
</tr>
<tr>
<td><strong>Tailwater Channel Flow Depth:</strong></td>
<td>3.55 Ft.</td>
</tr>
<tr>
<td><strong>2-Year Outlet Velocity:</strong></td>
<td>11.16 Ft/Sec.</td>
</tr>
<tr>
<td><strong>Natural Channel Bed Material:</strong></td>
<td>Fine Sand</td>
</tr>
<tr>
<td><strong>Mean Particle size of Bed Material:</strong></td>
<td>0.00082 - 0.0041</td>
</tr>
<tr>
<td><strong>Non-scour Velocity for Soil Type:</strong></td>
<td>1.24 Ft/Sec.</td>
</tr>
</tbody>
</table>

### SCOUR HOLE SIZE

| Depth | 1.91 Ft. |
| Width | 5.98 Ft. |
| Length| 13.63 Ft.|
| Plasticity Index | 5.00 |
| Location of Max. Scour | 5.45 Ft. |

### VDOT METHOD

<table>
<thead>
<tr>
<th>Outlet Protection Type</th>
<th>Class I</th>
</tr>
</thead>
<tbody>
<tr>
<td>(See VDOT Design Standards for Details)</td>
<td></td>
</tr>
<tr>
<td>Length of Apron</td>
<td>30.00 Ft.</td>
</tr>
<tr>
<td>Width of Apron</td>
<td>24.00 Ft.</td>
</tr>
<tr>
<td>Thickness of Apron</td>
<td>2.00 Ft.</td>
</tr>
</tbody>
</table>

### RECOMMENDATIONS:
## Chapter 3 – Documentation

**LD-268**

| STA TO STA | FLOW | LENGTH | 0.9 | 0.5 | 0.3 | CA | Tc | I2 | Q2 | TYPE SECTION | ALLOW VEL | VEL | DEP | VEL | DEP | I10 | Q10 | DEP | REMARKS |
|------------|------|--------|-----|-----|-----|----|----|----|----|-------------|-----------|-----|-----|-----|-----|-----|-----|-----|------|---------|
| 100+00     | 100+50 | 50     | 0.41| 0.365| 0.322| 0.161| 2.656| 0.887| 1.417| 1.417| 12.0  | 3.83 | 5.43 | 2   | 0.030| 2.0 | 4.14 | 2.82 | 0.98 | 0.00 | 5.13 | 7.77 | 0.98 | EC-2   |
| 100+50     | 101+00 | 50     | 0.02 | 0.018| 0.021| 0.01 | 0.095| 0.029| 0.056| 1.473| 12.7  | 3.74 | 5.509| 2  | 0.030| 2.0 | 4.18 | 2.85 | 0.98 | 0.00 | 5.01 | 7.39 | 0.98 | EC-2   |
| 101+00     | 101+50 | 50     | 0.02 | 0.018| 0.021| 0.01 | 0.095| 0.029| 0.056| 1.531| 13.5  | 3.64 | 5.566| 2  | 0.039| 2.0 | 3.56 | 2.44 | 0.96 | 0.00 | 4.89 | 7.47 | 1.07 | EC-2   |
| 101+50     | 102+00 | 50     | 0.02 | 0.018| 0.021| 0.01 | 0.095| 0.029| 0.056| 1.588| 14.1  | 3.57 | 5.556| 2  | 0.034| 2.0 | 4.36 | 2.97 | 0.98 | 0.00 | 4.79 | 7.00 | 0.98 | EC-2   |
| 102+00     | 102+50 | 50     | 0.02 | 0.018| 0.021| 0.01 | 0.095| 0.029| 0.056| 1.643| 14.7  | 3.50 | 5.747| 2  | 0.039| 2.0 | 4.44 | 3.03 | 0.98 | 0.00 | 4.71 | 7.73 | 0.97 | EC-2   |
| 102+50     | 103+00 | 50     | 0.02 | 0.018| 0.021| 0.01 | 0.095| 0.029| 0.056| 1.699| 15.4  | 3.42 | 5.815| 2  | 0.027| 2.0 | 4.03 | 2.75 | 0.92 | 0.00 | 4.61 | 7.83 | 1.03 | EC-2   |
| 103+00     | 103+50 | 50     | 0.02 | 0.018| 0.021| 0.01 | 0.095| 0.029| 0.056| 1.756| 16.1  | 3.38 | 5.881| 2  | 0.031| 2.0 | 4.28 | 2.92 | 0.90 | 0.00 | 4.52 | 7.93 | 1.00 | EC-2   |
| 103+50     | 104+00 | 50     | 0.02 | 0.018| 0.021| 0.01 | 0.095| 0.029| 0.056| 1.812| 16.8  | 3.28 | 5.945| 1  | 0.025| 2.0 | 4.10 | 2.79 | 1.03 | 0.03 | 4.43 | 8.03 | 1.15 | EC-2   |
| 104+00     | 104+50 | 50     | 0.02 | 0.018| 0.021| 0.01 | 0.095| 0.029| 0.056| 1.869| 17.5  | 3.21 | 6.008| 1  | 0.027| 2.0 | 4.24 | 2.89 | 1.02 | 0.03 | 4.35 | 8.12 | 1.14 | EC-2   |
| 104+50     | 105+00 | 50     | 0.02 | 0.018| 0.021| 0.01 | 0.095| 0.029| 0.056| 1.923| 18.0  | 3.17 | 6.102| 1  | 0.050| 2.0 | 5.61 | 3.82 | 0.89 | 0.09 | 4.29 | 8.25 | 1.00 | EC-2   |
| 105+00     | 105+50 | 50     | 0.02 | 0.018| 0.021| 0.01 | 0.095| 0.029| 0.056| 1.980| 18.6  | 3.13 | 6.194| 1  | 0.064| 2.0 | 5.92 | 4.04 | 0.88 | 0.00 | 4.23 | 8.39 | 0.98 | EC-2   |
| 105+50     | 106+00 | 50     | 0.02 | 0.018| 0.021| 0.01 | 0.095| 0.029| 0.056| 2.038| 19.0  | 3.08 | 6.264| 1  | 0.059| 2.0 | 5.71 | 3.91 | 0.90 | 0.00 | 4.18 | 8.51 | 1.00 | EC-2   |
| 106+00     | 106+50 | 50     | 0.02 | 0.018| 0.021| 0.01 | 0.095| 0.029| 0.056| 2.095| 19.4  | 3.05 | 6.389| 1  | 0.059| 2.0 | 7.01 | 4.78 | 0.92 | 0.00 | 4.14 | 8.66 | 0.92 | EC-3A  |

**Diagnoses:**

- **Location:** Prince Edward
- **Date:** 8/21/2014
- **Sheet:** 1
<table>
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<tr>
<th>STA. TO STA.</th>
<th>Flow (cfs)</th>
<th>0.9</th>
<th>0.5</th>
<th>0.3</th>
<th>CA</th>
<th>Tc</th>
<th>I2</th>
<th>TOP SECTION</th>
<th>SLOPE</th>
<th>VEL.</th>
<th>DEP</th>
<th>VEL.</th>
<th>DEP</th>
<th>H0</th>
<th>Q10</th>
<th>DEP</th>
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<tr>
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<td>108+00</td>
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<td>0.022</td>
<td>0.014</td>
<td>0.007</td>
<td>0.219</td>
<td>0.068</td>
<td>0.094</td>
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<td>0.007</td>
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<td>0.094</td>
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<td>0.094</td>
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### Chapter 3 – Documentation

#### LD-268

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**PROJECT** 0643-073-124, M501

**BY** RJW

**LOCATION** Prince Edward

**DATE** 8/31/2014

**SHEET** 1
### Chapter 3 – Documentation

#### LD-268

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**Typ. Section 2**

**Typ. Section 3**
# Chapter 3 – Documentation

## LD-268

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**PROJECT**: 80068

**LOCATION**: Prince Edward

**DATE**: 8/31/2014

**SHEET**: 1
Appendix 3A-4 PAC Milestone Deliverables

COMMONWEALTH of VIRGINIA
DEPARTMENT OF TRANSPORTATION
LOCATION AND DESIGN DIVISION

REPORT COVER SHEET

Final Hydraulic Design Computations
03/01/2016

Drainage Report Prepared By: Rebecca J. Worley, P.E.
Drainage Report Checked By: Victoria A. Bains, P.E.

Project Description: Route 17/Hook Road Signalization; sidewalk construction
From: Intersection of Route 17/Route 1216
To: 0.05 Mi. South of Intersection of Route 17/1216
Project UPC No.: 98806

Responsible for Pages 1 to 18
## TABLE OF CONTENTS

- Sealing and Signing of Responsible Person Cover Sheet ................................................................. 1
- Table of Contents .............................................................................................................................. 2
- Hydraulic Design Narrative .............................................................................................................. 3-4
- Erosion and Sediment Control/Stormwater Management Certification ....................................... 5-6
- Appendices ....................................................................................................................................... 7-18

Appendix A: Water Quantity – Outfall Analysis

Appendix B: Water Quality – Virginia Runoff Reduction Method Spreadsheet
Hydraulic Design Narrative

General Description:

VDOT Central Office Hydraulics Section has prepared a drainage design and associated computations for the Route 17/Route 1216 intersection project located in Gloucester County. The purpose of this project is to coordinate signals to improve traffic flow and safety and reduce congestion. Additionally, sidewalk will be constructed. The drainage plans and computations address the hydraulic changes to the roadway and comply with all regulatory requirements. A detailed description of Water Quantity and Water Quality compliance is shown in Appendix A and Appendix B respectively.

Existing Conditions:

The project site is located in Gloucester County. The properties along Route 17 are businesses. The southwest corner of the intersection currently drains via sheet flow, eventually draining to the Route 17 roadway storm sewer system. The southeast corner of the intersection currently drains via sheet flow, eventually draining to the Route 216 roadway storm sewer system. The median to the south of the intersection currently drains via a graded swale which outfalls into the Route 17 roadway storm sewer system. Both storm sewer systems ultimately outfall to Sarah Creek. There are no wetlands or water quality structures located within the project limits.

Proposed Drainage System:

The project will maintain the same general flow patterns. Runoff from both the southeast and southwest corners of the intersection will sheet flow from the project limits. Runoff from the median will flow through a graded grass swale and tie into the existing concrete median ditch approximately 250 feet south of the intersection. All runoff from the project limits will ultimately outfall to Sarah Creek.

Erosion and Sediment Control (ESC):

This ESC Plan has been designed, prepared, reviewed, and approved in accordance with the VDOT’s approved ESC & Stormwater Management (SWM) Program Standards and Specifications. A copy of the Certification Form is part of this document.
Appendix 3A-4 PAC Milestone Deliverables

**Hydrology:**

The Rational Method of calculating discharge is used exclusively on this project because all drainage areas are less than 200 acres in size. Drainage areas were calculated with the use of electronic survey and design data.

**Hydraulic Design Software:**

The design software utilized on this project includes:

- Virginia Runoff Reduction Method Spreadsheet
EROSION & SEDIMENT CONTROL/
STORMWATER MANAGEMENT
CERTIFICATION
Appendix 3A-4 PAC Milestone Deliverables

LD-445C (10/20/14)

VIRGINIA DEPARTMENT OF TRANSPORTATION
LOCATION AND DESIGN
EROSION AND SEDIMENT CONTROL (ESC) AND STORMWATER MANAGEMENT
(SWM) CERTIFICATION FORM

From: Plan Reviewer Rebecca Worley
To: Project Manager Nathan Huber

District: Fredericksburg Residency: Saluda

UPC Number: 98806 VDOT Project Number: 0017-036-577, M-501

Area to be Disturbed (to the nearest one-hundredth acre): 0.17

This form shall be completed by the Plan Reviewer and provided to the ESC/SWM Plan Designer. The ESC & SWM Plan Designer shall forward this form to the Project Authority for use in completing the application for a VPDES Construction Permit (if applicable).

This form serves to ensure that a project specific ESC Plan and SWM Plan has been designed/prepared, reviewed, and approved in accordance with the Virginia Department of Transportation’s approved ESC & SWM Standards and Specifications.

ESC Plan Reviewer*
The ESC Plan for the project listed above has been reviewed and approved in accordance with the VDOT’s approved ESC Standards and Specifications.

Signature: 
Title: Hydraulics Engineer

Printed name: Rebecca Worley Date: 3/1/2016

*DEQ Certified Plan Reviewer for ESC or Professional Engineer, Land Surveyor, Landscape Architect or Architect with expertise in the field of ESC.

SWM Plan Reviewer**
The SWM Plan for the project listed above has been reviewed and approved in accordance

Signature: 
Title: Hydraulics Engineer

Printed name: Rebecca Worley Date: 3/1/2016

**DEQ Certified Plan Reviewer for SWM: Individuals seeking SWM certification will be considered provisionally certified for two Years from the date they complete their first required training course.
APPENDIX A:
WATER QUANTITY
Appendix 3A-4  PAC Milestone Deliverables

**Water Quantity:**

This project utilizes the technical criteria of Part IIB (9VAC25-870-62) for determining its post-development stormwater management design. Using the DEQ Runoff Reduction spreadsheet for redevelopment, the total phosphorus load reduction required was found to be 0.07 lb/yr. Nutrient credits will be purchased to meet the water quality criteria.

Each outfall was analyzed individually for compliance with the water quantity criteria. Detailed information about each outfall analysis follows.

**Outfall 1 – Southwest corner (Route 17 George Washington Memorial Highway & Route 1219 Hook Road)**

Land disturbance at this outfall is associated with the construction of sidewalk and totals 0.0132 acres. The project area will flow into the gutter pan on Route 17, where the total drainage area is 2.2615 acres. Because the disturbed area is less than one-percent \( \left( \frac{0.0132}{2.2615} \times 100 = 0.58\% \right) \) of the total drainage area at the outfall and the spread from the gutter pan does not exceed the allowable, no water quantity controls are required for channel or flood protection. Calculated spread from the gutter pan at the limits of disturbance is 5.37’, which is less than the allowable spread of 8’. The drainage area map on the following page demonstrates compliance with the one-percent rule.
Appendix 3A-4 PAC Milestone Deliverables

Disturbed area = 0.0132 acres
Total drainage area = 2.2615 acres
0.58%
Outfall 2 – Route 17 George Washington Memorial Highway Median (South of intersection)

Land disturbance at this outfall is associated with the construction of a raised concrete median and pipe extension. Additionally, the grass channel downstream of the outfall pipe will be graded approximately 250 feet to tie into the existing concrete ditch. The total land disturbance is 0.1376 acres and the total drainage area at the outfall is 18.2121 acres. Because the disturbed area is less than one-percent ($\frac{0.1376}{18.2121} \times 100 = 0.76\%$) of the total drainage area at the outfall, no water quantity controls are required for channel or flood protection. The drainage area map on the following page demonstrates compliance with the one-percent rule.
Disturbed area = 0.1376 acres
Total drainage area = 18.2121 acres
0.76%
Appendix 3A-4 PAC Milestone Deliverables

Outfall 3 – Southeast corner pedestrian ramp (Route 17 George Washington Memorial Highway & Route 216 Guinea Road)

Land disturbance at this outfall is associated with the construction of a pedestrian ramp and totals 0.0041 acres. The disturbed area outfalls to the gutter and drains along Guinea Road, where the total drainage area at the first drop inlet is 0.4241 acres. Because the disturbed area is less than one-percent \( \frac{0.0041}{0.4241} \times 100 = 0.97\% \) of the total drainage area at the outfall and the spread does not exceed the allowable from the disturbed area to the first drop inlet, no water quantity controls are required for channel or flood protection. Calculated spread at the first drop inlet along Guinea Road is 4.91’, which is less than allowable spread of 8’. The drainage area map on the following page demonstrates compliance with the one-percent rule.
Appendix 3A-4  PAC Milestone Deliverables

Disturbed area = 0.0041 acres
Total drainage area = 0.4241 acres
0.97%
Appendix 3A-4  PAC Milestone Deliverables

Outfall 4 – Southeast corner sidewalk construction (Route 17 George Washington Memorial Highway & Route 216 Guinea Road)
Land disturbance at this outfall is associated with the construction of sidewalk and totals 0.0131 acres. Runoff draining from this land disturbance will not be concentrated, but will maintain runoff as sheet flow from the sidewalk area. Due to the limited increase in impervious area leading to a very slight increase in runoff, it is not anticipated that the project will cause flooding, erosion, or sedimentation to the downstream area. Therefore, this outfall is in compliance with the guidance in section D of the water quantity regulations for sheet flow.
Appendix 3A-4  PAC Milestone Deliverables

Disturbed area = 0.0131 acres
APPENDIX B: WATER QUALITY
Appendix 3A-4 PAC Milestone Deliverables

Water Quality:
This project utilizes the technical criteria of Part IIB (9VAC25-870-62) for determining its post-development stormwater management design. Using the DEQ Runoff Reduction spreadsheet for redevelopment, the total phosphorus load reduction required was found to be 0.07 lb/yr. Nutrient credits will be purchased to meet the water quality requirements for this project.

The DEQ Runoff Reduction spreadsheet below demonstrates compliance with the water quality requirements for this project.
### Virginia Runoff Reduction Method ReDevelopment Worksheet - v2.8 - June 2014

**To be used w/2011 BMP Standards and Specifications**

**Appendix 3A-4 PAC Milestone Deliverables**

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**Chapter 3 - Documentation**

**Site Data**

- **Project Name:** SBO10 Rte. 17 at Hook Road
- **Date:** 3/17/2014

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**Soil Characteristics**

- **A soils**: 0.30
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**Land Use Summary**

- Total Ro Development: 3.07 acres
- Post-Ro Development: 1.50 acres
- New Impervious: 1.57 acres

**Pre-Ro Development Treatment Volume (acre-ft)**

- **Ro Development Treatment Volume (acre-ft)**: 3.07
- **Post-Ro Development Treatment Volume (acre-ft)**: 0.30

**Pre-Ro Development Load (TPB (lbs))/**

- **Pre-Ro Development Load (TPB (lbs))**: 0.50
- **Post-Ro Development Load (TPB (lbs))**: 0.30

**A Parker Land Cover Summary reflects the pre - redeveloped land cover minus the porous land cover (forested/open space or managed turf) area.**

- **Total Acreage**: 3.07 acres
- **New Impervious Area**: 1.57 acres
- **Ro Development Area**: 1.50 acres

**Appendix 3A-4 PAC Milestone Deliverables**

---

**Page 18 of 18**
### Appendix 3A-5 Drainage Summary

<table>
<thead>
<tr>
<th>Reservoir</th>
<th>Capacity (m³)</th>
<th>Inflow (m³/day)</th>
<th>Outflow (m³/day)</th>
<th>Storage (m³)</th>
<th>Uplift (m³)</th>
<th>Erosion (m³)</th>
<th>Sediment (m³)</th>
<th>Clogging (m³)</th>
<th>Maintenance (m³)</th>
<th>Total (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reservoir 1</td>
<td>1000</td>
<td>500</td>
<td>200</td>
<td>800</td>
<td>50</td>
<td>10</td>
<td>100</td>
<td>50</td>
<td>20</td>
<td>1370</td>
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<tr>
<td>Reservoir 2</td>
<td>1500</td>
<td>700</td>
<td>300</td>
<td>1200</td>
<td>60</td>
<td>20</td>
<td>150</td>
<td>70</td>
<td>30</td>
<td>1960</td>
</tr>
<tr>
<td>Reservoir 3</td>
<td>2000</td>
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<td>1600</td>
<td>80</td>
<td>30</td>
<td>200</td>
<td>80</td>
<td>40</td>
<td>2320</td>
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</tbody>
</table>

Note: All values are approximate and subject to change based on environmental factors and maintenance activities.

*Maintenance and maintenance activities should be scheduled at the end of each month.*
Example for “No Plan”

<table>
<thead>
<tr>
<th>Storm Sewer Pipe</th>
<th>POST INSTALLATION INSPECTION</th>
<th>Pipe (All pipe installation on plans not identified as storm sewer pipe)</th>
<th>Individual Installation</th>
<th>Quantity to inspect (LF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size</td>
<td>LF</td>
<td>Size</td>
<td>LF</td>
<td>10% of Total</td>
</tr>
<tr>
<td>12&quot;</td>
<td>788</td>
<td>12&quot;</td>
<td>79</td>
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<td>15&quot;</td>
<td>228</td>
<td>15&quot;</td>
<td>898</td>
<td>40,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24,24</td>
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<tr>
<td>18&quot;</td>
<td>36</td>
<td>18&quot;</td>
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<tr>
<td>23&quot; x 14&quot;</td>
<td>2394</td>
<td>23&quot;</td>
<td>106</td>
<td>36,36,34</td>
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<tr>
<td>30&quot; x 9&quot;</td>
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<td>45&quot; x 29&quot;</td>
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<td>Subtotals</td>
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<td>Totals to be inspected</td>
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</table>
## Field Inspection Data Sheet

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<tr>
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<th><strong>SUBMITTAL DATE:</strong></th>
<th>10/21/01</th>
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<tbody>
<tr>
<td>Project:</td>
<td>Route 33 West Point Bridges over Pamunkey and Mattaponi Rivers</td>
<td>Scheduled Advertisement:</td>
<td>May 2002</td>
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<td>Revised:</td>
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<td>Agreement Date:</td>
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<td>Location:</td>
<td>New Kent County King William County Town of West Point King and Queen County</td>
<td>Scheduled / actual milestones Preliminary Field Review:</td>
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<tr>
<td></td>
<td></td>
<td>To:</td>
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<tr>
<td></td>
<td>Submitted</td>
<td>Company/Agency</td>
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<td></td>
<td>By:</td>
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<tr>
<td></td>
<td>Submitted</td>
<td>VDOT C.O.</td>
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</tr>
<tr>
<td></td>
<td>To:</td>
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<td></td>
<td>To:</td>
<td>Design Assignments</td>
<td>Project Manager Hydraulics Task Leader</td>
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Note: Sheet to be filled out and included in H&H Report. Blank sheet provided on next page.
### Documentation Data Sheet for Hydrologic and Hydraulic Computations

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<td></td>
<td>Project Manager</td>
<td>Hydraulics Task Leader</td>
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Note: This sheet to be filled out and included in H&H Report.
Appendix 3B-2 Suggested Outline for VDOT Hydrologic and Hydraulic Analysis Reports

Cover for H&HA Report describing project, submittal, and schedule

Section I - Project Description and Requirements

Section II - Hydrology
A. Criteria
B. Methodology
C. Peak Discharge Computations and Summary Table
D. FEMA Flood Maps
E. Previous Studies
F. Data Gathering

Section III - Open Channel Hydraulics
A. Criteria
B. Methodology
C. Typical Roadway Ditch Sections
D. Roadway Ditch Computations and Summary Table
E. Existing Stream Inventory
F. Data Gathering

Section IV - Culverts Hydraulics
A. Criteria
B. Methodology
C. Culvert Computations and Summary Table
D. Data Gathering

Section V - Storm Sewer Hydraulics
A. Criteria
B. Methodology
C. Spread Computations
D. Storm Sewer and Hydraulic Grade Line Computations
E. Data Gathering

Section VI - Stormwater Management
A. Criteria
B. Methodology
C. Stormwater Management Plan Summary
D. Detention Basin Computations
E. Data Gathering

Section VII – Erosion and Sediment Control
A. Criteria
B. Methodology
C. Sediment Basin Plan Summary
D. Phase I Narrative
E. Phase II Narrative
F. Data Gathering

Note: This a suggested format and does not attempt to identify all the elements necessary for adequate analysis or documentation
A separate form should be submitted for each appropriate site on this project.

I. Hydrologic History of Site (District Drainage Engineer)

State any unusual hydrologic occurrences of which you have or can acquire knowledge.

_____________________________________________________________________________
_____________________________________________________________________________
_____________________________________________________________________________

II. Hydraulic History of Site (District Bridge & Drainage Engrs.)

_____________________________________________________________________________
_____________________________________________________________________________
_____________________________________________________________________________

III. Comment of the relative importance and/or value of private or public property adjacent to this site (up and downstream) and the general affect of floods thereon. (Dist. Drainage Engr.)

_____________________________________________________________________________
_____________________________________________________________________________
_____________________________________________________________________________
_____________________________________________________________________________
### Chapter 3 - Documentation

#### Appendix 3B-3 Field Engineer’s Hydraulic Report

<table>
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<tr>
<th>IV.</th>
<th>State rip rap and/or scour protection recommendations and justification for these recommendations. (Dist. Bridge Engr.).</th>
</tr>
</thead>
<tbody>
<tr>
<td>V.</td>
<td>Provide a basic assessment of the environmental, ecological, historical and economic considerations, which may exert an influence on this site. (District Drainage Engineer)</td>
</tr>
<tr>
<td>VI.</td>
<td>Make note of any flood plain zoning and/or flood plain studies in existence or eminently proposed. (Dist. Drainage Engr.)</td>
</tr>
<tr>
<td>VII.</td>
<td>Other Special Considerations and Remarks (District Bridge and Drainage Engineers)</td>
</tr>
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# Chapter 4 – Legal Aspects

**TABLE OF CONTENTS**

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
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<tbody>
<tr>
<td>4.1</td>
<td>Overview</td>
<td>4-1</td>
</tr>
<tr>
<td>4.1.1</td>
<td>Introduction</td>
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</tr>
<tr>
<td>4.1.2</td>
<td>Order of Authority</td>
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</tr>
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<td>4.1.3</td>
<td>Related Publications</td>
<td>4-2</td>
</tr>
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<td>4.2</td>
<td>Federal Laws</td>
<td>4-3</td>
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<td>4.2.1</td>
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</tr>
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<td>4.3</td>
<td>Environmental Permits</td>
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<td>4.3.1</td>
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<td>4.4</td>
<td>Floodplain Laws and Regulations</td>
<td>4-5</td>
</tr>
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<td>4.4.1</td>
<td>FHWA Regulations, Policy, and Guidance</td>
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<td>Flood Disaster Protection</td>
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Chapter 4 - Legal Aspects

4.1 Overview

4.1.1 Introduction

Various drainage laws and rules applicable to highway facilities are discussed in this chapter. The purpose of this chapter is to provide information and guidance on the engineer's role in the legal aspects of highway drainage. This chapter should not be treated as a manual upon which to base legal advice or make legal decisions. It is also not a summary of all existing drainage laws, and most emphatically, this chapter is not intended as a substitute for legal counsel.

The following generalizations can be made in reaching the proper conclusion regarding liability:

- A goal in highway drainage design should be to perpetuate natural drainage, insofar as practicable.
- In general, the courts tend to look with disfavor upon infliction of injury or damage that reasonably could have been avoided by a prudent designer, even where some alteration in flow is legally permissible.
- The laws relating to the liability of government entities are undergoing change, with a trend toward increased government liability.

4.1.2 Order of Authority

The descending order to law supremacy is Federal, State, and local, and, except as provided for in the statutes or constitution of the higher level of government, the superior level is not bound by laws, rules, or regulations of a lower level. State permit requirements are an example of law supremacy. Federal agencies do not secure permits issued by State agencies, except as required by Federal law. Many laws of one level of government are passed for the purpose of enabling that level to comply with or implement provisions of laws of the next higher level. In some instances, however, a lower level of government may promulgate a law, rule or regulation that would require an unreasonable or even illegal action by a higher level. An example is a local ordinance that would require an expenditure of State funds for a purpose not intended in the agency’s revenue appropriation.

The rule of legal supremacy is interpreted to mean that VDOT policies and criteria are not subject to ordinances and regulations promulgated by local governing bodies, except in those specific instances and for specific purposes where the Department has expressly agreed to abide by the local criteria. In situations where there are no conflicts with matters of legal supremacy, the Department recognizes the prudence of supporting the objectives of local ordinances and regulations.
VDOT will administer the design, construction, and maintenance of highways accordingly, to the extent practicable.

When ordinances, criteria, regulations, etc. of local governing bodies are more restrictive than State law and/or VDOT policies and criteria, the Department is not legally bound to observe the local mandate except as noted above.

Many of the questions relative to conflicts in laws of different levels of government involve constitutional interpretation and must be determined case by case. Such conflicts should be referred to legal counsel before any action is taken.

### 4.1.3 Related Publications

There are numerous publications on the legal aspects of drainage and water laws. For additional information on the legal aspects of highway drainage the reader is referred to the following publications:

- Legal Research Digest, Transportation Research Board
- Highway Laws of Virginia, issued by the Virginia Department of Transportation, reprinted from the Code of Virginia (current edition), copyright by the Michie Company, Charlottesville, Virginia
4.2 Federal Laws

4.2.1 General Laws

Federal law consists of the Constitution of the United States; Acts of Congress; regulations that government agencies issue to implement these acts; Executive Orders issued by the President; and case law. Acts of Congress are published immediately upon issuance and are cumulated for each session of Congress and published in the United States Statutes At Large. Compilations of Federal Statutory Law, revised annually, are available in the United States Code (USC) and the United States Code Service (USCS).

The Federal Register, which is published daily, provides a uniform system for making regulations and legal notices available to the public. Presidential Proclamations and Executive Orders, Federal agency regulations and documents having general applicability and legal effect, documents required to be published by an act of Congress and, other Federal agency documents of public interest are published in the Federal Register. Compilations of Federal regulatory material revised annually are available in the Code of Federal Regulations (CFR).

4.2.2 Drainage

Federal law does not deal with drainage per se, but many laws have implications that affect drainage design. The laws include those concerning:

- Flood insurance and construction in flood hazard areas
- Navigation and construction in navigable waters
- Environmental protection
- Protection of fish and wildlife
- Coastal zone management
- Clean Water Act
4.3 Environmental Permits

4.3.1 Permits Affecting Streams, Wetlands, and Navigable Waters

In 1977, in cooperation with Norfolk District Corps of Engineers, Environmental Protection Agency, U. S. Fish and Wildlife Service and other Federal and State agencies, VDOT initiated an integrated environmental process for project early coordination and permit acquisition. This integrated process created the opportunity for State and Federal environmental agencies to meet, discuss and influence transportation projects in Virginia, while ensuring the appropriate protection and management of Virginia’s cultural and natural resources and water resources. Since that time, the Interagency Coordination Meeting (IACM) process has become an effective mechanism to coordinate the development of projects with Federal and State agencies in order to secure the appropriate permits and environmental approvals for work in surface waters and wetlands.

The Environmental Permit Manual and Document Handbook produced by the VDOT Environmental Division, contains current information and requirements regarding permits and agency coordination.
4.4 Floodplain Laws and Regulations

4.4.1 FHWA Regulations, Policy, and Guidance

The FHWA regulations on the location and design of floodplain encroachments (23CFR650 Subpart A - Location and Hydraulic Design of Encroachments on Flood Plans) include a requirement for project-by-project risk assessment or analyses of floodplain and floodway encroachments associated with highway projects. To supplement 23CFR650A, FHWA published Hydraulic Engineering Circular 17 (HEC-17) “The Design of Encroachments on Flood Plains using Risk Analysis” to direct planners and designers to consider floodplain and floodway encroachments in highway design.

The Federal Highway Administration (FHWA) develops regulations, policies, and guidance balancing environmental stewardship, flood risks, and the costs in locating, designing, constructing, operating, and maintaining transportation systems with national floodplain goals and requirements to keep the public safe. FHWA recognizes the risks of locating highways in floodplains; therefore, regulations, policies, and procedures apply to all base floodplain encroachments and not just for floodway encroachments regulated by FEMA in the National Flood Insurance Program (NFIP).

To assist state DOT’s to be compliant with FHWA policies, FHWA developed a procedure for coordinating with FEMA that addresses encroachments in floodways, regulated floodplains with a detailed study, and regulated floodplains that have approximate flood zones (“Procedures for Coordinating Highway Encroachments on Floodplains with the Federal Emergency Management Agency (FEMA)”, 1982). FEMA reviewed the FHWA working operating procedure and believes it meets the requirements of Executive Order 11988 by requiring compliance with NFIP standards and regulations, in order to facilitate the objective of flood loss reduction (see correspondence dated June 7, 1982).

The following sub-sections provide background and summarize the program and procedures adopted by VDOT to provide statewide consistency with FHWA and FEMA floodplain programs. Later chapters in this Drainage Manual provide details on VDOT technical procedures for analyzing, documenting, reporting, and coordinating encroachments on a project-by-project basis.

4.4.2 Flood Disaster Protection

The Flood Disaster Protection Act of 1973 (Pl 93-234, 87 Stat. 975) denies Federal financial assistance to flood prone communities that fail to qualify for flood insurance. Formula grants to States are excluded from the definition of financial assistance, and the definition of construction in the Act does not include highway construction; therefore, Federal aid for highways is not affected by the Act. The Act does require communities to adopt certain land use controls in order to qualify for flood insurance. These land use controls...

* Due to the magnitude of the changes to this sub-section, shading has been omitted.
requirements could impose restrictions on the construction of highways in floodplains and floodways in communities that have qualified for flood insurance. A floodway, as used here and as used in connection with the National Flood Insurance Program, is that portion of the floodplain required to pass a flood that has a 1 percent chance of occurring in any 1 year period without cumulatively increasing the water surface elevation more than 1 foot. *

4.4.3 Flood Insurance

The National Flood Insurance Act of 1968, as amended, (42 U.S.C. §§ 4001 through 4129) requires that communities adopt adequate land use and control measures to qualify for insurance. Federal criteria promulgated to implement this provision contain the following requirements that can affect certain highways:

- In riverine situations, when the Administrator of the Federal Insurance Administration has identified the flood prone area, the community must require that, until a floodway has been designated, no use, including land fill, be permitted within the floodplain area having special flood hazards for which base flood elevations have been provided, unless it is demonstrated that the cumulative effect of the proposed use, when combined with all other existing and reasonably anticipated uses of a similar nature, will not increase the water surface elevation of the 100-year flood more than 1 foot at any point within the community.
- After the floodplain area having special flood hazards has been identified and the water surface elevation for the 100-year flood and floodway data have been provided, the community must designate a floodway which will convey the 100-year flood without increasing the water surface elevation of the flood more than 1 foot at any point and prohibit, within the designated floodway, fill, encroachments, and new construction and substantial improvements of existing structures which would result in any increase in flood heights within the community during the occurrence of the 100-year flood discharge.
- The participating cities and/or counties agree to regulate new development in the designated floodplain and floodway through regulations adopted in a floodplain ordinance. The ordinance requires that development in the designated floodplain be consistent with the intent, standards and criteria set by the National Flood Insurance Program.

4.4.4 Local Community

The local community with land use jurisdiction, whether it is a city, county, or State, has the responsibility for enforcing National Flood Insurance Program (NFIP) regulations in that community if the community is participating in the NFIP. Therefore, essential first steps in conducting location hydraulic studies and preparing environmental documents include familiarity with the NFIP program and published maps and studies for a locality.

* Due to the magnitude of the changes to this sub-section, shading has been omitted.
VDOT may also coordinate with the local NFIP community when an encroachment does not require the revision of NFIP maps, but does result in an increase in flooding that is consistent with the NFIP. VDOT coordination with a local community will occur when an encroachment for a VDOT project results in the need to revise NFIP maps, as the local community will have an opportunity to review the map revision and provide FEMA and VDOT with comments. *

Consistency with NFIP standards is a requirement for FHWA Federal-Aid highway actions involving regulatory floodways. As per FHWA guidance, VDOT will submit proposals to Federal Emergency Management Agency (FEMA) for revisions to NFIP maps in a community, should it be necessary. VDOT will also pay fees for FEMA review of map revisions and respond directly to questions and comments from FEMA on map revisions. VDOT will communicate with the local NFIP community regarding potential impacts to NFIP maps during the project NEPA phase, and during map revision activities when seeking community official signature on form MT-2.

4.4.5 NFIP Maps

Where NFIP maps are available, their use is mandatory in determining whether a highway location alternative will include an encroachment on the base floodplain. FEMA Map Modernization efforts resulted in NFIP maps being updated and now being called Flood Insurance Rate Maps (FIRMs), containing all of the information previously included on historic flood boundary and floodway maps, when applicable.

Communities may or may not have published FIRMs, depending on their level of participation in the NFIP. Information on community participation in the NFIP is provided in the "National Flood Insurance Program Community Status Book" which is published on the internet at the following location: https://www.fema.gov/national-flood-insurance-program-community-status-book.

FIRM, FIS, and other related documents can be obtained from the FEMA Map Service Center (MSC) at the following internet location: https://msc.fema.gov/portal/home.

4.4.6 Coordination with FEMA

VDOT should coordinate with FEMA in situations where administrative determinations are needed involving a regulatory floodway or where flood risks in NFIP communities are significantly impacted. The circumstances which would ordinarily require coordination with FEMA include the following:

- When a proposed crossing encroaches on a regulatory floodway and, as such, would require an amendment to the floodway map.

* Due to the magnitude of the changes to this sub-section, shading has been omitted.
• When a proposed crossing encroaches on a floodplain where a detailed study has been performed, but no floodway designated, and the maximum 1 foot increase in the base flood elevation would be exceeded.
• When a local community is expected to enter into the regular program within a reasonable period and detailed floodplain studies are underway.
• When a local community is participating in the emergency program and base FEMA flood elevation in the vicinity of insurable buildings is increased by more than 1 foot. Where insurable buildings are not affected, it is sufficient to notify FEMA of changes to base flood elevations as a result of highway construction. *

Coordination means furnishing to FEMA and the community a preliminary site plan and water surface elevation information and technical data in support of a floodway revision request as required. Otherwise, this later coordination may be postponed until the design phase.

Floodplains are an issue discussed in NEPA documents. For projects that will be processed with a categorical exclusion, coordination may be carried out during design. However, the outcome of the coordination at this time could change the class of environmental processing.

4.4.6.1 Highway Encroachment on a Riverine Floodplain without a Detailed Study (Zone A)
In communities where detailed flood insurance studies have not been performed, VDOT must generate its own technical data to determine the base floodplain elevations and design encroachments in accordance with 23CFR650A. A LOMR must be submitted to FEMA as outlined previously where the increase in base flood elevations in the vicinity of insurable buildings exceeds 1 foot, but a CLOMR would not be required. VDOT will coordinate with the local community when base flood elevations increase more than 1 foot and a LOMR is required. Base floodplain elevations and encroachment information shall be furnished to the local community upon request.

4.4.6.2 Highway Encroachment on a Riverine Floodplain - Detailed Study with No Floodway (Zone AE)
In communities where a detailed flood insurance study has been performed but no regulatory floodway designated, the highway crossing should be designed to allow no more than 1 foot cumulative increase in the base flood elevation based on technical data from the flood insurance study and any construction occurring after the effective flood insurance study but before the new highway encroachment study. For base flood elevation increases less than 1 foot, the local NFIP community shall be notified and technical data supporting the increased flood elevation will be shared with the local community upon request. Increases greater than 1 foot shall be submitted to FEMA through the CLOMR/LOMR process and require coordination with the local community as discussed above in 4.4.6 and in Chapter 17.

* Due to the magnitude of the changes to this sub-section, shading has been omitted.
4.4.6.3 Highway Encroachment on a Riverine Floodplain - Detailed Study with a Floodway (Zone AE)

In the case where a highway project requires revisions to a mapped floodway, VDOT is responsible for submitting a conditional letter of map revision (CLOMR) prior to construction of the encroachment, and a letter of map revision (LOMR) after construction. Both the CLOMR and LOMR are submitted to FEMA for review and processing. Coordination with the local NFIP community during preparation of the CLOMR and LOMR is required. *

If a project element encroaches on the floodway, but has a very minor effect on the floodway water surface elevation (such as piers in the floodway that do not cause the baseflood to extend beyond the right of way), the project may normally be considered as being consistent with FHWA and NFIP standards. Also, if hydraulic conditions can be improved so that no water surface elevation increase is reflected in the hydraulic analysis of the new conditions, the project would normally be considered as being consistent with the standards. However, if the project causes the baseflood to extend beyond the right of way, especially upstream of an encroachment, then the project may not be consistent with the floodway, requiring evaluation of other alternatives or a revision to the mapped floodway.

Where it is not cost-effective to design a highway crossing to avoid encroachment on an established floodway, a second alternative would be a modification of the floodway itself. Often, FEMA will be willing to accept an alternative floodway configuration to accommodate a proposed crossing provided NFIP limitations on increases in the base flood elevation are not exceeded. This approach is useful where the highway crossing does not cause more than a 1-foot rise in the base flood elevation. In some cases, it may be possible to enlarge the floodway or otherwise increase conveyance in the floodway above and below the crossing in order to allow greater encroachment.

When it would be demonstrably inappropriate to design a highway crossing to avoid encroachment on the floodway and where the floodway cannot be modified such that the structure could be excluded, FEMA may approve an alternate floodway with backwater in excess of the 1 foot maximum only when the following conditions have been met:

- A location hydraulic study has been performed in accordance with "Location and Hydraulic Design of Encroachments on Floodplains" (23 CFR 650.113) and FHWA finds the encroachment is the only practicable alternative.
- The constructing agency has made appropriate arrangements with affected property owners and the community to obtain flooding easements or otherwise compensate them for future flood losses due to the effects of increased backwater elevations greater than 1 foot.

*[Due to the magnitude of the changes to this sub-section, shading has been omitted.]*
• The constructing agency has made appropriate arrangements to assure that the National Flood Insurance Program and Flood Insurance Fund will not incur any liability for additional future flood losses to existing structures which are insured under the Program and grandfathered in under the risk status existing prior to the construction of the structure.

4.4.6.4 Highway Encroachment on Coastal Flooding (Zones AE, V, VE,...)
In communities where coastal flooding is an issue due to storm surge and waves, FEMA conducts studies and collects historic flood information to complete FISs and/or FIRMs published with Zones V by approximate methods and Zone AE or VE by detailed study. As roadway projects will not impact flooding elevations due to storm surge detailed analysis may not be required. The information available from the FIS will be used to inform other aspects of the design.

4.4.6.5 Highway Encroachment on Unidentified Floodplains
Encroachments that are outside of NFIP communities or NFIP identified flood hazard areas should be designed in accordance with 23CFR650A.

4.4.7 Flood Control Systems

FHWA published a memorandum titled "Highway Embankments versus Levees and other Flood Control Structures" in 2008 to address the distinction between highway embankments and levees or other flood control systems (https://www.fhwa.dot.gov/engineering/hydraulics/policymemo/20080910.cfm). In general, the FHWA does not endorse or permit the consideration or use of highway embankments as levees or other flood control structures. However, when a highway embankment is coincident with a dam or levee system FHWA and VDOT will not take ownership of the embankment portion of the project and limit VDOT’s responsibility to any roadway pavement sections on the embankment. The ownership and maintenance of the flood control features and functions shall remain the responsibility of other Federal, State or Local agencies.

* Due to the magnitude of the changes to this sub-section, shading has been omitted.
4.5 Presidential Executive Orders

4.5.1 Background

Presidential Executive Orders (E.O.) have the effect of law in the administration of programs by Federal agencies. While executive orders do not directly apply to State Departments of Transportation, these requirements are usually implemented through general regulations or other Agency documents.

4.5.2 Executive Order 11988 (E.O. 11988)

Executive Order 11988, May 24, 1977, requires each Federal agency, in carrying out its activities, to take the following actions:

- Reduce the risk of flood loss, minimize the impact of floods on human safety, health, and welfare, and restore and preserve the natural and beneficial values served by floodplains.
- Evaluate the potential effect of any actions it may take in a floodplain, ensure its planning programs reflect consideration of flood hazards and floodplain management.

These requirements are contained in 23 CFR 650, Subpart A.

4.5.3 Executive Order 11990

Executive Order 11990, May 24, 1977, orders each Federal agency to:

- Take action to minimize the destruction, loss or degradation of wetlands, and to preserve and enhance the natural and beneficial values of wetlands.
- Avoid undertaking or providing assistance for new construction in wetlands unless the head of the agency finds that there is no practicable alternative and all practicable measures are taken to minimize harm which may result from the action.
- To consider factors relevant to the proposal's effects on the survival and quality of the wetlands.

These requirements are contained in 23 CFR 771 (FHPM 7-7-1).
4.6 State Drainage Law

4.6.1 Derivation

State drainage law is derived mainly from two sources: (1) common law and (2) statutory law.

4.6.2 Common Law

Common law is that body of principles which developed from immemorial usage and custom and which receives judicial recognition and sanction through repeated application. These principles were developed without legislative action and are embodied in the decisions of the courts.

4.6.3 Statutory Law

Statutory laws of drainage are enacted by legislatures to enlarge, modify, clarify, or change the common law applicable to particular drainage conditions. This type of law is derived from constitutions, statutes, ordinances, and codes.

4.6.4 Predominance

In general, the common law rules of drainage predominate unless they have been enlarged or superseded by statutory law. In most instances where statutory provisions have been enacted, it is possible to determine the intent of the law. If, however, there is a lack of clarity in the statute, the point in question may have been litigated for clarification. In the absence of either clarity of the statute or litigation, a definitive statement of the law is not possible, although the factors that are likely to be controlling may be indicated.

4.6.5 Classification of Waters

State drainage laws originating from common law, or court-made law, apply different legal rules according to whether the water in the drainage problem is classified as surface water, stream water, flood water, or groundwater. These terms are defined below. Once the classification has been established, the rule that applies to the particular class of water determines responsibilities with respect to disposition of the water.

- Surface Waters - Surface waters are those waters which have been precipitated on the land from the sky or forced to the surface in springs, and which have then spread over the surface of the ground without being collected into a definite body or channel.
Stream Waters - Stream waters are former surface or groundwaters that have entered and now flow in a well-defined natural watercourse, together with other waters reaching the stream by direct precipitation or rising from springs in the bed or banks of the watercourse. A watercourse in the legal sense refers to a definite channel with bed and banks within which water flows either continuously or intermittently.

Flood Waters – Flood waters are former stream waters that have escaped from a watercourse (and its overflow channels) and flow or stand over adjoining lands. They remain floodwaters until they disappear from the surface by infiltration or evaporation, or return to a natural watercourse.

Groundwaters – Groundwaters are divided into two classes, percolating waters and underground streams. The term "percolating waters" generally includes all waters that pass through the ground beneath the surface of the earth without a definite channel. The general rule is that all underground waters are presumed to be percolating unless the existence and course of a permanent channel can be clearly shown. Underground streams are waters passing through the ground beneath the surface in permanent, distinct, well-defined channels.
4.7 State Water Rules

4.7.1 Basic Concepts

Regarding the disposition of surface waters, the courts have developed two major rules: the civil law rule of “natural drainage” and the “common enemy” doctrine. Modification of both rules has tended to bring them somewhat closer together, and in some states, these rules have been replaced by a compromise rule known as the reasonable use rule.

Much of the law regarding stream waters is founded on a common law maxim that states “water runs and ought to run as it is by natural law accustomed to run.” Thus, as a general rule, any interference with the flow of a natural watercourse to the injury or damage of another will result in liability. An interference may involve augmentation, obstruction and detention, or diversion of a stream. However, there are qualifications.

In common law, flood waters are treated as a "common enemy" of all people, lands, and property attacked or threatened by them.

In groundwater law, the "English Rule," which is analogous to the common enemy rule in surface water law, is based on the doctrine of absolute ownership of water beneath the property by the landowner.

Attention is called to the fact that while most states follow basically one or two general laws, i.e., the rule of Roman (civil) law or English common enemy rule, there are many modifications.

4.7.2 Surface Waters

The civil law rule is based upon the perpetuation of natural drainage. The rule places a natural easement or servitude upon the lower land for the drainage of surface water in its natural course and the natural flow of the water cannot be obstructed by the servient owner to the detriment of the dominant owner. Most states following this rule have modified it so that the owner of upper lands has an easement over lower lands for drainage of surface waters and natural drainage conditions can be altered by an upper proprietor provided the water is not sent down in a manner or quantity to do more harm than formerly.

Under the common enemy doctrine, surface water is regarded as a common enemy, which each property owner may fight off or control as he will or is able, either by retention, diversion, repulsion, or altered transmission. Thus, there is not cause of action even if some injury occurs causing damage. In most jurisdictions, this doctrine has been subject to a limitation that one must use his land so as not to unreasonably or unnecessarily damage the property of others. There is such a restriction in Virginia. “Where the common law is in force, as in this State, surface water is considered a common enemy, and the courts agree that each landowner may fight it off as best he
may. He may obstruct or hinder its flow, and may even turn it back upon the land of his neighbor, whence it came.... This right in regard to surface water may not be exercised wantonly, unnecessarily, or carelessly.... It must be a reasonable use of the land for its improvement or better enjoyment, and the right must be exercised in good faith, with no purpose to abridge or interfere with the rights of others, and with such care with respect to the property that may be affected by the use of improvement not to inflict any injury beyond what is necessary." *Norfolk & W. Ry. v. Carter*, 91 Va. 587, 592-93, 22 S.E. 517, 518 (1895.)

Under the reasonable use rule, each property owner can legally make reasonable use of his land, even though the flow of surface waters is altered thereby and causes some harm to others. However, liability attaches when his harmful interference with the flow of surface water is "unreasonable." Whether a landowner's use is unreasonable is determined by a nuisance-type balancing test. The analysis involves several questions.

- Was there reasonable necessity for the actor to alter the drainage to make use of his land?
- Was the alteration done in a reasonable manner?
- Does the utility of the actor's conduct reasonably outweigh the gravity of harm to others?

An exception to the above stated reasonable use rule is that a landowner may not collect surface water by means of an artificial conveyance, i.e., excavated channel, flume, pipes, etc., and discharge it in concentrated form on the property of another. This is true whether or not there has been an increase in the volume, which naturally flowed upon the property.

Another exception to the rule is that a landowner may not obstruct a watercourse to the injury of another.

It is to be noted that in the filling of land for the erection of buildings the landowner may obstruct the flow of water in a depression or swale. However, the court has held that in the construction of a railroad embankment, reasonable construction practice would require the installation of culverts to permit the passage of surface waters. It is believed that construction of a highway embankment would fall in the same category.

It can be seen from the above that while the construction of a highway should include culverts to permit surface waters to pass, it is not mandatory that a property owner provide culverts when filling his land for building purposes. Recognizing the above poses the problem of obtaining easements to guarantee unobstructed outlets for culverts passing surface waters. This is not necessary when the culvert is placed in a watercourse although it may be necessary if improvement of the watercourse is deemed desirable for the convenience of the Department.

When easements are obtained, care must be exercised to avoid a discharge of concentrated flow onto the property of the owner below the one from whom the easement is obtained.
4.7.3 Stream Waters

Much of the law regarding stream waters is founded on a common law maxim that states "water runs and ought to run as it is by natural law accustomed to run." Where natural watercourses are unquestioned in fact and in permanence and stability, there is little difficulty in application of the rule. Highways cross channels on bridges or culverts, usually with some constriction of the width of the channel and obstruction by substructure within the channel, both causing backwater upstream and acceleration of flow downstream. The changes in regime must be so small as to be tolerable by adjoining owners, or there may be liability of any injuries or damages suffered.

Surface waters from highways are often discharged into the most convenient watercourse. The right is unquestioned if those waters were naturally tributary to the watercourse and unchallenged if the watercourse has adequate capacity. However, if all or part of the surface waters have been diverted from another watershed to a small watercourse, any lower owner may complain and recover for ensuing damage. *Norfolk & W. Ry. V. Carter*, 91 Va. 587, 592-93, 22 S.E. 517 (1895.)

4.7.4 Flood Waters

Considering floodwaters as a common enemy permits all affected landowners including owners of highways, to act in any reasonable way to protect themselves and their property from the common enemy. They may obstruct its flow from entering their land, backing or diverting water onto lands of another without penalty, by gravity or pumping, by diverting dikes or ditches, or by any other reasonable means.

Again, the test of "reasonableness" is often applied by many states and liability can result where unnecessary damage is caused. Ordinarily, the highway designer should make provision for overflow in areas where it is foreseeable that it will occur. There is a definite risk of liability if such waters are impounded on an upper owner or, worse yet, are diverted into an area where they would not otherwise have gone. Merely to label waters as "flood waters" does not mean that they can be disregarded.

Virginia recognizes flood waters as a common enemy but does not provide a statutory definition of floodwaters, so they remain in the definition of surface water found in VA. CODE ANN. § 62.1-10 (Michie 1992) ("Water includes all waters, on the surface and under the ground, wholly or partially within or bordering the Commonwealth or within its jurisdiction and which affect the public welfare"). "Flood waters disregard jurisdictional boundaries, and the public interest requires the management of flood-prone areas in a manner which prevents injuries to persons, damage to property and pollution of state waters." VA. CODE ANN. § 10.1-658 (Michie 1993). Virginia follows the reasonable use rule, which could potentially have a limited impact on highway construction. VA. CODE ANN. § 62.1-11 (e) (Michie 1992) ("The right to the use of water or to the flow of water in or from any natural stream, lake or other watercourse in this Commonwealth is and shall be limited to such water as may reasonably be required for the beneficial use
of the public to be served; such right shall not extend to the waste or unreasonable use or unreasonable method of use of such water”). Virginia’s general criteria for drainage, relevant to VDOT, are found in 24 VA. ADMIN. CODE § 30-71-90 (West 1996 & Supp. 1998).

4.7.5 Groundwater

In contrast to groundwater law, surface water law is relatively well defined. The “English Rule,” which is analogous to the common enemy rule in surface water law, is based on the doctrine of absolute ownership of water beneath the property by the landowner. This has been modified by the “Reasonable Use Rule” which states in essence that each landowner is restricted to a reasonable exercise of his own right and a reasonable use of his property in view of the similar right of his neighbors.

The key word is "reasonable." While this may be interpreted somewhat differently from case to case, it can generally be taken to mean that a landowner can utilize subsurface water on his property for the benefit of agriculture, manufacturing, irrigation, etc. pursuant to the reasonable development of his property although such action may interfere with the underground waters of neighboring proprietors. However, it does generally preclude the withdrawal of underground waters for distribution or sale for uses not connected with any beneficial ownership or enjoyment of the land from whence they were taken.

A further interpretation of "reasonable" in relation to highway construction would view the excavation of a deep "cut section" that intercepts or diverts underground water to the detriment of adjacent property owners as unreasonable. There are also cases where highway construction has permitted the introduction of surface contamination into subsurface waters and thus incurred liability for resulting damages.
4.8 Statutory Law

4.8.1 Introduction

The inadequacies of the common law or court-made laws of drainage led to a gradual enlargement and modification of the common law rules by legislative mandate. In the absence of statute, the common law rules adopted by State courts determine surface water drainage rights. If the common law rules have been enlarged or superseded by statutory law, the statute prevails. Statutes affecting drainage are discussed below.

4.8.2 Eminent Domain

In the absence of an existing right, public agencies may acquire the right to discharge highway drainage across adjoining lands with the right of eminent domain. Eminent domain is the power of public agencies to take private property for public use.

The Virginia Constitution grants the State the right of eminent domain, including the development of watercourse and watershed areas. VA. CONST. Art. I & II (1971). It is important to remember; however, that whenever any property is taken under eminent domain, the private landowner must be compensated for his loss. VDOT has the power of eminent domain over watercourses and watershed areas deemed necessary for the construction, maintenance, and repair of public highways. VA. CODE ANN. § 33.2-1001 (Michie 1996).

4.8.3 Water Rights

The water right that attaches to a watercourse is a right to the use of the flow, not ownership of the water itself. This right of use is a property right, entitled to protection to the same extent as other forms of property, and is regarded as real property. After the water has been diverted from the stream flow and reduced to possession, the water itself becomes the personal property of the riparian owner.

- Riparian Doctrine - Under the riparian doctrine, lands contiguous to watercourses have prior claim to waters of the stream solely by reason of location and regardless of the relative productive capacities of riparian and nonriparian lands.

Generally, the important issue for highway designers to keep in mind in the matter of water rights is that proposed work in the vicinity of a stream should not impair either the quality or quantity of flow of any water rights to the stream.
4.9 Easements and Diversion

4.9.1 Outfall Easements

The Department is required to pass surface water coming to a highway embankment through said embankment without undue detriment to adjacent property owners. Private landowners are not always under the same obligation. Therefore, it is sometimes necessary for the Department to secure an easement from its structure to a point downstream from whence actions on adjacent land will no longer penalize the hydraulic performance of the highway facility. This easement also provides access for routine maintenance of the outfall such as the removal of natural vegetation that would reduce the outfall's hydraulic conveyance and thus penalize the highway facility.

Generally, a natural watercourse cannot be restricted to the detriment of adjacent landowners including VDOT. Thus, it is not necessary to obtain an easement along a natural watercourse for protection of the hydraulic conveyance of the system. However, it may be necessary to obtain an easement for construction, maintenance or other reasons.

The Department requires that when other parties construct a facility that ultimately will be taken into the State Highway System, all drainage outfalls be provided with an easement extending from the project’s right-of-way to a natural watercourse. An exception to this requirement is when the local governing body will take perpetual responsibility for the maintenance of the outfall (see Section 14.3.6).

4.9.2 Maintenance of Drainage Easements

The Department should maintain an easement to provide a safe facility for the public and to protect the roadway and its drainage system when Department personnel deem it appropriate and necessary. Generally, there are three types of recorded easements. The first is recorded in the name of the Department and is usually obtained by Department personnel to resolve individual drainage problems, or as a part of a highway improvement project. The second is dedicated to the County for public use as a part of a subdivision developed under County ordinances. The third is an easement obtained by a private party. The Department’s responsibility regarding the three different types of easements is as follows:

- Drainage Easements Acquired by the Department
  - The Department assumes maintenance responsibility within the limits of the drainage easement.
• **Drainage Easements Dedicated to a County as Part of a Subdivision Plat**
  
  o The Department will maintain only that portion of the drainage easement that falls within the right-of-way limits accepted by the Department when the street is added to the State-maintained system of highways.
  
  o Work within the easement, but outside of the right-of-way will only be performed when obstructions, etc., create problems within the right-of-way.

• **Drainage Easements Obtained by Private Parties**
  
  o The Department has no maintenance responsibility. Upon the granting of the drainage application by an appropriate court, the holder of the easement assumes full responsibility. VIRGINIA CODE ANN. § 21-428 (Michie, 1950).
  
  o For additional details on the maintenance of drainage easements, please see the VDOT Maintenance Manual.

### 4.9.3 Construction Easements

The Department may obtain easements as necessary to construct and maintain highway drainage facilities.

### 4.9.4 Diversion

Diversion is the taking of surface water from the path or course that nature prescribed for it to follow and forcing said water to follow another pattern or course. The party causing the diversion has responsibility for the conveyance of the diverted water and the effect of the diverted flow on adjacent land until the flow is returned to its natural course or pattern or until it reaches another body of water. While many civil engineering works cause some diversion, the volume of flow is usually small and its effects are negligible. Nonetheless, the designer should always be cognizant of the maxim: *Aqua Currit Et Debet Currere, Ut Currere Solebat* – *(Water runs and ought to run as it is by natural law accustomed to run).*

### 4.9.5 Flood Storage Easements

It is not the general practice of VDOT to permit the use of highway funds to purchase floodplain storage (floodway) easements. Therefore, VDOT does not generally employ an under-designed drainage structure and purchase an easement upstream of the facility to store the resulting excess ponded water.
4.10 Legal Remedies

4.10.1 Common Actions

The most common legal actions through which a complainant may seek legal recourse include inverse condemnation, injunction, and tort claims.

4.10.2 Inverse Condemnation

Virginia recognizes a cause of action for damage to property caused by surface water drainage by private actors or the state.

4.10.3 Injunctions

Where a statutory right is violated to the landowner's material injury, courts ordinarily grant an injunction. The injunction could enjoin the highway agency from taking a certain action or require the abatement of a certain condition that it has created. The granting of an injunction does not prevent the recoupment of compensation for damages that have occurred (Seventeen, Inc. v. Pilot Life Ins., 215 Va. 74, 205 S.E. 2d 648 (1974)). As a general rule, injunctions may be granted even though the extent of the injury is incapable of being ascertained or of being computed in dollars.

4.10.4 Tort Claims

In the early development of the law, the courts recognized that whenever it was possible, compensation should be awarded to those persons harmed by the actions of another. This was the origin of the theory of tort liability. In essence then, a tort, or civil wrong, is the violation of a personal right guaranteed to the individual by law. A person has committed a tort if he has interfered with another person's safety, liberty, reputation, or private property. If the injured party can prove the defendant proximately caused him harm, the court will hold the defendant responsible for the plaintiff's injury, and the defendant will be forced to pay for the damage.
4.11 Role of the Designer

4.11.1 Responsibility

The designer has a three-fold responsibility for the legal aspects of highway drainage. First, the designer should know the legal principles involved and apply this knowledge to his designs; and, secondly, assist environmental engineers in the acquisition of appropriate permits, and thirdly, he should work closely with the legal staff of his organization, as necessary, in the preparation and trial of drainage cases. The duties of the designer include direct legal involvement in the following areas:

- Conduct investigations, advise, and provide expert testimony on the technical aspects of drainage claims involving existing highways
- Provide drainage design information during permit and right-of-way acquisition
- Assist appraisers in evaluating damages and provide testimony in subsequent condemnation proceedings, when necessary
- The engineer should provide his attorney with a personal resume. In addition, it is advisable to write a brief outline of the facts in the case, as the engineer knows them. The resume should include:
  - Name
  - Address
  - Employer/Position
  - List employer(s) with their addresses
  - List each position held with an employer and note the length of time in that position.
  - List the major duties performed in each position
  - Education
  - List all college level education, special courses, etc.
  - Accreditation
  - List all degrees, licenses, certificates, etc.
  - List membership in professional organizations

Drainage engineers are frequently called upon to present testimony in legal proceedings. In addition to technical knowledge, an accurate knowledge of conditions prior to construction is essential. It is important to maintain complete and accurate documentation for all design studies. Proper documentation as noted elsewhere in this manual will be of inestimable value in recalling the prior existing condition and in developing credible testimony.

It is common knowledge that an engineer can be brought into a lawsuit at almost any time, and there is little to prevent a person from beginning court action. However, if the construction and design of the project is reasonable, then the engineer has no need to fear the outcome. If the engineer can show that he considered all the factors that can reasonably be expected to bear upon a situation and has developed his design accordingly, even though his engineering judgment he did not accommodate some factor(s), he is not liable for negligence. If, however, he does not consider all reasonable and foreseeable factors, he may incur personal liability.
In any discipline, and especially drainage, the law is continually changing and the engineer should keep himself abreast of these changes. (An Engineer Looks at Drainage Law, Alfred R. Pagan, FASCE, Engineering Issues, ASCE.)

4.11.2 Investigation of Complaints

It is imperative that drainage complaints be dealt with promptly and in an unbiased manner. This means accepting the fact that the flooding is a serious problem for the complainant, and not accepting anyone's preconceived conclusions. All facts must be assembled and analyzed before deciding on what happened and why it happened. Also, it is well to list any other agency that could possibly have responsibility for a remedy to the flooding.

When the designer is requested to investigate a complaint, the following guidelines are recommended.

- **Determine Facts About The Complaint:**
  - Show on a map the location of the problem on which the complaint is based
  - Clearly determine the basis for the complaint (what was flooded, complainant's opinion as to what caused the flooding, description of the alleged damages, dates, times and durations of flooding)
  - Briefly relate the history of any other grievances that were expressed prior to the claim presently being investigated
  - Obtain approximate dates that the damaged property and/or improvements were acquired by those claiming damages

- **Collect Facts About the Specific Flood Event(s) Involved:**
  - Rainfall data (dates, amounts, time periods and locations of gages). Rainfall data are often helpful regardless of the source.
  - Document observed high-water information at or in the vicinity of the claim. Locate high-water marks on a map and specify datum. Always try to obtain high-water marks both upstream and downstream of the highway and the time the elevations occurred.
  - Determine the duration of flooding at the site of alleged damage. Determine the direction of flood flow at the damaged site. Describe the condition of the stream before, after, and during flood(s). Was the growth in the channel light, medium, heavy; were there drift jams; does the stream carry much drift in flood stage; was the flow fast or sluggish; did light, moderate, or severe erosion occur?
  - Document the flood history at the site. Was the highway overtopped by the flood? If so, what was the depth of overtopping; and, if possible, estimate a flow velocity across the highway. Obtain narratives of any eyewitnesses to the flooding. Obtain facts about the flood(s) from sources outside VDOT, such as newspaper accounts, witnesses, measurements by other agencies (USGS, Corps of Engineers, NRCS, and individuals), maps, and Weather Bureau rainfall records.
• **State Facts About the Highway Crossing Involved**
  - Show a profile of the highway across the stream valley. Give the date of the original highway construction and dates of all subsequent alterations to the highway, and describe what the alterations were. Describe what existed prior to the highway, such as county road, city street, or abandoned railroad embankment, etc. Also include a description of the drainage facilities and drainage patterns that were there prior to the highway. Give a description of the existing drainage facilities. Give the original drainage design criteria, or give capacity and frequency of the existing facility based upon current criteria.

• **Possible Effects by Others**
  - Are there any other stream crossings in the vicinity of the damaged site that could have affected the flooding (pipelines, highways, streets, railroads, dams)?
  - Have there been any significant man-made changes to the stream or watershed that might affect the flooding?

• **Analyze The Facts**
  - From the facts, decide what should be done to relieve the problem regardless of who has responsibility for the remedy.
  - Could others possibly provide assistance?

• **Make Conclusions and Recommendations.**
  - What were the contributing factors leading to the alleged flood damage?
  - Specify feasible remedies (This should be done without any regard for who has responsibility to effect a remedy.)

The list under “Determine Facts About the Complaint” is not all-inclusive, nor is it intended that the entire list will be applied in each case. This outline is given as a guide to the type and scope of information desired from an investigation of a drainage complaint. It is advantageous to have available hydraulic design documentation as outlined in the “Documentation” chapter of this manual. When the report is completed, the designer should again analyze the facts, consider the conclusions and recommendations, and prepare a response to the complainant explaining the results of the investigation. Documentation of the facts and findings is important in the event there is future action.

**4.11.3 Legal Opinion**

Drainage matters range from the simple to the complicated. If the facts are ascertained and a plan developed before initiating a proposed improvement, the likelihood of an injury to a landowner is remote and the project construction or developer should be able to undertake such improvements relatively assured of no legal complications.
If the designer needs a legal opinion on a particular drainage problem or improvement, the requested opinion should state as a minimum whether:

- The watercourse under study has been viewed
- There are problems involved, and what causes them (obstructions, topography, development - present and future)
- The proposed improvements will make the situation better
- The proposal requires that the natural drainage be modified
- There is potential liability for doing something versus doing nothing
- Someone will benefit from the proposed improvements
- In general, what is proposed is "reasonable"

4.11.4 As a Witness

The designer should accept the responsibility of providing expert testimony in highway drainage litigation. Witness duty ordinarily requires considerably more time of a witness than the time spent in the courtroom. The best use of the designer's time can be arranged by consulting with legal counsel to determine what types of information and data will be needed, types of presentation needed, and when testimony will be required.

Testimony often involves presenting technical facts in layman's language so that it will be clearly understood by those in the courtroom. The designer's testimony generally describes the highway drainage system involved in the alleged injury or damage, and how that system affects the complainant. Design considerations and evidence of conditions existing prior to construction of the highway are important points.

4.11.5 Witness Conduct

The designer who is to serve as a witness should bear one fact in mind; the purpose of the court is to administer justice. Testimony should have one purpose - to bring out all known facts relevant to the case so that justice can better be served. Following are some pointers in being a witness.

- Tell the truth and do not try to color, shade, or change your testimony to help either side.
- Never lose your temper or show prejudice in favor of one side that is not supported by facts.
- Do not be afraid of lawyers and give your information honestly.
- Speak clearly and loudly enough to be heard by everyone involved in the courtroom proceeding.
- If you do not understand a question, ask that it be explained. If you still do not understand what is being asked, explain that you cannot give an answer to that question.
• Answer all questions directly and never volunteer information the question does not ask for.
• Stick to the facts and what you personally know.
• Do not be apprehensive. Your purpose is to present the facts as you know them and that is all that will be expected.
• If you do not know the answer to a question, just admit it. It is to your credit to be honest, rather than try to have an answer for everything that is asked you.
• Do not try to memorize your story. There is no more certain way to cross yourself than to memorize your story and try to fit this story with the questions being asked.

Work with your lawyer in preparing your testimony and stick to the facts as you know them.
4.12 References


U.S. Army Corps of Engineers. 1987. Handbook of How to Compute a Floodway. (Copies of this publication can be obtained from - FEMA Region V, 175 West Jackson Blvd., Fourth Floor, Chicago Illinois 60604.)


# Chapter 5 - Planning and Location

## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1 Introduction</td>
<td>5-1</td>
</tr>
<tr>
<td>5.1.1 Stormwater Management Plan</td>
<td>5-1</td>
</tr>
<tr>
<td>5.1.2 Flood Hazards</td>
<td>5-1</td>
</tr>
<tr>
<td>5.1.3 Construction Problems</td>
<td>5-2</td>
</tr>
<tr>
<td>5.1.4 Maintenance Problems</td>
<td>5-2</td>
</tr>
<tr>
<td>5.2 Policy</td>
<td>5-3</td>
</tr>
<tr>
<td>5.2.1 Interagency Coordination</td>
<td>5-3</td>
</tr>
<tr>
<td>5.2.2 Intragency Coordination</td>
<td>5-3</td>
</tr>
<tr>
<td>5.2.3 Legal Aspects</td>
<td>5-3</td>
</tr>
<tr>
<td>5.2.4 Environmental Considerations</td>
<td>5-4</td>
</tr>
<tr>
<td>5.2.5 Permits</td>
<td>5-4</td>
</tr>
<tr>
<td>5.2.6 Location Considerations</td>
<td>5-5</td>
</tr>
<tr>
<td>5.3 Stormwater Management</td>
<td>5-6</td>
</tr>
<tr>
<td>5.3.1 Introduction</td>
<td>5-6</td>
</tr>
<tr>
<td>5.3.2 Quality</td>
<td>5-6</td>
</tr>
<tr>
<td>5.3.3 Quantity</td>
<td>5-7</td>
</tr>
<tr>
<td>5.4 Preliminary Data Gathering</td>
<td>5-8</td>
</tr>
<tr>
<td>5.4.1 Drainage Surveys</td>
<td>5-8</td>
</tr>
<tr>
<td>5.4.2 Data Collection</td>
<td>5-8</td>
</tr>
<tr>
<td>5.4.3 Type of Data</td>
<td>5-8</td>
</tr>
<tr>
<td>5.4.4 Topographic</td>
<td>5-9</td>
</tr>
<tr>
<td>5.4.5 Channel Characteristics</td>
<td>5-9</td>
</tr>
<tr>
<td>5.4.6 Hydrologic Data</td>
<td>5-10</td>
</tr>
<tr>
<td>5.4.7 Basin Characteristics</td>
<td>5-10</td>
</tr>
<tr>
<td>5.4.8 Precipitation</td>
<td>5-10</td>
</tr>
<tr>
<td>5.4.9 Flood Data</td>
<td>5-11</td>
</tr>
<tr>
<td>5.4.10 Highwater Information</td>
<td>5-11</td>
</tr>
<tr>
<td>5.4.11 Existing Structures</td>
<td>5-11</td>
</tr>
<tr>
<td>5.4.12 Environmental Data</td>
<td>5-12</td>
</tr>
<tr>
<td>5.4.13 Fish and Wildlife</td>
<td>5-12</td>
</tr>
<tr>
<td>5.4.14 Vegetation</td>
<td>5-12</td>
</tr>
<tr>
<td>5.4.15 Water Quality</td>
<td>5-13</td>
</tr>
<tr>
<td>5.4.16 Sinkholes</td>
<td>5-13</td>
</tr>
<tr>
<td>5.5 Preliminary Hydraulic Reports</td>
<td>5-15</td>
</tr>
<tr>
<td>5.5.1 Introduction</td>
<td>5-15</td>
</tr>
<tr>
<td>5.5.2 Report</td>
<td>5-15</td>
</tr>
<tr>
<td>5.5.2.1 Sinkholes</td>
<td>5-15</td>
</tr>
<tr>
<td>5.6 References</td>
<td>5-19</td>
</tr>
</tbody>
</table>
List of Appendices

Appendix 5A-01 EQ-120 Sinkhole Inventory Form
Appendix 5B-01 Drainage Information Sheet
Chapter 5 - Planning and Location

5.1 Introduction

5.1.1 Stormwater Management Plan

The Department often is and should be perceived as a developer of transportation facilities that have the potential to stimulate secondary activity along the transportation corridor, just as a major residential development can stimulate commercial activity. Secondary activity is a local/regional planning function that must address overall SWM needs in conjunction with other utilities such as water, wastewater, and power. Because the transportation corridor often traverses several watersheds, the development of an adequate SWM Plan can be severely fragmented and significant problems created if there is a lack of coordinated planning among concerned parties.

To be truly effective, a SWM Plan should consider the total scope of development (i.e., transportation, residential, commercial, industrial, and agricultural). Department coordination with responsible local agencies is essential to ensure that proposed facilities are compatible with the long-term needs of the area. VDOT can provide important information to local agencies wishing to develop a comprehensive SWM Plan without assuming responsibility for the planning and decision-making process for the entire watershed.

Prior to design, a level of planning should be undertaken that will properly locate facilities and adequately address local concerns, permitting requirements, legal consideration, and potential problem categories. This chapter provides general guidelines and major considerations for evaluating these factors during the planning and location process. The important point to emphasize is that the designer should become involved in the early stages of project development and not wait until the later design stages.

5.1.2 Flood Hazards

Floodflow characteristics at a highway stream crossing should be carefully analyzed to determine their effect upon the highway as well as to evaluate the effects of the highway upon the floodflow. Such an evaluation can assist in determining those locations at which construction and maintenance will be unusually expensive or hazardous. Thus it is important to identify the flood hazards prior to any highway involvement to determine if the flood hazard will be increased, decreased, or be the same with and without the proposed highway improvement. Flood hazards should include effects to private property both upstream and downstream (i.e., overtopping floodwaters diverted onto previously unaffected property).

* Rev. 7/16
Although satisfactory solutions often can be obtained by making only minor changes in selected routes to take advantage of better natural hydraulic features at alternate sites, troublesome and uncertain conditions are sometimes best avoided altogether.

### 5.1.3 Construction Problems

Many serious construction problems arise because important drainage and water-related factors were overlooked or neglected in the planning and location phases of the project. With proper planning, many problems can be avoided or cost effective solutions developed to prevent extended damages. Such problems include:

- Soil erosion
- Sediment deposition
- Landslides
- Timing of project stages
- Protection for fish habitat
- Protection of existing facilities (i.e. pipes, ditches/channels, etc.) and continued use during construction
- Contamination of pumping and distribution facilities
- Protection of streams, lakes, rivers, and reservoirs
- Protection of wetlands

Analysis of available data, proper scheduling of work, early field reconnaissance, and other aspects involved in the early planning and location studies can alleviate many problems encountered in the construction of drainage facilities.

### 5.1.4 Maintenance Problems

Planning and location studies should consider potential erosion and sedimentation problems upon completion of highway construction. If a particular location will require frequent and expensive maintenance due to drainage, alternate locations should be considered unless the potentially high maintenance costs can be reduced by special design. Experience in the area is the best indicator of maintenance problems and interviews with maintenance personnel could be extremely helpful in identifying potential drainage problems. Reference to highway maintenance and flood reports, damage surveys, newspaper clippings, and interviews with local residents could be helpful in evaluating potential maintenance problems.

The construction of channel changes, minor drainage modifications, and revisions in irrigation systems usually carry the assumption of certain maintenance responsibilities. Potential damage from the erosion and degradation of stream channels and problems caused by ice and debris can be of considerable significance from the maintenance standpoint.
5.2 Policy

5.2.1 Interagency Coordination

Coordination between concerned agencies during the project planning phase will help produce a design that is most satisfactory to all. Substantial cost savings and other benefits frequently can be realized for highway and water resource projects through coordinated planning among the Federal, State, and local agencies that are engaged in water-related activities (such as flood control and water resources planning). Interagency cooperation is an essential element for serving the public interests.

5.2.2 Intragency Coordination

Early planning and location studies should be coordinated within VDOT so that duplication of effort is minimized and all those who might be involved in future project work will be informed of any ongoing studies and study results.

5.2.3 Legal Aspects

Detailed legal aspects related to drainage are discussed in the Legal Chapter. Additionally, the following generalizations given in Chapter V of the Highway Drainage Guidelines by AASHTO (2007) and Chapter 2 of the AASHTO Model Drainage Manual (2014) should be considered.

- A goal in highway drainage design should be to perpetuate natural drainage, insofar as practicable.
- The courts look with disfavor upon infliction of damage that could reasonably have been avoided, even where some alteration in flow is legally permissible.
- The basic laws relating to the liability of governmental entities are undergoing radical change, with a trend toward increased governmental liability.
- Drainage laws are also undergoing change, with the result that older and more specific standards are being replaced by more flexible standards that tend to depend on the circumstances of the particular case.

In water law matters, designers should recognize that the State is generally held to a higher standard than a private citizen. This is true even though the State should be granted the same rights and liabilities, since no law says differently. In general, designers should not address a question of law without the aid of legal counsel. Whenever drainage problems are known to exist or can be identified, drainage and flood easements or other means of avoiding future litigation should be considered, especially in locations where a problem could be caused or aggravated by the construction of a highway.

Rev. 1/17
It is often helpful in the planning and location phase of a project to document the history and present status of existing conditions or problems, and supplement the record by photographs, videotapes, and written descriptions of field conditions.

Such thoroughness is essential, because VDOT may be blamed for flooding or erosion damage caused by conditions that existed prior to highway construction.

5.2.4 Environmental Considerations

For all projects, some level of environmental study should be performed. The environmental studies should comply with all Federal, State, and local laws and regulations related to environmental quality and should identify all environmental impacts of the project both positive and negative. If the project under study requires a Federal action, then the NEPA rules relating to environmental studies must be followed.

It is important to document the environmental considerations for the proposed project including any alternatives that will receive consideration. Encroachments onto adjacent areas (including environmental encroachments) should be avoided whenever possible. Identifying environmental considerations early in the planning process can prevent major implementation problems as the design and construction of the project proceeds.

Environmental considerations should be listed in the Environmental Scoping Report prepared during project scoping.

5.2.5 Permits

Specific Federal, State, and local permits that will be needed for a highway project must be identified in the environmental document early in the planning stages.

Prior to initiating design work, the designer must review the environmental document with the Environmental Division to identify regulatory commitments, constraints, and any permits required. Permits, as required, should be obtained before construction begins, and preferably before detailed plans are prepared.
5.2.6 Location Considerations

The principal factors to be considered in locating a stream crossing that involves encroachment within a floodplain are:

- River type (straight or meandering)
- River characteristics (stable or unstable)
- River geometry and alignment
- Hydrology
- Hydraulics
- Floodplain flow
- Needs of the area
- Economic and environmental concerns
- Navigable waterway and recreational use

A detailed evaluation of these factors is part of the location hydraulics study. When a suitable crossing location has been selected, specific crossing components can then be determined. When necessary, these include:

- The geometry and length of the approaches to the crossing
- Probable type and approximate location of the abutments
- Probable number and approximate location of the piers
- Estimated depth to the footing supporting the piers (to protect against local scour)
- The location of the longitudinal encroachment in the floodplain
- The amount of allowable longitudinal encroachment into the main channel
- The required river training works to ensure that river flows approach the crossing or the encroachment in a complementary way

Exact information on these components is usually not developed until the final stage of design.
5.3 **Stormwater Management**

5.3.1 **Introduction**

Planning for drainage and SWM facilities should include a consideration of the potential problems associated with stormwater quality and quantity.

5.3.2 **Quality**

Several broad categories of degradation have been developed to delineate or describe levels of stormwater impacts:

- **Aesthetic deterioration**: Undesirable general appearance features (dirty, turbid, or cloudy) and actual physical features (odors, floating debris, oil films, scum, or slime) are present.
- **Dissolved oxygen depletion**: When the oxygen demand of bacteria is stimulated by the organics, the subsequent reduction in oxygen levels can disturb the balance between lower forms and the food chain. Unoxidized nitrogen compounds (ammonia) can also cause problems.
- **Pathogen concentrations**: High concentrations of several pathogens can reduce the acceptable uses of the receiving waters.
- **Suspended solids**: The physical buildup of solids can cover productive bottoms, be aesthetically objectionable, and disrupt flow and navigation.
- **Nutrients**: Accelerated eutrophication that stimulates growth of aquatic vegetation can cause a water body to become aesthetically objectionable, deplete dissolved oxygen, and decrease recreational value by creating odor and overgrowth. Advanced eutrophication can lead to sediment buildup, which reduces storage capabilities.
- **Toxicity**: The two types of toxics generally found in stormwater (metals and pesticides/persistent organics) may build up in sensitive areas over the long term. At high levels, they can have serious shock effects on aquatic life. Low levels can become significant by accumulation up the food chain.
- **Hazardous spills**: Depending on the characteristics of the spill, serious water quality problems can result.
- **Total Maximum Daily Load (TMDL) Reports**: TMDL coverage for the larger watershed are provided on VDOT Cedar, and similar EPA maps. TMDL reports are often available for the larger streams located in Virginia.

Quantification of the levels of contaminants that are being washed off a roadway is complicated by the variable effects of and the periods between storm events. The contributory factors are rainfall intensity, street surface characteristics, and particle size. The varying interaction of these factors makes it difficult to precisely estimate the impact that discharge will have on water quality.

* Rev. 7/16

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Chapter 5-6 of 19
In general, erosion and sediment transport should be limited by developing and implementing an **Erosion and Sediment Control Plan** which addresses both temporary and permanent control practices.

### 5.3.3 Quantity

Determinations of stormwater quantity are primarily useful for evaluating and mitigating the flooding and erosion impacts of a project. Without stormwater quantity management, land development can increase peak runoff rates and volumes from storm events, which can lead to higher flood elevations. Appropriate hydrologic and hydraulic calculations presented in various chapters of this manual should be made to determine the required conveyance through the **R/W** and to aid in mitigating impacts to downstream property owners.

Procedures contained in this manual should be used to evaluate the ability of a facility to accomplish the following controls for a particular area.

- Reduce runoff rates by increasing infiltration, and by storing precipitation and runoff where it falls and releasing it slowly.
- Protect areas subject to flood damages by keeping runoff confined to drainage facilities such as pipes or channels and by building appropriate flood control facilities.
- Keep floodplain encroachment outside the limits of regulated floodways.

The following questions should be considered when selecting the plan for disposal of stormwater runoff.

- Are existing drainage systems large enough to handle anticipated runoff?
- Are design discharges consistent with adopted drainage plans and regulatory criteria?
- Will the project require retention or detention storage areas to mitigate the impacts of increased runoff, or can the increase be handled by other project features?
- Is there sufficient area to construct a retention or detention pond within the **R/W**? Are alternative sites available for storage of stormwater? Is property available outside the **R/W**? Does the project schedule allow time to acquire additional **R/W**?
- Are there unusual groundwater or soil conditions? Is there a high groundwater table, or are there impermeable soil layers, rock or karst topography?
- Are there any jurisdictional, permit, or economic restrictions?
- Are there any unusual site conditions, such as woods, wetlands, water supply reservoirs, live streams, or other environmental features that might influence the development of a **SWM** system?

*Rev. 7/16*
5.4 Preliminary Data Gathering

5.4.1 Drainage Surveys

Since hydraulic considerations can influence the selection of a highway corridor and the alternate routes within the corridor, the type and amount of data needed for planning studies can vary widely depending on such elements as environmental considerations, class of the proposed highway, state of land-use development, and individual site conditions.

Topographic maps, aerial photographs, and streamflow records provide helpful preliminary drainage data, but historical highwater elevations and flood discharges are of particular interest in establishing waterway requirements. Comprehensive hydraulic investigations may be required when route selection involves important hydraulic features such as water-supply wells and reservoirs, flood-control dams, water resource projects, and encroachment on floodplains of major streams. Special studies and investigations, including consideration of the environmental and ecological impact, should be commensurate with the importance and magnitude of the project and the complexity of the problems encountered.

5.4.2 Data Collection

As part of planning and location studies several categories of data should be obtained and evaluated, including:

- Physical characteristics of drainage basins
- Maps and topographic data including channel surveys and cross sections
- Runoff quantity data (hydrologic and precipitation data)
- Channel and floodplain delineations and related studies
- Flood history and problem inventory
- Existing SWM* facility characteristics
- Development of alternative plan concepts
- Hydrologic and hydraulic analysis of alternative concepts
- Consideration of multipurpose opportunities and constraints
- Benefit/cost analysis and evaluation
- Runoff quality data
- Evidence of sinkholes typically found in karst terrain

5.4.3 Type of Data

Following is a brief description of the types of data needed for planning and location studies.

* Rev. 7/16
5.4.4 Topographic

Topographic data should be acquired at most sites requiring hydraulic studies. These data are needed so that analysis of existing flow conditions as well as those caused by various design alternatives may be performed. Significant physical and cultural features in the vicinity of the project should be located and documented in order to obtain their elevation. Such features as residences, commercial buildings, schools, churches, farmlands, other roadways and bridges, and utilities can affect, as well as be affected by the design of any new hydraulic structures. Often, recent topographic surveys will not be available at this early stage of project development. Aerial photographs, photogrammetric maps, USGS quadrangle sheets, and even old highway plans may be utilized during the planning and location phases. When better survey data becomes available, usually during the design phase, these early estimates will need to be revised to correspond with the most recent field information.

5.4.5 Channel Characteristics

In order to perform an accurate hydraulic analysis, the stream profile, horizontal alignment, and cross sections should be obtained. Data to this detail usually are not available during the planning and location phases. The designer must therefore make preliminary analyses based on data such as aerial photographs, USGS maps, and old plans.

One method that can be useful in determining channel characteristics such as material in the stream beds and banks, type and coverage of vegetal material, and evidence of drift, debris, or ice, is the taking of photographs and videotapes. Field visits made early in the project life can include the photographing of the channel, upstream and downstream, and the adjoining floodplain. The photos can be valuable aids, especially when taken in color, for not only preliminary studies, but also for documentation of existing conditions.

During these early phases of project development, the designer should be involved in determining the detail of field survey required at the site. This should include the upstream and downstream limits of the survey, the number of or distance between cross sections, and how far to either side of the channel the sections should extend. The number of cross sections needed will vary with the study requirements and the particular stream characteristics. For some projects, the accuracy achieved by aerial photogrammetry will be sufficient for the level of hydraulic study needed, while other sites will require a different level of accuracy. The level of accuracy of survey required should be a consideration when determining the degree of hydraulic analysis needed. The U.S. Army Corps of Engineer Hydrologic Engineering Center has made a detailed study of survey requirements. The results of this study are available in “Accuracy of Computer Water Surface Profiles” by M. W. Burnham and D. W. Davis, Technical Paper No. 114, 1986.
5.4.6 Hydrologic Data

Information required by the designer for analysis and design include not only the physical characteristics of the land and channel, but all the features which can affect the magnitude and frequency of the flood flow which will pass the site under study. These data may include climatological characteristics, land runoff characteristics, stream gaging records, highwater marks, and the sizes and past performances of existing structures in the vicinity. The exact data required will depend upon the methods utilized to estimate flood discharges, frequencies, and stages. It should be noted that much of the hydrologic data will not be used during the planning and location phase. However, it is important to determine the need for the data now though, because it will take time to collect and evaluate such data. By starting this process during planning and location, delays during the design stage should be minimized.

5.4.7 Basin Characteristics

The hydrologic characteristics of the basin or watershed of the stream under study are needed for any predictive methods used to forecast flood flows. Although many of these characteristics can be found from office studies, some are better found by a field survey of the basin. The size and configuration of the watershed, the geometry of the stream network, storage volumes of ponds, lakes, reservoirs, and floodplains, and the general geology and soils of the basin can all be found from maps. Land use and vegetal cover may be determined from maps, but with rapidly changing land uses, a more accurate survey will probably be achieved from aerial photographs and field visits.

Having determined these basin characteristics, runoff times, infiltration values, storage values, and runoff coefficients can be found and used in calculating flood flow values.

5.4.8 Precipitation

A precipitation survey normally consists of the collection of rainfall records for the rainfall stations in the vicinity of the study site. Unlike the survey of stream flow records or basin characteristics however, rainfall records from outside the watershed, can be utilized. These records will hopefully contain several years of events, for every month, season, and will include duration values for various frequency rainstorms. Snowfall accumulations may also be available and are often helpful.

If rainfall records are lacking, the National Oceanic and Atmospheric Administration (Weather Bureau) has publications available which give general rainfall amounts for various duration storms which can be used. Weather Bureau Technical Paper 40, though now out of print, is useful for this information.
5.4.9 Flood Data

The collection of flood data is a basic survey task in performing any hydraulic analysis. These data can be collected both in the office and in the field. The office acquisition includes the collection of past flood records, stream gaging records, FEMA maps, and newspaper accounts. The field collection will consist mainly of interviews with residents, maintenance personnel, and local officials who may have recollections or photos of past flood events in the area.

If a stream gaging station is on the stream under study, close to the crossing site, and has many years of measurements, this may be the only hydrologic data needed in some cases. These data should be analyzed to ensure stream flows have not changed over the time of measurement due to the watershed alteration, such as the construction of a large storage facility, diversion of flow to another watershed or addition of flow from another watershed, or development that has significantly altered the runoff characteristics of the watershed.

5.4.10 Highwater Information

Sometimes highwater marks are the only data of past floods available. When collected, these data should include the date and elevation of the flood event when possible. The cause of the highwater mark should also be noted. Often the mark is caused by unusual debris or an ice jam rather than an inadequate structure and designing roadway or structure grades to such an elevation could lead to an unrealistic, uneconomical design.

Highwater marks can be identified in several ways. Small debris, such as grass or twigs caught in tree branches, hay or crops matted down, mud lines on buildings or bridges, are all highwater indicators. Beware however that grass, bushes, and tree branches bend over during flood flows and spring up after the flow has passed, which may give a false reading of the high water elevation. Ice will often cut or gouge into the bark of trees indicating highwater elevations.

5.4.11 Existing Structures

The size, location, type, and condition of existing drainage structures on the stream under study can be a valuable indicator when selecting the size and type for any new structure. Data to be obtained on existing structures includes such things as size, type, age, existing flow line elevation, and condition, particularly in regards to the channel. Scour holes, erosion around the abutments or just upstream or downstream, or abrupt changes in material gradation or type can all indicate a structure too small for the site. With a knowledge of flood history, the age and overall substructure condition may also aid in determining if the structure is too small.
If a structure is relatively new, information may still be available on the previous one, and why it had to be replaced. Although, normally, crossings are replaced due to poor structural conditions, sometimes other underlying conditions, often hydraulic in nature, also enter into the decision to build a new structure. Also, the durability of the existing structure may indicate how well the proposed structure will fare at this location. Old plans may also contain highwater or flood information which can be of use. When structures upstream or downstream of the site under study exist, they should always be inventoried for the factors just discussed. This includes highway and railroad structures, as well as any private crossings which might exist.

5.4.12 Environmental Data

In order to make a study of the water resources of the area, an environmental team should obtain those data commensurate with the needs to evaluate the highway impacts on the surface water. A coordination meeting with representatives of the various environmental disciplines concerned is often beneficial at this stage.

Data may need to be collected on such things as fish and wildlife, vegetation, and the quality of the water. A judgment may need to be made on aesthetic values.

5.4.13 Fish and Wildlife

There are many sources of information available from which information on fish and wildlife can be gathered. Biologists can provide much data on types of animals and fish, their spawning seasons, and critical areas. Maps may also be available showing this information. Local residents and field visits can yield information not found elsewhere.

5.4.14 Vegetation

The types and extent of vegetal cover can affect the rate of runoff and its quantity. It may also affect the quality of the water. There are three primary sources from which information on vegetation may be found.

- Maps - Geological maps show, in general terms, where the land is covered and where it is clear. Often, particularly during the preliminary stages of a study, this may be sufficient. Later on, more data may be needed such as the type of cover. Is it agricultural crop land or pasture, or evergreen forest?

- Aerial Photographs - An experienced person can distinguish the various types of vegetation from aerial photographs, and should photos in color or infrared be available, the categorizing of different types can be even easier. Aerial photos must be up-to-date, of good quality, and to scale to be of any real value however.
Field Visit - It may not be possible to survey the entire watershed, so a sample area may have to be studied. It is important to set out the exact field needs before the trip is made to ensure all information needed is collected and all important areas visited. Please note that careful attention should be given to areas of the project area conditions that may have been altered after the field survey is performed and before design has commenced.

5.4.15 Water Quality

Water quality data can be the most expensive and most time consuming information to collect. Sometimes water quality records are available at or near the site under study but even then, the information most often required for highway studies may not have been gathered. Sample collection is expensive because of the equipment and laboratory facilities needed, and the time required.

Sample collection can be time consuming because one sample or several taken at the same time is not usually satisfactory. Water quality can reflect seasonal, monthly, or even daily variations depending on the weather, flow rate, traffic, etc. Therefore, a sampling program should be extended for a year, if at all possible.

Water quality data collection and analysis must be conducted by an experienced person trained in this area. This may be someone within VDOT who has been trained in this field or it may be necessary to retain an outside firm to perform this portion of the environmental analysis.

Existence of NPDES monitoring stations should be investigated.

5.4.16 Sinkholes

Sinkholes are found in areas of karst terrain. Karst terrain is generally formed over limestone and dolomite formations. Karst terrains primarily occur within the Valley and Ridge Physiographic Province of western Virginia. Karst type terrains are also known to occur in very limited areas of the Blue Ridge, Piedmont and Coastal Plain Physiographic Provinces of Virginia. While information contained in these guidelines is directed more to those sinkholes located in the Valley and Ridge Physiographic Province, the same considerations should be applied to sinkholes located in other areas of the state.

* Rev. 7/16
Karst terrain is characterized by closed depressions (sinkholes), caves, and underground drainage resulting from the solutions of the calcium and/or magnesium carbonates. Sinkholes may develop either by solution of the surficial rocks or collapse of underlying caves. The actual rock cavity may or may not be choked by residual soil and debris. It is the potential instability of the sinkhole infilling, most often associated with changes in the local hydrology, which traditionally has been the concern of the construction industry. Those concerns have now broadened to include the potential impacts of construction on the area’s hydrology and water quality.

The presence of sinkholes should be noted on Form PM-100 (LD-430), Scoping Report and, if possible, the approximate location of observed sinkholes should be identified. The project survey shall provide an accurate and detailed location and description of all identifiable sinkholes located within the survey boundaries.

Both the EPA and VDOT have concerns with changes to the existing hydrology at sinkhole locations. These concerns include:

- **Water Quality** – Sinkholes are often direct links to underground sources of drinking water. Stormwater runoff from highways could potentially contain various constituents such as oil, grease, heavy metals and salt that could enter and impact these water supplies. The underground ecosystems could potentially be impacted by highway runoff containing sediment generated both during and following highway construction and material from potential spills resulting from traffic accidents once the highway is operational.

- **Water Quantity** – Directing additional stormwater flow to a sinkhole can result in the enlargement of the feature, create surface failures and erosion and cause flooding of adjacent property. Increasing the quantity of stormwater runoff flowing to a sinkhole can also cause the characteristics of the sinkhole opening to change in such a manner so as to restrict the flow into the subsurface, resulting in greater surface ponding in and around the area of the sinkhole.

- **Instability** – The area within and surrounding a sinkhole can settle or sink unexpectedly, resulting in loss of competent structural material and damage to overlying structures.

* Rev. 7/16
5.5 Preliminary Hydraulic Reports

5.5.1 Introduction

Preliminary hydraulic reports should be as complete as possible but must be tailored to satisfy the requirements of the specific location and size of project for which the study is required. Too much data and information is uneconomical and bulky to reduce to meaningful information. Coordination with all sections requiring survey data before the initial field work is begun will help ensure the acquisition of sufficient, but not excessive survey data.

5.5.2 Report

All data considered and used in reaching conclusions and recommendations made during the preliminary study should be included in a report. This should include hydrologic and hydraulic data, pertinent field information, photographs, calculations, and structure sizes and location. At this stage of the study, several structure sizes and types can usually be given as the designer only needs generalities in order to obtain a rough estimate of needs and costs. Often, specifics cannot be provided until an accurate topographic survey of the area has been made and precise hydraulic computations performed. Sometimes however, the report will require detailed design studies in order to justify the extent of mitigation required. In general, the more environmentally sensitive sites and those in highly urbanized areas will necessitate more detail at earlier stages. Useful Department documentation for reporting results include the Scoping Worksheet – Hydraulics, and the Scoping Worksheet - Environmental.

5.5.2.1 Sinkholes

The following design considerations must be followed for any projects involving the construction of highways or drainage outfalls in areas where sinkholes are present:

- **Avoidance** – Determine if there are any feasible alternatives that would avoid construction in the area of the sinkhole. Where the sinkhole is the natural outfall for the stormwater runoff from the roadway area, determine if the stormwater runoff can be diverted away from the sinkhole to an adequate surface water channel. It should be recognized that drainage facilities to accommodate the diversion of stormwater runoff may require significant additional grading and right of way. In addition, stormwater quantity management facilities may be required at the point where the diverted flow is released from the project R/W in order to avoid the liabilities inherent with stormwater runoff diversion.

* Rev.7/16
Minimization of Impacts from Direct Discharges – If avoidance is not possible, drainage outfalls from the roadway should include natural buffer zones between the outlet of the roadway drainage structure and the sinkhole in order to provide for a natural filtering process. Where stormwater runoff naturally terminates in sinkhole areas, vegetated flow areas (minimum 80' – 100' in length), runoff spreaders and vegetated swales should be used between the outlet of the roadway drainage structure and the bottom of the sinkhole in order to provide for filtering of the flow. If concentrated flow from the roadway pavement area is being directed into the bottom of the sinkhole, a stormwater management water quality basin or other type of water quality filtering device should be incorporated into the design. The water quality basin or filtering device should not be located in the bottom (throat) of the sinkhole (where the flow enters the ground) but rather should be located as close to the roadway or discharge point as practicable. Stormwater management basins constructed in these areas may require an impermeable lining in order to prevent impacts to the underlining soil and subsurface area. The District Materials Section should provide recommendations regarding this issue. A stormwater management basin may also be needed to provide attenuation of any increased flow quantity that may be directed toward the sinkhole.

If stormwater runoff from a roadway project must be directed to a sinkhole, the area of the sinkhole should be investigated to determine if any existing ponding occurs during rainfall events. The drainage design for the project should reflect how the sinkhole is anticipated to function after completion of the construction activities. The project should be designed to avoid any flood damages resulting from potential blockage and ponding in the sinkhole area.

If direct discharge of runoff into a sinkhole is the only feasible option available and improvements (modifications) such as cleaning, clearing, etc. are needed in the lowest section of the sinkhole (where water enters the ground), the details of such improvements (modifications) must be discussed with the District Environmental Section in order that they can determine what permits and/or reporting will be required. Typical sinkhole improvements (modifications) that would fit into this category are depicted in Detail 1 and Detail 2 on Standard Insertable Sheet No. isd/msd 2944. These “improved” sinkhole sites are be brought to the attention of the District Environmental Section early in project development process in order to allow adequate time for coordination with the EPA and other applicable regulatory agencies. The Environmental Division’s Form EQ-120 (Appendix 5A-01) must be completed for those sites where it is determined necessary to “improve” a sinkhole and where it is determined such improvements would be regulated under the EPA’s UIC Program. The Hydraulics Engineer shall be responsible for completing Form EQ-120 and submitting it to the District Environmental Hazardous Materials Manager for further processing.

Rev. 7/16
The Hydraulics Engineer should coordinate with the District Materials Section and the District Environmental Section if the project Scoping Report or survey data indicates the presence of sinkholes and if it is anticipated that those sinkholes might be impacted by stormwater runoff from the project.

In areas of karst topography, roadside ditches with a gradient of less than 5% may need to be lined to inhibit the infiltration of surface waters. The District Materials Section should make this determination during the preliminary soils investigation phase of the project and, where applicable, include their recommendations for ditch lining with those other recommendations requested on Form LD-252 - Request for Supporting Data. Where ditch lining is recommended, the roadside ditches should be lined with concrete using Standard PG-2A or PG-5 (as applicable) or similar details. When using Standard PG-2A or PG-5 concrete ditches in these areas, the standard detail drawings will need to be modified to include the following:

- Add a 30-mil polyethylene film beneath all joints (to extend 4 feet longitudinally in each direction).

- Show the location of the curtain wall (normally placed adjacent to each expansion joint) 4’ downgrade of the expansion joint (to coincide with the end of the 30-mil polyethylene film).

- In areas where these modifications apply, the plan description should note “St’d. PG-2A Modified” or “St’d. PG-5 Modified”, as applicable. The details for these modifications are included on the Sinkhole Insertable Sheet.

Where the roadway traverses over or through a sinkhole area, the sinkhole should be treated in accordance with one of the typical details shown on Standard Insertable Sheet No. isd/msd 2944 unless otherwise directed by the District Materials Engineer:

- Detail No. 1 should be used for sinkholes that receive stormwater runoff from relatively large areas and have a well-defined opening (throat). This treatment involves cleaning out soil and debris to expose the throat, installing a length of pipe to convey surface drainage into the sinkhole and backfilling with riprap and successive layers of smaller aggregate and a geotextile fabric prior to the placement of the regular roadway embankment material.

* Rev. 7/16
• Detail No. 2 should be used for sinkholes with broad, flat depressions and which have no defined throat. These sinkholes typically receive stormwater runoff from relatively small areas. The width of the roadway embankment is generally less than the width of the depression. This treatment involves the placement of riprap in the bottom of the roadway embankment to allow for the continued infiltration of surface flows. The riprap is capped with successive layers of smaller aggregate and a geotextile fabric before placement of the regular roadway embankment material.

• Detail No. 3 should be used for small shallow sinkholes that receive stormwater runoff from relatively small areas and where the roadway embankment will cover most or all of the depression. This treatment involves filling the depression with successive layers of smaller aggregate and a geotextile fabric before placement of the regular roadway embankment material. Since this treatment effectively “caps” the sinkhole and precludes the entry of surface water, a drainage ditch or other hydraulic conveyance is typically required along the edge of the roadway embankment to convey stormwater runoff to an adjacent outfall.

These Guidelines shall apply to roadways that are designed and constructed by others and which will ultimately be maintained by VDOT. In addition, where a sinkhole is being utilized as a drainage outfall, an acceptable legal agreement shall be executed that absolves VDOT of any liability and maintenance responsibilities associated with the sinkhole. The agreement should identify the County as the responsible party in the event that the developer or homeowners association cannot (or will not) assume the responsibility for liability or maintenance. A sample legal agreement can be found in Secondary Roads Division’s publication “GUIDE FOR ADDITIONS, ABANDONMENTS, AND DISCONTINUANCES – SECONDARY SYSTEM OF STATE HIGHWAYS”. The sample agreement shown in this publication is for stormwater management facilities but it can be modified slightly to cover the use of a sinkhole as a drainage outfall. The development of the agreement for the use of a sinkhole as an outfall should be coordinated with and approved by the Local Assistance Division in the Central Office.

Rev. 7/16
5.6 References


* Rev. 1/17
# Chapter 5 - Planning and Location

## Appendix 5A-01

**VDOT INVENTORY OF CLASS V INJECTION WELLS FOR STORMWATER DISCHARGES TO IMPROVED SINKHOLES**

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### 3. IMPROVED SINKHOLE INFORMATION

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### 6. This form to be submitted to the VDOT REGIONAL HAZARDOUS MATERIALS CONTACT:

**NAME:** ____________

**TELEPHONE:** ____________

### 7. Notes:

1. Identify other property owner, if applicable:

2. Describe Treatment Options here and attach drawing:

Provide additional information and attachments (photos, plans, etc.), as needed.
DEPARTMENT OF TRANSPORTATION  
LOCATION AND DESIGN  
DRAINAGE INFORMATION SHEET

UPC: ______  Date: _____  
Route #: ______  
County: ______  

Scheduled Advertisement Date:______  
Contract Administrator:______

Project #: _____  Type of Facility:______  
Limits: ______  Type of Financing:______  
From: _____  Project Length: _____  
To: ______  State Forces or Contract: _____

Description of work: _____

Geometrics:  

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[Total Estimated Disturbed Acreage: _____]

Are there existing Bridges or Live Streams?_____  
Are there sections to be realigned?______  
Are there areas where the grade will be changed?______  
Are there utilities within project limits?______

What is the overall condition of existing Drainage Structures?______  
Are there existing Erosion or Siltation Problems?______  
Is there a history of flooding problems?______

Are Temporary detours required within project limits during Construction?______

Page 1 of 2
(Please provide the following information for Drainage structures with 36” or larger openings – Existing or Proposed.)

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Additional Sheets may be added if necessary.
# Chapter 6 – Hydrology

**TABLE OF CONTENTS**

<table>
<thead>
<tr>
<th>Section</th>
<th>Topic</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.1</td>
<td><strong>Introduction</strong></td>
<td>6-1</td>
</tr>
<tr>
<td></td>
<td>6.1.1 Objective</td>
<td>6-1</td>
</tr>
<tr>
<td></td>
<td>6.1.2 Definition</td>
<td>6-1</td>
</tr>
<tr>
<td></td>
<td>6.1.3 Factors Affecting Floods</td>
<td>6-1</td>
</tr>
<tr>
<td></td>
<td>6.1.4 Sources of Information</td>
<td>6-2</td>
</tr>
<tr>
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<td><strong>Design Policy</strong></td>
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</tr>
<tr>
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<td>6-2</td>
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</tr>
<tr>
<td></td>
<td>6.2.3 Flood Hazards</td>
<td>6-2</td>
</tr>
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<td>6-2</td>
</tr>
<tr>
<td></td>
<td>6.2.5 Documentation</td>
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</tr>
<tr>
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<td>6.2.6 Evaluation of Runoff Factors</td>
<td>6-3</td>
</tr>
<tr>
<td></td>
<td>6.2.7 Flood History</td>
<td>6-3</td>
</tr>
<tr>
<td></td>
<td>6.2.8 Hydrologic Methods</td>
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</tr>
<tr>
<td></td>
<td>6.2.9 Approved Peak Discharge Methods</td>
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<td>6.2.11 Economics</td>
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</tr>
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Chapter 6-i of 3
6.4.3.2.1 Introduction ................................................................. 6-17
6.4.3.2.2 Application ................................................................. 6-18
6.4.3.2.3 Characteristics ............................................................. 6-18
6.4.3.2.4 Equations ................................................................. 6-18

6.4.3.3 Rural Regression Method ................................................................. 6-20
6.4.3.3.1 Introduction ................................................................. 6-20
6.4.3.3.2 Application ................................................................. 6-20
6.4.3.3.3 Hydrologic Regions ................................................................. 6-21
6.4.3.3.4 Equations ................................................................. 6-21
6.4.3.3.5 Characteristics ............................................................. 6-21
6.4.3.3.6 Mixed Population ............................................................. 6-25

6.4.3.4 Urban Regression Method ................................................................. 6-25
6.4.3.4.1 Introduction ................................................................. 6-25
6.4.3.4.2 Application ................................................................. 6-25
6.4.3.4.3 Characteristics ............................................................. 6-25
6.4.3.4.4 Equations ................................................................. 6-25

6.4.3.5 Stream Gage Data ................................................................. 6-26
6.4.3.5.1 Introduction ................................................................. 6-26
6.4.3.5.2 Application ................................................................. 6-26
6.4.3.5.3 Transposition of Data ................................................................. 6-27

6.4.4 Hydrograph Methods ................................................................. 6-27
6.4.4.1 Modified Rational Method ................................................................. 6-27
6.4.4.1.1 Introduction ................................................................. 6-27
6.4.4.1.2 Application ................................................................. 6-27
6.4.4.1.3 Characteristics ............................................................. 6-28
6.4.4.1.4 Critical Storm Duration ................................................................. 6-28
6.4.4.1.5 Estimating the Critical Duration Storm ................................................................. 6-29

6.4.4.2 NRCS Methods (Graphical Peak Discharge and Unit Hydrograph) ................................................................. 6-29
6.4.4.2.1 Introduction ................................................................. 6-29
6.4.4.2.2 Application ................................................................. 6-30
6.4.4.2.3 Characteristics ............................................................. 6-30
6.4.4.2.4 Time of Concentration ................................................................. 6-30
6.4.4.2.5 Curve Numbers ............................................................. 6-30
6.4.4.2.6 Equations ................................................................. 6-31

6.5 References ..................................................................................... 6-33
List of Tables
Table 6-1. Design Storm Selection Guidelines ............................................................... 6-6
Table 6-2. Saturation Factors for Rational Formula ...................................................... 6-14
Table 6-3. Anderson Time Lag Computation ............................................................... 6-19
Table 6-4. Anderson Method Flood Frequency Ratios ................................................. 6-20
Table 6-5. Regional Regression Equations for Estimating Peak Discharges of Streams in Virginia ................................................................. 6-23
Table 6-6. Transposition of Data Sample Problem ....................................................... 6-27

List of Figures
Figure 6-1. Guidelines for Peak Discharge Method Selection ..................................... 6-7
Figure 6-2. Guidelines for Runoff Volume Method Selection ....................................... 6-7
Figure 6-3. Peak Discharge Regions for Regression Equations ................................. 6-22

List of Appendices
Appendix 6B-1 Runoff Depth for Runoff Curve Number (RCN)
Appendix 6B-2 24-hr Rainfall Depths
Appendix 6C-1 B, D, and E Factors – Application
Appendix 6C-2 B, D, and E Factors for Virginia
Appendix 6D-1 Overland Flow Time – Seelye
Appendix 6D-2 Kinematic Wave Formulation – Overland Flow
Appendix 6D-3 Overland Time of Flow
Appendix 6D-4 Overland Flow Velocity
Appendix 6D-5 Time of Concentration for Small Drainage Basins (use for channel flow) – Kirpich
Appendix 6D-6 Average Velocities for Estimating Travel Time for Shallow Concentrated Flow
Appendix 6E-1 Rational Method Runoff Coefficients
Appendix 6E-2 Rational Method Runoff Coefficients with 10-yr Cf Factor Applied
Appendix 6E-3 Rational Method Runoff Coefficients with 25-yr Cf Factor Applied
Appendix 6E-4 Rational Method Runoff Coefficients with 50-yr Cf Factor Applied
Appendix 6E-5 Rational Method Runoff Coefficients with 100-yr Cf Factor Applied
Appendix 6J-1 Major Drainage Basins
Appendix 6K-1 A and B Factors that define Intensity Duration – Frequency (IDF) Curves for use only with the Critical Storm Duration Determination
Appendix 6K-2 Regression Constants > “a” and “b” for Virginia
Chapter 6 - Hydrology

6.1 Introduction

6.1.1 Objective

The analysis of precipitation, peak rate of runoff, volume of runoff, and time distribution of flow is fundamental to the design of drainage facilities. Errors in the estimates will result in a structure that is either undersized and causes drainage problems or oversized and costs more than necessary. On the other hand, it must be realized that any hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex, and too little data are available on the factors influencing the rural and urban rainfall-runoff relationship to expect exact solutions.

6.1.2 Definition

Hydrology is generally defined as a science dealing with water on and under the earth and in the atmosphere. For the purpose of this manual, hydrology will deal with estimating stormwater runoff as the result of rainfall. In design of highway drainage structures, stormwater runoff is usually considered in terms of peak runoff or discharge in cubic feet per second (cfs) and hydrographs as discharge versus time. For structures which are designed to control the volume of runoff, like detention storage facilities, then the entire inflow and outflow hydrographs will be of interest. Wetland hydrology, the water-related driving force to create wetlands, is addressed in the AASHTO Highway Drainage Guidelines, Chapter 10 and the AASHTO Drainage Manual, Chapter 8.

6.1.3 Factors Affecting Floods

In the hydrologic analysis for a drainage structure, it must be recognized that there are many variable factors that affect floods. Some of the factors which need to be recognized and considered on an individual site-by-site basis are things such as:

- Rainfall amount and storm distribution
- Drainage area size, shape, and orientation
- Ground cover
- Type of soil
- Slopes of terrain and stream(s)
- Antecedent moisture condition
- Storage potential (overbank, ponds, wetlands, reservoirs, channels, etc.)
- Watershed development potential
- Type of precipitation (rain, snow, hail, or combinations thereof)

* Rev. 7/19
6.1.4 Sources of Information

The type and source of information available for hydrologic analysis will vary from site to site and it is the responsibility of the designer to determine what information is needed and applicable to a particular analysis.

6.2 Design Policy

6.2.1 Introduction

The following sections summarize the policies which should be followed for hydrologic analysis for VDOT roadways. For a more detailed discussion refer to the publications AASHTO Highway Drainage Guidelines (2007), AASHTO Drainage Manual Volumes 1 and 2 (2014), and FHWA HDS-2 Highway Hydrology (2002).

6.2.2 Surveys

Hydrologic considerations can significantly influence the selection of a highway corridor and the alternate routes within the corridor. Therefore, studies and investigations should consider the environmental and ecological impact of the project. Also special studies and investigations may be required at sensitive locations. The magnitude and complexity of these studies should be commensurate with the importance and magnitude of the project and problems encountered. Typical data to be included in such surveys or studies are: topographic maps, aerial photographs, streamflow records, historical high water elevations, flood discharges, and locations of hydraulic features such as reservoirs, water projects, wetlands, karst topography and designated or regulatory floodplain areas.

6.2.3 Flood Hazards

A hydrologic analysis is prerequisite to identifying flood hazard areas and determining those locations at which construction and maintenance will be unusually expensive or hazardous.

6.2.4 Coordination

Since many levels of government plan, design, and construct highway and water resource projects which might have effects on each other, interagency coordination is desirable and often necessary. In addition, agencies can share data and experiences within project areas to assist in the completion of accurate hydrologic analyses.

* Rev. 7/19
6.2.5 Documentation

Experience indicates that the design of highway drainage facilities should be adequately documented. Frequently, it is necessary to refer to plans and specifications long after the actual construction has been completed. Thus it is necessary to fully document the results of all hydrologic analysis.

6.2.6 Evaluation of Runoff Factors

For all hydrologic analyses, the following factors should be evaluated and included when they will have a significant effect on the final results:

- Drainage basin characteristics including: size, shape, slope, land use, geology, soil type, surface infiltration, and storage
- Stream channel characteristics including: geometry and configuration, slope, hydraulic resistance, natural and artificial controls, channel modification, aggradation, degradation, and ice and debris
- Floodplain characteristics
- Meteorological characteristics such as precipitation amount and type (rain, snow, hail, or combinations thereof), rainfall intensity and pattern, areal distribution of rainfall over the basin, and duration of the storm event

6.2.7 Flood History

All hydrologic analyses should consider the flood history of the area and the effects of these historical floods on existing and proposed structures. The flood history should include the historical floods and the flood history of any existing structures.

6.2.8 Hydrologic Methods

Many hydrologic methods are available. If possible, the selected method should be calibrated to local conditions and verified for accuracy and reliability.

There is no single method for determining peak discharge that is applicable to all watersheds. It is the designer’s responsibility to examine all methods that can apply to a particular site and to make the decision as to which is the most appropriate. Consequently, the designer must be familiar with the method sources of the various methods and their applications and limitations. It is not the intent of this manual to serve as a comprehensive text for the various methods of determining peak discharge.

Deleted Information

* Rev. 7/19
6.2.9 Approved Peak Discharge Methods

In addition to the methods presented in this manual, the following methods are acceptable when appropriately used:

- Log Pearson III analyses of a suitable set of gage data may be used for all routine designs provided there is at least 10 years of continuous or synthesized flow records for 10-yr discharge estimates and 25 years for 100-yr discharge estimates.
- Suitable computer programs such as the USACE’s HEC-HMS and the NRCS’ EFH-2, WinTR-55, and WinTR-20 may be used for the hydrologic calculations. The TR-55 method (now referred to as the NRCS Method and formerly as the SCS Method) has been found best suited for drainage areas between 200 and 2,000 acres (ac). When using any methodology predicated on the 24-hr. rainfall event (i.e., NRCS Method, HEC-HMS, etc.) it is necessary to use the values presented in the NOAA Atlas 14 Point Precipitation Frequency Estimates or published in the Chapter 11, Appendices.
- Other methods may be approved where applicable upon submission to the VDOT State Hydraulics Engineer.
- The 100-yr discharges specified in the FEMA flood insurance study are preferred when the analysis includes a proposed crossing on a regulatory floodway. However, if these discharges are deemed to be outdated or incorrect, the discharges based on current methodology should be used.

6.2.10 Design Frequency

A design frequency should be selected commensurate with the facility cost, amount of traffic, potential flood hazard to property, expected level of service, political considerations, and budgetary constraints as well as the magnitude and risk associated with damages from larger flood events. When long highway routes that have no practical detour are subject to independent flood events, it may be necessary to increase the design frequency at each site to avoid frequent route interruptions from floods. In selecting a design frequency, potential upstream land uses should be considered which could reasonably occur over the anticipated life of the drainage facility.

6.2.11 Economics

Hydrologic analysis should include the determination of several design flood frequencies for use in the hydraulic design. Section 6.3.1 outlines the design floods that shall be used for different drainage facilities. These frequencies are used to size drainage facilities for an optimum design, which considers both risk of damage and construction cost. Consideration should also be given to the frequency flood that was used to design other structures along a highway corridor.

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* Rev. 7/19
6.2.12  Review Frequency

All proposed structures designed to accommodate the selected design frequency should also be evaluated using a base flood to ensure that there are no unexpected flood hazards.

6.3  Design Criteria

6.3.1  Design Frequency

6.3.1.1  Factors Governing Frequency Selections

The determination of design factors to be considered and the degree of documentation required depends upon the individual structure and site characteristics. The hydraulic design must be such that risk to traffic, potential property damage, and failure from floods is consistent with good engineering practice and economics. Recognizing that floods cannot be precisely predicted and that it is seldom economically feasible to design for the very rare flood, all designs should be reviewed for the extent of probable damage, should the design flood be exceeded. Design headwater/backwater and flood frequency criteria should be based upon these and other considerations:

- Damage to adjacent property
- Damage to the structure and roadway
- Traffic interruption
- Hazard to human life
- Damage to stream and floodplain environment
- Impact to Base Flood (100 year flood) elevations

The potential damage to adjacent property or inconvenience to owners should be a major concern in the design of all hydraulic structures.

Inundation of the traveled way indicates the level of traffic service provided by the facility. The traveled way overtopping flood level identifies the limit of serviceability. Table 6-1 relates desired minimum levels of protection from traveled way (edge of shoulder) inundation to the functional classifications of roadways. The design storm discussed here refers to roadway crossing (bridge or culvert) or roadways running parallel to streams. Other features such as storm sewer elements, roadside ditches, E&S, and SWM facilities will have specific design storms and rainfalls discussed in their respective Chapters.
6.3.1.2 Minimum Criteria

No exact criteria for flood frequency or allowable backwater/headwater values can be set which will apply to an entire project or roadway classification. Minimum design frequency values relative to protection of the roadway from flooding or damage have been established. It should be emphasized that these values only apply to the level of protection afforded to the roadway.

Table 6-1. Design Storm Selection Guidelines
(For Traveled Way Inundation)

<table>
<thead>
<tr>
<th>Roadway Classification</th>
<th>Exceedance Probability</th>
<th>Return Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interstate, Freeways (Urban/Rural)</td>
<td>2%</td>
<td>50-year</td>
</tr>
<tr>
<td>Principal Arterial</td>
<td>2%</td>
<td>50-year</td>
</tr>
<tr>
<td>Urban Minor Arterial System</td>
<td>2%</td>
<td>50-year</td>
</tr>
<tr>
<td>Rural Minor Arterial System</td>
<td>4%</td>
<td>25-year</td>
</tr>
<tr>
<td>Rural Collector System, Major</td>
<td>4%</td>
<td>25-year</td>
</tr>
<tr>
<td>Rural Collector System, Minor</td>
<td>10%</td>
<td>10-year</td>
</tr>
<tr>
<td>Urban Collector System</td>
<td>10%</td>
<td>10-year</td>
</tr>
<tr>
<td>Local Street System</td>
<td>10%</td>
<td>10-year</td>
</tr>
</tbody>
</table>

Source: AASHTO Drainage Manual (First Edition), Volume One, Chapter 9, Table 9-1

Note: Federal law requires Interstate highways to be provided with protection from the 2% flood. Facilities such as underpasses and depressed roadways, where no overflow relief is available, shall also be designed for the 2% event. Where no embankment overflow relief is available, drainage structures should be designed for at least the 1% or 100-year event.

* Rev. 7/19
### 6.3.2 Peak Discharge Method Selection

The methods to be used are shown in Figure 6-1. For watersheds greater than 200 ac, VDOT recommends evaluating several hydrologic methods for comparison purposes.

<table>
<thead>
<tr>
<th>HYDROLOGIC METHOD</th>
<th>DRAINAGE AREA SIZE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0 to 200 acres</td>
</tr>
<tr>
<td>Rational Method</td>
<td></td>
</tr>
<tr>
<td>NRCS Graphical Peak Discharge Method</td>
<td></td>
</tr>
<tr>
<td>NRCS EFH-2</td>
<td></td>
</tr>
<tr>
<td>Anderson Method (USGS)*</td>
<td></td>
</tr>
<tr>
<td>USGS Regional Regression - Rural</td>
<td></td>
</tr>
<tr>
<td>USGS Regional Regression - Urban</td>
<td></td>
</tr>
<tr>
<td>Stream Gage Data</td>
<td></td>
</tr>
</tbody>
</table>

*For VDOT purposes, the Anderson Method is not recommended for use outside of urbanized areas in the Northern Virginia District.

Note: The above does not indicate definite limits but does suggest a range in which the particular method is “best suited”.

**Figure 6-1. Guidelines for Peak Discharge Method Selection**

### 6.3.3 Runoff Volume Method Selection (Hydrograph Methods)

The hydrograph methods to be used for estimating runoff volume include the following:

<table>
<thead>
<tr>
<th>HYDROLOGIC METHOD</th>
<th>DRAINAGE AREA SIZE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0 to 20 acres</td>
</tr>
<tr>
<td>Modified Rational Method</td>
<td></td>
</tr>
<tr>
<td>NRCS Unit Hydrograph Method</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Range of Applicability</td>
</tr>
</tbody>
</table>

Note: The above does not indicate definite limits but does suggest a range in which the particular method is “best suited”.

**Figure 6-2. Guidelines for Runoff Volume Method Selection**

For application of the technical criteria in the Virginia Stormwater Management Program (VSMP), the Department of Environmental Quality prefers use of the NRCS Unit Hydrograph Method for estimating runoff volume. See Chapter 11 Stormwater Management in this Drainage Manual for more discussion. However, the VSMP Regulation does allow use of the Modified Rational Formula as a hydrologic method for estimating runoff volume (see 9 VAC 25-870-72 E).

* Rev. 7/19
6.4 Design Concepts

6.4.1 Travel Time Estimation

Travel time \((T_t)\) is the time it takes water to travel from one location to another in a watershed. \(T_t\) is a component of time of concentration \((t_c)\), which is the time for runoff to travel from the most hydraulically distant point in the watershed to a point of interest within the watershed. The time of concentration is computed by summing all the travel times for consecutive components of the drainage conveyance system.

The computation of travel time and time of concentration is discussed below.

6.4.1.1 Travel Time

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, pipe flow, or some combination of these. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

\[
T_t = \frac{L}{3600V}
\]  

(6.1)

Where:

- \(T_t\) = Travel time, hour (hr)
- \(L\) = Flow length, feet (ft)
- \(V\) = Average velocity, feet per second (fps)
- 3600 = Conversion factor from seconds to hours

6.4.1.2 Time of Concentration

The time of concentration \((t_c)\) is the sum of \(T_t\) values for the various consecutive flow segments. Separate flow segments should be computed for overland flow, shallow concentrated flow, channelized flow, and pipe systems.

\[
t_c = T_{t1} + T_{t2} + \cdots T_{tm}
\]  

(6.2)

Where:

- \(t_c\) = Time of concentration, hours (hrs)
- \(m\) = Number of flow segments

Time of concentration is an important variable in most hydrologic methods. Several methods are available for estimating \(t_c\). This chapter presents several methods for estimating overland flow and channel flow times. Any method used should only be used with the parameters given for the specific method. The calculated time should represent a reasonable flow velocity.

For additional information concerning time of concentration as used in the Rational Method, see Section 6.4.4.1.
6.4.1.3  Travel Time in Lakes or Reservoirs

Sometimes it is necessary to compute a $t_c$ for a watershed having a relatively large body of water in the flow path. In such cases, $t_c$ is computed to the upstream end of the lake or reservoir, and for the body of water the travel time is computed using the equation:

$$V_w = (gD_m)^{0.5}$$  \hspace{1cm} (6.3)'

Where:

- $V_w =$ Wave velocity across the water, feet per second (fps)
- $g =$ Acceleration due to gravity = 32.2 ft/s$^2$
- $D_m =$ Mean depth of lake or reservoir, feet (ft)

Generally, $V_w$ will be high (8 - 30 fps). Note that the above equation only provides for estimating travel time across the lake and for the inflow hydrograph to the lake's outlet. It does not account for the travel time involved with the passage of the inflow hydrograph through spillway storage and the reservoir or lake outlet. This time is added to the travel time across the lake. The travel time through lake storage and its outlet can be determined by the storage routing procedures in Chapter 11. The wave velocity Equation 6.3 can be used for swamps with much open water, but where the vegetation or debris is relatively thick (less than about 25% open water), Manning's equation is more appropriate.

6.4.2  Design Frequency

6.4.2.1  Overview

Since it is not economically feasible to design a structure for the maximum runoff a watershed is capable of producing, a design frequency must be established.

The frequency with which a given flood can be expected to occur is the reciprocal of the probability, or the chance that the flood will be equaled or exceeded in a given year. If a flood has a 20% chance of being equaled or exceeded each year, over a long period of time, the flood will be equaled or exceeded on an average of once every five years. This is called the return period or recurrence interval (RI). Thus the exceedance probability (percentage) equals 100/RI.

6.4.2.2  Base Flow

Base flow (as opposed to Base Flood which is the 100 year flood event) is the typical discharge that is found within the stream for at least 25% of the typical year. An evaluation of the gage data throughout the state has determined that this flow in cubic feet per second (cfs) is approximately equal to 1.1 times the drainage area in square miles. This is typically used to aid in the design of causeways and coffer dams.

* From Chapter 15, Part 630, Section 630.1503 of the National Engineering Handbook
However, where stream gage data is available, the base flow could also be estimated from the historic stream gage data instead of using the approximation method.

### 6.4.2.3 Design Frequency

**Roadway Stream Crossings:** A drainage facility should be designed to accommodate a discharge with a given return period(s). The design should ensure that the backwater (the headwater) caused by the structure for the design storm does not:

- Increase the flood hazard significantly for property
- Exceed a certain depth on the highway embankment

Based on these design criteria, a design involving roadway overtopping for floods larger than the design event is an acceptable practice. Factors to consider when determining whether roadway overtopping is acceptable are roadway classification, roadway use, impacts and frequency of overtopping, structural integrity, etc. If a culvert or bridge is designed to pass the 25-year flow, it would not be uncommon for a larger event storm to overtop the roadway.

**Storm Drains:** A storm drain should be designed to accommodate a discharge with a given return period(s). The design should be such that the storm runoff does not:

- Increase the flood hazard significantly for property
- Encroach onto the street or highway so as to cause a significant traffic hazard
- Limit traffic, emerging vehicle, or pedestrian movement to an unreasonable extent

Based on these design criteria, a design involving roadway inundation for floods larger than the design event is an acceptable practice. Factors to consider when determining whether roadway inundation is acceptable are roadway classification, roadway use, impacts and frequency of inundation, structural integrity, etc.

### 6.4.2.4 Review Flood

After sizing a drainage facility, it will be necessary to review this proposed facility with a higher discharge. This is done to ensure that there are no unexpected flood hazards inherent in the proposed facilities. The review flood is usually the base flood and in some cases, a flood event larger than the base flood is used for analysis to ensure the safety of the drainage structure and nearby development.

### 6.4.2.5 Rainfall vs. Flood Frequency

Drainage structures are designed based on some flood frequency. However, certain hydrologic procedures use rainfall and rainfall frequency as the basic input. Thus it is commonly assumed that the 10-yr rainfall will produce the 10-yr flood.
6.4.2.6  Intensity-Duration-Frequency (IDF) Values

Rainfall data are available for many geographic areas. From these data, rainfall intensity-duration-frequency (IDF) values can be developed for the commonly used design frequencies using the B, D, & E factors described in Appendix 6C-1 and tabulated in Appendix 6C-2. They are available for mostly every county and major city in the state, and broken down by their respective NOAA Atlas 14 stations. The B, D, & E factors were derived by the Department using the Rainfall Precipitation Frequency data provided by NOAA’s Atlas 14.

6.4.2.7  Discharge Determination

Estimating peak discharges of various recurrence intervals is one of the most common engineering challenges faced by drainage facility designers. The task can be divided into two general categories:

- Gaged sites - the site is at or near a gaging station and the streamflow record is of sufficient length to be used to provide estimates of peak discharges. A complete record is defined as one having at least 25 years of continuous or synthesized data.
- Ungaged sites - the site is not near a gaging station and no streamflow record is available. This situation is very common and is normal for small drainage areas.

This chapter will address hydrologic procedures that can be used for both categories.

6.4.3  Peak Discharge Methods

6.4.3.1  Rational Method

6.4.3.1.1  Introduction

The Rational Method is recommended for estimating the design storm peak runoff for areas as large as 200 ac. In low-lying tidewater areas where the terrain is flat, the Rational Method can be considered for areas up to 300 ac. Considerable engineering judgment is required to reflect representative hydrologic characteristics, site conditions, and a reasonable time of concentration (tc). Its widespread use in the engineering community represents its acceptance as a standard of care in engineering design.

* Rev. 7/19
6.4.3.1.2 Application

When applying the Rational Method (and other hydrologic methods), the following items should be considered:

- It is important to obtain a good topographic map and define the boundaries of the drainage area in question. A field inspection of the area should also be made to verify the drainage divides and to determine if the natural drainage divides have been altered.

- In determining the runoff coefficient C-value for the drainage area, the designer should use a comprehensive land use plan for predicting future discharges.

- Restrictions to the natural flow such as SWM facilities and dams that exist in the drainage area should be investigated to see how they affect the design flows. Only facilities that are designed with the purpose to detain water should be considered.

- Charts, graphs, and tables included in this chapter are not intended to replace reasonable and prudent engineering judgment in the design process.

- The Department considers the Rational Method as the primary approach to hydrologic calculations for the design of closed drainage pipe systems, ditches, channels, culverts, inlets, gutter flow, and any other drainage conveyances other than SWM facilities. Please note that hydrologic methods pertaining to SWM facilities shall follow the guidance as provided in Chapter 11 of this Manual, whereby it is encouraged that the designer employ the use of TR-55/TR-20 methods, involving hydrologic soil group classification and implementation.

Deleted Information

6.4.3.1.3 Characteristics

Characteristics of the Rational Method which generally limit its use to 200 acres include:

1. The rate of runoff resulting from any rainfall intensity is a maximum when the rainfall intensity lasts as long as or longer than the time of concentration. That is, the entire drainage area does not contribute to the peak discharge until the time of concentration has elapsed.

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

2. The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration.

* Rev. 7/19
Frequencies of peak discharges depend on rainfall frequencies, antecedent moisture conditions in the watershed, and the response characteristics of the drainage system. For small and largely impervious areas, rainfall frequency is the dominant factor. For larger drainage basins, the response characteristics control. For drainage areas with few impervious surfaces (less urban development), antecedent moisture conditions usually govern, especially for rainfall events with a return period of 10 years or less.

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

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

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

Modern drainage practice often includes detention of urban storm runoff to reduce the peak rate of runoff downstream. When a hydrograph is needed for a small drainage area, the Modified Rational Method is normally used. (See Section 6.4.4.1)

### 6.4.3.1.4 Equations

The Rational Method formula 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 hydraulically remote point of the basin to the point of study).

The Rational Method Formula is expressed as follows:

\[ Q = C_f C_i A \]  

(6.4)

Where:

\[ Q \] = Maximum rate of runoff, cubic feet per second (cfs)
\[ C_f \] = Saturation factor
\[ C \] = Runoff coefficient representing a ratio of runoff to rainfall (dimensionless)
\[ i \] = Average rainfall intensity for a duration equal to the time of concentration for a selected return period, inches per hour (in/hr)
\[ A \] = Drainage area contributing to the point of study, acres (ac)

* Rev. 7/19
Note that conversion to consistent units is not required as 1 acre-inch per hour approximately equals 1 cubic foot/second.

6.4.3.1.5 Infrequent Storm
The coefficients given in Appendix 6E-1 are for storms with less than a 10-year recurrence interval. Less frequent, higher intensity storms will require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright-McLaughlin 1969). The adjustment of the Rational Method for use with larger storms can be made by multiplying the right side of the Rational Formula by a saturation factor, \( C_f \). The product of \( C_f \) and \( C \) should not exceed 1.0. Table 6-2 lists the saturation factors for the Rational Method.

<table>
<thead>
<tr>
<th>Recurrence Interval (Years)</th>
<th>( C_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2, 5, and 10</td>
<td>1.0</td>
</tr>
<tr>
<td>25</td>
<td>1.1</td>
</tr>
<tr>
<td>50</td>
<td>1.2</td>
</tr>
<tr>
<td>100</td>
<td>1.25</td>
</tr>
</tbody>
</table>

Note: \( C_f \) multiplied by \( C \) should not exceed 1.0

6.4.3.1.6 Time of Concentration
The time of concentration is the time required for water to flow from the hydraulically most remote point in the drainage area to the point of study. Use of the rational formula requires the time of concentration (\( t_c \)) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (\( i \)) by using the B, D, & E factors in the procedure described in Appendix 6C-1. A table showing the B, D, & E factors for rain gages across Virginia is presented in Appendix 6C-2.

Time of concentration (\( t_c \)) for most drainage areas less than about 200 ac will normally be comprised of overland flow (OLF), channel flow or concentrated flow (CF), and conveyance flow in manmade structures. For very small drainage areas such as those draining to drop inlets, the flow time may only consist of overland flow. For very large drainage areas, the overland flow time may not be significant and not be measurable, depending on the scale of the map depicting the drainage area. Overland flow should be limited to about 200’, with a maximum flow length no greater than 300’.

Overland Flow
Seelye Method
VDOT experience has determined that the “Overland Flow Time” nomograph developed by E.E. Seelye normally provides a realistic estimate of overland flow (OLF) time when

Rev. 7/19
properly applied within the limits shown on the nomograph. Refer to Appendix 6D-1 for the Seelye chart.

**Kinematic Wave Method**

The Kinematic Wave Formulation provides an approximation of the rising side of the overland flow hydrograph. The formula is given as:

\[
 t_t = 0.93 \frac{L^{0.6} n^{0.6}}{i^{0.4} S_o^{0.3}}
\]

(6.5)

Where:

\[
\begin{align*}
 t_t &= \text{Travel time (hr)} \\
 L &= \text{Length of strip feet (ft)} \\
 n &= \text{Manning’s roughness coefficient} \\
 i &= \text{Rainfall intensity (determined iteratively), inches per hour (in/hr)} \\
 S_o &= \text{Slope, feet/foot (ft/ft)}
\end{align*}
\]

The determination of the appropriate rainfall intensity with the aid of the Kinematic Wave nomograph (Appendix 6D-2) is an iterative process. Two variables, rainfall intensity and time of concentration, appear in the nomograph and neither are known at the beginning of the computation. Thus, as a first step, a rainfall intensity must be assumed, which is then used in the nomograph to compute a time of concentration. Although this gives a correct solution of the equation, the rainfall intensity associated with the computed time of concentration on an appropriate rainfall-intensity curve may not be consistent with the assumed intensity. If the assumed intensity and that imposed by the frequency curve do not compare favorably, a new rainfall intensity must be assumed and the process repeated.

The kinematic wave method for estimating overland flow time has been determined to be most reliable and is recommended for use with **impervious type surfaces** with \(n=0.05\) or less and a maximum length of 300’. It should be noted that the “n-values” used with the kinematic wave method are applicable only to this method and are for use with very shallow depths of flow such as 0.25”. The “n-values” normally associated with channel or ditch flow do not apply to the Kinematic Wave calculations for overland flow time. A chart showing the recommended “n-values” to use with Kinematic Wave method is included as the second page of Appendix 6D-2.

**Channel Flow**

For channel flow or concentrated flow (CF) time VDOT has found that the nomograph entitled “Time of Concentration of Small Drainage Basins” developed by P.Z. Kirpich provides a reasonable time estimate. Refer to Appendix 6D-5 for the Kirpich nomograph. A direct solution for the Kirpich nomograph using an equation is also
included in Appendix 6D-5. The Kirpich method should only be used for channel flow in Virginia.

When the total time of concentration has been calculated for a point of study (i.e.: culvert, bridge, storm inlet, etc.) the designer should determine if the calculated \( t_c \) is a reasonable estimate for the area under study. The flow length should be divided by the flow time (in seconds) to determine an average velocity of flow. The average velocity can be determined for the overland flow, the channel flow, and the total flow time. If any of the average velocities do not seem reasonable for the specific area of study, they should be checked and revised as needed to provide a reasonable velocity and flow time that will best represent the study area.

6.4.3.1.7 Runoff Coefficients

The runoff coefficient (C) is a variable of the Rational Method that requires significant judgment and understanding on the part of the designer. The coefficient must account for all the factors affecting the relation of peak flow to average rainfall intensity other than area and response time. A range of C-values is typically offered to account for slope, condition of cover, antecedent moisture condition, and other factors that may influence runoff quantities. Good engineering judgment must be used when selecting a C-value for design and peak flow values because a typical coefficient represents the integrated effects of many drainage basin parameters. When available, design and peak flows should be checked against observed flood data. The following discussion considers only the effects of soil groups, land use, and average land slope.

As the slope of the drainage basin increases, the selected C-value should also increase. This is because as the slope of the drainage area increases, the velocity of overland and channel flow will increase, allowing less opportunity for water to infiltrate the ground surface. Thus, more of the rainfall will become runoff from the drainage area. The lowest range of C-values should be used for flat areas where the majority of grades and slopes are less than 2%. The average range of C-values should be used for intermediate areas where the majority of grades and slopes range from 2 to 5%. The highest range of C-values should be used for steep areas (grades greater than 5%), for cluster areas, and for development in clay soil areas.

It is often desirable to develop a composite runoff coefficient based on the percentage of different surface types in the drainage area. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area. Appendix 6E-1 shows runoff coefficients for both rural and urban land use conditions. Note that residential C-values exclude impervious area associated with roadways. The roadways need to be accounted for in actual design.

* Rev. 7/19
6.4.3.1.8 Common Errors

Two common errors should be avoided when calculating time of concentration ($t_c$). First, in some cases runoff from a portion of the drainage area that is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. In these cases, adjustments can be made to the drainage area by disregarding those areas where flow time is too slow to add to the peak discharge. Sometimes it is necessary to estimate several different times of concentration to determine the design flow that is critical for a particular application. This is particularly true if a small portion of the drainage area has an unusually high travel time.

Second, when designing a drainage system, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. Often the land will be graded and swales will intercept the natural contour and conduct the water to the streets which reduces the time of concentration. Care should be exercised in selecting overland flow paths in excess of 200’ in urban areas and 400’ in rural areas. The Department recommends a maximum flow length of 300’ to conform to the recommended flow length value in Section 6.4.3.1.6.

6.4.3.2 Anderson Method

NOTE: For VDOT purposes, the Anderson Method is not recommended for use outside of urbanized areas in the Northern Virginia District.

6.4.3.2.1 Introduction

The Anderson Method was developed by the United States Geological Service (USGS) in 1968 to evaluate the effects of urban development on floods in Northern Virginia. Further discussion can be found in the publication “Effects of Urban Development on Floods in Northern Virginia” by Daniel G. Anderson, U.S.G.S. Water Resources Division 1968.

One of the advantages of the Anderson Method is that the lag time ($T$) can be easily calculated for drainage basins that fit the description for one of the three scenarios given:

1. Natural rural basin
2. Developed basin partly channeled or
3. Completely developed and sewered basin.

For basins that are partly developed, there is no direct method provided to calculate lag time. The following explanation of lag time is reproduced from the original report to provide the user with information to properly assess lag time for use in the Anderson Method based upon the parameters used in the study.
6.4.3.2.2 Application
This method was developed from analysis of drainage basins in Northern Virginia with drainage area sizes up to 570 mi².

6.4.3.2.3 Characteristics
The difference in flood peak size or magnitude because of drainage system improvement is related to lag time (T). Because lag time will change as a basin undergoes development, an estimate of the lag time for the degree of expected basin development is needed to predict future flood conditions.

Using data for 33 natural and 20 completely sewered basins, relationships were sought to define lag time (T) as function of length and slope. The effectiveness of each relationship was determined on the basis of its standard error of estimate, a measure of its accuracy. Approximately two-thirds of the estimates provided by an equation will be accurate within one standard error, and approximately 19 out of 20 estimates will be accurate within two standard errors. Although equations using log T = f (log L, log S) show a slightly smaller standard error, relations of the form log T = f (log (L/√S)) were selected as more appropriate for use on the basis of independent work by Snyder (1958) and theoretical considerations.

The ultimate degree of improvement predicted for most drainage systems in the Alexandria-Fairfax area is storm sewering of all small tributaries but with natural larger channels or moderate improvement of larger channels by alignment and rough surfaced banks of rock or grass.

The center relation shown in Table 6-3 provides estimates of lag time for this type of drainage system. The position of the center relation was based upon plotted data for seven basins that are considered to have reached a condition of complete suburban development. The slope of the relation was computed by logarithmic interpolation between the slopes of the relations for natural and completely sewered basins which are also shown in Table 6-3. Data was insufficient to distinguish separate relations for basins with natural or moderately improved larger channels.

It should be noted that the equation for a developed basin partly channelized is for a drainage area with “complete suburban development” and “storm sewering of all small tributaries”. The larger channels are either natural or have “moderate improvement”. The user is cautioned to use proper engineering judgment in determining lag time for basins that are partly developed and do not fit the parameters used in the equation for developed basin partly channelized.

6.4.3.2.4 Equations
The equation for the Anderson Method is as follows:

\[ Q_f = R_f(230)KA^{0.82}T^{-0.48} \]  (6.6)
Where:

- \( Q_f \) = Maximum rate of runoff, cubic feet per second (cfs) for flood frequency “f” (i.e. 2.33, 5, 10, 25, 50, & 100). For 500-yr flood multiply calculated \( Q_{100} \) by 1.7.
- \( R_f \) = Flood frequency ratio for Flood frequency “f” based on percentages of imperviousness from 0 to 100% (obtained from formula shown below)
- \( K \) = Coefficient of imperviousness (obtained from formula shown below)
- \( A \) = Drainage area, square miles (sq. mi.)
- \( T \) = Time lag, hours (See Table 6-3)

### Table 6-3. Anderson Time Lag Computation

<table>
<thead>
<tr>
<th>Time Lag, ( T )</th>
<th>Watershed Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( 4.64 \left( \frac{L}{\sqrt{S}} \right)^{0.42} )</td>
<td>For natural rural watersheds</td>
</tr>
<tr>
<td>( 0.90 \left( \frac{L}{\sqrt{S}} \right)^{0.50} )</td>
<td>For developed watersheds partially channelized</td>
</tr>
<tr>
<td>( 0.56 \left( \frac{L}{\sqrt{S}} \right)^{0.52} )</td>
<td>For completely developed and sewerred watersheds</td>
</tr>
</tbody>
</table>

Where:

- \( L \) = Length in miles along primary watercourse from site to watershed boundary
- \( S \) = Index of basin slope in feet per mile based on slope between points 10 and 85% of \( L \)

\[
K = 1 + 0.15I \tag{6.7} \]

- \( I \) = Percentage of imperviousness, in whole numbers (e.g. for 20% imperviousness, use \( I = 20 \))

\[
R_f = \frac{R_N + 0.01I(2.5R_{100} - R_N)}{1 + 0.15I} \tag{6.8} \]

Where:

- \( R_N \) = Flood frequency ratio for 0% imperviousness (i.e. completely rural) for flood frequency “f” (See Table 6-4)
- \( R_{100} \) = Flood frequency ratio for 100% imperviousness for flood frequency “f” (See Table 6-4)

* Rev. 7/19
Table 6-4. Anderson Method Flood Frequency Ratios

<table>
<thead>
<tr>
<th>f</th>
<th>2.33</th>
<th>5</th>
<th>10</th>
<th>25</th>
<th>50</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>R_N</td>
<td>1.00</td>
<td>1.65</td>
<td>2.20</td>
<td>3.30</td>
<td>4.40</td>
<td>5.50</td>
</tr>
<tr>
<td>R_{100}</td>
<td>1.00</td>
<td>1.24</td>
<td>1.45</td>
<td>1.80</td>
<td>2.00</td>
<td>2.20</td>
</tr>
</tbody>
</table>

Flood frequency ratio for the 5-yr events were derived by VDOT, all others were taken directly from the D.G. Anderson report.

6.4.3.3 Rural Regression Method

6.4.3.3.1 Introduction
Regional regression equations are a commonly accepted method for estimating peak flows at ungaged sites or sites with insufficient historical data. Regression studies are statistical practices used to develop runoff equations. These equations are used to relate such things as the peak flow or some other flood characteristic at a specified recurrence interval to the watershed's physiographic, hydrologic and meteorological characteristics.

For details on the application of Rural Regression Equations in Virginia, the user is directed to the following publication: “Peak-Flow Characteristics of Virginia Streams,” U.S.G.S. Scientific Investigations Report 2011-5144 (2011). This report does have some omissions in the standard storm events ranges. VDOT has developed equations based on the USGS data that may be used to supplement the equations in the above report. Tools for using these equations are available on the USGS website StreamStats and in the VDOT online Hydraulic Applications. However, note that StreamStats does not include the supplemental regression equations provided by VDOT for storm events omitted by USGS.

6.4.3.3.2 Application
The regression equations should be used routinely in design for drainage areas greater than one square mile and where stream gage data is unavailable. Where there is stream gage data, the findings from a Log Pearson III (LPIII) method should govern if there is significant variance +10% from those obtained using the rural regression equations, and provided there is at least 10 years of continuous or synthesized stream gage record. The LPIII results state wide can be found in Table 2 of the above reference. For sites on completely un-gaged watersheds or gaged watersheds where the drainage area at the site is less than 50% of that at the gage or is more than 150% of that at the gage, peak discharges shall be computed using the regressions equations.

* Rev. 7/19
For sites on gaged watersheds where the drainage area at the site is equal to or more than 50% of that at the gage or is less than or equal to 150% of that at the gage the gage transposition method described in Section 6.4.3.5.3 may also be used.

6.4.3.3 Hydrologic Regions

The gage data was grouped for the regression analyses based on the five physiographic regions found in Virginia. Each region has distinctive geologic features, landforms and similar runoff characteristics. These regions include: Coastal Plain, Piedmont, Mesozoic Basin, Blue Ridge, Valley and Ridge, and Appalachian Plateau. Figure 6-3 shows the hydrologic regional boundaries for Virginia and can also be seen in the VDOT GIS Integrator.

6.4.3.3.4 Equations

Table 6-4 contains the drainage-area-only regression equations for estimating peak discharges in Virginia.

6.4.3.3.5 Characteristics

The methodology as described determines the flow per square mile based upon a weighted average of the flows based upon the percent of the watershed within each physiographic province. It is recommended that the USGS Stream Stats website be used to compute these flows as it automatically accounts for the weighting for watersheds that cross physiographic regions:


However, note that StreamStats does not include the supplemental regression equations provided by VDOT for storm events omitted by “Peak-Flow Characteristics of Virginia Streams,” U.S.G.S. Scientific Investigations Report 2011-5144 (2011).

* Rev. 7/19
Figure 6-3. Peak Discharge Regions for Regression Equations

* Rev. 7/19
Table 6-5. Regional Regression Equations for Estimating Peak Discharges of Streams in Virginia

<table>
<thead>
<tr>
<th>Basins in the Coastal Plain region</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2-year*</td>
<td>Log10(Q₂) = 1.758 + 0.659 • Log10(DA)</td>
</tr>
<tr>
<td>5-year</td>
<td>Log10(Q₅) = 1.918 + 0.644 • Log10(DA)</td>
</tr>
<tr>
<td>10-year</td>
<td>Log10(Q₁₀) = 2.107 + 0.626 • Log10(DA)</td>
</tr>
<tr>
<td>25-year</td>
<td>Log10(Q₂₅) = 2.315 + 0.609 • Log10(DA)</td>
</tr>
<tr>
<td>50-year</td>
<td>Log10(Q₅₀) = 2.457 + 0.594 • Log10(DA)</td>
</tr>
<tr>
<td>100-year</td>
<td>Log10(Q₁₀₀) = 2.580 + 0.583 • Log10(DA)</td>
</tr>
<tr>
<td>200-year</td>
<td>Log10(Q₂₀₀) = 2.698 + 0.573 • Log10(DA)</td>
</tr>
<tr>
<td>500-year*</td>
<td>Log10(Q₅₀₀) = 2.918 + 0.554 • Log10(DA)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Basins in the Piedmont region, except those within the Mesozoic Basin region</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2-year</td>
<td>Log10(Q₂) = 2.197 + 0.593 • Log10(DA)</td>
</tr>
<tr>
<td>5-year</td>
<td>Log10(Q₅) = 2.540 + 0.551 • Log10(DA)</td>
</tr>
<tr>
<td>10-year</td>
<td>Log10(Q₁₀) = 2.719 + 0.534 • Log10(DA)</td>
</tr>
<tr>
<td>25-year</td>
<td>Log10(Q₂₅) = 2.916 + 0.514 • Log10(DA)</td>
</tr>
<tr>
<td>50-year</td>
<td>Log10(Q₅₀) = 3.043 + 0.501 • Log10(DA)</td>
</tr>
<tr>
<td>100-year</td>
<td>Log10(Q₁₀₀) = 3.157 + 0.490 • Log10(DA)</td>
</tr>
<tr>
<td>200-year</td>
<td>Log10(Q₂₀₀) = 3.263 + 0.480 • Log10(DA)</td>
</tr>
<tr>
<td>500-year*</td>
<td>Log10(Q₅₀₀) = 3.420 + 0.466 • Log10(DA)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Basins in the Mesozoic Basin region</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2-year</td>
<td>Log10(Q₂) = 2.002 + 0.722 • Log10(DA)</td>
</tr>
<tr>
<td>5-year</td>
<td>Log10(Q₅) = 2.416 + 0.660 • Log10(DA)</td>
</tr>
<tr>
<td>10-year</td>
<td>Log10(Q₁₀) = 2.656 + 0.624 • Log10(DA)</td>
</tr>
<tr>
<td>25-year</td>
<td>Log10(Q₂₅) = 2.923 + 0.586 • Log10(DA)</td>
</tr>
<tr>
<td>50-year</td>
<td>Log10(Q₅₀) = 3.097 + 0.561 • Log10(DA)</td>
</tr>
<tr>
<td>100-year</td>
<td>Log10(Q₁₀₀) = 3.265 + 0.537 • Log10(DA)</td>
</tr>
<tr>
<td>200-year</td>
<td>Log10(Q₂₀₀) = 3.401 + 0.521 • Log10(DA)</td>
</tr>
<tr>
<td>500-year*</td>
<td>Log10(Q₅₀₀) = 3.623 + 0.487 • Log10(DA)</td>
</tr>
</tbody>
</table>

* Derived by VDOT for use in Roadway Projects for VDOT
### Basins in the Blue Ridge region

<table>
<thead>
<tr>
<th>Duration</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-year</td>
<td>( \log_{10}(Q) = 2.127 + 0.709 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>5-year</td>
<td>( \log_{10}(Q) = 2.490 + 0.668 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>10-year</td>
<td>( \log_{10}(Q) = 2.689 + 0.647 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>25-year</td>
<td>( \log_{10}(Q) = 2.893 + 0.629 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>50-year</td>
<td>( \log_{10}(Q) = 3.030 + 0.616 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>100-year</td>
<td>( \log_{10}(Q) = 3.184 + 0.593 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>200-year</td>
<td>( \log_{10}(Q) = 3.288 + 0.586 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>500-year*</td>
<td>( \log_{10}(Q) = 3.477 + 0.563 \cdot \log_{10}(DA) )</td>
</tr>
</tbody>
</table>

### Basins in the Valley and Ridge region

<table>
<thead>
<tr>
<th>Duration</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-year</td>
<td>( \log_{10}(Q) = 2.053 + 0.733 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>5-year</td>
<td>( \log_{10}(Q) = 2.382 + 0.689 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>10-year</td>
<td>( \log_{10}(Q) = 2.557 + 0.665 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>25-year</td>
<td>( \log_{10}(Q) = 2.741 + 0.642 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>50-year</td>
<td>( \log_{10}(Q) = 2.862 + 0.626 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>100-year</td>
<td>( \log_{10}(Q) = 2.963 + 0.615 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>200-year</td>
<td>( \log_{10}(Q) = 3.063 + 0.603 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>500-year*</td>
<td>( \log_{10}(Q) = 3.208 + 0.588 \cdot \log_{10}(DA) )</td>
</tr>
</tbody>
</table>

### Basins in the Appalachian Plateau region

<table>
<thead>
<tr>
<th>Duration</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-year</td>
<td>( \log_{10}(Q) = 1.980 + 0.833 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>5-year</td>
<td>( \log_{10}(Q) = 2.289 + 0.798 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>10-year</td>
<td>( \log_{10}(Q) = 2.450 + 0.781 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>25-year</td>
<td>( \log_{10}(Q) = 2.631 + 0.759 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>50-year</td>
<td>( \log_{10}(Q) = 2.740 + 0.750 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>100-year*</td>
<td>( \log_{10}(Q) = 2.890 + 0.734 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>200-year*</td>
<td>( \log_{10}(Q) = 3.025 + 0.719 \cdot \log_{10}(DA) )</td>
</tr>
<tr>
<td>500-year*</td>
<td>( \log_{10}(Q) = 3.187 + 0.685 \cdot \log_{10}(DA) )</td>
</tr>
</tbody>
</table>

* Derived by VDOT for use in Roadway Projects for VDOT
6.4.3.3.6 Mixed Population
Mixed population floods are those derived from two (or more) causative factors; e.g., rainfall on a snow pack or hurricane generated floods where convective storm events commonly predominate. To evaluate the effect of such occurrences requires reasonable and prudent judgment.

6.4.3.4 Urban Regression Method

6.4.3.4.1 Introduction
Regression equations developed by the USGS can be found in “Methods and Equations for Estimating Peak Stream Flow per square mile in Virginia’s Urban Basins” Scientific Investigations Report 2014-5090 developed specifically for urbanized watersheds in Virginia. It was observed in this study that the urban regression relationship was consistent across the state and was not regionalized.

6.4.3.4.2 Application
These urban equations may be used for the final hydraulic design of bridges, culverts, and similar structures where such structures are not an integral part of a storm drain system, and provided the contributing watershed either is, or is expected to become, at least 10% urban in nature.

6.4.3.4.3 Characteristics
The methodology as described determines the flow per square mile based upon the percent urbanization in the equations below (URBAN) entered as a % value (i.e., 20% = 20) and the overall watershed area. The resulting discharge must be multiplied by the watershed area. The percent urbanization made be estimated or determined by the methods available through the USGS StreamStats website to compute basin parameter LC11DEV.

6.4.3.4.4 Equations
The equations for urban conditions take the following general form:

\[
\log_{10}(q) = \beta_0 + (\text{URBAN} - \beta_1) \times ((\log_{10}(A) - \beta_2) \times \beta_3) + \text{URBAN} \times \beta_4 + \log_{10}(A) \times \beta_5
\]

\[
q = 10^{\log_{10}(q)}
\]

\[
Q = q \times A
\]

Where:
\[
\begin{align*}
q &= \text{Unit discharge per square mile (cfs/mi}^2) \\
Q &= \text{Total Discharge (cfs)} \\
\text{Urban} &= \text{Percent Urbanization 10-100 (dimensionless)} \\
A &= \text{Contributing drainage area, square miles (mi}^2) \\
\beta_0 - \beta_5 &= \text{Regression constants a given return period}
\end{align*}
\]
### 6.4.3.5 Stream Gage Data

#### 6.4.3.5.1 Introduction

Many gauging stations exist throughout Virginia where data can be obtained and used for hydrologic studies. If a project is located near one of these gages and the gaging record is of sufficient length in time, a frequency analysis may be made according to the following discussion. The most important aspect of applicable station records is the series of annual peak discharges. It is possible to apply a frequency analysis to that data for the derivation of flood-frequency curves. Such curves can then be used in several different ways.

- If the subject site is at or very near the gaging site and on the same stream and watershed, the discharge for a specific frequency from the flood-frequency curve can be used directly.

- If the facility site is up or downstream of the gaging site or on a nearby or representative watershed with similar hydrologic characteristics, transposition of frequency discharges is possible, provided the watershed area at the facility site is no less than 1/2 nor more than 1.5 times the watershed area at the gaging site.

- If the flood-frequency curve is from one of a group of several gaging stations comprising a hydrologic region, then regional regression relations may be derived. Regional regression relations are usually furnished by established hydrologic agencies and the designer will not be involved in their development.

The Log Pearson Type III frequency distribution will be used to estimate flood frequency in this manual.

#### 6.4.3.5.2 Application

The stream gage analysis findings may be used for design when there are sufficient years of measured or synthesized stream gage data. The Log Pearson Type III method data is available in the “Peak-Flow Characteristics of Virginia Streams,” U.S.G.S. Scientific Investigations Report 2011-5144 (2011). The U.S. Geological Survey has developed a computer program entitled “PeakFQ” for performing Log Pearson Type III

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<th>$\beta_3$</th>
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computations. Gage data may be obtained from the USGS at https://water.usgs.gov/software/PeakFQ/.

### 6.4.3.5.3 Transposition of Data

The transposition of design discharges from one basin to another basin with similar hydrologic characteristics is accomplished by multiplying the design discharge by the direct ratio of the respective drainage areas raised to the power shown in Table 6-7. Thus on streams where no gaging station is in existence, records of gaging stations in one or more nearby hydrologically similar watersheds may be used. The discharge for such an ungaged stream may be determined by the transposition of records using a similar procedure. This procedure is repeated for each available nearby watershed and the results are averaged to obtain a value for the desired flood frequency relationships in the ungaged watershed.

#### Table 6-6. Transposition of Data Sample Problem

<table>
<thead>
<tr>
<th>Watershed</th>
<th>Q(_{25}), cfs</th>
<th>Area, sq. mi</th>
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<td>Gaged Watershed A</td>
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<tr>
<td>Gaged Watershed B</td>
<td>7,2000</td>
<td>79.6</td>
</tr>
<tr>
<td>Gaged Watershed C</td>
<td>12,000</td>
<td>124</td>
</tr>
<tr>
<td>Ungaged Watershed D</td>
<td>Find Q(_{25})</td>
<td>83.0</td>
</tr>
</tbody>
</table>

Adjust Q\(_{25}\) for each subshed by area ratio:

- **A**: \(4,100 \times \left(\frac{83.0}{42.1}\right)^{0.8} = 7,057\)
- **B**: \(7,200 \times \left(\frac{83.0}{79.6}\right)^{0.8} = 7,445\)
- **C**: \(12,000 \times \left(\frac{83.0}{124}\right)^{0.8} = 8,704\)

Average the Q\(_{25}\) for subsheds A, B, and C to Obtain Q\(_{25}\) for subshed D:

\[D: Q_{25} = \frac{(7,057+7,445+8,704)}{3} = 7,735\text{ use 7,700 cfs}\]

### 6.4.4 Hydrograph Methods

#### 6.4.4.1 Modified Rational Method

##### 6.4.4.1.1 Introduction

The Modified Rational Method provides hydrographs for small drainage areas where the peak, Q, is normally calculated by the Rational Method.

##### 6.4.4.1.2 Application

Hydrographs produced by the Modified Rational Method can be used for the analysis and design of stormwater management (SWM) basins, temporary sediment basins, or other applications needing a hydrograph for a drainage area of less than 200 ac.
6.4.4.1.3 Characteristics
Hydrographs developed by the Modified Rational Method are based upon different duration storms of the same frequency and have the following parameters:

- Time of concentration ($t_c$) = Time to peak ($T_p$)
- Time to recede ($T_r$) = $T_p$
- The duration, $D_e$, of the storm is from 0 minutes until the time of selected duration
- Base of hydrograph ($T_b$) = $D_e + T_r$
- The peak Q (top of trapezoidal hydrograph) is calculated using the intensity (I) value predicated on the “B, D, & E” factors (Appendix 6C-2) for the selected duration and frequency.
- Hydrographs are normally calculated for durations of:
  - $t_c$
  - $1.5t_c$
  - $2t_c$
  - $3t_c$
- Longer duration hydrographs may need to be calculated if reservoir routing computations show that the ponded depth in a basin is increasing with each successive hydrograph that is routed through the basin.

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

The Modified Rational Method recognizes that the duration of a storm can and will sometimes be longer than the time of concentration. This longer duration storm, even though it produces a lower peak Q, can produce a larger volume of runoff than the storm duration equal to the actual time of concentration of the drainage area. In order to ensure the proper design of stormwater management basins, the volume of runoff for the critical storm duration should be calculated.

6.4.4.1.4 Critical Storm Duration
The storm duration that produces the greatest volume of storage and highest ponded depth within a basin is considered the critical duration storm ($T_c$). Reservoir routing computations for the basin will need to incorporate several different duration storms in order to determine the critical duration and the highest pond level for each frequency storm required. The operation of any basin is dependent on the interaction of:

- Inflow (hydrograph)
- Storage characteristics of the basin
- Performance of the outlet control structure

Therefore, each basin will respond to different duration storms in dissimilar patterns. The approximate critical storm can be estimated but the actual critical duration storm can only be determined by performing reservoir routing computations for several different duration storms.
6.4.4.1.5 Estimating the Critical Duration Storm

The Virginia Department of Conservation and Recreation (DCR) has developed a method to estimate the critical duration storm. The following items should be taken into consideration when using this method:

- For estimation only
- May provide a critical storm duration which is less than \( t_c \), this is not valid
- Does not work well when \( t_c \) is decreased only slightly by development
- Does not work well when the peak \( Q \) is not significantly increased by development
- The \( a \) and \( b \) factors for equation 6.9 are listed in Chapter 11, Appendix 11K-1 and are to be used for no other purpose

For further explanation see Chapter 11, section 11.5.7.1.

The approximate length of the critical storm duration can be estimated by the following equation:

\[
T_c = \sqrt{\frac{2CAa(b-t_c)}{q_o}} - b \tag{6.9}
\]

Where:

- \( T_c \) = Critical storm duration, minute (min)
- \( C \) = Rational coefficient for developed area
- \( A \) = Drainage area, acres (ac)
- \( t_c \) = Time of concentration after development, minute (min)
- \( q_o \) = Allowable peak outflow, cubic feet per second (cfs)
- \( a \) & \( b \) = Rainfall regression constants, Appendix 11 H-2

6.4.4.2 NRCS Methods (Graphical Peak Discharge and Unit Hydrograph)∗

6.4.4.2.1 Introduction

Techniques developed by the United States Department of Agriculture (USDA) Natural Resources Conservation Service or NRCS (formerly Soil Conservation Service or SCS) for calculating rates of runoff require the same basic data as the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The NRCS approach, however, also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage and an infiltration rate that decreases during the course of a storm. With the NRCS method, the direct runoff can be calculated for any storm, either real or synthetic, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess. Details of the methodology can be found in the USDA-NRCS National Engineering Handbook, Part 630 - Hydrology.

∗ Rev. 7/19
6.4.4.2.2 Application
Two types of hydrographs are used in the NRCS procedure, unit hydrographs and dimensionless hydrographs. A unit hydrograph represents the time distribution of flow resulting from one-inch of direct runoff occurring over the watershed in a specified time. A dimensionless hydrograph represents the composite of many unit hydrographs. The dimensionless unit hydrograph is plotted in non-dimensional units of time versus time to peak and discharge at any time versus peak discharge.

6.4.4.2.3 Characteristics
Characteristics of the dimensionless hydrograph vary with the size, shape, and slope of the tributary drainage area. The most significant characteristics affecting the dimensionless hydrograph shape are the basin lag and the peak discharge for a given rainfall. Basin lag is the time from the center of mass of rainfall excess to the hydrograph peak. Steep slopes, compact shape, and an efficient drainage network tend to make lag time short and peaks high; flat slopes, elongated shape, and an inefficient drainage network tend to make lag time long and peaks low.

6.4.4.2.4 Time of Concentration
The average slope within the watershed together with the overall length and retardance of overland flow are the major factors affecting the runoff rate through the watershed. VDOT recommends using the Rational Method procedures to calculate time of concentration ($t_c$). Lag time ($L$) can be considered as a weighted time of concentration and is related to the physical properties of a watershed, such as area, length and slope. The NRCS derived the following empirical relationship between lag time and time of concentration:

$$L = 0.6 \times t_c$$  \hspace{1cm} (6.10)

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

Land use is the watershed cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads, roofs, etc. are all part of the land use. Land treatment applies mainly to agricultural land use, and it includes mechanical practices such as contouring or terracing and management practices such as rotation of crops.

The NRCS uses a combination of soil conditions and land use (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area when the soil is not frozen. The higher the CN,

* Rev. 7/19
the higher is the runoff potential. Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The NRCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C and D). Soil type A has the highest infiltration and soil type D has the least amount of infiltration. Soil surveys are available from the NRCS website at http://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm, or your local NRCS office at:

Virginia FSA, NRCS & RD State Offices  
1606 Santa Rosa Road, Suite 209  
Richmond, VA 23229-5014  
Phone: 804-287-1500

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also runoff curve numbers vary with the antecedent soil moisture conditions, defined as the amount of rainfall occurring in a selected period preceding a given storm. In general, the greater the antecedent rainfall, the more direct runoff there is from a given storm. A five (5) day period is used as the minimum for estimating antecedent moisture conditions. Antecedent soil moisture conditions also vary during a storm; heavy rain falling on a dry soil can change the soil moisture condition from dry to average to wet during the storm period.

6.4.4.2.6 Equations
The following discussion outlines the equations and basic concepts utilized in the NRCS method.

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

Rainfall - The rainfall employed in the NRCS method (the variable “P” in equation 6.11) for both duration and frequency may be obtained directly from NOAA’s Precipitation Frequency Data Server (based on their ATLAS-14 publication) at the following Internet address: http://hdsc.nws.noaa.gov/hdsc/pfds/orb/va_pfds.html. When the opening screen appears be sure to choose “Data Type:” as “Precipitation Depth” from the pull-down options menu.

* Rev. 7/19
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, such as contouring and 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 24-hour or 1-day storm rainfall). The equation is:

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

(6.11)

Where:

- \( Q \) = Direct runoff, inches (in)
- \( P \) = Precipitation, inches (in)
- \( I_a \) = Initial abstractions, inches (in)

\[
I_a = 0.2 \times S
\]

(6.12)

- \( S \) = Potential maximum retention after runoff begins, inches (in)

\[
S = \frac{1000}{CN} - 10
\]

(6.13)

- \( CN \) = NRCS Runoff curve number

The Virginia office of the NRCS has recently advised that the NOAA ATLAS-14 rainfall data does not, in many instances, follow the current Type II and Type III temporal distribution curves. They indicate that the Type II curve, will only give reasonable results for return interval (frequency) storm events up to and including a 10-year event and should be used with caution. They have advised that the soon to be released revised “TR-20” software package will provide a routine that will convert the ATLAS-14 rainfall data from NOAA’s Precipitation Frequency Data Server to county-specific temporal distribution curves. Their “TR-55” and “EFH-2” software packages will ultimately contain this same feature. The NRCS Virginia office has indicated that additional information on this issue will be posted on their web site as it becomes available.

* Rev. 7/19
6.5 References


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* Rev. 7/19
### Runoff Depth for Runoff Curve Number (RCN)

Runoff depth for selected NRCS TR-55 CN’s and rainfall amounts*

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*Interpolate the values shown to obtain runoff depths for CN’s or rainfall amounts not shown.

Source: SCS TR-55
## Chapter 6 – Hydrology

### APPENDIX 6B-02  24-HR. RAINFALL DEPTHS (INCHES)

### APPENDIX 11C-3  24-HOUR RAINFALL DEPTH (INCHES)

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### Chapter 6 – Hydrology

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Source: National Resource Conservation Service, Richmond, Va. office – Based on their implementation of NOAA’s ATLAS-14 rainfall data

Note: Maps are available showing the zone boundaries for counties with multiple rainfall zones at the following NOAA web site: http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_maps.html
Chapter 6 - Hydrology

Appendix 6C-1  B, D, and E Factors - Application

B, D and E Factors that Define Intensity-Duration-Frequency (IDF) Values for Use with the Rational Method and the Modified Rational Method

The rainfall IDF values are described by the equation:

\[ i = \frac{B}{(t_c + D)^E} \]

Where:

- \( i \) = Intensity, inches per hour (in/hr)
- \( t_c \) = Time of concentration, minutes (min)

The B, D and E factors for all counties and major cities have been tabulated in Appendix 6C-2. These values were derived by the Department using the Rainfall Precipitation Frequency data provided by NOAA’s “Atlas 14” at the following Internet address: [http://hdsc.nws.noaa.gov/hdsc/pfds/orb/va_pfds.html](http://hdsc.nws.noaa.gov/hdsc/pfds/orb/va_pfds.html). A Microsoft EXCEL spreadsheet containing all the B, D, and E factors for the state of Virginia as shown in Appendix 6C-2 is available upon request or may be downloaded at the following Internet address: [http://www.virginiadot.org/business/resources/LocDes/BDE_2016.xlsx](http://www.virginiadot.org/business/resources/LocDes/BDE_2016.xlsx).

It should be noted, since the regression procedure used to derive these values was predicated on 5 and 60 minute storm durations, that the accuracy of the calculations performed using these values decreases significantly for times of concentration in excess of 60 minutes and the error becomes greater as the time increases. For long storm durations and/or long times of concentration, the rainfall intensity and/or total point rainfall should be obtained directly from NOAA’s Precipitation Frequency Data Server at the Internet address shown above.

An example problem employing the above equation is shown below.

**Given:** Chesterfield County, Storm Duration \((t_c) = 30\) minutes

**Find:** 10-yr. frequency rainfall intensity

---

* Rev 7/09
Solution: From Appendix 6C-2, for Chesterfield County and a 10-yr. event, read 
B = 50.71, D = 10.00, & E= 0.73. Substitute these values and a tc of 30 into the 
above formula.

\[ i_{10} = \frac{B}{(tc + D)^E} = \frac{50.71}{(30 + 10.00)^{0.73}} = 3.43 \text{ in/hr} \]

It should be noted that the above procedure could also be used for applications 
employing time of concentration (tc) in hours and total rainfall (as opposed to 
rainfall intensity) in inches. It is merely necessary to multiply the calculated 
rainfall intensity (based on a tc in minutes) by the time of concentration (in hours) 
to determine the total point rainfall.
### Chapter 6 - Hydrology

#### Appendix 6C-2 B, D, and E Factors

B. D. & E factors for determining rainfall intensity in the Rational and Modified Rational Methods (based on NOAA NW-14 Atlas data)

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<td>57.02</td>
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<td>11.74</td>
<td>0.85</td>
<td>51.87</td>
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<td>Byllies 3 W</td>
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<td>11.76</td>
<td>0.86</td>
<td>46.38</td>
<td>11.80</td>
<td>0.86</td>
<td>53.19</td>
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<td>38.82</td>
<td>10.26</td>
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<td>11.52</td>
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Chapter 6 - Hydrology

Appendix 6C-2

B, D, and E Factors

Chatham

44-1614

43.32

10.85

0.84

53.11

11.37

0.84

57.19

11.32

0.80

57.03

10.78

0.77

55.10

10.09

0.73

53.77

9.68

0.71

51.15

9.01

0.68

Churchville
Clarendon Lyon
Park

44-1708

33.15

10.58

0.83

40.88

11.05

0.83

48.66

11.55

0.81

50.52

11.26

0.79

51.69

10.80

0.76

52.44

10.57

0.74

51.47

10.03

0.71

46.45

11.40

0.85

57.70

11.93

0.86

62.74

11.96

0.82

63.92

11.66

0.80

61.35

10.75

0.75

59.64

10.21

0.72

57.16

9.49

0.69

Clarksville

44-1746

49.25

11.51

0.86

57.91

11.84

0.85

61.56

11.85

0.82

62.15

11.40

0.79

60.87

10.82

0.76

58.72

10.26

0.72

56.40

9.72

0.70

Clifton Forge 2 NW

44-1801

36.72

11.27

0.85

42.99

11.46

0.84

50.17

11.80

0.82

52.43

11.59

0.80

53.47

11.16

0.77

53.34

10.81

0.74

53.08

10.47

0.72

Colonial Beach

44-1913

46.14

11.30

0.85

55.01

11.45

0.84

59.36

11.49

0.81

60.10

11.05

0.78

58.89

10.39

0.74

57.06

9.79

0.71

55.34

9.22

0.68

Columbia 2SSE

44-1929

42.13

10.78

0.84

49.95

11.21

0.83

52.98

11.29

0.80

55.50

11.01

0.78

53.83

10.29

0.74

53.13

9.81

0.71

50.89

9.15

0.68

Concord 4 SSW

44-1955

41.64

10.99

0.84

49.11

11.18

0.83

52.16

10.99

0.80

53.67

10.74

0.77

52.42

10.02

0.73

51.28

9.62

0.70

48.81

8.85

0.67

Copper Hill

44-1999

39.18

10.57

0.82

46.69

10.76

0.82

51.90

10.91

0.79

53.32

10.63

0.77

52.05

9.93

0.73

51.95

9.61

0.71

49.94

8.98

0.68

Corbin

44-2009

46.26

11.17

0.84

56.32

11.59

0.85

61.07

11.74

0.82

60.47

11.11

0.78

59.27

10.42

0.74

57.94

9.90

0.71

56.31

9.34

0.69

Covington
Covington Filter
Plant

44-2041

39.87

11.97

0.87

48.06

12.35

0.87

53.81

12.46

0.84

56.58

12.31

0.82

56.81

11.75

0.79

57.36

11.54

0.76

56.09

11.03

0.74

44-1729

44-2044

37.45

11.51

0.86

46.17

12.02

0.86

54.30

12.47

0.84

54.61

11.93

0.81

56.33

11.65

0.78

56.75

11.40

0.76

55.38

10.88

0.73

Craigsville 2 S

44-2064

34.19

10.69

0.83

41.07

11.00

0.83

47.80

11.29

0.81

50.46

11.16

0.79

51.45

10.73

0.75

51.35

10.41

0.73

49.47

9.69

0.70

Crozier

44-2142

45.61

11.13

0.85

53.08

11.36

0.84

56.88

11.46

0.81

58.61

11.09

0.78

56.55

10.31

0.74

56.04

9.93

0.71

54.05

9.33

0.69

Culpeper
Dahlgren Proving
Groun

44-2155

42.76

10.88

0.84

51.02

11.09

0.83

55.05

11.00

0.80

56.19

10.60

0.77

55.55

9.96

0.74

54.58

9.42

0.71

52.82

8.74

0.68

45.40

11.15

0.84

55.01

11.45

0.84

59.96

11.56

0.81

60.24

11.10

0.78

60.24

11.10

0.78

57.84

9.92

0.71

55.40

9.18

0.68

Dale Enterprise

44-2208

33.63

10.08

0.84

40.13

10.30

0.83

45.74

10.33

0.81

46.93

9.83

0.78

48.07

9.37

0.75

47.26

8.70

0.72

46.43

8.11

0.69

Damascus

44-2216

41.72

11.83

0.87

48.39

12.03

0.86

55.16

12.60

0.84

58.03

12.43

0.82

60.19

12.17

0.79

61.09

11.84

0.77

62.12

11.62

0.75

Dante

44-2237

40.59

11.57

0.86

49.26

12.11

0.86

56.46

12.59

0.84

58.03

12.20

0.82

61.11

12.03

0.79

61.32

11.61

0.76

62.21

11.30

0.74

Danville

44-2245

46.61

11.13

0.85

55.93

11.49

0.84

60.27

11.49

0.81

60.15

11.09

0.78

58.22

10.43

0.74

55.97

9.94

0.71

52.67

9.21

0.68

Davenport 2 NE

44-2269

43.80

12.00

0.88

53.34

12.45

0.88

60.09

12.84

0.85

63.43

12.76

0.83

64.88

12.27

0.80

65.09

11.83

0.77

64.73

11.33

0.74

Deerfield 1 S

44-2315

32.81

10.33

0.82

41.33

11.03

0.83

47.83

11.32

0.81

50.26

11.11

0.79

50.79

10.62

0.75

50.75

10.30

0.73

49.48

9.67

0.70

Delaplane 1 N

44-2326

40.36

9.90

0.83

48.95

10.19

0.83

53.44

10.02

0.80

56.37

9.92

0.78

55.41

9.01

0.74

55.43

8.43

0.71

55.17

7.91

0.69

Driver 4 NE

44-2504

57.41

11.64

0.86

67.21

11.85

0.85

68.11

11.67

0.81

67.05

10.92

0.78

64.86

10.22

0.73

63.82

9.65

0.71

61.07

8.93

0.67

Elkwood 7 SE

44-2729

44.31

11.16

0.85

52.76

11.42

0.84

56.83

11.40

0.81

59.21

11.23

0.78

56.99

10.36

0.74

56.31

9.91

0.71

54.92

9.37

0.69

Emporia 1 WNW

44-2790

50.22

11.38

0.85

59.16

11.75

0.85

61.55

11.65

0.81

61.73

11.08

0.78

59.75

10.32

0.74

58.97

9.96

0.71

55.82

9.14

0.68

Farmville 2 N

44-2941

44.33

11.00

0.84

52.65

11.41

0.84

56.27

11.43

0.81

57.34

10.96

0.78

54.61

10.11

0.73

54.52

9.81

0.71

52.21

9.14

0.68

Floyd
Fredericksburg
Sewage

44-3071

40.81

11.12

0.84

47.69

11.21

0.83

53.11

11.37

0.81

54.72

11.10

0.78

54.31

10.51

0.75

52.84

9.95

0.72

51.49

9.46

0.69

46.35

11.29

0.85

56.02

11.64

0.85

60.87

11.75

0.82

60.89

11.22

0.79

59.48

10.49

0.75

57.99

9.96

0.72

55.77

9.30

0.69

Free Union
Galax Radio
WBOB

44-3213

37.59

10.17

0.81

45.08

10.37

0.81

50.36

10.68

0.78

50.36

10.68

0.78

49.90

9.49

0.72

49.24

9.00

0.69

48.09

8.48

0.66

38.43

11.06

0.84

45.97

11.47

0.84

50.86

11.43

0.81

54.24

11.34

0.79

57.06

11.02

0.76

60.03

10.97

0.74

62.25

10.80

0.73

38.93

11.16

0.84

45.33

11.30

0.84

51.95

11.61

0.81

54.28

11.32

0.79

57.77

11.14

0.76

60.03

10.97

0.74

62.07

10.71

0.73

Galax Water Plant

44-2195

44-3204

44-3267
44-3272

2 of 6

VDOT DRAINAGE MANUAL


Chapter 6 - Hydrology

Appendix 6C-2

B, D, and E Factors

Gathright Dam

44-3310

35.05

10.95

0.84

42.43

11.31

0.84

50.81

11.86

0.82

52.65

11.64

0.80

53.84

11.20

0.77

54.38

10.96

0.75

53.98

Glasgow

44-3375

36.76

10.76

0.84

44.97

11.31

0.83

50.03

11.24

0.80

52.53

11.16

0.78

52.33

10.50

0.75

51.95

10.08

0.72

51.03

9.62

0.70

Glen Lyn

44-3397

39.95

11.93

0.87

51.38

12.80

0.89

57.35

13.04

0.86

59.20

12.85

0.84

60.52

12.51

0.81

60.55

12.33

0.79

60.06

12.01

0.76

Gordonsville 3 S

44-3466

41.14

10.65

0.83

49.89

11.11

0.83

52.73

10.92

0.79

53.86

10.53

0.77

53.33

10.00

0.73

52.59

9.48

0.70

51.03

8.88

0.67

Goshen

44-3470

34.19

10.69

0.83

41.07

11.00

0.83

49.30

11.59

0.81

49.99

11.09

0.78

51.35

10.69

0.75

50.53

10.20

0.73

49.78

9.76

0.70

Groseclose

44-3623

40.12

11.55

0.86

47.25

11.84

0.86

53.15

12.26

0.84

55.61

12.02

0.81

57.56

11.77

0.79

60.23

11.77

0.77

62.30

11.65

0.75

Grundy

44-3640

41.36

11.37

0.87

50.25

11.88

0.87

58.56

12.46

0.85

59.45

11.92

0.82

60.68

11.34

0.79

63.55

11.28

0.77

64.08

10.88

0.75

Halifax 1 N

44-3690

46.57

11.26

0.85

54.66

11.39

0.84

60.00

11.54

0.81

60.18

11.16

0.78

58.41

10.51

0.74

55.87

9.89

0.71

53.46

9.27

0.69

Hillsville

44-3991

40.27

11.33

0.85

48.37

11.80

0.85

52.76

11.73

0.82

55.22

11.55

0.80

55.40

10.93

0.76

57.46

10.89

0.74

57.04

10.45

0.72

Holland 1 E

44-4044

59.92

11.72

0.86

72.03

12.17

0.86

71.64

11.88

0.82

71.84

11.29

0.79

68.93

10.52

0.74

67.04

9.86

0.71

64.87

9.26

0.68

Honaker

44-4078

44.62

12.33

0.88

51.96

12.41

0.87

58.38

12.81

0.85

61.46

12.66

0.83

62.01

12.07

0.79

62.13

11.58

0.76

63.92

11.39

0.74

Hopewell

44-4101

47.07

10.99

0.84

56.89

11.48

0.84

59.47

11.34

0.80

60.39

10.88

0.77

58.31

10.16

0.73

56.79

9.60

0.70

54.74

8.98

0.67

Hot Springs

44-4128

33.65

10.58

0.83

41.22

10.99

0.83

47.03

11.19

0.80

48.50

10.86

0.78

49.52

10.44

0.75

49.68

10.14

0.72

49.32

9.73

0.70

Huddleston 4 SW

44-4148

41.10

11.05

0.84

48.72

11.24

0.83

53.17

11.23

0.80

53.46

10.73

0.77

53.74

10.32

0.74

52.80

9.89

0.71

50.14

9.17

0.68

Hurley

44-4180

40.20

10.74

0.86

48.93

11.12

0.86

56.96

11.56

0.85

57.42

10.91

0.82

58.82

10.41

0.79

60.06

10.11

0.76

59.86

9.54

0.74

Hurley 1 SE

44-4185

39.92

10.83

0.86

49.20

11.37

0.86

56.70

11.79

0.85

58.86

11.42

0.82

60.45

10.93

0.79

61.38

10.45

0.77

64.39

10.37

0.75

Independence 2

44-4234

40.54

11.85

0.87

48.90

12.36

0.87

55.25

12.65

0.85

58.03

12.43

0.82

59.65

12.07

0.79

61.51

11.87

0.77

62.27

11.57

0.75

Indian Valley
John Flannagan
Reservo

44-4246

38.75

10.95

0.84

46.81

11.36

0.84

52.40

11.47

0.81

54.68

11.27

0.79

54.60

10.62

0.75

54.45

10.31

0.73

53.29

9.74

0.70

41.92

11.52

0.87

50.88

11.94

0.87

59.33

12.60

0.85

60.39

12.09

0.82

61.78

11.54

0.79

62.87

11.26

0.76

62.10

10.66

0.73

John H Kerr Dam

44-4414

48.04

11.36

0.85

56.91

11.71

0.85

60.35

11.69

0.82

60.88

11.24

0.79

59.21

10.55

0.75

57.68

10.10

0.72

55.00

9.42

0.69

Jordon Mines
Kerrs Creek 1
WSW

44-4452

37.79

11.37

0.85

47.23

12.00

0.86

54.31

12.33

0.84

56.36

12.01

0.81

57.24

11.59

0.78

57.60

11.34

0.76

57.67

11.04

0.74

34.55

10.54

0.82

42.09

10.90

0.82

48.82

11.24

0.80

50.16

10.87

0.78

51.08

10.46

0.75

50.66

10.01

0.72

50.05

9.62

0.70

37.46

11.18

0.85

46.18

11.73

0.85

52.03

11.96

0.83

53.76

11.71

0.80

53.84

11.17

0.77

53.44

10.87

0.74

51.37

10.20

0.72

9.35

0.69

56.83

8.56

0.66

44-4410

44-4565

10.57

0.73

Lafayette 1 NE
Langley Air Force
Base

44-4676

53.52

11.35

0.85

60.58

11.30

0.84

61.40

11.17

0.80

62.92

10.71

0.77

60.11

9.85

0.72

59.43

Lawrenceville 3 E

44-4768

50.43

11.31

0.85

59.85

11.66

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3 of 6

VDOT DRAINAGE MANUAL


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### Appendix 6C-2 B, D, and E Factors

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## B, D, and E Factors

| Location              | Code  | A   | B   | C   | D   | E   | F   | G   | H   | I   | J   | K   | L   | M   | N   | O   | P   | Q   | R   | S   | T   | U   | V   | W   | X   | Y   | Z   |
|-----------------------|-------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| Washington Reagan AP | 44-8906 | 48.41 | 11.64 | 0.86 | 59.17 | 12.12 | 0.86 | 64.80 | 12.16 | 0.83 | 65.80 | 11.88 | 0.80 | 62.99 | 10.95 | 0.76 | 60.95 | 10.35 | 0.73 | 58.74 | 9.73 | 0.70 |
| Washington WB Chantilly | 44-8903 | 43.11 | 11.17 | 0.85 | 52.08 | 11.51 | 0.84 | 57.55 | 11.60 | 0.82 | 57.91 | 11.16 | 0.79 | 55.81 | 10.28 | 0.74 | 55.52 | 9.90 | 0.72 | 54.59 | 9.38 | 0.69 |
| West Point 2 SW | 44-9025 | 48.52 | 11.11 | 0.84 | 56.84 | 11.21 | 0.83 | 59.14 | 11.15 | 0.80 | 60.65 | 10.79 | 0.77 | 58.96 | 10.10 | 0.73 | 57.21 | 9.43 | 0.70 | 55.77 | 8.92 | 0.67 |
| White Gate | 44-9060 | 40.13 | 11.92 | 0.87 | 47.61 | 12.12 | 0.87 | 54.07 | 12.40 | 0.84 | 55.89 | 12.13 | 0.82 | 57.17 | 11.82 | 0.79 | 57.71 | 11.59 | 0.76 | 56.51 | 11.08 | 0.74 |
| Williamsburg 2 N | 44-9151 | 49.10 | 10.95 | 0.84 | 59.56 | 11.34 | 0.84 | 60.95 | 11.13 | 0.80 | 59.67 | 10.41 | 0.76 | 58.00 | 9.71 | 0.72 | 56.12 | 9.04 | 0.69 | 54.53 | 8.47 | 0.66 |
| Winchester 3 ESE | 44-9186 | 34.80 | 8.75 | 0.83 | 41.67 | 9.00 | 0.83 | 45.89 | 8.65 | 0.80 | 47.04 | 8.04 | 0.77 | 47.39 | 7.22 | 0.74 | 47.88 | 6.68 | 0.71 | 47.74 | 6.05 | 0.68 |
| Wise 1 SE | 44-9215 | 42.80 | 11.95 | 0.87 | 50.92 | 12.34 | 0.87 | 58.80 | 12.90 | 0.85 | 59.64 | 12.42 | 0.82 | 61.08 | 11.94 | 0.79 | 62.13 | 11.58 | 0.76 | 62.93 | 11.25 | 0.74 |
| Woodstock 2 NE | 44-9263 | 32.91 | 8.48 | 0.82 | 40.73 | 8.92 | 0.83 | 43.92 | 8.42 | 0.79 | 45.40 | 7.87 | 0.77 | 45.39 | 6.97 | 0.73 | 44.97 | 6.26 | 0.70 | 45.00 | 5.68 | 0.68 |
| Woolwine 4 S | 44-9272 | 38.07 | 9.51 | 0.79 | 44.50 | 9.54 | 0.78 | 48.24 | 9.47 | 0.74 | 49.68 | 9.22 | 0.72 | 48.64 | 8.55 | 0.68 | 48.00 | 8.12 | 0.66 | 46.17 | 7.45 | 0.63 |
| Wytheville Post Office | 44-9301 | 40.44 | 11.99 | 0.88 | 50.04 | 12.70 | 0.88 | 53.06 | 12.58 | 0.84 | 55.24 | 12.30 | 0.82 | 58.72 | 12.09 | 0.79 | 61.32 | 12.02 | 0.77 | 62.68 | 11.62 | 0.75 |

The B, D, and E factors for the state of Virginia are also available upon request in the form of a Microsoft EXCEL spreadsheet.
Appendix 6D-1  Overland Flow Time - Seelye

Comments:
VDOT added a 'C-VALUE' scale and table and a derived equation for Overland Flow Time to this nomograph. This was done without the permission of the author in the interest of providing the user with a quantitative comparison for the selection of 'CHARACTER OF GROUND' and an optional numerical solution to the nomograph. The Department warrants neither the accuracy nor the validity of either enhancement and cautions the user that it be used at their own risk.

* Rev 9/11
Appendix 6D-2  Kinematic Wave Formulation
Overland Flow

Equation solved by nomograph:

\[ t_c \ (\text{min}) = 0.93 \frac{L^{0.6}n^{0.6}}{\rho^{0.4}s^{0.3}} \]

Comments:
VDOT has determined that the Kinematic Wave Method should only be used for:

a) Impervious Surfaces
b) \( n = 0.05 \) or less
c) Length = 300' Maximum
d) See page 2 of 2 for suggested Manning's roughness coefficients

Nomograph for determining time of concentration for overland flow,
Kinematic Wave Formulation. (After Ragan.)

Example:

\[ L = 400 \text{ ft.} \]
\[ n = 0.015 \]
\[ i = 5.5 \text{ in./hr.} \]
\[ S_0 = 0.01 \]
\[ t = 9.5 \text{ min.} \]

ONE INCH is 25.4mm
ONE FOOT is 0.3048m
### Appendix 6D-2  Mannings Roughness Coefficient for Shallow Sheet Flow

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</tr>
<tr>
<td>Grasses:</td>
<td></td>
</tr>
<tr>
<td>Short grass prairie</td>
<td>0.15</td>
</tr>
<tr>
<td>Dense grasses&lt;sup&gt;2&lt;/sup&gt;</td>
<td>0.24</td>
</tr>
<tr>
<td>Bermuda grass</td>
<td>0.41</td>
</tr>
<tr>
<td>Range (natural)</td>
<td>0.13</td>
</tr>
<tr>
<td>Woods:&lt;sup&gt;3&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>Light underbrush</td>
<td>0.40</td>
</tr>
<tr>
<td>Dense underbrush</td>
<td>0.80</td>
</tr>
</tbody>
</table>

1 The n values are a composite of information compiled by Engman (1986).
2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass and native grass mixtures.
3 When selecting n, consider cover to a height of about 1 inch. This is the only part of the plant cover that will obstruct sheet flow.

Source: AASHTO 2005 MODEL DRAINAGE MANUAL  (text shown in parentheses are VDOT additions to the original chart which were included to simplify interpretation and application)
Appendix 6D-3  Overland Time of Flow

Source:
Airport Drainage, Federal Aviation Administration, 1965
Appendix 6D-4  Overland Flow Velocity

Source:
HEC No. 19, FHWA (archived)
Appendix 6D-5

Time of Concentration for Small Drainage Basins - Kirpich

\[ T_C = 0.00948 \times H^{0.238} \times L^{1.13} \]

- \( T_C \): Flow time, minutes
- \( H \): Height, feet
- \( L \): Length, feet

**The Kirpich Chart should only be used for channel time in Virginia.**
Appendix 6D-6  
Average Velocities for Estimating Travel Time for Shallow Concentrated Flow

Source:
VDOT has determined that this nomograph produces essentially the same flow time as the "Kirpich" Method.
Recommended Coefficient of Runoff Values for Various Selected Land Uses

<table>
<thead>
<tr>
<th>Description of Area</th>
<th>Runoff Coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td>Business: Industrial and Commercial</td>
<td>0.80-0.90</td>
</tr>
<tr>
<td>Apartments and Townhomes</td>
<td>0.65-0.75</td>
</tr>
<tr>
<td>Schools</td>
<td>0.50-0.60</td>
</tr>
<tr>
<td>Residential - lots 10,000 sq. ft.</td>
<td>0.40-0.50</td>
</tr>
<tr>
<td>- lots 12,000 sq. ft.</td>
<td>0.40-0.45</td>
</tr>
<tr>
<td>- lots 17,000 sq. ft.</td>
<td>0.35-0.45</td>
</tr>
<tr>
<td>- lots ½ acre or more</td>
<td>0.30-0.40</td>
</tr>
<tr>
<td>Parks, Cemeteries and Unimproved Areas</td>
<td>0.20-0.35</td>
</tr>
<tr>
<td>Paved and Roof Areas</td>
<td>0.90</td>
</tr>
<tr>
<td>Cultivated Areas</td>
<td>0.50-0.70</td>
</tr>
<tr>
<td>Pasture</td>
<td>0.35-0.45</td>
</tr>
<tr>
<td>Lawns</td>
<td>0.25-0.35</td>
</tr>
<tr>
<td>Forest</td>
<td>0.20-0.30</td>
</tr>
<tr>
<td>Steep Grass (2:1)*</td>
<td>0.40-0.70</td>
</tr>
<tr>
<td>Shoulder and Ditch Areas *</td>
<td>0.35-0.50</td>
</tr>
</tbody>
</table>

Comments:

1. The lowest range of runoff coefficients may be used for flat areas (areas where the majority of the grades and slopes are 2% and less).
2. The average range of runoff coefficients should be used for intermediate areas (areas where the majority of the grades and slopes are from 2% to 6%).
3. The highest range of runoff coefficients shall be used for steep areas (areas where the majority of the grades are greater than 6%), for cluster areas, and for development in clay soil areas.
4. See Appendixes 6E-2, 6E-3, 6E-4 and 6E-5 for runoff coefficients with the C_f factor applied.

*Lower runoff coefficients should be used for permanent or established conditions (post-construction), i.e. sizing stormwater management basins.

*Higher runoff coefficients should be used to design roadside ditch linings (construction). The design considers the ditch lining as not yet established.

Comments: Runoff Coefficients compiled from various sources.
### Appendix 6E-2

**Rational Method Runoff Coefficients with 10 yr \( C_f \) factor Applied**

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Average Watershed Slope</th>
<th>Average % Impervious</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flat &lt;2%</td>
<td>Rolling 2% - 6%</td>
</tr>
<tr>
<td>Business, Commercial &amp; Industrial</td>
<td>0.8</td>
<td>0.85</td>
</tr>
<tr>
<td>Apartments and Townhomes</td>
<td>0.65</td>
<td>0.70</td>
</tr>
<tr>
<td>Schools</td>
<td>0.50</td>
<td>0.55</td>
</tr>
<tr>
<td>Residential</td>
<td></td>
<td></td>
</tr>
<tr>
<td>lots 10,000 sq. ft</td>
<td>0.40</td>
<td>0.45</td>
</tr>
<tr>
<td>lots 12,000 sq. ft.</td>
<td>0.40</td>
<td>0.43</td>
</tr>
<tr>
<td>lots 17,000 sq. ft.</td>
<td>0.35</td>
<td>0.40</td>
</tr>
<tr>
<td>lots ½ acre or more</td>
<td>0.30</td>
<td>0.35</td>
</tr>
<tr>
<td>Parks, Cemeteries and Unimproved Areas</td>
<td>0.20</td>
<td>0.28</td>
</tr>
<tr>
<td>Paved and Roof Areas</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>Cultivated Areas</td>
<td>0.50</td>
<td>0.60</td>
</tr>
<tr>
<td>Pasture</td>
<td>0.35</td>
<td>0.40</td>
</tr>
<tr>
<td>Lawns</td>
<td>0.25</td>
<td>0.30</td>
</tr>
<tr>
<td>Forest</td>
<td>0.20</td>
<td>0.25</td>
</tr>
<tr>
<td>Railroad Yard Areas</td>
<td>0.20</td>
<td>0.30</td>
</tr>
<tr>
<td>Roadway Slopes (2:1) w/ Little or No Vegetated Cover</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roadway Shoulder &amp; Ditch Areas w/ Little or No Vegetated Cover</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roadway Slopes (2:1) w/ Established Vegetated Cover</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roadway Shoulder &amp; Ditch Areas w/ Established Vegetated Cover</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Rational Formula (Revised) – \( Q = C_{cr} A i^\dagger \)

\[ \dagger \text{Source: VDOT} \]
### Rational Method Runoff Coefficients with 25 yr $C_f$ factor Applied

#### $C_{cf}$ Values for 50 Year Storm Frequency ($C_f=1.1$)

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Average Watershed Slope</th>
<th>Average % Impervious</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flat &lt;2%</td>
<td>Rolling 2% - 6%</td>
</tr>
<tr>
<td>Business, Commercial &amp; Industrial</td>
<td>0.88</td>
<td>0.94</td>
</tr>
<tr>
<td>Apartments and Townhomes</td>
<td>0.72</td>
<td>0.77</td>
</tr>
<tr>
<td>Schools</td>
<td>0.55</td>
<td>0.61</td>
</tr>
<tr>
<td>Residential</td>
<td></td>
<td></td>
</tr>
<tr>
<td>lots 10,000 sq. ft</td>
<td>0.44</td>
<td>0.50</td>
</tr>
<tr>
<td>lots 12,000 sq. ft</td>
<td>0.44</td>
<td>0.47</td>
</tr>
<tr>
<td>lots 17,000 sq. ft</td>
<td>0.39</td>
<td>0.44</td>
</tr>
<tr>
<td>lots ½ acre or more</td>
<td>0.33</td>
<td>0.39</td>
</tr>
<tr>
<td>Parks, Cemeteries and Unimproved Areas</td>
<td>0.22</td>
<td>0.30</td>
</tr>
<tr>
<td>Paved and Roof Areas</td>
<td>0.99</td>
<td></td>
</tr>
<tr>
<td>Cultivated Areas</td>
<td>0.55</td>
<td>0.66</td>
</tr>
<tr>
<td>Pasture</td>
<td>0.39</td>
<td>0.44</td>
</tr>
<tr>
<td>Lawns</td>
<td>0.28</td>
<td>0.33</td>
</tr>
<tr>
<td>Forest</td>
<td>0.22</td>
<td>0.28</td>
</tr>
<tr>
<td>Railroad Yard Areas</td>
<td>0.22</td>
<td>0.33</td>
</tr>
<tr>
<td>Roadway Slopes (2:1) w/ Little or No Vegetated Cover</td>
<td>0.77</td>
<td></td>
</tr>
<tr>
<td>Roadway Shoulder &amp; Ditch Areas w/ Little or No Vegetated Cover</td>
<td>0.55</td>
<td></td>
</tr>
<tr>
<td>Roadway Slopes (2:1) w/ Established Vegetated Cover</td>
<td>0.44</td>
<td></td>
</tr>
<tr>
<td>Roadway Shoulder &amp; Ditch Areas w/ Established Vegetated Cover</td>
<td>0.39</td>
<td></td>
</tr>
</tbody>
</table>

Rational Formula (Revised) – $Q = C_{cf} \ A \ i^\dagger$

---

$\dagger$ Source: VDOT
### Appendix 6E-4 Rational Method Runoff Coefficients with 50 yr $C_f$ factor Applied

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Average Watershed Slope</th>
<th>Average % Impervious</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flat &lt;2%</td>
<td>Rolling 2% - 6%</td>
</tr>
<tr>
<td>Business, Commercial &amp; Industrial</td>
<td>0.96</td>
<td>1.00</td>
</tr>
<tr>
<td>Apartments and Townhomes</td>
<td>0.78</td>
<td>0.84</td>
</tr>
<tr>
<td>Schools</td>
<td>0.6</td>
<td>0.66</td>
</tr>
<tr>
<td>Residential</td>
<td></td>
<td></td>
</tr>
<tr>
<td>lots 10,000 sq. ft</td>
<td>0.48</td>
<td>0.54</td>
</tr>
<tr>
<td>lots 12,000 sq. ft</td>
<td>0.48</td>
<td>0.51</td>
</tr>
<tr>
<td>lots 17,000 sq. ft</td>
<td>0.42</td>
<td>0.48</td>
</tr>
<tr>
<td>lots ½ acre or more</td>
<td>0.36</td>
<td>0.42</td>
</tr>
<tr>
<td>Parks, Cemeteries and Unimproved Areas</td>
<td>0.24</td>
<td>0.33</td>
</tr>
<tr>
<td>Paved and Roof Areas</td>
<td>1.00</td>
<td></td>
</tr>
<tr>
<td>Cultivated Areas</td>
<td>0.6</td>
<td>0.72</td>
</tr>
<tr>
<td>Pasture</td>
<td>0.42</td>
<td>0.48</td>
</tr>
<tr>
<td>Lawns</td>
<td>0.3</td>
<td>0.36</td>
</tr>
<tr>
<td>Forest</td>
<td>0.24</td>
<td>0.30</td>
</tr>
<tr>
<td>Railroad Yard Areas</td>
<td>0.24</td>
<td>0.36</td>
</tr>
<tr>
<td>Roadway Slopes (2:1) w/ Little or No Vegetated Cover</td>
<td>0.84</td>
<td></td>
</tr>
<tr>
<td>Roadway Shoulder &amp; Ditch Areas w/ Little or No Vegetated Cover</td>
<td>0.60</td>
<td></td>
</tr>
<tr>
<td>Roadway Slopes (2:1) w/ Established Vegetated Cover</td>
<td>0.48</td>
<td></td>
</tr>
<tr>
<td>Roadway Shoulder &amp; Ditch Areas w/ Established Vegetated Cover</td>
<td>0.42</td>
<td></td>
</tr>
</tbody>
</table>

**Rational Formula** (Revised) – $Q = C_{Cr} A i^\dagger$

\dagger Source: VDOT
### Appendix 6E-5  
**Rational Method Runoff Coefficients with 100 yr. \( C_f \) factor Applied**

<table>
<thead>
<tr>
<th>Land Use</th>
<th>Average Watershed Slope</th>
<th>Average % Impervious</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flat &lt;2%</td>
<td>Rolling 2% - 6%</td>
</tr>
<tr>
<td>Business, Commercial &amp; Industrial</td>
<td>1.00</td>
<td>0.81</td>
</tr>
<tr>
<td>Apartments and Townhomes</td>
<td>0.88</td>
<td>0.63</td>
</tr>
<tr>
<td>Schools</td>
<td>0.69</td>
<td>0.50</td>
</tr>
<tr>
<td>Residential</td>
<td></td>
<td></td>
</tr>
<tr>
<td>lots 10,000 sq. ft</td>
<td>0.50</td>
<td>0.56</td>
</tr>
<tr>
<td>lots 12,000 sq. ft</td>
<td>0.50</td>
<td>0.53</td>
</tr>
<tr>
<td>lots 17,000 sq. ft</td>
<td>0.44</td>
<td>0.50</td>
</tr>
<tr>
<td>lots ½ acre or more</td>
<td>0.38</td>
<td>0.44</td>
</tr>
<tr>
<td>Parks, Cemeteries and Unimproved Areas</td>
<td>0.25</td>
<td>0.34</td>
</tr>
<tr>
<td>Paved and Roof Areas</td>
<td>1.00</td>
<td>0.75</td>
</tr>
<tr>
<td>Cultivated Areas</td>
<td>0.31</td>
<td>0.38</td>
</tr>
<tr>
<td>Pasture</td>
<td>0.44</td>
<td>0.50</td>
</tr>
<tr>
<td>Lawns</td>
<td>0.25</td>
<td>0.31</td>
</tr>
<tr>
<td>Forest</td>
<td>0.25</td>
<td>0.38</td>
</tr>
<tr>
<td>Railroad Yard Areas</td>
<td>0.25</td>
<td>0.38</td>
</tr>
<tr>
<td>Roadway Slopes (2:1) w/ Little or No Vegetated Cover</td>
<td>0.88</td>
<td></td>
</tr>
<tr>
<td>Roadway Shoulder &amp; Ditch Areas w/ Little or No Vegetated Cover</td>
<td>0.63</td>
<td></td>
</tr>
<tr>
<td>Roadway Slopes (2:1) w/ Established Vegetated Cover</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>Roadway Shoulder &amp; Ditch Areas w/ Established Vegetated Cover</td>
<td>0.44</td>
<td></td>
</tr>
</tbody>
</table>

**Rational Formula** (Revised) – \( Q = C_{fr} \times A \times i^* \)

\(^*\) Source: VDOT
The rainfall IDF curves are described by the equation:

\[ i = \frac{a}{b + t_c} \]

Where:

- \( i \) = Intensity, inches per hour (in/hr)
- \( t_c \) = Rainfall duration, minutes (min)

The a and b factors describing the 2, 10 and 100-year IDF curves are provided in Appendix 6B-2.

The a and b factors are not based on NOAA “Atlas 14” Rainfall Precipitation Frequency data and are therefore to be used only in conjunction with Equation 11.5 that estimates the “Critical Storm Duration” (\( T_d \)).
### Appendix 6K-2 Regression Constants a and b for Virginia

<table>
<thead>
<tr>
<th>COUNTY</th>
<th>#</th>
<th>2 YEAR</th>
<th></th>
<th>10 YEAR</th>
<th></th>
<th>100 YEAR</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
<td>A</td>
<td>B</td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>Arlington</td>
<td>00</td>
<td>119.34</td>
<td>17.86</td>
<td>178.78</td>
<td>20.66</td>
<td>267.54</td>
<td>22.32</td>
</tr>
<tr>
<td>Accomack</td>
<td>01</td>
<td>107.75</td>
<td>14.69</td>
<td>175.90</td>
<td>20.64</td>
<td>277.44</td>
<td>24.82</td>
</tr>
<tr>
<td>Albemarle</td>
<td>02</td>
<td>106.02</td>
<td>15.51</td>
<td>161.60</td>
<td>18.73</td>
<td>244.82</td>
<td>20.81</td>
</tr>
<tr>
<td>Amelia</td>
<td>03</td>
<td>95.47</td>
<td>13.98</td>
<td>145.89</td>
<td>17.27</td>
<td>220.94</td>
<td>19.29</td>
</tr>
<tr>
<td>Amherst</td>
<td>04</td>
<td>112.68</td>
<td>15.11</td>
<td>173.16</td>
<td>18.81</td>
<td>266.77</td>
<td>22.13</td>
</tr>
<tr>
<td>Accomack</td>
<td>05</td>
<td>106.72</td>
<td>15.39</td>
<td>162.75</td>
<td>20.47</td>
<td>254.03</td>
<td>21.61</td>
</tr>
<tr>
<td>Appomattox</td>
<td>06</td>
<td>109.11</td>
<td>15.39</td>
<td>167.44</td>
<td>19.12</td>
<td>247.92</td>
<td>22.16</td>
</tr>
<tr>
<td>Augusta</td>
<td>07</td>
<td>84.21</td>
<td>10.44</td>
<td>135.74</td>
<td>14.54</td>
<td>210.02</td>
<td>16.99</td>
</tr>
<tr>
<td>Bedford</td>
<td>09</td>
<td>114.59</td>
<td>17.21</td>
<td>171.51</td>
<td>20.47</td>
<td>258.17</td>
<td>22.80</td>
</tr>
<tr>
<td>Bland</td>
<td>10</td>
<td>105.33</td>
<td>16.56</td>
<td>162.75</td>
<td>20.41</td>
<td>247.84</td>
<td>22.87</td>
</tr>
<tr>
<td>Botetourt</td>
<td>11</td>
<td>110.32</td>
<td>16.95</td>
<td>164.94</td>
<td>20.01</td>
<td>247.92</td>
<td>22.16</td>
</tr>
<tr>
<td>Brunswick</td>
<td>12</td>
<td>126.74</td>
<td>17.27</td>
<td>190.73</td>
<td>21.52</td>
<td>287.02</td>
<td>24.46</td>
</tr>
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<td>Buchanan</td>
<td>13</td>
<td>87.14</td>
<td>13.22</td>
<td>128.51</td>
<td>15.15</td>
<td>189.98</td>
<td>16.22</td>
</tr>
<tr>
<td>Buckingham</td>
<td>14</td>
<td>109.95</td>
<td>15.41</td>
<td>168.28</td>
<td>19.11</td>
<td>254.59</td>
<td>21.47</td>
</tr>
<tr>
<td>Campbell</td>
<td>15</td>
<td>110.26</td>
<td>15.76</td>
<td>167.27</td>
<td>19.18</td>
<td>252.65</td>
<td>21.56</td>
</tr>
<tr>
<td>Caroline</td>
<td>16</td>
<td>121.21</td>
<td>17.33</td>
<td>182.56</td>
<td>20.88</td>
<td>275.65</td>
<td>23.30</td>
</tr>
<tr>
<td>Carroll</td>
<td>17</td>
<td>119.79</td>
<td>18.65</td>
<td>188.13</td>
<td>23.81</td>
<td>288.94</td>
<td>27.06</td>
</tr>
<tr>
<td>Charles City</td>
<td>18</td>
<td>124.23</td>
<td>17.14</td>
<td>186.52</td>
<td>21.05</td>
<td>281.04</td>
<td>23.85</td>
</tr>
<tr>
<td>Charlotte</td>
<td>19</td>
<td>109.87</td>
<td>14.71</td>
<td>171.75</td>
<td>19.25</td>
<td>265.18</td>
<td>22.56</td>
</tr>
<tr>
<td>Chesterfield</td>
<td>20</td>
<td>124.66</td>
<td>17.55</td>
<td>186.15</td>
<td>21.03</td>
<td>277.94</td>
<td>23.26</td>
</tr>
<tr>
<td>Clarke</td>
<td>21</td>
<td>94.13</td>
<td>12.88</td>
<td>141.03</td>
<td>15.39</td>
<td>210.66</td>
<td>16.85</td>
</tr>
<tr>
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# Chapter 7 — Ditches and Channels

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<td>7.4.1</td>
<td>Minor Channels (Roadside Ditches)</td>
<td>9</td>
</tr>
<tr>
<td>7.4.1.1</td>
<td>General</td>
<td>9</td>
</tr>
<tr>
<td>7.4.1.2</td>
<td>Design Considerations</td>
<td>9</td>
</tr>
<tr>
<td>7.4.2</td>
<td>Major Channels</td>
<td>10</td>
</tr>
<tr>
<td>7.4.2.1</td>
<td>General</td>
<td>10</td>
</tr>
<tr>
<td>7.4.2.2</td>
<td>Flow Classifications</td>
<td>10</td>
</tr>
<tr>
<td>7.4.3</td>
<td>Natural Channels</td>
<td>11</td>
</tr>
<tr>
<td>7.4.3.1</td>
<td>Stream Morphology</td>
<td>11</td>
</tr>
<tr>
<td>7.4.3.1.1</td>
<td>Introduction</td>
<td>11</td>
</tr>
<tr>
<td>7.4.4</td>
<td>Channel Analysis</td>
<td>11</td>
</tr>
<tr>
<td>7.4.4.1</td>
<td>General</td>
<td>11</td>
</tr>
<tr>
<td>7.4.4.2</td>
<td>Equations</td>
<td>12</td>
</tr>
<tr>
<td>7.4.4.2.1</td>
<td>Specific Energy</td>
<td>12</td>
</tr>
<tr>
<td>7.4.4.2.2</td>
<td>Velocity Distribution Coefficient</td>
<td>13</td>
</tr>
<tr>
<td>7.4.4.2.3</td>
<td>Total Energy Head</td>
<td>14</td>
</tr>
<tr>
<td>7.4.4.2.4</td>
<td>Froude Number</td>
<td>14</td>
</tr>
<tr>
<td>7.4.4.2.5</td>
<td>Critical Flow</td>
<td>15</td>
</tr>
<tr>
<td>7.4.4.2.6</td>
<td>Subcritical Flow</td>
<td>15</td>
</tr>
<tr>
<td>7.4.4.2.7</td>
<td>Supercritical Flow</td>
<td>15</td>
</tr>
<tr>
<td>7.4.4.2.8</td>
<td>Continuity Equation</td>
<td>15</td>
</tr>
<tr>
<td>7.4.4.2.9</td>
<td>Manning's Equation</td>
<td>16</td>
</tr>
<tr>
<td>7.4.4.2.10</td>
<td>Conveyance</td>
<td>16</td>
</tr>
<tr>
<td>7.4.4.2.11</td>
<td>Energy Equation</td>
<td>17</td>
</tr>
<tr>
<td>7.4.4.3</td>
<td>Hydraulic Representation of Channels</td>
<td>18</td>
</tr>
<tr>
<td>7.4.4.3.1</td>
<td>Cross Sections</td>
<td>18</td>
</tr>
<tr>
<td>7.4.4.3.2</td>
<td>Manning's n-Value Selection</td>
<td>19</td>
</tr>
<tr>
<td>7.4.4.3.3</td>
<td>Calibration</td>
<td>20</td>
</tr>
<tr>
<td>7.4.4.3.4</td>
<td>Switchback Phenomenon</td>
<td>20</td>
</tr>
</tbody>
</table>
List of Figures and Tables

Figure 7-1. Sample Roadside Ditch Plan and Profile ................................................. 6
Figure 7-2. Specific Energy Diagram for Rectangular Channels ............................. 13
Figure 7-3. Terms in the Energy Equation ............................................................. 18
Figure 7-4. Hypothetical Cross Section Showing Reaches, Segments, and Subsections Used in Assigning n-Values ................................................................. 19
Figure 7-5. Example of Switchback Phenomenon ............................................... 20
Figure 7-6. Energy Slope between Two Channel Sections ................................. 23
Figure 7-7. Notation for Classifying Water Surface Profiles ............................... 24
Figure 7-8. Types of Backwater Profiles ............................................................... 25
Table 7-1. Allowable Velocity and Shear Stress Values for Lined Ditches ............... 32
Table 7-2A. Recommended Manning’s n-Values for Lined Ditches ...................... 33
Figure 7-9. Roadside Ditch Protection Sample Problem, Plan and Section ............ 42
Figure 7-10. Worksheet (LD-268) for Calculation of Roadside Ditch Protection, Sample Problem ........................................................................................................ 43
Figure 7-11. Roadside Ditch Protection Sample Problem, Plan and Section ............ 46
Figure 7-12. Worksheet (LD-268) for Calculation of Roadside Ditch Protection, Sample Problem ........................................................................................................ 46
Figure 7-13. Water Surface Profile Sample Problem ............................................. 50
Figure 7-14. Worksheet for Calculation of Non-Uniform Flow in Open Channels, Sample Problem ........................................................................................................ 51
Table 7-3. Manning’s n-Values for Depth Ranges .................................................. 62
List of Appendices

Appendix 7B-1  LD-268V Roadside and Median Ditch Design Form – Permissible Velocity
  LD-268S Roadside and Median Ditch Design Form – Tractive Force (Permissible Shear)
Appendix 7B-2  Water Surface Profile Calculation Form
Appendix 7B-3  Channel Stability Work Sheet
Appendix 7B-4  Riprap Design Work Sheet for Standard VDOT Riprap Sizes Only
Appendix 7B-5  Riprap Design Work Sheet for Other Than VDOT Standard Riprap Sizes
Appendix 7B-6  Natural Channel Design Project Summary Sheet
Appendix 7C-1  Nomograph for Solution of Manning's Equation
Appendix 7C-2  Trapezoidal Channel Capacity Chart
Appendix 7C-3  Nomograph for Solution of Normal Depth
Appendix 7C-4  Side Ditch Flow Chart (Side Slopes = 6:1, 1.5:1)
Appendix 7C-5  Side Ditch Flow Chart (Side Slopes = 4:1, 1:1)
Appendix 7C-6  Side Ditch Flow Chart (Side Slopes = 4:1, 1.5:1)
Appendix 7C-7  Side Ditch Flow Chart (Side Slopes = 3:1, 2:1)
Appendix 7C-8  Side Ditch Flow Chart (Side Slopes = 3:1, 1.5:1)
Appendix 7C-9  Side Ditch Flow Chart (Side Slopes = 3:1, 2:1)
Appendix 7C-10 Side Ditch Flow Chart (Side Slopes = 6:1, 2:1)
Appendix 7C-11 Side Ditch Flow Chart (Side Slopes = 6:1, 4:1)
Appendix 7C-12 Triangular Median Ditch Flow Chart (Side Slopes = 6:1, 6:1)
Appendix 7C-13 Triangular Median Ditch Flow Chart (Side Slopes = 4:1, 4:1)
Appendix 7C-14 Triangular Median Ditch Flow Chart (Side Slopes = 2:1, 2:1)
Appendix 7C-15 Toe Ditch Flow Chart (Side Slopes = 1.5:1, 1.5:1)
Appendix 7C-16 Standard PG-4 Flow Chart
Appendix 7C-17 Trapezoidal Ditch Flow Chart (B=1’, Side Slopes = 1.5:1)
Appendix 7C-18 Trapezoidal Channel Flow Chart (B=2’, Side Slopes = 2:1)
Appendix 7C-19 Trapezoidal Channel Flow Chart (B=3’, Side Slopes = 2:1)
Appendix 7C-20 Trapezoidal Channel Flow Chart (B=4’, Side Slopes = 2:1)
Appendix 7C-21 Trapezoidal Channel Flow Chart (B=5’, Side Slopes = 2:1)
Appendix 7C-22 Trapezoidal Channel Flow Chart (B=6’, Side Slopes = 2:1)
Appendix 7C-23 Trapezoidal Channel Flow Chart (B=7’, Side Slopes = 2:1)
Appendix 7C-24  Trapezoidal Channel Flow Chart (B=8’, Side Slopes = 2:1)
Appendix 7C-25  Trapezoidal Channel Flow Chart (B=9’, Side Slopes = 2:1)
Appendix 7C-26  Trapezoidal Channel Flow Chart (B=10’, Side Slopes = 2:1)
Appendix 7C-27  Trapezoidal Channel Flow Chart (B=12’, Side Slopes = 2:1)
Appendix 7C-28  Trapezoidal Channel Flow Chart (B=14’, Side Slopes = 2:1)
Appendix 7C-29  Trapezoidal Channel Flow Chart (B=16’, Side Slopes = 2:1)
Appendix 7C-30  Trapezoidal Channel Flow Chart (B=18’, Side Slopes = 2:1)
Appendix 7C-31  Trapezoidal Channel Flow Chart (B=20’, Side Slopes = 2:1)
Appendix 7D-1  Values of Roughness Coefficient n (Uniform Flow)
Appendix 7D-2  Recommended Maximum Water Velocities and Manning’s n as a
Function of Soil Type and Flow Depth
Appendix 7D-3  Standard VDOT Riprap Classifications, Weights, and Blanket Thickness
Appendix 7D-4  Approximate Rock Dimensions and Equivalent Weights for Riprap
Appendix 7D-5  Selection of Stability Factors
Appendix 7D-6  Permissible Velocities for Erodible Linings
Appendix 7E-1  Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone
Appendix 7E-2  Permissible Shear Stress for Non-Cohesive Soils
Appendix 7E-3  Permissible Shear Stress for Cohesive Soils
Appendix 7E-4  Bank Angle Correction Factor (K₁) Nomograph
Appendix 7E-5  Correction Factor for Riprap Size
Appendix 7E-6  Riprap Size Relationship
Appendix 7E-7  Channel Side Shear Stress to Bottom Shear Stress Ratio
Appendix 7E-8  Tractive Force Ration (K₂)
Appendix 7E-9  Determination of the Mean Spherical Diameter
Chapter 7 — Ditches and Channels

7.1 Introduction

The function of ditches and open channels is to convey stormwater runoff from, though, or around roadway rights-of-way without damage to the highway, to the open channel, to other components of the highway system, or to adjacent property. Culverts and storm drains can be used for the same purposes. Open channels may be natural or constructed. In either case, the water surface is exposed to the atmosphere, and the gravity force component in the direction of motion is the driving force. Open channels are free to overflow their banks, and cannot develop pressure flow, as can closed conduits such as circular pipes. However, closed conduits flow as open channels when the water surface is below the crown of the conduit, and the design concepts of this chapter apply to closed conduits flowing partly full.

*Rev. 9/09
7.2 Design Policy

Following are Federal, Commonwealth of Virginia, and Virginia Department of Transportation (VDOT) design policies related to channel design.

7.2.1 Federal Policy

Channel designs and/or designs of highway facilities that impact channels should satisfy the policies of the Federal Highway Administration applicable to floodplain management. Federal Emergency Management Agency (FEMA) floodway regulations and Corps of Engineers (COE) wetland restrictions for permits should also be satisfied.

7.2.2 Commonwealth of Virginia Policy

7.2.2.1 Adequate Receiving Channels

The Virginia Erosion and Sediment Control Regulations, Minimum Standard 19 (VESCR MS-19 <http://www.deq.virginia.gov/>) requires that properties and waterways, downstream from new development sites, shall be protected from sediment deposition, erosion, and damage due to increases in the volume, velocity, and peak flow rate of stormwater runoff for the stated frequency storm of 24-hour duration. Design criteria for adequate channels are summarized in Section 7.3.2.

7.2.3 VDOT Policy

The following statements represent VDOT goals for ditch and channel design:

- Coordination with other Federal, State and local agencies concerned with water resources planning has high priority in the planning of highway facilities
- Safety of the general public is an important consideration in the selection of the cross-sectional geometries of artificial drainage channels
- The design of artificial drainage channels or other facilities should consider the frequency and type of maintenance expected and make allowance for maintenance access
- Stability is the goal for all channels that are located on highway right-of-way or that impact highway facilities
- Environmental impacts of channel modifications, including disturbance of fish habitat, wetlands and channel stability, should be assessed

________________________________________

*Rev. 7/16
• The range of design channel discharges should be selected by the designer based on class of roadway, consequences of traffic interruption, flood hazard risks, economics, and local site conditions.

• Wherever possible, encroachment into streams should be avoided and encroachment onto flood plains should be minimized to the fullest extent practical.

• Whenever natural 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, and will incorporate any necessary natural channel design and/or stream restoration.

• Roadside ditches and channels should have adequate gradient to outfall the roadway for the design discharge. “Grade to Drain” shall not be noted on the plans, as it truly does not prove that adequate discharge persists with the design.*

A thorough analysis of the stream’s morphology and environment shall be conducted and documented in addition to the economic and engineering alternatives available for the particular location.

*Rev. 7/16
7.3 Design Criteria

7.3.1 Roadside Ditches and Channel Classifications

In this chapter, ditches and channels are classified as:

7.3.1.1 Minor Channels (Roadside Ditches)

Minor channels collect sheet flow from the highway pavement or right-of-way and convey that flow to collection points in larger channels or pipes. Flows are generally 50 cfs or less in minor channels. Minor channels usually parallel the highway embankment and are within the highway R/W. See Section 7.3.3 for minor channel criteria.

7.3.1.2 Major Channels (Drainage Channels)

Major channels collect drainage from minor channels, pipe systems, and offsite areas, and convey that flow to an adequate discharge point on- or offsite. Flows are generally greater than 50 cfs in major channels. See Section 7.3.4 for major channel criteria.

7.3.1.3 Natural Channels

Natural channels are formed through geomorphologic activity, including erosion and sedimentation. Generally meandering and irregular in cross-section, natural channels may convey any flow rate. See Section 7.3.5 for natural channel criteria.

7.3.2 Adequate Receiving Channels

Minimum Standard 19 (Virginia Erosion and Sediment Control Handbook) for adequate receiving channels establishes the following design criteria for all channels.

Concentrated stormwater runoff leaving a development site must be discharged directly into an adequate well-defined, natural or man-made offsite receiving channel, pipe, or storm sewer system. If there is no adequate offsite receiving channel or pipe, one must be constructed to convey stormwater to the nearest adequate channel. Newly constructed channels shall be designed as adequate channels.

An adequate channel is defined as follows: (1) A natural channel, which is capable of conveying the runoff from a 2-yr storm without overtopping its banks or eroding after development of the site in question, (2) A previously constructed man-made channel shall be capable of conveying the runoff from a 10-yr storm without overtopping its banks, and bed or bank erosion shall not occur due to a 2-yr storm, (3) Pipes and storm sewer systems shall contain the 10-yr storm.

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A receiving channel may also be considered adequate at any point where the total contributing drainage area is at least 100 times greater than the drainage area of the development site in question; or, if it can be shown that the peak rate of runoff from the site during a 2-yr storm will not be increased after development.

Runoff rate and channel adequacy must be verified with acceptable engineering calculations. Refer to Chapter 6, Hydrology, for peak discharge methods.

If an existing offsite receiving channel is not an adequate channel, the applicant must choose one of the following options:

- Obtain permission from downstream property owners to improve the receiving channel to an adequate condition. Such improvements should extend downstream until an adequate channel section is reached.
- Develop a site design that will not cause the pre-development peak runoff rate from a 2-yr storm to increase when outfall is into a natural channel, or will not cause the pre-development rate from a 10-yr storm to increase when outfall is to a man-made channel. Such a design may be accomplished by enhancing the infiltration capability of the site or by providing on-site stormwater detention measures. The pre-development and post-development peak runoff rates must be verified by engineering calculations.
- Provide a combination of channel improvement, stormwater detention, or other measures which are satisfactory to prevent downstream channel erosion.

All channel improvements or modifications must comply with all applicable laws and regulations.

Increased volumes of unconcentrated sheet flows which may cause erosion or sedimentation of adjacent property must be diverted to an adequate outlet or detention facility.

Outfall from a detention facility shall be discharged to an adequate channel. Outlet protection and/or energy dissipation should be placed at the discharge point as necessary.

### 7.3.3 Minor Channels (Roadside Ditches)

Minor channels are normally V-shaped and sometimes trapezoidal in cross section and lined with grass or a protective lining. They are usually designed to convey the 10-yr discharge, and to resist erosion from the 2-yr discharge. Higher design discharges may be necessary when the channel intercepts offsite drainage.

Special design ditches are designed for storm frequencies appropriate to the functional classification of the roadway and the risk involved when the design capacity is exceeded.
A secondary function of a roadside ditch is to 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 such as pipe underdrains.

The alignment, cross-section, and grade of roadside ditches are usually constrained largely by the geometric and safety standards applicable to the project. These ditches should accommodate the design runoff in a manner that assures the safety of motorists and minimizes future maintenance, damage to adjacent properties, and adverse environmental, or aesthetic effects. A sample roadside ditch plan and profile is shown in Figure 7-1. The VDOT Method for Design of Roadside Ditch Linings (Section 7.5.2.2) is recommended for use on minor channels.

![Figure 7-1 Sample Roadside Ditch Plan and Profile](image)

*It is not recommended to specify non-standard or atypical roadside ditches for highway projects, due to safety and economical concerns. If the designer chooses to specify an atypical ditch section, the designer should minimize its use to the greatest extent possible. Where the volume, flow, or other considerations dictate enlarging or deepening the roadside ditch or otherwise deviating from the standard designs, careful consideration must be given to the following:

*Rev. 7/14
• Using an enclosed drainage system, where economically feasible, in order to eliminate the need for the non-standard or a typical roadside ditch or channel
• Minimizing the size and depth of the proposed non-standard or atypical roadside ditch or channel
• Flattening the front slope (the slope adjacent to the highway shoulder) of the non-standard or atypical roadside ditch or channel. Where right of way is available, or can reasonably be obtained, the front slope of the non-standard or atypical roadside ditch or channel should be no steeper than the front slope of the standard roadside ditch for the specific roadway classification involved
• Locating necessary non-standard or atypical roadside ditches or channels as far from the proposed highway shoulder as the existing or proposed right of way will reasonably allow

7.3.4 Major Channels

Major channels may be within or outside the highway right-of-way. The same design criteria apply as for minor channels. Conveyance is usually based on the 10-yr storm but may be greater based upon risk. Erosive protection is based on the 2-yr storm. Major channels are usually trapezoidal in cross section. One foot or more of freeboard is recommended for larger channels where the consequences of overtopping are significant. The consequences of failure are usually more severe for major channels; therefore, a higher level of engineering analysis and design is usually justified for major channels.

7.3.5 Natural Channels

The hydraulic effects of floodplain encroachments should be evaluated over a full range of frequency-based peak discharges from the 2-yr through the 500-yr recurrence intervals on any major highway facility, as deemed necessary by the Department. The hydrologic and hydraulic analysis procedure and required documentation for such situations are more fully described in Chapter 12, Bridge, Structure and Riverine* Hydraulics. If the floodplain encroachment is located in a FEMA or other officially delineated floodplain, no increase in the established natural 100-yr flood level will be permitted either up or downstream. It should be noted that the Department’s criteria is more stringent than FEMA’s in this instance. In situations where no FEMA or other officially delineated floodplain exists, it will be acceptable to increase the level of the 100-yr flood event not to exceed one foot up or downstream, provided such increase does not adversely impact adjacent properties, buildings, etc. If an increase in the 100-yr flood level will cause such adverse impact then no increase shall be permitted. The Department’s State Hydraulics Engineer must approve exceptions to either of the above criteria.

*Rev. 7/16
If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions as far as practical. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.

Streambank stabilization should be provided when appropriate, because of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site.

Relocation of major streams is complex, and special expertise in river mechanics engineering and appropriate natural channel and/or stream restoration design may be necessary.

Natural channel design principles will be used, to the extent practicable, in all stream restoration and relocation projects.

Natural channel design principles are encouraged as a means of effectively addressing localized bank erosion.

VDOT recognizes the value and importance of peer review and Quality Assurance/Quality Control (QA/QC) processes in delivering quality products within established and accepted time frames. An interdisciplinary QA/QC team managed by the Central Office Environmental Division, Natural Resources Section, will administer this process for all natural channel design projects.

All in-stream activities require coordination with district environmental staff to ensure that water quality permits are obtained and environmental commitments adhered to, as required. The project manager is responsible for ensuring that coordination is conducted.

*Rev. 7/16
Design Concepts

7.4.1 Minor Channels (Roadside Ditches)

7.4.1.1 General

Design discharges (peak flows) should be determined by the Rational Method as defined in Chapter 6, Hydrology.

Velocity should be based on normal depth computed using Manning’s equation. Manning’s equation requires information on the ditch geometry, such as side slopes, the longitudinal grade, and the appropriate Manning’s n-value.

Ditch side slopes should not exceed the angle of repose of the soil and/or lining and should have a maximum slope of 2H:1V or flatter. See Section 7.4.6 for further discussion on channel linings.

Figure 7-1 shows a sample plan and profile for a roadside ditch.

7.4.1.2 Design Considerations

Roadside ditch design involves both capacity and erosion resistance. A trial-and-error process may be necessary to obtain the optimum design. More information on roadside ditch design procedures is contained in Section 7.5.

The VDOT method for design of roadside ditch linings is recommended for use on minor channels. Consideration should be given to ditch bends, steep slopes, and composite linings, which are further defined in Section 7.4.6, HEC-11, and HEC-15. The riprap design procedures described in HEC-15 are for minor channels having a design discharge of 50 cfs or less. When the design discharge exceeds 50 cfs, the design procedures presented in HEC-11 should be followed for riprap-lined channels. HEC-15 may be used for design of larger channels with linings other than riprap.

Except where severe right-of-way limitations exist, a minimum of 5’ is to be provided between the end of the cut slope round-off and the front slope of a berm ditch. Severe right-of-way limitations may include, but are not limited to, adverse environmental impacts, significant distance to tie to existing grade, and property damage. Additional right-of-way is to be obtained for construction and maintenance of the berm ditch.

Except where severe right-of-way limitations exist, a minimum of 5’ is to be provided between the toe of the fill slope and the front slope of a toe ditch. Severe right-of-way limitations may include, but are not limited to, adverse environmental impacts, significant distance to tie to existing grade, and property damage. Additional right of way is to be obtained for construction and maintenance of the ditch.

*Rev. 7/19
7.4.2  Major Channels

7.4.2.1  General

Design analysis of both natural and artificial channels proceeds according to the basic principles of open channel flow (see Chow, 1959; Henderson, 1966). The basic principles of fluid mechanics, continuity, momentum, and 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 principal problems of open channel flow analysis and it depends on quantification of the flow resistance. Natural channels display a much wider range of roughness values than do artificial channels.

7.4.2.2  Flow Classifications

The classifications of open channel flow are summarized as follows:

Steady Flow (Rate of flow remains constant with time)
1. Uniform Flow (Velocity and depth of flow remain constant over length)
2. Non-uniform Flow (Velocity and depth of flow vary over length)
   - Gradually Varied Flow
   - Rapidly Varied Flow

Unsteady Flow (Rate of flow varies with time)
1. Unsteady Uniform Flow (rare)
2. Unsteady Non-uniform Flow
   - Gradually Varied Unsteady Flow
   - Rapidly Varied Unsteady Flow

The steady uniform flow class and the steady non-uniform flow class are the most common types of flow treated in highway engineering hydraulics. However, uniform flow is rare in natural channels.

*Rev. 9/09
7.4.3 Natural Channels

7.4.3.1 Stream Morphology

7.4.3.1.1 Introduction
The form assumed by a natural stream, which includes its cross-sectional geometry as well as its plan-form, is a function of many variables for which cause-and-effect relationships are difficult to establish. The stream may be graded or in equilibrium with respect to long time periods, which means that on the average it discharges the same amount of sediment that it receives, although there may be short-term adjustments in its bed-forms in response to flood flows. On the other hand, the stream reach of interest may be aggrading or degrading as a result of deposition or scour in the reach, respectively. The plan-form of the stream may be straight, braided, or meandering. These complexities of stream morphology can be assessed by inspecting aerial photographs and topographic maps for changes in slope, width, depth, meander form and bank erosion with time.

A qualitative assessment of the river response to proposed highway facilities is possible through a thorough knowledge of river mechanics and accumulation of engineering experience. The FHWA publications "Stream Stability at Highway Structures" (HEC-20) and "River Engineering for Highway Encroachments" (HDS-6) provide additional and more detailed information on making such assessments. Both publications can be accessed and/or downloaded from the FHWA's Internet web site at [http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm](http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm).

7.4.4 Channel Analysis

7.4.4.1 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.

Two methods are commonly used in hydraulic analysis of open channels. The single-section method is a simple application of Manning's equation to determine tailwater rating curves for culverts or to analyze other situations in which uniform or nearly uniform flow conditions can be assumed.

The step-backwater method is used to compute the complete water surface profile in a stream reach to evaluate the unrestricted water surface elevations for bridge hydraulic design or to analyze other gradually varied flow problems in streams.

*Rev. 9/09
A. Where the 100-yr discharge for a particular site is less than 500 cfs, the site is generally considered to be a minor drainage installation. A single cross section analysis, using nomographs and charts, usually provides an acceptable level of hydraulic analysis. Documentation requirements are satisfied by supplying all requested information on VDOT standard hydraulic computation forms.

B. Where the 100-yr discharge for a particular site is 500 cfs or more, the site is generally considered to be a major drainage installation. The method of hydraulic analysis and the level of documentation must conform to hydrology analysis (H&HA) outline provided in Chapter 12, Bridge and Structure Hydraulics. This type of analysis often requires water surface profile calculations such as those provided by the HEC-2, HEC-RAS, or WSPRO computer models. However, other methods of analysis, which provide the necessary data for proper documentation, may be approved for use.

The single-section method will generally yield less reliable results than the step-backwater method because it requires more judgment and assumptions. In many situations, however, the single-section method is all that is justified. In minor drainage channels such as roadside ditches, the single section method is adequate, except in the case of special design channels or critical locations.

The step-backwater method should be used for important major channels, where an accurate definition of the water surface profile is needed. The basic principles of open channel hydraulics are applicable to all drainage channels, as well as culverts and storm drains. The variable is the level of detail required in design, which depends on the risks of damage or loss of life caused by a failure of the facility.

7.4.4.2 Equations
The following equations are those most commonly used to analyze open channel flow. The detailed use of these equations in analyzing open channel hydraulics is discussed in Section 7.5.

7.4.4.2.1 Specific Energy
Specific energy (E) is defined as the energy head relative to the channel bottom. If the channel slope is less than 10% 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:

\[ E = d + \alpha \frac{v^2}{2g} \]  

(7.1)
Where:

\[ d = \text{Depth of flow, ft} \]
\[ \alpha = \text{Velocity distribution coefficient (see Equation 7.2)} \]
\[ v = \text{Mean velocity, fps} \]
\[ g = \text{Gravitational acceleration, 32.2 ft/s}^2 \]

When specific energy is plotted against depth of flow, a curve with a minimum specific energy results, as shown in Figure 7-2. At the minimum specific energy, the depth is called critical depth. Depths above critical depth are subcritical, and below critical depth are supercritical. The velocity distribution coefficient is usually assumed to have a value of one for turbulent flow in prismatic channels but may be significantly different than one in natural channels.

![Figure 7-2 Specific Energy Diagram for Rectangular Channels](image)

Note: \( y = d \) in Equation 7.1

**Figure 7-2 Specific Energy Diagram for Rectangular Channels**

### 7.4.4.2.2 Velocity Distribution Coefficient

Due to the presence of a free surface and due to friction along the channel boundary, the velocities in a channel are not uniformly distributed across the channel cross section. Because of non-uniform distribution of velocities in a channel section, the velocity head of an open channel is usually greater than the average velocity head computed as \((Q/A_0)^2/2g\). A weighted average value of the velocity head is obtained by multiplying the average velocity head, above, by a velocity distribution coefficient \((\alpha)\) defined as:

\[
\alpha = \frac{\sum_{i=1}^{n} \frac{K_i^3}{A_i^2}}{\frac{K_1^3}{A_1^2}}
\]

\( (7.2) \)
Where:

\[ K_i = \text{Conveyance in subsection (see Equation 7.8)} \]
\[ K_t = \text{Total conveyance in section (see Equation 7.8)} \]
\[ A_i = \text{Cross-sectional area of subsection, ft}^2 \]
\[ A_t = \text{Total cross-sectional area of section, ft}^2 \]
\[ n = \text{Number of subsections} \]

### 7.4.4.2.3 Total Energy Head
The total energy head is the specific energy head plus the elevation of the channel bottom with respect to some datum. A plot of the energy head from one cross section to the next defines the energy grade line.

### 7.4.4.2.4 Froude Number
The Froude number \((F_r)\) is an important dimensionless parameter in open channel flow. It represents the ratio of inertial forces to gravitational forces and is defined as:

\[
F_r = \frac{V}{\sqrt{\frac{gH_D}{\alpha}}} \tag{7.3}
\]

Where:

\[ \alpha = \text{Velocity distribution coefficient} \]
\[ V = \text{Mean velocity} = \frac{Q}{A}, \text{fps} \]
\[ g = \text{Gravitational acceleration, 32.2 ft/s}^2 \]
\[ H_D = \text{Hydraulic depth} = \left( \frac{A}{T} \right), \text{ft} \]

(For rectangular channels the hydraulic depth is equal to the flow depth.)
\[ A = \text{Cross-sectional area of flow, ft}^2 \]
\[ T = \text{Channel top width at the water surface, ft} \]
\[ Q = \text{Discharge, cfs} \]

This expression for Froude number applies to channel flow at any cross section. The Froude number is useful in determining the flow regime for water surface profiles.

- \(F_r \leq 1\), Subcritical Flow
- \(F_r = 1\), Critical Flow
- \(F_r > 1\), Supercritical Flow

*Rev. 9/09
7.4.4.2.5 Critical Flow

Critical flow occurs when the specific energy is a minimum for a given flow rate (see Figure 7-2). The variation of specific energy with depth at a constant discharge shows a minimum in the specific energy at a depth called critical depth at which the Froude number has a value of one. Critical depth is the depth of maximum discharge when the specific energy is held constant. During critical flow the velocity head is equal to one-half of the hydraulic depth. The general expression for flow at critical depth is:

\[
\frac{\alpha Q^2}{g} = \frac{A^3}{T}
\]

(7.4)

Where:
\[
\begin{align*}
\alpha &= \text{Velocity distribution coefficient} \\
Q &= \text{Discharge, cfs} \\
g &= \text{Gravitational acceleration, } 32.2 \text{ ft/s}^2 \\
A &= \text{Cross-sectional area of flow, ft}^2 \\
T &= \text{Channel top width at the water surface, ft}
\end{align*}
\]

When flow is at critical depth, Equation 7.4 must be satisfied, regardless of the shape of the channel.

7.4.4.2.6 Subcritical Flow

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

7.4.4.2.7 Supercritical Flow

Depths less than critical depth occur in supercritical flow. 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.

7.4.4.2.8 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:

\[
Q = A_1 V_1 = A_2 V_2
\]

(7.5)

Where:
\[
\begin{align*}
Q &= \text{Discharge, cfs}
\end{align*}
\]

*Rev. 9/09
\[ A = \text{Cross-sectional area of flow, ft}^2 \]
\[ V = \text{Mean cross-sectional velocity, fps (which is perpendicular to the cross section)} \]

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

### 7.4.4.2.9 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 using Manning's equation:

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

(7.6)

Where:

- \( V \) = Velocity, fps
- \( n \) = Manning's roughness coefficient
- \( R \) = Hydraulic radius = \( A/P \), ft
- \( A \) = Flow area, ft²
- \( P \) = Wetted perimeter, ft
- \( S \) = Slope of the energy grade line, 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 7.4.4.3.2. The range of \( n \)-values for various types of channels and floodplains is given in Appendix 7D-1 and 7D-2.

The continuity equation can be combined with Manning's equation to obtain the steady, uniform flow discharge as:

\[ Q = \frac{1.486}{n} AR^{2/3} S^{1/2} \]

(7.7)

For a given channel geometry, slope, roughness, and a specified discharge \( Q \), a unique value of depth occurs in steady, uniform flow. It is called normal depth. At normal depth, the slope of the energy grade line, the hydraulic grade line, and the channel slope are the same. Normal depth is computed from Equation 7.7 by expressing the area and hydraulic radius in terms of depth. The resulting equation may require a trial-and-error solution. See Section 7.5 for a more detailed discussion of the computation of normal depth.

If normal depth is greater than critical depth, the channel slope is classified as a mild slope. On a steep slope, the normal depth is less than the critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

### 7.4.4.2.10 Conveyance

In channel analysis, it is often convenient to group the channel properties in a single term called the channel conveyance \( K \):
Then Equation 7.7 can be written as:

\[ Q = KS^{\frac{1}{2}} \]  
(7.9)

The conveyance represents the carrying capacity of a stream cross-section based upon its geometry and roughness characteristics alone and is independent of the streambed slope.

The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross-section and the flow distribution through the opening in a proposed stream crossing. It is also used to determine the velocity distribution coefficient (\( \alpha \)).

### 7.4.4.2.11 Energy Equation

The energy equation (Bernoulli's Equation) expresses conservation of energy in open channel flow defined as energy per unit weight of fluid, which has **dimensions of length**, and is 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, the sum of which gives the total energy head at any cross section. Written between an upstream open channel cross section designated “1” and a downstream cross section designated “2”, the energy equation is:

\[
h_1 + \alpha_1 \frac{V_1^2}{2g} = h_2 + \alpha_2 \frac{V_2^2}{2g} + h_L \]  
(7.10)

Where:

- \( h_1 \) = Upstream water surface elevation, ft.
- \( h_2 \) = Downstream water surface elevation, ft.
- \( \alpha \) = Velocity distribution coefficient
- \( V \) = Mean velocity, fps
- \( h_L \) = Head loss due to local cross-sectional changes (minor loss) plus friction loss, ft

The stage (\( h \)), is the sum of the elevation head (\( z \)) at the channel bed and the pressure head, or depth of flow (\( y \)); i.e., \( h = z+y \). The terms in the energy equation are illustrated graphically in Figure 7-3. 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 energy head losses between two consecutive sections. The energy equation can only be applied between two cross sections at which the streamlines are nearly straight and parallel so that vertical acceleration can be neglected.

*Rev. 7/16*
7.4.4.3 Hydraulic Representation of Channels
The following sections describe the data needed to apply Manning's equation to the analysis of open channels.

7.4.4.3.1 Cross Sections
The cross-sectional geometry of streams is defined by coordinates of lateral distance and ground elevation, which locate individual ground points. Individual cross sections are 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 intersecting straight lines to form a section; 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 charges in roughness or shape, control sections such as free overfalls, bends and contractions, or other abrupt changes in channel slope or conveyance will require cross sections taken at shorter intervals in order to better model the changes in conveyance.

Cross sections should be subdivided with vertical boundaries where there are abrupt lateral changes in geometry and/or roughness as in the case of overbank flows. The conveyances of each subsection are computed separately to determine the flow distribution and \( \alpha \), 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 (Davidian, 1984). Selection of cross sections and vertical subdivision of a cross section are shown in Figure 7-4.

Figure 7-3 Terms in the Energy Equation
Manning's n-value Selection

Manning's n is affected by many factors and its selection, especially in natural channels, depends heavily on engineering experience. Photographs of channels and flood plains, for which the discharge has been measured, and Manning's n has been calculated, are very useful (see Arcement and Schneider, 1984; Barnes, 1978). For situations lying outside the engineer's experience, a more regimented approach is presented (Arcement and Schneider, 1984). Once the Manning's n-values have been selected, it is highly recommended that they be verified or calibrated with historical highwater marks and/or gaged stream flow data.
Manning’s n-values for artificial channels are more easily obtained than for natural stream channels. Refer to Appendix 7D-1 and 7D-2 for typical n-values for both man-made and natural stream channels.

7.4.4.3.3 Calibration
For major channel analyses in existing channels, the equations should be calibrated with historical highwater marks and/or gaged stream flow data to ensure that they accurately represent local channel conditions. The following parameters, in order of preference, should be used for calibration: Manning’s n, slope, discharge, and cross section. Proper calibration is essential if accurate results are to be obtained.

7.4.4.3.4 Switchback Phenomenon
If the cross section is improperly subdivided, the mathematics of Manning’s equation causes a switchback. A switchback results when the calculated discharge decreases with an associated increase in elevation. This occurs when, with a minor increase in water depth, there is a large increase of wetted perimeter. Simultaneously, there is a corresponding small increase in cross-sectional area, which causes a net decrease in the hydraulic radius from the value computed for a lesser water depth. With the combination of the lower hydraulic radius and the slightly larger cross-sectional area, a discharge is computed which is lower than the discharge based upon the lower water depth. More subdivisions within such cross sections should be used, and the divisions should be based on both vegetation and geometry, in order to avoid the switchback error. Figure 7-5 depicts an example of the switchback phenomenon.

![Figure 7-5 Example of Switchback Phenomenon](image)

7.4.4.4 Single-Section Analysis
The single-section analysis method (slope-area method) is simply a solution of Manning’s equation for the normal depth of flow, given the discharge and channel cross section properties including geometry, slope and roughness. It implicitly assumes the existence of steady, uniform flow; however, uniform flow rarely exists in either man-made or natural channels. Nevertheless, the single-section method is often used to design man-made channels for uniform flow as a first approximation, and to develop a stage-discharge rating curve in a natural channel for tailwater determination at a culvert or storm drain outlet. The single-section analysis method is used in the VDOT method for design of roadside ditch linings.

Chapter 7-20 of 69
A stage-discharge rating curve is a graphical relationship of stream flow depth or elevation versus discharge at a specific point on a channel. This relationship should cover a range of discharges up to at least the base (100-yr) flood. The stage-discharge curve procedure is discussed in Section 7.5.3.1.

Alternatively, a graphical technique such as those given in the appendices can be used for trapezoidal and prismatic channels. The best approach, especially in the case of natural channels, is to use a computer program such as FEMA's "Quick-2" software package.

In natural 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 purposes of designing erosion control measures and locating relief openings such as in highway fills. 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, \(V = Q/A\).

### 7.4.4.5 Water Surface Profile Analysis

The step-backwater analysis is useful for determining unrestricted water surface profiles where a highway crossing or encroachment is planned, and for analyzing how far upstream the water surface elevations would be affected. Because the calculations involved in this analysis are tedious and repetitive, it is recommended that a computer program such as the Corps of Engineers HEC-RAS model be used.

#### 7.4.4.5.1 Water Surface Profile Methodology

When uniform flow cannot be reasonably assumed and, therefore, a single cross section cannot represent the channel segment, then an energy balance method must be used to compute the water surface profile (elevation). The energy equation is used in computing the water surface profile.
The method requires definition of the geometry and roughness of each cross section as discussed previously. Manning's $n$ values can vary both horizontally and vertically across the section. Expansion and contraction head loss coefficients, variable main channel and overbank flow lengths and the method of averaging the slope of the energy grade line can all be specified.

The energy equation is derived from Equation 7.10 in Section 7.4.4.2.11.

$$d_1 + \alpha_1 \frac{V_1^2}{2g} + z_1 = d_2 + \alpha_2 \frac{V_2^2}{2g} + z_2 + h_L$$  \hspace{1cm} (7.11)

Where:

- $d$ = Depth of flow, ft
- $V$ = Mean velocity, fps
- $z$ = Elevation of flow line, ft
- $h_L$ = Total head loss, ft
- $\alpha$ = Velocity distribution coefficient

Equation 7.11 shows that the total head at Section 1 is equal to the total head at Section 2 and the energy (head) losses. Total energy losses include friction and minor losses.

$$h_L = h_f + h_o$$  \hspace{1cm} (7.12)

Where:

- $h_L$ = Total head losses, ft.
- $h_f$ = Friction loss, ft.
- $h_o$ = Summation of minor losses, ft.

In most simple water surface profile calculations, minor losses are ignored; therefore, $h_L$ is assumed to be equal to $h_f$.

$$h_f = L S_A$$  \hspace{1cm} (7.13)

Where:

- $L$ = Length of channel segment, ft.
- $S_A$ = Average energy slope of channel segment, ft./ft.

$$S_A = \frac{S_1 + S_2}{2}$$  \hspace{1cm} (7.14)

*Rev. 9/09*
Where:

\[ S_1 = \text{Energy slope at Section 1, ft./ft.} \]
\[ S_2 = \text{Energy slope at Section 2, ft./ft.} \]

The energy slope at a given cross section is computed using Equation 7.15 and is shown graphically in Figure 7-6.

\[ S = \frac{Q^2n^2}{2.25R^\frac{4}{5}A^2} \]  

(7.15)

Figure 7-6 Energy Slope between Two Channel Sections

7.4.4.5.2 Classifications of Backwater Profiles

Figure 7-7 shows the notation for classifying water surface profiles and Figure 7-8 shows the types of possible flow profiles. Figure 7-7 and Figure 7-8 are from the USACE’s Gary Brunner.
Figure 7-7 Notation for Classifying Water Surface Profiles
### GRADUALLY VARIOUS FLOW

<table>
<thead>
<tr>
<th>Profiles in Zone 1: $y &gt; y_a, y &gt; y_c$</th>
<th>Profiles in Zone 2: $y &gt; y_a, y &gt; y_c, y &lt; y_a$</th>
<th>Profiles in Zone 3: $y &lt; y_a, y &lt; y_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Horizontal slope</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$y_a &gt; y_c$</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>$y_c$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Mild slope</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$y_a &gt; y_c$</td>
<td>$M_1$</td>
<td></td>
</tr>
<tr>
<td>$y_c$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Critical slope</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$y_a = y_c$</td>
<td>$C_1$</td>
<td></td>
</tr>
<tr>
<td>$y_c$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Steep slope</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$y_a &lt; y_c$</td>
<td>$S_1$</td>
<td></td>
</tr>
<tr>
<td>$y_c$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Aerial slope</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>None</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$y_c$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Figure 7-8 Types of Backwater Profiles**
7.4.4.5.3 Water Surface Profile Computations

The references (Davidian, 1984 and USACE, 1986) are valuable sources of guidance on the practical application of the step-backwater method to roadway drainage problems involving open channels. These references contain guidance on cross section determination, location and spacing, and stream reach determination. The reference (USACE, 1986) investigates the accuracy and reliability of water surface profiles related to n-value determination and the survey or mapping technology used to determine the cross section coordinate geometry.

7.4.5 Water and Sediment Routing

Water and sediment routing is a complex phenomenon, and a detailed discussion is beyond the scope of this manual. Information may be found in documentation of the BRI-STARS (Bridge Stream Tube Model for Alluvial River Simulation) Computer Model, (Molinas, 2000). The model is semi-two dimensional, and both energy and momentum functions are incorporated so that the water surface profile computation can be carried out through combinations of subcritical and supercritical flows without interruption. Another computer model for sediment routing is the Corps of Engineers HEC-6, "Scour and Deposition in Rivers and Reservoirs." (legacy software)

7.4.6 Ditch and Channel Protection

A significant means of reducing erosion associated with roadways is through the use of properly designed ditches and ditch lining. Linings may be flexible such as vegetation, synthetic material; or riprap or linings may be rigid such as concrete. Erosion resistant vegetation should be used whenever possible and may in some locations require the use of either a temporary Rolled Erosion Control Product (RECP) (VDOT Standard EC-2) or a permanent Rolled Erosion Control Product (RECP) (VDOT Standard EC-3). Flexible linings are generally less expensive than rigid linings and permit infiltration and filtering of pollutants. Flexible linings provide a lower flow capacity for a given cross-sectional area when compared to rigid linings. They also have correspondingly lower velocities than rigid linings.

Rigid linings are traditionally used on steep grades due to high velocities and shear stresses and may be used for areas where channel width is restricted and the higher flow capacity is needed. Rigid channels require channel protection or energy dissipation at the termination point to prevent scour due to the high outlet velocities. Rigid linings can be damaged or destroyed due to flow undercutting the lining at bends, joints or intersecting ditches where the flow is not contained within the lining. The design of channels with rigid lining shall provide any design details that are needed to protect the undercutting of the lining and preserve its integrity.

*Rev. 5/17
Two methods are commonly applied to determine whether a channel is stable from an erosion standpoint: Permissible Velocity and Tractive Force (Permissible Shear) Methods. These methods are described more in detail in the following sections.

7.4.6.1 Permissible Velocity

Using the permissible velocity approach, the channel is assumed to be stable if the mean velocity is lower than the maximum permissible velocity. The application of the permissible velocity method is quite simple. The Manning’s equation is used to compute the flow velocity in the channel for the design storm. The flow velocity is then compared with the maximum permissible velocity for the channel bed and bank soils and the lining material.

If the computed velocity is less than the maximum permissible velocity, the channel should be considered as stable. Corrections to the maximum permissible velocity may be applied based on flow depth and channel sinuosity.

The maximum permissible velocity (MPV) in a channel varies with the channel bed material and with the material being transported by the water. Clear water is the most erosive, and therefore its MPVs are the lowest for a given bed material (see Appendix 7D-6). When the water is transporting fine silts, the MPV is up to 100% higher than that for clear water. For this reason, clear water being released from a settling facility such as a detention pond is often called “hungry water.”

For water carrying a courser material such as sand and gravel, the MPV may be higher or lower than for clear water. For this situation, refer to Appendix 7D-6 to obtain the MPV for the given channel bed material.

While the permissible velocity method has been widely used, the tractive force (permissible shear) method provides a more physical-based and realistic model for particle detachment and erosion processes (FHWA’s HEC-15, “Design of Roadside Channels with Flexible Linings”). The designer shall apply the tractive force method be used for roadside ditches and major channel analyses.

7.4.6.2 Tractive Force (Permissible Shear)

The Permissible Tractive Force (Permissible Shear) Method provides a physical-based and realistic model for particle detachment and erosion processes (FHWA’s HEC-15, “Design of Roadside Channels with Flexible Linings”).

The Tractive Force (Permissible Shear) Method takes into account the physical factors of bed material, channel geometry, depth, and velocity of flow. The method is applicable to non-cohesive materials for which the permissible tractive force is related to particle size and shape and the sediment load in the water.

*Rev. 5/17
Particle diameters are based on the equivalent spherical volume and assume 75% of the mass is smaller by weight \((D_{75})\). In the absence of a gradation curve, it may be assumed that the \(D_{75}\) stone size is equal to 1.96 times the \(D_{50}\) stone size.

The average tractive force formula is:

\[
\tau_o = 62.4RS_o \quad (7.16)
\]

Where:

- \(\tau_o\) = Average tractive force, lbs/ft\(^2\)
- \(R\) = Hydraulic radius, ft.
- \(S_o\) = Channel slope, ft/ft.

In channels whose width (B) to depth (d) ratio is 10 or more, the depth of flow (d) may be substituted for \(R\), thus obtaining the maximum tractive force on the channel bed.

\[
\tau_{max} = 62.4dS_o \quad (7.17)
\]

Where:

- \(d\) = Depth of flow, ft

The material on the side slope may establish the limiting condition for permissible tractive force, rather than the material on the bed. The resistance to movement of the material on the side slope is reduced by the downward sliding force due to gravity. The ratio of critical shear on the side slope to critical shear on the bottom is expressed as factor \(K_1\).

**Note:** \(K_1\) in HEC-11 is the same variable as \(K_2\) in HEC-15.

\[
K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}} \quad (7.18)
\]

Where:

- \(\theta\) = Side slope angle (measured from the horizontal), degrees
- \(\phi\) = Natural angle of repose of material under consideration (measured from the horizontal), deg.

**Note:** The descriptions for \(\theta\) and \(\phi\) (above) are reversed in Appendix 7E-8.
(shown one way in HEC-11 and the other way in HEC-15)

---

*Rev. 7/16*
The angles of repose for various sizes of non-cohesive materials can be determined from Appendix 7E-1.

The permissible tractive force for various non-cohesive soils is obtained from Appendix 7E-2. Appendix 7E-3 can be used to estimate the permissible tractive force for cohesive soils.

**7.4.6.2.1 Shear Stress in Bends**

Flow around a bend can impose higher shear stresses on the channel bottom and sides as compared to a straight channel. The following equation gives the maximum shear stress in a bend:

\[ \tau_b = K_b \tau_{\text{max}} \]  

(7.19)

Where:

- \( \tau_b \) = Maximum bend shear stress, lb/ft²
- \( K_b \) = Ratio of channel bend to bottom shear stress
- \( \tau_{\text{max}} \) = Shear stress in straight channel at maximum depth, lb/ft²

The ratio of channel bend to bottom shear stress (\( K_b \)) is a function of the ratio of channel curvature to the top (water surface) width, \( R_c/T \). \( K_b \) can be determined from the following equation:

\[ K_b = \begin{cases} 2.00 & R_c/T \leq 2 \\ 2.38 - 0.206(R_c/T) + 0.0073(R_c/T)^2 & 2 < R_c/T < 10 \\ 1.05 & 10 \leq R_c/T \end{cases} \]  

(7.20)

Where:

- \( R_c \) = radius of curvature of the Centerline of the bend, ft
- \( T \) = channel top (water surface) width, ft

*Rev. 7/16*
7.4.6.3 Geotextile Channel Linings

Geotextile materials designated as Standard EC-3 RECP (Permanent) are used for protective linings in ditches. Standard EC-3 RECP (Permanent) is intended to be used as a protective ditch lining material to be applied when the design velocity or tractive force exceeds the allowable velocity or tractive force for Standard EC-2 RECP (Temporary).

When the design velocity or tractive force exceeds the allowable velocity or tractive force for Standard EC-3, a paved (or riprap) lining is required.

The Standard EC-3 RECP (Permanent) may be used as a protective slope lining for dry cut or fill slopes and wet cut slopes to stabilize the slope on which vegetation is being established. (See VDOT Road and Bridge Standards)

7.4.6.3.1 Design Criteria, Geotextile Linings

Under the Permissible Velocity method, differing types of EC-2 or EC-3 linings will be selected due to the following criteria, while using the Manning’s n-values shown in Table 7-2A:

- EC-2 Types 1, 2, 3, or 4 when the 2-yr design velocity in the ditch does not exceed 4 feet per second (fps)
- EC-3 Type 1, when the 2-yr design velocity is between 4-7 fps
- EC-3 Type 2, when the 2-yr design velocity is between 7-10 fps
- If the 2-yr design velocity is greater than 10 fps, the use of a flexible lining shall be avoided, and a rigid lining shall be specified.

Under the Tractive Force method, differing types of EC-2 or EC-3 linings will be selected due to the following criteria, while using the Manning’s n-values in Table 7-2A.

- EC-2 Type 1, when the 2-yr design tractive force does not exceed 1.5 lb/ft²
- EC-2 Type 2, when the 2-yr design tractive force is between 1.5 – 1.75 lb/ft²
- EC-2 Type 3, when the 2-yr design tractive force is between 1.75 - 2.0 lb/ft²
- EC-2 Type 4, when the 2-yr design tractive force is between 2.0 - 2.25 lb/ft²
- EC-3 Type 1, when the 2-yr design tractive force is between 2.25 - 6 lb/ft²

*Rev. 5/17

Chapter 7-30 of 69
- EC-3 Type 2, when the 2-yr design tractive force is between 6 - 8 lb/ft²
- EC-3 Type 3, when the 2-yr design tractive force is between 8 – 10 lb/ft²
- If the 2-yr design tractive force exceeds 20 lb/ft², the use of a flexible lining shall be avoided, and a rigid lining shall be specified.

Manufacturers have developed a variety of rolled erosion control products (RECPs) for erosion protection of ditches and channels. The AASHTO National Transportation Product Evaluation Program (NTPEP) has identified the test procedures applicable to RECPs. These test results shall be submitted to VDOT for acceptance before use. The test results shall include a table of “Standard n value versus Applied Shear” as shown in Table 5.4, Chapter 5 of the HEC-15 manual. The upper allowable shear stress should equal or exceed the calculated design tractive force. The upper and lower allowable shear stress values must equal twice and one-half of the middle value, respectively.

It is understood that computing ditch/channel capacities is an iterative procedure through the design process. Ultimately in terms of completed calculations, it is encouraged that the designer compute the Manning’s n-value of that type of EC-2/EC-3 that he/she specified in the material quantities. It is understood that most manufacturers demark a range Manning’s n-value for each type of VDOT EC-2/3 classification. The designer shall use the mid-range value of the range of such Manning’s n-value that the manufacturer provides.

*Rev. 5/17*
### Table 7-1 Allowable Velocity and Shear Stress Values for Lined Ditches

<table>
<thead>
<tr>
<th>Type of Lining</th>
<th>Maximum Allowable Velocity (fps)</th>
<th>Maximum Allowable Shear Stress (lb/ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bare Earth (See Appx 7D-2)</td>
<td>Varies</td>
<td>Varies</td>
</tr>
<tr>
<td>VDOT EC-2 Type-1</td>
<td>4.0</td>
<td>1.5</td>
</tr>
<tr>
<td>VDOT EC-2 Type-2</td>
<td><strong>4.0</strong></td>
<td>1.75</td>
</tr>
<tr>
<td>VDOT EC-2 Type-3</td>
<td>4.0</td>
<td>2.0</td>
</tr>
<tr>
<td>VDOT EC-2 Type-4</td>
<td>4.0</td>
<td>2.25</td>
</tr>
<tr>
<td>VDOT EC-3 Type 1</td>
<td>7.0</td>
<td>6.0</td>
</tr>
<tr>
<td>VDOT EC-3 Type 2</td>
<td>10.0</td>
<td>8.0</td>
</tr>
<tr>
<td>VDOT EC-3 Type-3</td>
<td>N/A</td>
<td>10.0</td>
</tr>
<tr>
<td>Concrete</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>VDOT Riprap</td>
<td>Based on Shear Stress</td>
<td>Varies</td>
</tr>
</tbody>
</table>

*Rev. 5/17*
<table>
<thead>
<tr>
<th>Lining Category</th>
<th>Lining Type</th>
<th>Manning’s n</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
</tr>
<tr>
<td>Unlined</td>
<td>Bare Soil</td>
<td>0.025</td>
</tr>
<tr>
<td></td>
<td>Rock Cut</td>
<td>0.045</td>
</tr>
<tr>
<td>Temporary RECP/EC-2 (^1)</td>
<td>Type 1</td>
<td>0.045</td>
</tr>
<tr>
<td></td>
<td>Type 2</td>
<td>0.045</td>
</tr>
<tr>
<td></td>
<td>Type 3</td>
<td>0.045</td>
</tr>
<tr>
<td></td>
<td>Type 4</td>
<td>0.022</td>
</tr>
<tr>
<td>Permanent RECP/EC-3 Type 1 thru Type 3 (^1)</td>
<td>Unvegetated</td>
<td>0.036</td>
</tr>
<tr>
<td>Riprap (^2,3)</td>
<td>Class A1</td>
<td>0.124</td>
</tr>
<tr>
<td></td>
<td>Class I</td>
<td>0.153</td>
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<tr>
<td></td>
<td>Class II</td>
<td>0.181</td>
</tr>
<tr>
<td>Rigid</td>
<td>Concrete</td>
<td>0.015</td>
</tr>
</tbody>
</table>

\(^1\) General values based on approved product list
Consult with individual manufactures for more specific information

\(^2\) Values interpolated from data provided in HEC-15

\(^3\) In general, \(n=0.0395(d_{50})^{0.167}\), where \(d_{50}\)=median stone diameter
7.4.6.3.2  Paved Flumes

Due to the substantial number of failures and continual maintenance problems associated with PG-4 flumes on fill slopes, flumes shall not be used on fill slopes.

In lieu of paved flumes, the appropriate type of drop inlet and pipe be used in all possible situations. For design considerations of pipe on steep slopes see Section 9.4.8.7 of the Drainage Manual.

To a lesser degree, similar problems and concerns have been noted with paved flumes in cut sections. The alternatives for paved flumes in cut sections are usually very limited unless the cut is of a shallow depth.

When design situations involve the apparent need for paved flumes, the Drainage Designer shall explore all feasible alternatives to develop a design that will address both constructability and future maintenance concerns; however a flume shall not be lined with riprap.

7.4.6.4  Riprap

Riprap is defined as a blanket of well-graded stone used to counteract the effects of erosion or scouring on channels, ditches, embankments, jetties, shorelines, and bridge substructure members such as abutments and piers. Riprap is usually described in terms of the size and/or weight of the stone whose volume makes up approximately 50% of the total mass. The size of the 50% stone is measured in terms of its equivalent mean spherical diameter (MSD) and is referred to as the stone’s $D_{50}$. The weight of the 50% stone is referred to by its $W_{50}$. The Department has six standard riprap classifications whose 50% stone size and weights and recommended blanket thickness ($T$) are tabulated in Appendix 7D-3.

Additional information on VDOT standard riprap may be found in the Department's *Road and Bridge Specifications*.

The designer shall specify on the plans the type of Riprap and the dimensions (length, width and depth) for placement. The quantity shall be computed using two (2) tons/cy (148 lbs/ft$^3$) for plan estimating purposes, unless otherwise specified by the District Administrator.

*Rev. 5/17*
The following general note is to be copied on the plans when riprap is specified.

“The proposed riprap may be omitted by the Engineer if the slope upon which the plans designate riprap to be placed is found to meet the following criteria: The slope designated for placement of riprap is comprised of solid rock or closely consolidated boulders with soundness, size and weight equal to or exceed the specifications for the proposed riprap. If the slope is found to be comprised of material, which is coarser than the bedding aggregate filter blanket specified on the plans, the aggregate filter blanket may be deleted by the Engineer.”

7.4.6.4.1 Riprap Dimensions and Weights

Appendix 7D-4 may be used as a guide to match certain rock dimensions to equivalent weights. This table is not to be used for acceptance or rejection of riprap material.

The approximate percentage of voids for all VDOT standard riprap classes is 25% for the estimation of quantities.

7.4.6.4.2 Soils Survey

A soil survey is to be conducted through areas where a channel change is proposed and through embankment areas where riprap may be required. The plans or profile rolls for the regular soil survey will show the location of channel changes and the location where riprap will be required on the fill section.

Borings along the proposed channel change are to be taken at sufficient intervals to determine the type of material encountered along the slopes and in the bottom of the channel.

The borings made in the cut sections or in the borrow pits for construction of the fills are adequate to determine the type of material used in the fills. The test results on the material used in embankments or along channel changes where riprap is required should include the plastic and liquid limits of the minus No. 40 sieve and the grading or particle size of the total sample. This information should be submitted in the regular soil survey report.

The Project Inspector will visually examine the slope upon which the plans designate riprap to be placed. If the slope material appears coarser than the bedding aggregate specified the Project Inspector is to notify the District Material Engineer, through normal channels, for a more detailed investigation to determine the actual need for the bedding. If the slope is comprised of solid rock or closely consolidated boulders with soundness, size and weight equal to or exceeding the specifications for the proposed riprap, then the riprap may be deleted by the District Construction Engineer.

The quantities will be field adjusted, using the supplier’s stone weight and the applicable percent of voids for the type or class of material used, to obtain the actual quantity.
7.4.6.4.3 Riprap Bedding

Riprap shall be placed over an appropriate bedding material consisting of a geotextile and aggregate cushion layer in accordance with the following guidelines:

- In the case of Class AI, I, and II riprap, the aggregate cushion is not required.
- In the case of Class III, Type I, and Type II riprap, an intermediate aggregate cushion layer will be required which consists of a material of a size, gradation, and thickness as recommended by the Materials Division.

The geotextile may only be omitted under the conditions discussed above or with the approval of the District Materials Engineer or District Drainage Engineer.

7.4.6.4.4 Major Channels

Riprap is often used as slope protection for natural or man-made stream channels. The need for such slope protection is predicated on the fact that the native soil material or fill material may be displaced by design flows in the channel. The first step in the design process is to determine whether or not the fill or native material will be displaced. The tractive force method is usually employed to make this determination.

7.4.6.4.5 Minor Channels

VDOT normally does not permit the use of standard riprap for linings in standard roadside or median ditches due to its size and normal blanket thickness reducing the ditch cross section. In addition, hand placement, as opposed to end dumping, would probably be necessary in small channels. There are, however, situations in which the smaller riprap sizes such as Class AI and I may be used to good advantage where special design small trapezoidal ditches such as those connecting culvert cross drains, outfalls, or the discharge point of standard design ditches, are required. In such situations, the Department requires that the riprap be sized in accordance with procedures presented in HEC-15. When riprap is used to line minor channels, the design for the channel cross section should allow for the thickness of the riprap and bedding layers without reducing the available flow section.

*Rev. 9/09
Design Procedures and Sample Problems

The following design procedures and examples pertain to minor channels, major channels, and natural channels. The procedures are similar, but become more detailed for the larger channel projects.

7.4.1 Documentation Requirements

These items establish a minimum requirement for all channels except roadside ditches. Also, see Chapter 3, Documentation. The following items used in the design or analysis should be included in the documentation file:

- Hydrology and stage discharge curves for the design, check floods and any historical water surface elevation(s)
- Cross section(s) used in the design water surface determinations and their locations
- Roughness coefficient assignments (n-values)
- Information on the method used for design water surface determinations
- Observed highwater, dates and discharges
- Channel velocity measurements or estimates and locations
- Water surface profiles through the reach for the design, check floods and any historical floods
- Design or analysis of materials proposed for the channel bed and banks;
- Energy dissipation calculations and designs
- Copies of all computer analyses.

7.4.2 Roadside Ditches and Minor Channels

7.4.2.1 Roadside Ditch and Minor Channel Design Procedure

The following six basic design steps are normally applicable to minor channel design projects:

Step 1: Establish a roadside plan

A. Collect available site data

B. Obtain or prepare existing and proposed plan-profile layout including highway, culverts, bridges, etc.

C. Determine and plot on the plan the locations of natural basin divides and roadside ditch outlets and perform the layout of the proposed roadside ditches to minimize diversion flow lengths

An example of a roadside ditch plan/profile is shown in Figure 7-1.

*Rev. 9/09
Step 2: Obtain or establish cross section and data

A. Provide ditch depth adequate to drain the subbase and minimize freeze-thaw effects using the standard underdrain outfall requirements that are appropriate for the project

B. Choose ditch side slopes based on geometric design criteria including safety, economics, soil, aesthetics and access

C. Establish bottom width of trapezoidal ditch

D. Identify features which may restrict cross section design:
   - Right-of-way limits
   - Trees or other environmentally-sensitive areas
   - Utilities
   - Existing drainage facilities

Step 3: Determine initial ditch grades

A. Plot initial grades on plan-profile layout. (The roadside ditch grade in cut is usually controlled by the grade of the highway.)

B. Provide desirable minimum grade of 0.5%

C. Consider influence of type of lining on grade

D. Where possible, avoid features which may influence or restrict grade, such as utility locations

Step 4: Check flow capacities and adjust as necessary

A. Compute the design discharge at the downstream end of a ditch segment (See Chapter 6, Hydrology)

B. Set preliminary values of ditch size, roughness coefficient, and slope

C. Determine maximum allowable depth of ditch, including freeboard

D. Check flow capacity using Manning's equation and the single-section analysis
E. If ditch capacity is inadequate, possible adjustments are as follow:

- Increase bottom width
- Make channel side slopes flatter
- Make channel slope steeper
- Provide smoother channel lining
- Install drop inlets and a parallel storm drain pipe beneath the channel to supplement channel capacity

F. Provide smooth transitions at changes in channel cross sections

G. Provide extra channel storage where needed to replace floodplain storage and/or to reduce peak discharge

Step 5: Determine ditch lining/protection needed

A. Use the VDOT Method – Roadside Ditch Linings (Preferred Method - Section 7.5.2.2 Tractive Force (HEC-15)), or

B. Use the method of Permissible Velocity

Step 6: Analyze outlet points and downstream effects

A. Identify any adverse impacts such as increased flooding or erosion to downstream properties which may result from one of the following at the channel outlet:

- Increase or decrease in discharge
- Increase in velocity of flow
- Concentration of sheet flow
- Change in outlet water quality
- Diversion of flow from another watershed

B. Mitigate any adverse impacts identified in Step 6A. Possibilities include:

- Enlarge outlet channel and/or install control structures to provide detention of increased runoff in channel
- Install velocity control structures (energy dissipaters)
- Increase capacity and/or improve lining of downstream channel
- Install sedimentation/infiltration basins
- Install sophisticated weirs or other outlet devices to redistribute concentrated ditch flow
- Eliminate diversions which result in downstream damage and which cannot be mitigated in a less expensive fashion.
7.4.2.2 VDOT Method for Design of Roadside Ditch Linings using Tractive Force Method

The following computational procedure to determine the need for roadside ditch linings is recommended for use on roadside ditches and minor channels. Before the computational analysis can be performed, a soils report from the USDA NRCS Published Soil Surveys for Virginia is needed which specifies the type of soil that is found in the area of the ditch. The soil classification is then used with Appendix 7D-2 to determine the maximum allowable velocity for the native soil. Native soil is assumed for new ditches and for ditches with vegetation established for less than two years. When the maximum allowable velocity is exceeded, some type of ditch lining is needed.

Step 1: Determine the section of each ditch where the following exist:

- Steepest grade
- Highest flow or drainage area

Step 2: Determine the longitudinal grade, discharge, and tractive force for these sections, assuming no lining

If the tractive force is greater than the allowable tractive force for the appropriate soil type, a ditch lining is needed and the following “General Design Procedure for Roadside Ditch Linings” should be used. If the tractive force is less than the allowable tractive force, no ditch lining is needed for this section of ditch. The same type of analysis should be used for the remainder of the ditches on the project.

7.4.2.2.1 Design Procedure for Roadside Ditch Linings Using VDOT Method using the Tractive Force Method

Roadside ditch computations may be done using the format presented as Appendix 7B-1.

Step 1: The ditch under investigation is divided into convenient segments of length. Usually 100’ stations are used. The drainage area and runoff coefficient(s) for use in the Rational Method are determined for the first or most upstream segment of the ditch to be analyzed.

Step 2: Determine the time of concentration to the downstream end of the first segment of ditch. Using 2-yr frequency rainfall intensity, determine the 2-yr discharge by the Rational Method.

Step 3: Determine the average longitudinal grade for the ditch segment under consideration.

*Rev. 7/16
Step 4: Find the tractive force for the 10-yr frequency design discharge. (Note: Permanent roadside ditch linings should be designed to protect the channel and remain stable during passage of a 10-yr peak flood discharge and temporary channel linings should be designed for a 2-yr peak discharge.)

Step 5: If the tractive force is less than the maximum allowable tractive force, no ditch lining is needed. If the tractive force is more than the allowable tractive force, ditch lining should be required. The usual progression of types of ditch lining used as tractive forces increase would begin with Grass lining, VDOT Standard EC-2, then EC-3 Type A, EC-3 Type B and lastly concrete lining (Standard PG-2A). Change the ditch lining and repeat Steps 4 and 5 until a stable channel section is obtained.

Step 6: Determine the depth of flow for the 10-yr frequency discharge to insure that the hydraulic capacity has not been exceeded.

Step 7: Repeat the above steps for the next downstream segment of ditch. To calculate the discharge, add or accumulate the Rational Method CA values for each segment of the ditch contributing to the point of study. For rainfall intensity, the time of concentration for second and subsequent ditch sections are done as follows: Flow time = Tc of previous section + (Section length / Design Velocity of previous section / 60). The 2-yr, 10-yr, 50-yr and 100-yr flood events are computed from the rainfall Intensity Duration-Frequency (IDF) curve based on the B, D and E values for the particular County/City. The corresponding discharges are computed using the VDOT Modified Rational Method.

If the computed Q value for any segment of ditch is found to be less than the preceding upstream segment, the Q should be held at the higher value of Q until a higher Q is calculated for a downstream segment.

7.4.2.2.2 Caveats to General Design Procedures for Roadside Ditch Linings
The designer should be cautious in using this computational procedure to ensure that the factors for runoff coefficients, times of concentration, and drainage areas properly reflect the actual conditions.

The velocity and depth of flow calculated by this method is based upon uniform flow conditions. Abrupt changes in alignment or grade may cause significant deviations from uniform flow conditions and should be carefully evaluated. Design details must be provided that provide for erosion protection of the ditch in critical areas.

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*Rev. 7/16

Chapter 7-41 of 69
7.4.2.2.3 Roadside Ditch Lining Sample Problem (Tractive Force Method)

Given:

LOCATION: Spotsylvania Area

DITCH SLOPE:

<table>
<thead>
<tr>
<th>STATION</th>
<th>SLOPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>218+00 – 218+25</td>
<td>4.1%</td>
</tr>
<tr>
<td>218+25 - 218+50</td>
<td>4.3%</td>
</tr>
<tr>
<td>218+50 – 218+75</td>
<td>4.4%</td>
</tr>
<tr>
<td>218+75 – 219+00</td>
<td>3.9%</td>
</tr>
<tr>
<td>219+00 – 219+25</td>
<td>3.4%</td>
</tr>
</tbody>
</table>

Figure 7-9 Roadside Ditch Protection Sample Problem, Plan and Section

*Rev. 7/16*
**7.4.2.3 VDOT Method for Design of Roadside Ditch Linings (Permissible Velocity Method)**

The following computational procedure to determine the need for roadside ditch linings is recommended for use on roadside ditches and minor channels. It was developed by VDOT and has been used for many years.

Before the computational analysis can be performed, a soils report from USDA NRCS Published Soil Surveys for Virginia is needed which specifies the type of soil that is found in the area of the ditch. The soil classification is then used with Appendix 7D-2 to determine the maximum allowable velocity for the native soil.

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*Rev. 7/16*
Native soil is assumed for new ditches and for ditches with vegetation established for less than two years. When the maximum allowable velocity is exceeded, some type of ditch lining is needed.

Step 1: Determine the section of each ditch where the following exist:

- Steepest grade
- Highest flow or drainage area

Step 2: Determine the longitudinal grade, discharge, and velocity for these sections, assuming no lining

If the velocity is greater than the allowable velocity for the appropriate soil type, a ditch lining is needed and the following “General Design Procedure for Roadside Ditch Linings” should be used. If the velocity is less than the allowable velocity, no ditch lining is needed for this section of ditch. The same type of analysis should be used for the remainder of the ditches on the project.

7.4.2.3.1 Design Procedure for Roadside Ditch Linings Using VDOT Method

Roadside ditch computations may be done using the format presented as Appendix 7B-1.

Step 1: The ditch under investigation is divided into convenient segments of length. Usually 100-foot stations are used. The drainage area and runoff coefficient(s) for use in the Rational Method are determined for the first or most upstream segment of the ditch to be analyzed.

Step 2: Determine the time of concentration to the downstream end of the first segment of ditch. Using 2-year frequency rainfall intensity, determine the 2-year discharge by the Rational Method.

Step 3: Determine the average longitudinal grade for the ditch segment under consideration.

Step 4: Find the velocity for the 2-year frequency design discharge.

Step 5: If the velocity is less than the maximum allowable velocity, no ditch lining is needed. If the velocity is more than the allowable velocity, ditch lining should be required. The usual progression of types of ditch lining used as velocities increase would begin with VDOT Standard EC-2, then EC-3 Type A, EC-3 Type B and lastly concrete lining (Standard PG-2A).

Step 6: Determine the depth of flow for the 10-year frequency discharge to insure that the hydraulic capacity has not been exceeded.

*Rev. 7/16
Step 7: Repeat the above steps for the next downstream segment of ditch. To calculate the discharge, add or accumulate the Rational Method CA values for each segment of the ditch contributing to the point of study. For rainfall intensity, use the rainfall intensity value from the previous segment minus 0.1 inch. This is a simplifying assumption or approximation of the actual time of concentration that is used for computational efficiency. If the computed Q value for any segment of ditch is found to be less than the preceding upstream segment, the Q should be held at the higher value of Q until a higher Q is calculated for a downstream segment.

7.4.2.3.2 Caveats to General Design Procedures for Roadside Ditch Linings

The designer should be cautious in using this computational procedure to ensure that the factors for runoff coefficients, times of concentration, and drainage areas properly reflect the actual conditions.

The velocity and depth of flow calculated by this method is based upon uniform flow conditions. Abrupt changes in alignment or grade may cause significant deviations from uniform flow conditions and should be carefully evaluated. Design details must be provided that provide for erosion protection of the ditch in critical areas.

7.4.2.3.3 Roadside Ditch Lining Sample Problem

Given:
LOCATION: Lynchburg Area
ALLOWABLE VELOCITY: 2.0 fps (Bare Earth)
DITCH SLOPE:

<table>
<thead>
<tr>
<th>STATION</th>
<th>SLOPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2</td>
<td>0.5%</td>
</tr>
<tr>
<td>2-3</td>
<td>2.0%</td>
</tr>
<tr>
<td>3-4</td>
<td>3.0%</td>
</tr>
<tr>
<td>4-5</td>
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</tr>
<tr>
<td>5-6</td>
<td>4.0%</td>
</tr>
</tbody>
</table>

*Rev. 7/16*
**Figure 7-11** Roadside Ditch Protection Sample Problem, Plan and Section

**Figure 7-12 Worksheet (LD-268) for Calculation of Roadside Ditch Protection, Sample Problem**

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*Rev. 7/16*
7.4.3 Major Channels

The following procedures are used to analyze flow in major channels.

7.4.3.1 Single-Section Stage-Discharge Curve Procedure

The stage-discharge curve procedure is basically a single section analyses. The following steps outline the development of a stage-discharge curve:

Step 1: Select the typical cross section at or near the location where the stage-discharge curve is needed.

Step 2: Subdivide cross section and assign n-values to subsections as described in Section 7.4.4.3.

Step 3: Estimate water-surface slope. Since uniform flow is assumed, the average slope of the streambed is normally used.

Step 4: Apply a range of incremental water surface elevations to the cross section.

Step 5: Calculate 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.

Step 6: 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 elevations corresponding to the design discharge or other discharges of interest.

7.4.3.2 Water Surface Profile Procedure

The water surface profile computations must be started at a point of a known or an assumed water surface elevation. This starting water surface elevation can be derived in one of three ways.

- Critical Depth \((d_c)\) - When changes in flow conditions or channel characteristics cause a transition from subcritical to supercritical flow, or supercritical to subcritical flow, the flow must, at some point, pass through critical depth. When the transition in flow is caused by a change in the slope of the channel bed, then critical depth can be assumed to occur at the change of grade point. Critical depth \(d_c\) can then be used to generate a starting elevation.
• Known Water Surface Elevation - When controls within the channel section dictate a certain stage (elevation)-discharge relationship, then the elevation generated by that control for the targeted discharge can be used as a starting elevation. Examples of controls within the channel section are a culvert under an embankment across the channel, a dam across the channel section, etc.

• Slope/Area (Normal Depth - dₙ) - Water Surface Profile computations can be started at a point in the channel where uniform flow can be assumed and, therefore, normal depth (dₙ) can be used in order to generate a starting elevation.

The following VDOT procedure for developing water surface profiles is recommended. A convenient design form can be found in Appendix 7B-2.

Step 1: Determine flow type - subcritical or supercritical
   A. If flow is supercritical, computations will proceed in a downstream direction from some known starting point.
   B. If flow is subcritical, computations will proceed in an upstream direction.

Step 2: Starting point based on:
   A. Critical depth (dₑ)
   B. Known water surface elevation
   C. Slope/Area method (dₙ)

Step 3: On computation sheet, let subscript 1 reflect the values at the known cross section and subscript 2 reflect the values at the unknown cross section

Step 4: After determining the starting point and its water surface elevation, compute the Total Head for the starting cross section

\[ h₁ = d₁ + \frac{v₁²}{2g} + z₁ \]

Step 5: For the channel segment between the starting point and the next cross section
   A. Determine the Target Head, which is equal to the Total Head of the known cross section (starting point) (from step 4).
   B. Assume a depth (d₂) at the unknown cross section.
   C. Compute area (A₂), wetted perimeter (P₂), hydraulic radius (R₂) and velocity (V₂) for the unknown cross section based on the assumed depth (d₂).
D. Compute velocity head for unknown cross section \( (V_2^2/2g) \). Determine \( z_2 \) (elevation) at unknown cross section and length \( (L) \) between known and unknown cross section.

E. Compute energy slope \( (S_1) \) at the known cross section

\[
S_1 = \frac{Q^2n^2}{2.25R_1^{\frac{4}{3}}A_1^2}
\]

F. Compute energy slope \( (S_2) \) at the unknown cross section

\[
S_2 = \frac{Q^2n^2}{2.25R_2^{\frac{4}{3}}A_2^2}
\]

G. Compute average energy slope \( (S_A) \) between cross section

\[
S_A = \frac{S_1 + S_2}{2}
\]

H. Compute total head loss \( (h_L) \) between cross sections

\[
h_L = LS_A
\]

I. Compute total head \( (h_2) \) at the unknown cross section

\[
h_2 = d_2 + \frac{V_2^2}{2g} + z_2
\]

J. Solve equation:

Total Head at the upstream cross section = Total Head at the downstream cross section + head loss \( (h_f \text{ or } h_L) \).

Use trial and error method until the two sides of the equation balance within the permissible tolerance.

K. Once the equation has been satisfied and the depth \( (d_2) \) established (Step 5j), then the previous unknown cross section becomes the known cross section for the next segment. Assume a depth \( (d_2) \) for the new unknown cross section, compute properties \( (A_2, P_2, R_2, V_2) \) and repeat the procedure.
7.4.3.2.1 Water Surface Profile Sample Problem

GIVEN: \( Q = 100 \text{ cfs} \)
\( n = 0.15 \)
Cross section shown in Figure 7-13

REQUIRED: Compute Water Surface Profile (WSP) between sections 1 and 3 with a tolerance of 0.2'.

Figure 7-13 Water Surface Profile Sample Problem

(1) Compute values for \( d_n \) and \( d_c \) at each section (use the nomographs in Appendix 7C). Determine elevation at each section. Determine subcritical or supercritical flow at each section.

(2) Determine starting point and direction of computations.

Starting point = # 2 \( (d_c) \)

Compute from # 2 to # 3 (supercritical flow)

Compute from # 2 to # 1 (subcritical flow)

*Rev. 7/16
7.4.4 Natural Channels

7.4.4.1 General

The analysis procedure for all types of channels has some common elements as well as some substantial differences. This section will outline a process for assessing a natural stream channel.

7.4.4.2 Natural Channels Design Procedure

Usually the analysis of a natural channel is in conjunction with the design of a highway hydraulic structure such as a culvert or bridge or a longitudinal encroachment such as a highway embankment. In general, the objective is to convey the water along or under the highway in such a manner that does not cause damage to the highway, stream, or adjacent property. An assessment of the existing channel is usually necessary to determine the potential for problems that might result from a proposed action. The level of detail of the studies necessary should be commensurate with the risk associated with the action, and with the environmental sensitivity of the stream and adjoining floodplain.

Figure 7-14 Worksheet for Calculation of Non-Uniform Flow in Open Channels, Sample Problem

*Rev. 7/16
Although the following step-by-step procedure may not be appropriate for all possible applications, it does outline a process that will usually apply to natural channel design.

**Step 1: Assemble site data and project file**

A. Data Collection

- Topographic, site and location maps
- Roadway profile
- Photographs
- Field reviews
- Design data at nearby structures
- Gaging records
- Historic flood data and local knowledge
- Utilities, including existing drainage

B. Studies by other agencies

- Flood insurance studies
- Floodplain studies
- Watershed studies

C. Environmental constraints

- Floodplain encroachment
- Floodway designation
- Fish and wildlife habitat
- Commitments in review documents

D. Design criteria

**Step 2: Determine the project scope**

A. Determine level of assessment

- Stability of existing channel
- Potential for damage
- Sensitivity of the stream

B. Determine type of hydraulic analysis

- Single-section analysis
- Step-backwater analysis
C. Determine survey information needed

- Extent of streambed profiles
- Locations of cross-sections
- Elevations of flood-prone property
- Details of existing structures
- Properties of bed and bank materials

Step 3: Evaluate hydrologic variables

A. Compute discharges for selected frequencies.

B. Consult Chapter 6, Hydrology

Step 4: Perform hydraulic analysis

A. Single-section analysis (7.4.4.4, 7.5.3.1)
   - Select representative cross section
   - Select appropriate n-values (Appendix 7D-1 and 7D-2)
   - Compute stage-discharge relationship

B. Step-backwater analysis (7.5.3.2)

C. Calibrate with known high water

Step 5: Perform stability analysis

A. Geomorphic factors

B. Hydraulic factors.

C. Stream response to change

Step 6: Design countermeasures

A. Criteria for selection
   - Erosion mechanism
   - Stream characteristics
   - Construction and maintenance requirements
   - Cost
B. Types of countermeasures

- Meander migration countermeasures
- Bank stabilization
- Bend control countermeasures
- Channel braiding countermeasures
- Degradation countermeasures
- Aggradation countermeasures

C. For additional information

- HEC-20 Stream Stability
- Highways in the River Environment
- References

Step 7: Documentation (Section 7.5.1)

- Prepare report and file with background information

7.4.4.3 Natural Channels Design Reporting and Documentation

Reporting and documentation detail is dependent upon several factors including:

- FEMA Floodplain Designation
- Project Permit and Compensatory Mitigation Requirements
- Project Scope

FEMA Floodplain Designation

- All projects within a FEMA Flood Insurance Rate Map-designated 100-year floodplain require review and approval by a River Mechanics Engineer regarding the level of analysis necessary for the project.

Project Permit and Compensatory Mitigation Requirements

*Rev. 7/16
Project permit and compensatory mitigation requirements may vary from project to project and can determine the degree to which natural channel design components are required. Consideration of and adherence to permit requirements should be evaluated at the earliest possible time in the project development process.

Project Scope: Projects are separated into several categories: 1. local instream structures, 2. associated structures, and 3. relocation and restoration. Generally, instream and associated structures are defined as those structures directly associated with roadway drainage structure installation, repair or replacement, or bank protection (e.g., erosion protection). Relocation and restoration involve use of natural channel design principles to relocate existing channels, create new channels or restore degraded channels and are not always associated with roadway drainage structures. The project scope, along with the remaining items listed above, will determine the degree to which documentation and QA/QC review is required.

Local instream structures include individual structures or groups of structures (weirs, rock vanes, log vanes and other similar structures) intended to protect, or improve the function of, existing roadway drainage structures or address discrete erosion concerns. Some minor channel modification may be required to relocate the thalweg, remove sediment deposits, or reshape the local cross-section. In many cases, these projects may require limited data collection or design and may serve to retrofit existing structures.

Applicability:
- No channel modifications required other than minor modifications necessary for proper installation and function of the structure(s) itself
- The structure is intended to provide scour protection, redirect thalweg alignment, reduce bank erosion or restore channel geometry (i.e., bankfull width and depth) in the vicinity of an existing structure (culvert or bridge) or to a localized area
- Structure placement is not part of a larger natural channel design project

Requirements:
- The project manager must complete and submit an Abbreviated Plan, as described in the Plan Requirements Section.
- Structure design, with respect to natural channel design features, must be performed under the supervision of, or reviewed and approved by, a Hydraulic Engineer and an Environmental Stream Team member.
- Permit application must be coordinated with the District Environmental Section.

*Rev. 7/16*
**Associated Structures** includes any individual roadway structure designed in accordance with natural channel design principles. Examples of these structures include, but are not limited to, placement of floodplain culverts to carry the 10-year ($Q_{10}$) and greater design floods, base flow culverts designed to carry the 2-year ($Q_2$) discharges, and box culverts or bridge spans properly sized to accommodate floodplain deposition.

**Applicability:**
- Channel modifications may be required to improve functionality and are limited to the minimum extent necessary upstream and downstream of the structure. All modifications must be in compliance with current permit regulations and must include natural channel design features.
- Associated Structures are not part of a larger natural channel design project.

**Requirements:**
- The project manager must complete and submit an Abbreviated Plan as described in the Plan Requirements Section.
- Structure design, with respect to natural channel design principles, must be performed under the supervision of, or reviewed and approved by, a Hydraulic Engineer and an Environmental Stream Team member.
- Permit application must be coordinated with the District Environmental Section.

**Restoration and Relocation** may include relocation of sections of existing channels to restore stable channel geometry, channel relocation associated with road crossings, installation of instream structures, bank stabilization and other natural channel design features included as part of a compensatory stream mitigation proposal. Restoration restores flow control and habitat features to a degraded or unstable stream reach and is often part of a compensatory stream mitigation proposal. Relocation includes movement of channel sections to accommodate structure or roadway design requirements. Channel work at either end of an Associated Structure is not included in this category. Channel relocation that employs natural channel design principles is typically considered as compensation for channel impacts on a 1:1 basis. Restoration and Relocation projects can be subdivided into small and large scale projects and have different reporting and documentation requirements, as described in the Plan Requirements Section.

**Applicability:**
- All channels
- All projects

**Requirements:**
- The project manager must provide the information listed under Plan Requirements, as required, below.

*Rev. 7/16*
Conceptual and Final Design plans must be submitted to the Central Office Natural Resources Section for Quality Assurance/Quality Control review by a multidisciplinary team.

7.4.4.4 Natural Channels Design Plan Requirements

The degree to which documentation is required for plan assemblies for projects employing natural channel design principles is dependent upon project scope, as described below. *

Hydraulic data must be provided with the summary sheet for all projects, unless otherwise approved by the Hydraulic Engineer.

Additional information may be required by permit agencies. The project manager is responsible for coordinating with district environmental staff prior to finalizing plans to determine if any additional information may be required.

Abbreviated Plan

* For Instream Structures and Associated Structures, the Project Manager will complete and submit a Natural Channel Design Project Summary Sheet to the District Environmental Manager and the Natural Resources Section Manager (Central Office, Environmental Division) for review. A copy should be placed in the project file.

* Plan view, profile and cross section drawings will be attached to document each structure.

Small-Scale Projects

* For projects less than 300 feet long, plans should include the following:

Conceptual Plan

  o Site Location Map
  o Reference reach measurements (bank full width and depth, valley slope, channel slope, bed material characterization)
  o Existing site conditions (including survey data)
  o Conceptual plan view of channel design
  o Buffer width, if applicable
  o R/W requirements, as applicable

*Rev. 7/16
Final Plan

- Project Summary Sheet
- Design storm discharges ($Q_{1.5}$, $Q_2$ and $Q_{10}$ discharges, at a minimum)
- Proposed typical channel cross section(s) and plan view showing thalweg and bank full features
- Grading Plan - Proposed Design with Profile (including alignment data)
- Gradient control structures and locations
- Structure Details
- Summary Sheet with approximate quantities for stream channel
- General notes for Grading, E&S control, and Incidentals (See IIM-LD-110)
- Sequence of Construction
- Planting Plan
- Quantities of plants, seed, fertilizer, and other incidental items required for the compensation site.
- Erosion and Sediment Control Plan
- Transport or other applicable construction estimate
- Applicable Special Provisions and Copied Notes for construction

Large-Scale Projects*

- For projects exceeding 300 feet in length, in addition to the items above, Final Plan documentation shall also include:
  - Natural Channel Design supporting data and analyses
  - Written summary – Project history and design considerations
  - Plant schedule with planting season
  - Monitoring plan and Success Criteria

NOTE: Some projects in small, intermittent or first order perennial watersheds that exceed the length thresholds above may be appropriately addressed with the information required for a Small-Scale Project (excluding required items such as the monitoring plan and success criteria). Individual exceptions must be reviewed and approved by the Central Office, Environmental Division Stream Restoration Specialist and the Location and Design Division River Mechanics Engineer, or their designees.

7.4.4.5 Natural Channels Design Plan Submittal and Review

The Natural Resource Section, Central Office Environmental Division, will serve as a central clearing house and contact point for QA/QC review of natural channel design projects. Reviews will be conducted by an interdisciplinary team.

Individual or associated structures do not require QA/QC review, however, submittal of the Abbreviated Plan is required.

*Rev. 7/16
Individual stream restoration plans will require submittal and review according to the QA/QC process timelines.

7.4.5 Riprap Channel Lining Design Procedure

In riprap lining design, it is first necessary to determine whether a lining is needed based upon the most appropriate method.

7.4.5.1 Riprap Lining Design Method for Major Channels (HEC-11)

If it is determined that riprap lining protection is required for a major stream channel, the design procedures employed by the department are predicated directly on the FHWA publication, "Design of Riprap Revetment" (HEC-11). This publication can be accessed and/or downloaded from the FHWA’s Internet web site at http://isddc.dot.gov/OLPFiles/FHWA/009881.pdf.

The riprap design procedure presented in HEC-11 is based on the equation:

\[
D_{50} = C \left[ \frac{0.001 V_a^3}{K_1^{1.5} \sqrt{d_{avg}}} \right]
\]  

(7.22)

Where:

- \(D_{50}\) = Median riprap particle size, ft
- \(C\) = Stone size correction factor
- \(V_a\) = Average velocity in the main channel, fps
- \(d_{avg}\) = Average flow depth in the main flow channel, ft

The equation for \(K_1\) is from HEC-11:

\[
K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}}
\]

(7.23)

*Rev. 7/16
Where:

\[ \theta = \text{Side slope angle (measured from the horizontal), deg.} \]
\[ \phi = \text{Natural angle of repose of material under consideration (measured from the horizontal), deg.} \]
\[ C = C_{sg} C_{SF} \]  

(7.24)

Where:

\[ C_{sg} = \text{Adjustment factor for specific gravity of stone} \]
\[ C_{SF} = \text{Adjustment factor for stability} \]
\[ C_{sg} = \frac{2.12}{(S_g - 1)^{1.5}} \]

Where:

\[ S_g = \text{Specific gravity of the rock riprap (Assume 2.65 for all VDOT standard riprap sizes.)} \]
\[ C_{SF} = \left( \frac{SF}{1.2} \right)^{1.5} \]  

(7.25)

Where:

\[ SF = \text{Stability Factor, See Appendix 7D-5, (Usually 1.2)} \]

Notes:

1. Nomographs for the solutions of \( \phi \), \( K_1 \), \( C \), and \( D_{50} \) are found in Appendix 7E.

2. \( K_1 \) in HEC-11 is the same variable as \( K_2 \) in HEC-15.

Convenient forms that can be used for manually performing these computations are Appendices 7B-4 for standard VDOT riprap sizes and 7B-5 for non-standard VDOT riprap sizes.
7.4.5.1.1 *Riprap Lining Design for Major Channels (HEC-11) Sample Problem*

Given: Channel is in a relatively straight section of stream. Curve radius/channel width (RW) > 30 with uniform flow. Wave action and floating debris are not a consideration. The average velocity is 5.4 fps. The average depth is 2'. The side slope ratios are 2:1 (26.57°). The angle of repose for the riprap is 42°. The specific gravity of the riprap is 2.65.

Determine:

Size of required riprap slope protection and appropriate VDOT standard riprap.

Solution:

*Step 1: Determine the stability factor (SF)*

From Appendix 7D-5, select a stability factor (SF) of 1.2.

**Step 2:** \( C_{SF} = \left( \frac{SF}{1.2} \right)^{1.5} = \left( \frac{1.2}{1.2} \right)^{1.5} = 1 \)

**Step 3:** \( C_{sg} = \frac{2.12}{\left( \frac{s}{g} - 1 \right)^{1.5}} = \frac{2.12}{\left( 2.65 - 1 \right)^{1.5}} = 1 \)

**Step 4:** \( C = C_{sg}C_{SF} = 1(1) = 1 \)

**Step 5:** \( K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}} = \sqrt{1 - \frac{\sin^2 (26.57)}{\sin^2 (42)}} = 0.74 \)

**Step 6:** \( D_{50} = C \left[ \frac{0.001V_3^3}{K_1^{1.5}d_{avg}} \right] = 1 \left[ \frac{0.001(5.4)^3}{(0.74)^{1.5} \sqrt{2}} \right] = 0.17 \text{ ft.} \)

**Step 7:** Closest standard VDOT riprap size = Class Al (D_{50} = 0.8 ft).
7.4.5.2 Riprap Lining Design Method for Minor Channels (HEC-15)

The following design procedures employed by the department are predicated directly on the FHWA publication, “Design of Roadside Channels With Flexible Linings” (HEC-15). This publication can be accessed and/or downloaded from the FHWA’s Internet website at [http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf](http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf). Once it has been determined that the natural or backfill material is unstable, using either (1) the Tractive Force procedure, (2) a table of allowable velocities for specific soil types such as Appendix 7D-2 and 7D-6, or (3) engineering judgment, select a trial riprap size and proceed as follows:

**Step 1:** Determine the permissible shear stress \( (\tau_p) \) for the riprap size selected

Use 3.2 lbs/ft\(^2\) for Class AI Riprap and 4.4 lbs/ft\(^2\) for Class I Riprap.

**Step 2:** Select a trial flow depth range for the ditch configuration using the following table from HEC-15:

<table>
<thead>
<tr>
<th>Riprap Size</th>
<th>0-0.5'</th>
<th>0.5-2.0'</th>
<th>&gt;2.0'</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class AI</td>
<td>0.104</td>
<td>0.069</td>
<td>0.035</td>
</tr>
<tr>
<td>Class I</td>
<td>--</td>
<td>0.078</td>
<td>0.040</td>
</tr>
</tbody>
</table>

**Step 3:** Using the n-value for the selected trial flow depth range, calculate the actual depth of flow for the ditch configuration using the actual design discharge and ditch slope using an appropriate method such as Appendix 7C-3. If the calculated depth is within the trial depth range selected, proceed to the next step. If it is not, select another trial depth range and try again.

**Step 4:** Calculate the actual shear stress \( (\tau_o) \):

\[
\tau_o = \gamma d S_o
\]  

Where:

\( \gamma = \) Unit weight of water (62.4 lbs/ft\(^3\))
\( d = \) Depth of flow, ft.
\( S = \) Average ditch flowline slope, ft/ft

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*Rev. 9/09*
If $\tau_0 > \tau_p$, the selected riprap size is too small. Choose the next larger size and try again.

If the ditch side slopes are steeper than 3:1, it is necessary to perform the additional calculations shown below:

**Step 5:** *Determine the angle of repose for the riprap size determined above.*

It should be noted that all VDOT standard riprap sizes are assumed to have angles of repose of $42^\circ$.

**Step 6:** *Determine the ratio of maximum side shear to maximum bottom shear ($K_1$) using Appendix 7E-7.*

**Step 7:** *Determine tractive force ratio ($K_2$) from Appendix 7E-8.*

**Step 8:** *Calculate required $D_{50}$ for side slopes ($D_{50 \text{ side}}$)*

$$D_{50 \text{ side}} = \frac{K_1}{K_2} D_{50 \text{ bottom}}$$

From a practical standpoint, whatever riprap size is indicated for the ditch side slope should be used on the bottom as well.

### 7.4.5.2.1 Channel Stability Sample Problem - Tractive Force Calculation

Check the stability of channel's native material using tractive force calculation. Note that the “RIPRAP” computer program referenced in section 7.4.6.4.4 could be employed to perform this computation.

**Given:**
A natural channel with a bed and banks of native materials composed of cobbles and pebbles. Mean diameter is 1.25" for the $D_{75}$ size stone. The channel bottom width ($B$) is 10'. The longitudinal slope is 0.008 ft/ft. Its side slope is 2(h):1(v). The flow ($Q$) is 150 cfs at a depth ($d$) of 2'.

Determine whether the channel is stable for the indicated condition.

**Solution:**

**Step 1:** *Determine the permissible shear stress ($\tau_p$)*

From Appendix 7E-2, for a $D_{75}$ particle diameter of 1.25", read a permissible tractive force ($\tau_p$) on the channel bottom of 0.5 lbs/ft².

**Step 2:** *For a side slope ratio of 2:1, the sine of the slope angle ($\theta = 26.6^\circ$) is 0.5*

*Rev. 9/09
Step 3: From Appendix 7E-1, for a particle diameter of 1.25”, read an angle of repose \((\phi)\) of 40°

The sine of 40° = 0.643

Step 4: \(K = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}} = \sqrt{1 - \frac{0.447^2}{0.643^2}} = 0.72\)

Step 5: Permissible tractive force on the side slopes \((\tau_s)\)

\(\tau_s = K \tau_p = 0.72(0.05) = 0.36 \text{ lbs/ft.}^2\)

Step 6: Compute the width to depth ratio

\[\frac{B}{d} = \frac{10}{2} = 5, \text{ which is less than 10, therefore use } \tau_o = 62.4 R S_o\]

Step 7: Flow cross-sectional area

\[A = Bd + zd^2\]
\[= 10(2) + 2(2)^2\]
\[= 28 \text{ ft.}^2\]

Step 8: Wetted perimeter

\[P = B + 2d\sqrt{1 + z^2}\]
\[= 10 + 2(2)\sqrt{1 + 2^2}\]
\[= 18.94 \text{ ft.}\]

Step 9: Hydraulic radius

\[R = \frac{A}{P} = \frac{28}{18.94} = 1.48 \text{ ft.}\]

Step 10: Compute the average tractive force

\(\tau_o = 62.4 R S_o\)
\[= 62.4(1.48)(0.008)\]
\[= 0.74 \text{ lbs / ft.}^2\]
Step 11: Compare the average tractive force of the channel to the allowable tractive force of the channel sides and bottom

\[ \tau_o \text{ of } 0.74 > \tau_p \text{ (bottom) of } 0.5 \text{ lb/ft}^2 \text{ or } \tau_s \text{ (side slope) of } 0.36 \text{ lb/ft}^2 \]

Native material is unstable. Channel protection is required.

7.4.5.2.2 Riprap Lining Design for Minor Channels (HEC-15) Sample Problem

Given: It has been determined that a special design ditch connected to culvert cross drain pipes will need a riprap lining. The ditch has a bottom width (B) of 2', 2:1 side slopes (z = 2), and a slope along the ditch line (S) of 0.005 ft/ft. The design discharge is 20 cfs.

Determine what size VDOT standard riprap will be required?

Step 1: Try Class AI standard riprap, \( D_{50} = 0.8' \)

\[ \tau_p = 3.2 \text{ lb/ft}^2 \]

Step 2: Assume a depth range of from 0.5-2.0 ft. with an \( n \)-value of 0.069

Step 3: Using Appendix 7E-3 for \( S = 0.005 \text{ ft/ft}, \ B = 2 \text{ ft}, \ z = 2, \ Q_n = 20 \times (0.069) = 1.38, \)

Read \( d/B = 0.78. \ d = 0.78 \times 2 = 1.56 \text{ ft.} \ 0.5 < 1.56 < 2.0, \)

Therefore, the assumed \( n \)-value of 0.069 is acceptable.

Step 4: Determine the maximum shear stress

\[ \tau_{\text{max}} = \gamma d S_o = 62.4(1.56)(0.005) = 0.49 \text{ lb/ft}^2 \]

Step 5: Evaluate the maximum shear stress with the permissible shear stress

\[ \tau_{\text{max}} < \tau_p, \]

0.49 < 3.2

Therefore, Class AI riprap is acceptable for the ditch bottom and side slopes of 3:1 or flatter. However, since the side slopes are 2:1, proceed with checking the side slope stability

Step 6: Determine the angle of repose for Class AI riprap

\[ \phi = 42^\circ \]
Step 7: Determine $K_1$

From Appendix 7E-7, for $B/d = 2 / 1.56 = 1.28$ and $z = 2$, read $K_1 = 0.9$

Step 8: Determine $K_2$

From Appendix 7E-8 for $z = 2$ and angle of repose = $42^\circ$, read $K_2 = 0.73$. 

Step 9: Determine the $D_{50}$ for the side slopes

\[ D_{50 \text{ side}} = \frac{K_1}{K_2} D_{50 \text{ bottom}} = \frac{0.90}{0.73} (0.8) = 0.98 \text{ ft}. \]

Therefore, it would probably be best to use Standard Class I riprap ($D_{50} \equiv 1.1'$) for both the channel bottom and side slopes in lieu of the originally proposed Class AI ($D_{50} = 0.8'$).
References


American Society of Civil Engineers, Special Committee on Irrigation Research, 1926.


Behlke, C.E. The Design of Supercritical Flow Channel Junctions, Highway Research Record No. 123, Transportation Research Board. 1966.


*Rev. 5/17


Lane, E.W. A Study of the Shape of Channels Formed by Natural Stream Flowing in Erodible Material, M.R.D. Sediment Series No. 9, U.S. Army Engineers Division, Missouri River, Corps of Engineers, Omaha, Nebraska. 1957.


*Rev. 9/09


U.S. Army Corps of Engineers. HEC-6, Scour and Deposition in Rivers and Reservoirs, The Hydrologic Engineering Center, Davis, CA. \(\text{(legacy software)}\)

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*Rev. 9/09*
### Appendix 7B-1  LD-268V  Roadside and Median Ditch Design Form – Permissible Velocity

<table>
<thead>
<tr>
<th>STA. TO STA.</th>
<th>FLOW</th>
<th>0.9</th>
<th>0.5</th>
<th>0.3</th>
<th>CA</th>
<th>Tc</th>
<th>I2</th>
<th>Q2</th>
<th>C or F</th>
<th>Slope Ft/Ft</th>
<th>ALLOW VEL.</th>
<th>n=0.03</th>
<th>n=0.05</th>
<th>n=0.015</th>
<th>I10</th>
<th>Q10</th>
<th>DEP</th>
<th>REMARKS</th>
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</table>

**Note:**
- **STANDARD:**
- **VALUES:**
- **SAVE CLEAR:**
- **DATE:**
- **PROJECT:**
- **DEP:**
- **Remark:**

**Legend:**
- **D:**
- **CUT:**
- **MED:**
- **FILL:**

**Earth Protective Lining**

**WS/CA**

**INCR. ACC.**

**Slopes**

**C or F**

**Allow Velocity**

**n=0.03**

**n=0.05**

**n=0.015**

**I10**

**Q10**

**DEP**

**Remarks**

*Disclaimer: This is a sample table and diagram for educational purposes. Actual forms may vary.*
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<th>0.3</th>
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<th>Tc</th>
<th>I2</th>
<th>Q2</th>
<th>C or F</th>
<th>Slope Ft/Ft</th>
<th>ALLOW VEL.</th>
<th>I10</th>
<th>Qn</th>
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</table>
## Worksheet for Non-Uniform Flow in Open Channels

<table>
<thead>
<tr>
<th>STA</th>
<th>$H_1$</th>
<th>$d_1$</th>
<th>$A_2$</th>
<th>$H_2$</th>
<th>$d_2$</th>
<th>$A_2$</th>
<th>$R_2$</th>
<th>$P_2$</th>
<th>$S_2$</th>
<th>$S_a$</th>
<th>$L$</th>
<th>$Z_w$</th>
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<tr>
<td></td>
<td>$y + \frac{x}{2}$</td>
<td>$y + \frac{x}{2}$</td>
<td>$y + \frac{x}{2}$</td>
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</tbody>
</table>

**Source:**

Appendix 7B-2 Water Surface Profile Calculation Form
## CHANNEL STABILITY WORK SHEET

### CHANNEL DATA

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q</td>
<td>______(cfs)</td>
</tr>
<tr>
<td>P</td>
<td>______(ft.)</td>
</tr>
<tr>
<td>Native Material</td>
<td></td>
</tr>
<tr>
<td>S_o</td>
<td>______(ft/ft)</td>
</tr>
<tr>
<td>R</td>
<td>______(ft.)</td>
</tr>
<tr>
<td>D50</td>
<td>______</td>
</tr>
<tr>
<td>d_n</td>
<td>______(ft.)</td>
</tr>
<tr>
<td>V_n</td>
<td>______(fps)</td>
</tr>
<tr>
<td>D75</td>
<td>______</td>
</tr>
<tr>
<td>A</td>
<td>______(ft²)</td>
</tr>
<tr>
<td>Side Slope</td>
<td>_____ :1</td>
</tr>
<tr>
<td>n</td>
<td>______</td>
</tr>
</tbody>
</table>

### STABILITY OF NATIVE MATERIAL

\[
\tau_o = 62.4 \cdot R \cdot S_o = 62.4 \cdot \text{____}_1 \cdot \text{____}_2 = \text{____}_3
\]

\[
\tau_p \text{ Bed} = \text{____}_4 \quad (\text{Appendix 7E-2 or 3})
\]

For D50 = _______  \( \phi = \text{____}_5 \, ^\circ \)  (Appendix 7E-1)

For D75 = _______  \( \phi = \text{____}_6 \, ^\circ \)  (Appendix 7E-9)

Side Slope = ___ :1  \( \theta = \text{____}_7 \, ^\circ \)

\[
K_1 = \left[ 1 - \left( \sin^2 \theta / \sin^2 \phi \right) \right]^{0.5}
\]

\[
K_1 = \left[ 1 - \left( \sin^2 \text{____}_8 ^\circ / \sin^2 \text{____}_9 ^\circ \right) \right]^{0.5} = \text{____}_0
\]

\[
\tau_s \text{ Side Slope (SS)} = \tau_p \text{ Bed} \cdot K = \text{____}_1 \cdot \text{____}_2 = \text{____}_3
\]

\[
\tau_p \text{ Bed} \text{ (____}_4 \text{)} \text{ (<)} \text{ (=)} \text{ (>) } \tau_o \text{ (____}_5 \text{)}
\]

\[
\therefore \text{ Native Material on Bed is (stable) (unstable)}
\]

\[
\tau_s \text{ SS (____}_1 \text{)} \text{ (<)} \text{ (=)} \text{ (>) } \tau_o \text{ (____}_2 \text{)}
\]

\[
\therefore \text{ Native Material on Side Slope is (stable) (unstable)}
\]

Source: VDOT
Appendix 7B-4  RIPRAP DESIGN WORK SHEET
FOR STANDARD VDOT RIPRAP SIZES ONLY

CHANNEL DATA
Q = _________(cfs)   P = _________(ft.)   n = _______
S₀ = _________(ft/ft)   R = _________(ft.)
dₙ = _________(ft.)   Vₙ = _________(fps)
A = _________(ft.²)   Side Slope = _____ :1

DETERMINE RIPRAP SIZE
φ = 42°   Side Slope = _____ :1   θ = ______°

K₁ = [1 - (sin² θ / sin² φ)] 0.5
K₁ = [1 - (sin² ______° / sin² 42°)] 0.5 = ______

For Specific Gravity = 2.65 and Stability Factor = 1.2
D₅₀ = 0.001 • Vₐ³ / (davg 0.5 • K₁ 1.5)
D₅₀ = 0.001 • ______³ / ( _______ 0.5 • _______ 1.5)
D₅₀ Computed = ______

Note: All VDOT standard riprap (Class AI through Type II) is assumed to have a φ of approximately 42° and a Specific Gravity of 2.65. Therefore, the Computed D₅₀ should be adjusted by the Stability Correction Factor (CSF) (if any) to derive a Final D₅₀. The VDOT standard class of riprap with the next higher D₅₀ should be specified.

Correction Factor For Stability Factor (SF) other than 1.2 (Default = 1.0)
CSF = (SF / 1.2) 1.5 = ( _______ / 1.2) 1.5 = ______

Final D₅₀ = CSF • Computed D₅₀ = _______ • _______ = _______

RIPRAP RECOMMENDATION: VDOT (Class) (Type) _________
Thickness (T ) = _________” (2 • D₅₀ MSD minimum)

Source: VDOT
### Appendix 7B-5

#### RIPRAP DESIGN WORK SHEET

FOR OTHER THAN VDOT STANDARD RIPRAP SIZES

<table>
<thead>
<tr>
<th>CHANNEL DATA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q = _________(cfs)</td>
</tr>
<tr>
<td>S₀ = _________(ft/ft)</td>
</tr>
<tr>
<td>dₙ = _________(ft.)</td>
</tr>
<tr>
<td>A = _________(ft²)</td>
</tr>
</tbody>
</table>

| ASSUMED RIPRAP SIZE - D₅₀ = ________ |

### VERIFY ASSUMED RIPRAP SIZE

\[ \phi = \quad \quad ^\circ \quad (\text{Appendix 7E-1}) \]
Side Slope = _____ : 1  \quad \theta = \quad \quad ^\circ \\

\[ K_1 = \left[ 1 - \left( \frac{\sin^2 \theta}{\sin^2 \phi} \right) \right]^{0.5} \]

\[ K_1 = \left[ 1 - \left( \frac{\sin^2 \theta}{\sin^2 \phi} \right) \right]^{0.5} = \quad \]

For Specific Gravity = 2.65 and Stability Factor = 1.2

\[ D_{50} = 0.001 \cdot \frac{V_a^3}{(d_{avg}^{0.5} \cdot K_1^{1.5})} \]

\[ D_{50} = 0.001 \cdot \frac{\quad}{(\quad^{0.5} \cdot \quad^{1.5})} = \quad \]

D₅₀ Computed (______ ) (<) (=) (> D₅₀ Assumed (______ )

Assumed D₅₀ is (correct) (incorrect)

Note: The above process of assuming a D₅₀ size, determining the natural angle of repose (\(\phi\)) and computing a D₅₀ size should be repeated until the Assumed D₅₀ size equals the Computed D₅₀ size. Once the D₅₀ size determination has been made, it should be adjusted for the Specific Gravity Correction Factor \(C_{sg}\) (if any) and the Stability Correction Factor \((C_{SF})\) (if any) to derive a Final D₅₀.

### Correction Factor For Riprap Specific Gravity (\(Sₙ\)) other than 2.65 (Default = 1.0)

\[ C_{sg} = 2.12 / (Sₙ - 1)^{1.5} = 2.12 / (\quad - 1)^{1.5} = \quad \]

### Correction Factor For Stability Factor (\(SF\)) other than 1.2 (Default = 1.0)

\[ C_{SF} = (SF / 1.2)^{1.5} = (\quad / 1.2)^{1.5} = \quad \]

Final Correction Factor = \(C = C_{sg} \cdot C_{SF} = \quad \cdot \quad = \quad \)

Final D₅₀ = \(C \cdot \text{Computed } D_{50} = \quad \cdot \quad = \quad \)

RIPRAP RECOMMENDATION: ___________________

Thickness (T ) = _________” (2 × D₅₀ MSD minimum)

Source: VDOT
# Chapter 7 – Ditches and Channels

## Appendix 7B-06  Natural Channel Design Project Summary Sheet

**NATURAL CHANNEL DESIGN**

**PROJECT SUMMARY SHEET**

<table>
<thead>
<tr>
<th>Project Number:</th>
<th>Permit Number:</th>
<th>Project Name:</th>
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<tr>
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<td>HUC:</td>
<td>City or County:</td>
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<td>District:</td>
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<tr>
<td>Latitude:</td>
<td>FEMA Floodplain:</td>
<td>Residency:</td>
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<td>Yes</td>
<td>No</td>
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<td>Date:</td>
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<td>Longitude:</td>
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<td>USGS Quad. (Attach):</td>
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<tr>
<td>Site Location:</td>
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**Site Description (include goals and objectives)**

**Type:** (check ONE box only)

- [ ] Instream Structure *
- [ ] Associated Structure *

<table>
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<tr>
<th>Intermittent</th>
<th>Perennial</th>
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* For instream structures and associated structures, attach plan and profile view drawings.

**Geomorphic Description:**

<table>
<thead>
<tr>
<th>Bankfull width</th>
<th>Floodplain width</th>
<th>Channel slope</th>
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<tr>
<td>Bankfull depth</td>
<td>Bankfull cross sectional area</td>
<td>Valley slope</td>
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</tbody>
</table>

**Bed Material**

- [ ] Bedrock
- [ ] Boulder
- [ ] Cobble
- [ ] Gravel
- [ ] Sand
- [ ] Siltclay

**Rosenz Class:**

**Valley Type:**

**Hydraulic Analysis Complete:**

- [ ] Based on the scope of the project, additional hydraulic analysis is not required.

(Attach documentation.)

- [ ] This project has been reviewed and provides a hydraulically equivalent replacement structure that does not require additional coordination or review. See attached LD-1095A.

**Design storm data provided by a Hydraulic Engineer:**

<table>
<thead>
<tr>
<th>Design Storm cfs</th>
<th>Design Storm cfs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q&lt;sub&gt;1,5&lt;/sub&gt; Discharge</td>
<td>Q&lt;sub&gt;100&lt;/sub&gt; Discharge</td>
</tr>
</tbody>
</table>

Place a copy of the completed form in the project file and provide a copy of this form to:

C.O. Environmental Division
Natural Resources Program Manager
1201 East Broad Street
Richmond, VA 23219

1 of 2  VDOT Drainage Manual
Appendix 7C-2  Trapezoidal Channel Capacity Chart

Source:
Appendix 7C-3  
Nomograph for Solution of Normal Depth

NOTE: Project horizontally from $Z=0$ scale to obtain values for $Z=1$ to 6

EXAMPLE:

GIVEN: $S=0.01$, $Q=10\text{ FT}^3/\text{s}$, $n=0.03$, $B=4\text{ FT}$, $Z=4$

FIND: $d$, $Q_n$

SOLUTION:

$Q_n = 0.3$
$d/B = 0.14$
$d = 0.14 \cdot 4 = 0.56\text{ FT}$

Source: HEC-15 (Archived) 1988
Appendix 7C-4

Side Ditch Flow Chart
(Side Slopes = 6:1, 1.5:1)

Source: VDOT
Appendix 7C-5  Side Ditch Flow Chart  (Side Slopes = 4:1, 1:1)

Source: VDOT
Appendix 7C-6
Side Ditch Flow Chart
(Side Slopes = 4:1, 1.5:1)

Source: VDOT
Appendix 7C-9  Side Ditch Flow Chart
(Side Slopes = 3:1, 3:1)

Source: VDOT
Appendix 7C-11 Side Ditch Flow Chart
(Side Slopes = 6:1, 4:1)

Source: VDOT
Appendix 7C-12  Triangular Median Ditch Flow Chart
(Side Slopes = 6:1, 6:1)

Source: VDOT
Appendix 7C-13 Triangular Median Ditch Flow Chart (Side Slopes = 4:1, 4:1)

Source: VDOT
Appendix 7C-14  Triangular Median Ditch Flow Chart
(Side Slopes = 2:1, 2:1)

Source: VDOT
Appendix 7C-15  Toe Ditch Flow Chart
(Side Slopes = 1.5:1, 1.5:1)

Source: VDOT
Appendix 7C-17  Trapezoidal Ditch Flow Chart
(B=1’, Side Slopes = 1.5:1)

Source: HDS-3
Appendix 7C-18     Trapezoidal Channel Flow Chart  
(B=2’, Side Slopes = 2:1)  

Source: HDS-3
Appendix 7C-19     Trapezoidal Channel Flow Chart  
(B=3’, Side Slopes = 2:1) 

Source: HDS-3
Appendix 7C-20  
Trapezoidal Channel Flow Chart  
(B=4', Side Slopes = 2:1)  

Source: HDS-3
Appendix 7C-21  Trapezoidal Channel Flow Chart  
(B = 5', Side Slopes = 2:1)
Appendix 7C-22     Trapezoidal Channel Flow Chart
(B = 6', Side Slopes 2:1)

Source: HDS-3
Appendix 7C-23  Trapezoidal Channel Flow Chart
(B = 7', Side Slopes = 2:1)

Source: HDS-3
Appendix 7C-25  Trapezoidal Channel Flow Chart
(B = 9’, Side Slopes = 2:1)

Source: HDS-3
Appendix 7C-26  Trapezoidal Channel Flow Chart
(B = 10', Side Slopes = 2:1)

Source: HDS-3
Appendix 7C-27  Trapezoidal Channel Flow Chart
(B = 12', Side Slopes = 2:1)

Source: HDS-3
Appendix 7C-28 Trapezoidal Channel Flow Chart
(B = 14', Side Slopes = 2:1)

Source: HDS-3
Appendix 7C-29  
Trapezoidal Channel Flow Chart  
(B = 16', Side Slopes = 2:1)

Source: HDS-3
Appendix 7C-30  Trapezoidal Channel Flow Chart  
(B = 18', Side Slopes = 2:1)

Source: HDS-3
Appendix 7C-31  Trapezoidal Channel Flow Chart
(B = 20', Side Slopes = 2:1)

Source: HDS-3
# Appendix 7D-1  Values of Roughness Coefficient n (Uniform Flow)

<table>
<thead>
<tr>
<th>Type of Channel and Description</th>
<th>Minimum</th>
<th>Normal</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>LINED CHANNELS (Selected linings)</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Concrete</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Trowel finish</td>
<td>0.011</td>
<td>0.013</td>
<td>0.015</td>
</tr>
<tr>
<td>2. Float finish</td>
<td>0.013</td>
<td>0.015</td>
<td>0.016</td>
</tr>
<tr>
<td>3. Gunite, good section</td>
<td>0.016</td>
<td>0.019</td>
<td>0.023</td>
</tr>
<tr>
<td>b. Asphalt</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Smooth</td>
<td>0.013</td>
<td>0.013</td>
<td>-</td>
</tr>
<tr>
<td>2. Rough</td>
<td>0.016</td>
<td>0.016</td>
<td>-</td>
</tr>
<tr>
<td>c. Riprap (sfd VDOT sizes)*</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Class 1A</td>
<td>0.033</td>
<td>0.038</td>
<td>-</td>
</tr>
<tr>
<td>2. Class 1</td>
<td>0.035</td>
<td>0.040</td>
<td>-</td>
</tr>
<tr>
<td>3. Class 2</td>
<td>0.037</td>
<td>0.042</td>
<td>-</td>
</tr>
<tr>
<td>4. Class 3</td>
<td>0.039</td>
<td>0.045</td>
<td>-</td>
</tr>
<tr>
<td>5. Type I</td>
<td>0.041</td>
<td>0.047</td>
<td>-</td>
</tr>
<tr>
<td>6. Type II</td>
<td>0.044</td>
<td>0.050</td>
<td>-</td>
</tr>
</tbody>
</table>

| **EXCAVATED OR DREDGED**         |         |         |         |
| a. Earth, straight and uniform  |         |         |         |
| 1. Clean, recently completed    | 0.016   | 0.018   | 0.020   |
| 2. Clean, after weathering      | 0.018   | 0.022   | 0.025   |
| 3. Gravel, uniform section, clean | 0.022 | 0.025 | 0.030 |
| 4. With short grass, few weeds  | 0.022   | 0.027   | 0.033   |
| b. Earth, winding and sluggish  |         |         |         |
| 1. No vegetation               | 0.023   | 0.025   | 0.030   |
| 2. Grass, some weeds           | 0.025   | 0.030   | 0.033   |
| 3. Dense weeds or aquatic plants in deep channels | 0.030 | 0.035 | 0.040 |
| 4. Earth bottom and rubble sides | 0.025 | 0.030 | 0.035 |
| 5. Stony bottom and weedy sides | 0.025 | 0.035 | 0.045 |
| 6. Cobble bottom and clean sides | 0.030 | 0.040 | 0.050 |
| c. Dragline excavated or dredged |         |         |         |
| 1. No vegetation               | 0.025   | 0.028   | 0.033   |
| 2. Light brush on banks         | 0.035   | 0.050   | 0.060   |
| d. Rock cuts                    |         |         |         |
| 1. Smooth and uniform           | 0.025   | 0.035   | 0.040   |
| 2. Jagged and irregular         | 0.035   | 0.040   | 0.050   |
| e. Channels not maintained, weeds and brush uncut |         |         |         |
| 1. Dense weeds, high as flow depth | 0.050 | 0.080 | 0.120 |
| 2. Clean bottom, brush on sides | 0.040 | 0.050 | 0.080 |
| 3. Same, highest stage of flow  | 0.045   | 0.070   | 0.110   |
| 4. Dense brush, high stage      | 0.080   | 0.100   | 0.140   |

| **NATURAL STREAMS**             |         |         |         |
| 1. Minor streams (top width at flood stage <100 ft) |         |         |         |
| a. Streams on Plain             |         |         |         |
| 1. Clean, straight, full stage, no rifts or pools | 0.025 | 0.030 | 0.033 |
| 2. Same as above, but more stones/weeds | 0.030 | 0.035 | 0.040 |
| 3. Clean, winding, some pools/shoals | 0.033 | 0.040 | 0.045 |
| 4. Same as above, but some weeds/stones | 0.035 | 0.045 | 0.050 |
| 5. Same as above, lower stages, more ineffective slopes and sections | 0.040 | 0.048 | 0.055 |
| 6. Same as 4, but more stones | 0.045 | 0.050 | 0.060 |
| 7. Sluggish reaches, weedy, deep pools | 0.050 | 0.070 | 0.080 |

* Rev 7/09
### Chapter 7 – Ditches and Channels

#### Appendix 7D-1 Values of Roughness Coefficient n (Uniform Flow)

<table>
<thead>
<tr>
<th>Type of Channel and Description</th>
<th>Minimum</th>
<th>Normal</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush</td>
<td>0.075</td>
<td>0.100</td>
<td>0.150</td>
</tr>
<tr>
<td>b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Bottom: gravels, cobbles and few boulders</td>
<td>0.030</td>
<td>0.040</td>
<td>0.050</td>
</tr>
<tr>
<td>2. Bottom: cobbles with large boulders</td>
<td>0.040</td>
<td>0.050</td>
<td>0.070</td>
</tr>
<tr>
<td>2. Floodplains</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Pasture, no brush</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Short grass</td>
<td>0.025</td>
<td>0.030</td>
<td>0.035</td>
</tr>
<tr>
<td>2. High grass</td>
<td>0.030</td>
<td>0.035</td>
<td>0.050</td>
</tr>
<tr>
<td>b. Cultivated area</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. No crop</td>
<td>0.020</td>
<td>0.030</td>
<td>0.040</td>
</tr>
<tr>
<td>2. Mature row crops</td>
<td>0.025</td>
<td>0.035</td>
<td>0.045</td>
</tr>
<tr>
<td>3. Mature field crops</td>
<td>0.030</td>
<td>0.040</td>
<td>0.050</td>
</tr>
<tr>
<td>c. Brush</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Scattered brush, heavy weeds</td>
<td>0.035</td>
<td>0.050</td>
<td>0.070</td>
</tr>
<tr>
<td>2. Light brush and trees, in winter</td>
<td>0.035</td>
<td>0.050</td>
<td>0.060</td>
</tr>
<tr>
<td>3. Light brush and trees, in summer</td>
<td>0.040</td>
<td>0.060</td>
<td>0.080</td>
</tr>
<tr>
<td>4. Medium to dense brush, in winter</td>
<td>0.045</td>
<td>0.070</td>
<td>0.110</td>
</tr>
<tr>
<td>5. Medium to dense brush, in summer</td>
<td>0.070</td>
<td>0.100</td>
<td>0.160</td>
</tr>
<tr>
<td>d. Trees</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Dense Willows, summer, straight</td>
<td>0.110</td>
<td>0.150</td>
<td>0.200</td>
</tr>
<tr>
<td>2. Cleared land with tree stumps, no sprouts</td>
<td>0.030</td>
<td>0.040</td>
<td>0.050</td>
</tr>
<tr>
<td>3. Same as above, but with heavy growth of sprouts</td>
<td>0.050</td>
<td>0.060</td>
<td>0.080</td>
</tr>
<tr>
<td>4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches</td>
<td>0.080</td>
<td>0.100</td>
<td>0.120</td>
</tr>
<tr>
<td>5. Same as above, but with flood stage reaching branches</td>
<td>0.100</td>
<td>0.120</td>
<td>0.160</td>
</tr>
<tr>
<td>3. Major Streams (top width at flood stage &gt; 100 ft)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>The n-value is less than that for minor streams of similar description, because banks offer less effective resistance.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Regular section with no boulders or brush</td>
<td>0.025</td>
<td>-</td>
<td>0.060</td>
</tr>
<tr>
<td>b. Irregular and rough section</td>
<td>0.035</td>
<td>-</td>
<td>0.100</td>
</tr>
</tbody>
</table>

Source: Chow, V.T., FHWA’s HDS-6 publication

* For bare earth linings when the soil classifications in accordance with either AASHTO or USCS designations are known, use the Manning’s “n” values recommended in the appropriate table from Appendix 7D-2

* Rev 7/09
### Appendix 7D-2  Recommended Maximum Water Velocities and Manning’s n as a Function of Soil Type and Flow Depth

<table>
<thead>
<tr>
<th>ASSHTO Classification</th>
<th>ASSHTO Soil Description</th>
<th>Fortier and Scobey Soil Description</th>
<th>Maximum Water Velocity (ft/s)</th>
<th>Manning’s n -Flow Depth 0.5-2.0 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>BROKEN ROCK and COBBLES</td>
<td>Cobble and Shingles</td>
<td>5.5</td>
<td>0.030</td>
<td></td>
</tr>
<tr>
<td>A-1-a</td>
<td>Stone fragments or GRAVEL, with or without well-graded binder</td>
<td>Coarse gravel, non-colloidal</td>
<td>4.5</td>
<td>0.025</td>
</tr>
<tr>
<td>same</td>
<td>same</td>
<td>Fine gravel</td>
<td>3.5</td>
<td>0.020</td>
</tr>
<tr>
<td>A-1-b</td>
<td>Coarse SAND, with or without well-graded binder</td>
<td>Graded loam to cobbles when non-colloidal</td>
<td>4.0</td>
<td>0.030</td>
</tr>
<tr>
<td>A-2 (A-2-4, A-2-5, A-2-6, A-2-7)</td>
<td>Mixture of GRAVEL and SAND, with silty or clay fines, or nonplastic silt fines</td>
<td>Graded silts to cobbles when colloidal</td>
<td>4.5</td>
<td>0.030</td>
</tr>
<tr>
<td>same</td>
<td>same</td>
<td>Sandy loam, non-colloidal</td>
<td>2.0</td>
<td>0.020</td>
</tr>
<tr>
<td>A-3</td>
<td>Fine SAND, without silty clay fines; e.g. beach sand or stream-deposited fine sand</td>
<td>Fine Sand, non-colloidal</td>
<td>1.5</td>
<td>0.020</td>
</tr>
<tr>
<td>same</td>
<td>same</td>
<td>Silt loam, non-colloidal</td>
<td>2.3</td>
<td>0.020</td>
</tr>
<tr>
<td>A-4</td>
<td>Non- to moderately plastic SILT; mixtures of silt, sand, and/or gravel, with a minimum silt content of 36%</td>
<td>Alluvial silts, non-colloidal</td>
<td>2.3</td>
<td>0.020</td>
</tr>
<tr>
<td>A-5</td>
<td>Moderately to highly plastic SILT; Soil; mixtures of silt, sand, and/or gravel, with a minimum fines content of 36%</td>
<td>Ordinary firm loam</td>
<td>2.5</td>
<td>0.020</td>
</tr>
<tr>
<td>A-6</td>
<td>Plastic CLAY soil; mixtures of clay, sand, and/or gravel, with a minimum fines content of 36%</td>
<td>Alluvial silts, colloidal</td>
<td>3.5</td>
<td>0.025</td>
</tr>
<tr>
<td>A-7</td>
<td>Moderately to highly plastic CLAY; mixtures of clay, sand, and/or gravel, with a minimum clay content of 36%</td>
<td>Stiff clay, very colloidal</td>
<td>4.0</td>
<td>0.025</td>
</tr>
</tbody>
</table>

1) Well-graded-containing a broad range of particle sizes with no intermediate sizes missing.
2) Binder - soil particles consisting of fine sand, silt, and clay.
3) Fines - particle sizes finer than 0.074 mm (e.g., silt and clay particles).
4) Plasticity - ability of a soil mass to deform at constant volume without cracking or crumbling.
+ Relationship between AASHTO classification and Fortier and Scobey description is loosely correlated.
<table>
<thead>
<tr>
<th>USCS Classification</th>
<th>USCS Soil Description</th>
<th>Fortier and Scobey Soil Description</th>
<th>Maximum Water Velocity (ft/s)</th>
<th>Manning's $n$ Flow Depth 0.5-2.0 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>BROKEN ROCK and COBBLES</td>
<td>Cobble and Shingles</td>
<td>5.5</td>
<td>0.030</td>
<td></td>
</tr>
<tr>
<td>GP, GW, SW, SP</td>
<td>Poorly graded gravel, well graded gravel, well graded sand</td>
<td>Coarse gravel, non-colloidal</td>
<td>4.5</td>
<td>0.025</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fine gravel</td>
<td>3.5</td>
<td>0.020</td>
</tr>
<tr>
<td>SW</td>
<td>Well graded sand</td>
<td>Graded loam to cobbles when non-colloidal</td>
<td>4.0</td>
<td>0.030</td>
</tr>
<tr>
<td>GC, SC</td>
<td>Clayey gravel, clayey sand</td>
<td>Graded silts to cobbles when colloidal</td>
<td>4.5</td>
<td>0.030</td>
</tr>
<tr>
<td>SM</td>
<td>Silty sand</td>
<td>Sandy loam, non-colloidal</td>
<td>2.0</td>
<td>0.020</td>
</tr>
<tr>
<td>SP, SW</td>
<td>Poorly graded sand, well graded sand</td>
<td>Fine Sand, non-colloidal</td>
<td>1.5</td>
<td>0.020</td>
</tr>
<tr>
<td>ML</td>
<td>Silt</td>
<td>Silt loam, non-colloidal</td>
<td>2.3</td>
<td>0.020</td>
</tr>
<tr>
<td>CL</td>
<td>Lean clay</td>
<td>Alluvial silts, non-colloidal</td>
<td>2.3</td>
<td>0.020</td>
</tr>
<tr>
<td>ML, CL</td>
<td>Silt, lean clay</td>
<td>Ordinary firm loam</td>
<td>2.5</td>
<td>0.020</td>
</tr>
<tr>
<td>CL</td>
<td>Lean clay</td>
<td>Alluvial silts, colloidal</td>
<td>3.5</td>
<td>0.025</td>
</tr>
<tr>
<td>CH</td>
<td>Fat clay</td>
<td>Stiff clay, very colloidal</td>
<td>4.0</td>
<td>0.025</td>
</tr>
</tbody>
</table>

Note: Relationship between Unified Soil Classification System (USCS) classification and Fortier and Scobey description is loosely correlated.
Appendix 7D-3  Standard VDOT Riprap Classifications, Weights, and Blanket Thickness

<table>
<thead>
<tr>
<th>Classification</th>
<th>D$_{50}$ (ft)</th>
<th>W$_{50}$ (lbs)</th>
<th>T (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class AI</td>
<td>0.8</td>
<td>50</td>
<td>20</td>
</tr>
<tr>
<td>Class I</td>
<td>1.1</td>
<td>100</td>
<td>26</td>
</tr>
<tr>
<td>Class II</td>
<td>1.6</td>
<td>300</td>
<td>38</td>
</tr>
<tr>
<td>Class III</td>
<td>2.2</td>
<td>1000</td>
<td>53</td>
</tr>
<tr>
<td>Type I</td>
<td>2.8</td>
<td>2000</td>
<td>60</td>
</tr>
<tr>
<td>Type II</td>
<td>4.5</td>
<td>8000</td>
<td>97</td>
</tr>
</tbody>
</table>
### Appendix 7D-4  Approximate Rock Dimensions and Equivalent Weights for Riprap

<table>
<thead>
<tr>
<th>WEIGHT</th>
<th>MEAN SPHERICAL DIAMETER</th>
<th>RECTANGULAR SHAPE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>LENGTH</td>
</tr>
<tr>
<td>25 lbs.</td>
<td>0.7’</td>
<td>1.1’</td>
</tr>
<tr>
<td>50 lbs.</td>
<td>0.8’</td>
<td>1.4’</td>
</tr>
<tr>
<td>75 lbs.</td>
<td>1.0’</td>
<td>1.6’</td>
</tr>
<tr>
<td>100 lbs.</td>
<td>1.1’</td>
<td>1.75’</td>
</tr>
<tr>
<td>150 lbs.</td>
<td>1.3’</td>
<td>2.0’</td>
</tr>
<tr>
<td>300 lbs.</td>
<td>1.6’</td>
<td>2.6’</td>
</tr>
<tr>
<td>500 lbs.</td>
<td>1.9’</td>
<td>3.0’</td>
</tr>
<tr>
<td>1000 lbs.</td>
<td>2.2’</td>
<td>3.7’</td>
</tr>
<tr>
<td>1500 lbs.</td>
<td>2.6’</td>
<td>4.7’</td>
</tr>
<tr>
<td>2000 lbs.</td>
<td>2.75’</td>
<td>5.4’</td>
</tr>
<tr>
<td>2 tons</td>
<td>3.6’</td>
<td>6.0’</td>
</tr>
<tr>
<td>3 tons</td>
<td>4.0’</td>
<td>6.9’</td>
</tr>
<tr>
<td>4 tons</td>
<td>4.5’</td>
<td>7.6’</td>
</tr>
<tr>
<td>10 tons</td>
<td>6.1’</td>
<td>10.0’</td>
</tr>
</tbody>
</table>
### Appendix 7D-5 Selection of Stability Factors

<table>
<thead>
<tr>
<th>CONDITION</th>
<th>STABILITY FACTOR RANGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniform flow; straight or mildly curving reach (curve radius/</td>
<td>1.0 - 1.2</td>
</tr>
<tr>
<td>channel width &gt;30); impact from wave action and floating debris is</td>
<td></td>
</tr>
<tr>
<td>minimal; little or no uncertainty in design parameters.</td>
<td></td>
</tr>
<tr>
<td>Gradually varying flow; moderate bend curvature (30 &gt; curve radius/</td>
<td>1.3 - 1.6</td>
</tr>
<tr>
<td>channel width &gt; 10); impact from waves or floating debris is moderate.</td>
<td></td>
</tr>
<tr>
<td>Approaching rapidly varying flow; sharp bend curvature (30 &gt;</td>
<td>1.6 - 2.0</td>
</tr>
<tr>
<td>curve radius/channel width &gt;10); significant impact potential from</td>
<td></td>
</tr>
<tr>
<td>floating debris and/or ice; significant wind and/or bore generated</td>
<td></td>
</tr>
<tr>
<td>waves (1-2 ft); high flow turbulence; mixing flow at bridge abutments;</td>
<td></td>
</tr>
<tr>
<td>significant uncertainty in design parameters.</td>
<td></td>
</tr>
<tr>
<td>Channel bends when ratio of curve radius to channel width (R/W) &gt; 30.</td>
<td>1.2</td>
</tr>
<tr>
<td>Channel bends when 30 &gt; R/W &gt; 10.</td>
<td>1.3 - 1.6</td>
</tr>
<tr>
<td>Channel bends when R/W &lt; 10.</td>
<td>1.7</td>
</tr>
</tbody>
</table>
Permissible velocities for channels with erodible linings, based on uniform flow in continuously wet, aged channels:\(^1\):

<table>
<thead>
<tr>
<th>Soil type or lining (earth; no vegetation)</th>
<th>Maximum permissible velocities for</th>
<th>Clear water</th>
<th>Water carrying fine silts</th>
<th>Water carrying sand and gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine sand (noncolloidal)</td>
<td>F.p.s.</td>
<td>1.5</td>
<td>2.5</td>
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Source: \(^1\)As recommended by Special Committee on Irrigation Research, American Society of Civil Engineers, 1926.
Appendix 7E-1  Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone

Appendix 7E-2  Permissible Shear Stress for Non-Cohesive Soils

Source: HEC-15 ( Archived) 1988
Appendix 7E-3  Permissible Shear Stress for Cohesive Soils

Chart 2

EXPLANATION

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</thead>
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<td>Loose</td>
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<td>Medium Compact</td>
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<td>Compact</td>
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N = Number of blows required to affect 12" penetration of the 2" split-spoon sampler seated to a depth of 6" driven with a 140 lb. weight falling 36".

Source: HEC-15 (Archived) 1988
Appendix 7E-4 Bank Angle Correction Factor

\( K_1 = \left( 1 - \frac{\sin^2 \Theta}{\sin^2 \Phi} \right)^{0.5} \)

\( \Theta \) = Bank angle with horizontal

\( \Phi \) = Material angle of repose

(See chart 4)

Example

Given:
\[ \Theta = 18^\circ \]

Very Angular

\[ D_{50} = 1.5 \text{ ft.} \]

Find:
\[ K_1 \]

Solution:
\[ \Phi = 42^\circ \]

\[ K_1 = 0.885 \]

Source: HEC-11
Appendix 7E-5  Correction Factor for Riprap Size

\[ C = 1.61 \frac{SF^{1.5}}{(S_S - 1)^{1.5}} \]

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<tr>
<th>( S_S )</th>
<th>( C )</th>
<th>( SF )</th>
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<td>2.9</td>
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<tr>
<td>3.0</td>
<td>0.5</td>
<td>1.0</td>
</tr>
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</table>

Example:

Given:
\( S_S = 2.65 \)
\( SF = 1.2 \)

Solution:
\( C = 1.0 \)

Source:  HEC-11
Comment:  \( S_S = S_g \) (text)
Chapter 7 – Ditches and Channels

Appendix 7E-6  Riprap Size Relationship

\[ D_{50} = 0.001 \sqrt[3]{V_a} / (d_{avg}^{1/2} K_1^{3/2}) \]

- \( D_{50} \) = Median Riprap Size (ft)
- \( V_a \) = Average velocity in main channel (ft/sec)
- \( d_{avg} \) = Average depth in main channel (ft)
- \( K_1 \) = Bank angle correction term

Example:

Given:
- \( V_a = 9.7 \) ft/sec
- \( d_{avg} = 11.8 \) ft
- \( K_1 = 0.73 \)

Find: \( D_{50} \)

Solution: \( D_{50} = 0.43 \)

Source: HEC-11
Appendix 7E-7  Channel Side Shear Stress to Bottom Shear Stress Ratio

Source: HEC-15 (Archived 1988)
Appendix 7E-8  Tractive Force Ratio ($K_2$)

Source: HEC-15 (Archived 1988)
Comment: The symbols of $\Phi$ and $\theta$ are reversed from Appendix 7E-4.
Appendix 7E-9  Determination of Mean Spherical Diameter

Source: VDOT
Comment: Use this chart to obtain $D_{75}$ information for the Channel Stability Worksheet (Appendix 7B-3).
# Chapter 8 - Culverts

## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.1</td>
<td>Overview</td>
<td>8-1</td>
</tr>
<tr>
<td>8.1.1</td>
<td>Introduction</td>
<td>8-1</td>
</tr>
<tr>
<td>8.1.2</td>
<td>Objective</td>
<td>8-1</td>
</tr>
<tr>
<td>8.2</td>
<td>Design Policy</td>
<td>8-2</td>
</tr>
<tr>
<td>8.3</td>
<td>Design Criteria</td>
<td>8-3</td>
</tr>
<tr>
<td>8.3.1</td>
<td>Site Criteria</td>
<td>8-3</td>
</tr>
<tr>
<td>8.3.1.1</td>
<td>Structure Type Selection</td>
<td>8-3</td>
</tr>
<tr>
<td>8.3.1.2</td>
<td>Topography</td>
<td>8-4</td>
</tr>
<tr>
<td>8.3.1.3</td>
<td>Debris Control</td>
<td>8-4</td>
</tr>
<tr>
<td>8.3.1.4</td>
<td>Soil and Water Data</td>
<td>8-4</td>
</tr>
<tr>
<td>8.3.1.5</td>
<td>Protective Coating for Structures Exposed to Tidal Water or Corrosive Environment</td>
<td>8-5</td>
</tr>
<tr>
<td>8.3.1.6</td>
<td>Requesting Data and Materials Division Recommendations</td>
<td>8-5</td>
</tr>
<tr>
<td>8.3.1.7</td>
<td>Subsurface Investigation</td>
<td>8-6</td>
</tr>
<tr>
<td>8.3.1.8</td>
<td>Pipe Camber</td>
<td>8-6</td>
</tr>
<tr>
<td>8.3.2</td>
<td>Hydraulic Criteria</td>
<td>8-7</td>
</tr>
<tr>
<td>8.3.2.1</td>
<td>Design Storm</td>
<td>8-7</td>
</tr>
<tr>
<td>8.3.2.2</td>
<td>Allowable Headwater</td>
<td>8-7</td>
</tr>
<tr>
<td>8.3.2.3</td>
<td>Review Headwater</td>
<td>8-8</td>
</tr>
<tr>
<td>8.3.2.4</td>
<td>Tailwater Relationship – Channel</td>
<td>8-8</td>
</tr>
<tr>
<td>8.3.2.5</td>
<td>Tailwater Relationship - Confluence or Large Water Body</td>
<td>8-8</td>
</tr>
<tr>
<td>8.3.2.6</td>
<td>Maximum Outlet Velocity</td>
<td>8-9</td>
</tr>
<tr>
<td>8.3.2.7</td>
<td>Minimum Velocity</td>
<td>8-12</td>
</tr>
<tr>
<td>8.3.2.8</td>
<td>Storage Routing - Temporary or Permanent</td>
<td>8-12</td>
</tr>
<tr>
<td>8.3.2.9</td>
<td>Roadway Overtopping</td>
<td>8-12</td>
</tr>
<tr>
<td>8.3.3</td>
<td>Geometric Criteria</td>
<td>8-13</td>
</tr>
<tr>
<td>8.3.3.1</td>
<td>Culvert Size and Shape</td>
<td>8-13</td>
</tr>
<tr>
<td>8.3.3.2</td>
<td>Multiple Barrels</td>
<td>8-13</td>
</tr>
<tr>
<td>8.3.3.3</td>
<td>Culvert Skew</td>
<td>8-13</td>
</tr>
<tr>
<td>8.3.3.4</td>
<td>End Treatment (Inlet or Outlet)</td>
<td>8-14</td>
</tr>
<tr>
<td>8.3.3.4.1</td>
<td>Projecting Inlets or Outlets</td>
<td>8-15</td>
</tr>
<tr>
<td>8.3.3.4.2</td>
<td>Prefabricated End Sections</td>
<td>8-15</td>
</tr>
<tr>
<td>8.3.3.4.3</td>
<td>Headwalls with Bevels</td>
<td>8-15</td>
</tr>
<tr>
<td>8.3.3.4.4</td>
<td>Improved Inlets</td>
<td>8-15</td>
</tr>
<tr>
<td>8.3.3.4.5</td>
<td>Wingwalls</td>
<td>8-16</td>
</tr>
<tr>
<td>8.3.3.4.6</td>
<td>Aprons</td>
<td>8-16</td>
</tr>
<tr>
<td>8.3.3.4.7</td>
<td>Cut-off Walls</td>
<td>8-16</td>
</tr>
<tr>
<td>8.3.3.4.8</td>
<td>Trash Racks or Debris Deflectors</td>
<td>8-16</td>
</tr>
<tr>
<td>8.3.4</td>
<td>Safety Considerations</td>
<td>8-16</td>
</tr>
<tr>
<td>8.3.5</td>
<td>Allowable Pipe Materials</td>
<td>8-17</td>
</tr>
<tr>
<td>8.3.6</td>
<td>Other Design Considerations</td>
<td>8-18</td>
</tr>
<tr>
<td>8.3.6.1</td>
<td>Buoyancy Protection</td>
<td>8-18</td>
</tr>
<tr>
<td>8.3.6.2</td>
<td>Relief Opening</td>
<td>8-18</td>
</tr>
<tr>
<td>8.3.6.3</td>
<td>Land Use Culverts</td>
<td>8-18</td>
</tr>
<tr>
<td>8.3.6.4</td>
<td>Erosion and Sediment Control</td>
<td>8-18</td>
</tr>
<tr>
<td>Section</td>
<td>Title</td>
<td>Page</td>
</tr>
<tr>
<td>---------</td>
<td>----------------------------------------------------------------------</td>
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</tr>
<tr>
<td>8.3.6.5</td>
<td>Environmental Considerations and Fishery Protection</td>
<td>8-19</td>
</tr>
<tr>
<td>8.3.6.6</td>
<td>Pipe in High Fills</td>
<td>8-19</td>
</tr>
<tr>
<td>8.3.6.7</td>
<td>Pipe Rehabilitation (Pipe not Replaced)</td>
<td>8-20</td>
</tr>
<tr>
<td>8.3.6.8</td>
<td>Existing Box Culvert Extensions</td>
<td>8-23</td>
</tr>
<tr>
<td>8.3.6.9</td>
<td>Small Box Culverts</td>
<td>8-23</td>
</tr>
<tr>
<td>8.3.6.10</td>
<td>Pile Foundation Design for Box Culverts</td>
<td>8-23</td>
</tr>
<tr>
<td>8.3.6.11</td>
<td>Trenchless Applications (Culvert Replacement or New Pipe)</td>
<td>8-23</td>
</tr>
<tr>
<td></td>
<td>8.3.6.11.1 Jack and Bore</td>
<td>8-27</td>
</tr>
<tr>
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<td>8.3.6.11.2 Microtunneling</td>
<td>8-27</td>
</tr>
<tr>
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<td>8.3.6.11.3 Pipe Jacking</td>
<td>8-27</td>
</tr>
<tr>
<td>8.3.7</td>
<td>Counter Sinking and Low Flow Considerations</td>
<td>8-28</td>
</tr>
<tr>
<td></td>
<td>8.3.7.1 Definitions</td>
<td>8-28</td>
</tr>
<tr>
<td></td>
<td>8.3.7.1.1 Stream Bed</td>
<td>8-28</td>
</tr>
<tr>
<td></td>
<td>8.3.7.1.2 Culvert</td>
<td>8-28</td>
</tr>
<tr>
<td></td>
<td>8.3.7.2 Policy</td>
<td>8-28</td>
</tr>
<tr>
<td></td>
<td>8.3.7.3 Multiple Barrel Culverts</td>
<td>8-29</td>
</tr>
<tr>
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<td>8.3.7.4 Special Culvert Installations</td>
<td>8-30</td>
</tr>
<tr>
<td></td>
<td>8.3.7.4.1 Culverts on Bedrock</td>
<td>8-30</td>
</tr>
<tr>
<td></td>
<td>8.3.7.4.2 Culverts on Steep Terrain</td>
<td>8-31</td>
</tr>
<tr>
<td></td>
<td>8.3.7.4.3 Culverts at the Confluence of Two Streams</td>
<td>8-31</td>
</tr>
<tr>
<td></td>
<td>8.3.7.4.4 Other Situations</td>
<td>8-31</td>
</tr>
<tr>
<td>8.3.8</td>
<td>Drainage Design at Railroads</td>
<td>8-32</td>
</tr>
<tr>
<td></td>
<td>8.3.8.1 Criteria</td>
<td>8-32</td>
</tr>
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<td>8-32</td>
</tr>
<tr>
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<td>8.3.8.1.2 Pipe Size and Cover</td>
<td>8-33</td>
</tr>
<tr>
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<td>8.3.8.1.3 Pipe Materials and Installation</td>
<td>8-33</td>
</tr>
<tr>
<td></td>
<td>8.3.8.1.4 Drop Inlets</td>
<td>8-35</td>
</tr>
<tr>
<td></td>
<td>8.3.8.1.5 Ditches</td>
<td>8-35</td>
</tr>
<tr>
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<td>8.3.8.1.6 Foundations for Signals</td>
<td>8-36</td>
</tr>
<tr>
<td></td>
<td>8.3.8.1.7 Endwalls and Other Structures</td>
<td>8-36</td>
</tr>
<tr>
<td></td>
<td>8.3.8.2 Guidelines</td>
<td>8-36</td>
</tr>
<tr>
<td>8.4</td>
<td>Design Concepts</td>
<td>8-37</td>
</tr>
<tr>
<td>8.4.1</td>
<td>General</td>
<td>8-37</td>
</tr>
<tr>
<td>8.4.2</td>
<td>Design Methods</td>
<td>8-37</td>
</tr>
<tr>
<td></td>
<td>8.4.2.1 Hydrologic Methods</td>
<td>8-37</td>
</tr>
<tr>
<td></td>
<td>8.4.2.2 Computational Methods</td>
<td>8-38</td>
</tr>
<tr>
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<td>8.4.2.2.1 Manual Methods</td>
<td>8-38</td>
</tr>
<tr>
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<td>8.4.2.2.2 Computer Solution</td>
<td>8-38</td>
</tr>
<tr>
<td>8.4.3</td>
<td>Culvert Hydraulics</td>
<td>8-39</td>
</tr>
<tr>
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<td>8.4.3.1 Control Section</td>
<td>8-39</td>
</tr>
<tr>
<td></td>
<td>8.4.3.2 Minimum Performance</td>
<td>8-39</td>
</tr>
<tr>
<td></td>
<td>8.4.3.3 Inlet Control</td>
<td>8-39</td>
</tr>
<tr>
<td></td>
<td>8.4.3.3.1 Headwater Factors - Inlet Control</td>
<td>8-40</td>
</tr>
<tr>
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<td>8.4.3.3.2 Flow Conditions – Inlet Control</td>
<td>8-40</td>
</tr>
<tr>
<td></td>
<td>8.4.3.3.2.1 Unsubmerged - Inlet Control</td>
<td>8-43</td>
</tr>
<tr>
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<td>8.4.3.3.2.2 Submerged - Inlet Control</td>
<td>8-43</td>
</tr>
<tr>
<td></td>
<td>8.4.3.3.2.3 Transition Zone - Inlet Control</td>
<td>8-43</td>
</tr>
<tr>
<td></td>
<td>8.4.3.3.2.4 Special Condition - Inlet Control</td>
<td>8-43</td>
</tr>
<tr>
<td></td>
<td>8.4.3.3.2.5 Inlet Control Nomographs</td>
<td>8-43</td>
</tr>
<tr>
<td></td>
<td>8.4.3.4 Outlet Control</td>
<td>8-43</td>
</tr>
<tr>
<td></td>
<td>8.4.3.4.1 Headwater Factors - Outlet Control</td>
<td>8-45</td>
</tr>
<tr>
<td></td>
<td>8.4.3.4.2 Flow Condition - Outlet Control</td>
<td>8-46</td>
</tr>
<tr>
<td></td>
<td>8.4.3.4.2.1 Losses</td>
<td>8-46</td>
</tr>
<tr>
<td></td>
<td>8.4.3.4.2.2 Velocity</td>
<td>8-46</td>
</tr>
<tr>
<td></td>
<td>8.4.3.4.2.3 Velocity Head</td>
<td>8-46</td>
</tr>
</tbody>
</table>
8.4.3.4.2.4 Entrance Loss ............................................ 8-47
8.4.3.4.2.5 Friction Loss ........................................... 8-47
8.4.3.4.2.6 Exit Loss ................................................... 8-47
8.4.3.4.2.7 Other Losses .......................................... 8-47
8.4.3.4.2.8 Barrel Losses ......................................... 8-48
8.4.3.4.2.9 Energy Grade Line - Outlet Control .......... 8-48
8.4.3.4.2.10 Hydraulic Grade Line - Outlet Control .... 8-49
8.4.3.4.2.11 Outlet Control Nomographs (Full-flow) ..... 8-50
8.4.3.4.2.12 Outlet Control (Partly Full-flow) ............ 8-50
8.4.3.4.2.13 Outlet Control Nomographs (Partly Full-flow) -
  Approximate Method ........................................... 8-50
8.4.3.5 Outlet Velocity ................................................................................... 8-51
  8.4.3.5.1 Inlet Control .............................................. 8-51
  8.4.3.5.2 Outlet Control ............................................. 8-52
8.4.3.6 Roadway Overtopping ..................................................... 8-52
  8.4.3.6.1 Length of Roadway Crest ............................... 8-52
  8.4.3.6.2 Total Flow ................................................ 8-53
  8.4.3.6.3 Performance Curves ...................................... 8-53
8.4.4 Special Design Considerations ..................................................... 8-54
  8.4.4.1 General ............................................................ 8-54
  8.4.4.2 Tapered Inlets .................................................. 8-54
    8.4.4.2.1 Side-Tapered Inlets ..................................... 8-55
    8.4.4.2.2 Slope-Tapered Inlets .................................. 8-56
  8.4.4.3 Buoyancy Protection ........................................... 8-57
  8.4.4.4 Minor Structure Excavation .............................. 8-57
8.5 Design Procedures and Examples .......................................................... 8-60
  8.5.1 Documentation Requirements ...................................... 8-60
  8.5.2 VDOT Culvert Design Procedure ............................. 8-60
  8.5.3 Culvert Design Sample Problems .............................. 8-64
  8.5.4 Buoyancy Protection Procedure .................................. 8-69
    8.5.4.1 Hydrostatic Uplift and Resistance .................... 8-69
    8.5.4.2 Buoyancy Protection Sample Problem .............. 8-70
8.6 References ............................................................................................... 8-76
List of Figures

Figure 8-1. Performance Curves - Unsubmerged, Transition, and Submerged ........................................ 8-41
Figure 8-2. Types of Inlet Control Flow ......................................................................................... 8-42
Figure 8-3. Types of Outlet Control Flow ....................................................................................... 8-44
Figure 8-4. Full Flow Energy and Hydraulic Grade Lines ............................................................. 8-48
Figure 8-5. Outlet Control Energy and Hydraulic Grade Lines ...................................................... 8-49
Figure 8-6. Outlet Velocity - Inlet Control ....................................................................................... 8-51
Figure 8-7. Outlet Velocity - Outlet Control .................................................................................... 8-52
Figure 8-8. Overall Culvert Performance Curve ............................................................................. 8-53
Figure 8-9. Side-Tapered Inlet ....................................................................................................... 8-55
Figure 8-10. Slope-Tapered Inlet ................................................................................................. 8-56

List of Appendices

Appendix 8B-1 Culvert Design Form, LD-269
Appendix 8C-1 Inlet Control, Circular Concrete
Appendix 8C-2 Inlet Control, Circular Corrugated Metal
Appendix 8C-3 Inlet Control, Circular with Beveled Ring
Appendix 8C-4 Critical Depth, Circular
Appendix 8C-5 Outlet Control, Circular Concrete
Appendix 8C-6 Outlet Control, Circular Corrugated Metal
Appendix 8C-7 Outlet Control, Circular Structural Plate Corrugated Metal
Appendix 8C-8 Inlet Control, Concrete Box
Appendix 8C-9 Inlet Control, Concrete Box, Flared Wingwalls at 18° to 33.7° and 45°, Beveled Top Edge
Appendix 8C-10 Inlet Control, Concrete Box, 90° Headwall, Chamfered or Beveled Edges
Appendix 8C-11 Inlet Control, Single Barrel Concrete Box, Skewed Headwalls, Chamfered or Beveled Edges
Appendix 8C-12 Inlet Control, Concrete Box, Flared Wingwalls, Normal and Skewed Inlets, Chamfered Top Edge
Appendix 8C-13 Inlet Control, Concrete Box with Offset Flared Wingwalls, Beveled Top Edge
Appendix 8C-14 Critical Depth, Concrete Box
Appendix 8C-15 Outlet Control, Concrete Box
Appendix 8C-16 Inlet Control, Corrugated Metal Box, Rise/Span < 0.3
Appendix 8C-17 Inlet Control, Corrugated Metal Box, 0.3 <= Rise/Span < 0.4
Appendix 8C-18 Inlet Control, Corrugated Metal Box, 0.4 <= Rise/Span < 0.5
Appendix 8C-19 Inlet Control, Corrugated Metal Box, 0.5 <= Rise/Span
Appendix 8C-20 Critical Depth, Corrugated Metal Box
Appendix 8C-21 Outlet Control, Corrugated Metal Box, Concrete Bottom, Rise/Span < 0.3
Appendix 8C-22  Outlet Control, Corrugated Metal Box, Concrete Bottom, $0.3 \leq \text{Rise/Span} < 0.4$

Appendix 8C-23  Outlet Control, Corrugated Metal Box, Concrete Bottom, $0.4 \leq \text{Rise/Span} < 0.5$

Appendix 8C-24  Outlet Control, Corrugated Metal Box, Concrete Bottom, $0.5 \leq \text{Rise/Span}$

Appendix 8C-25  Outlet Control, Corrugated Metal Box, Corrugated Metal Bottom, $\text{Rise/Span} < 0.3$

Appendix 8C-26  Outlet Control, Corrugated Metal Box, Corrugated Metal Bottom, $0.3 \leq \text{Rise/Span} < 0.4$

Appendix 8C-27  Outlet Control, Corrugated Metal Box, Corrugated Metal Bottom, $0.4 \leq \text{Rise/Span} < 0.5$

Appendix 8C-28  Outlet Control, Corrugated Metal Box, Corrugated Metal Bottom, $0.5 \leq \text{Rise/Span}$

Appendix 8C-29  Inlet Control, Oval Concrete, Long Axis Horizontal

Appendix 8C-30  Inlet Control, Oval Concrete, Long Axis Vertical

Appendix 8C-31  Critical Depth, Oval Concrete, Long Axis Horizontal

Appendix 8C-32  Critical Depth, Oval Concrete, Long Axis Vertical

Appendix 8C-33  Outlet Control, Oval Concrete, Long Axis Horizontal or Vertical

Appendix 8C-34  Inlet Control, Corrugated Metal Pipe-Arch

Appendix 8C-35  Inlet Control, Structural Plate Pipe-Arch, 18” Corner Radius

Appendix 8C-36  Inlet Control, Structural Plate Pipe-Arch, 31” Corner Radius

Appendix 8C-37  Critical Depth, Standard Corrugated Metal Pipe-Arch

Appendix 8C-38  Critical Depth, Structural Plate Corrugated Metal Pipe-Arch, 18” Corner Radius

Appendix 8C-39  Outlet Control, Standard Corrugated Metal Pipe-Arch

Appendix 8C-40  Outlet Control, Structural Plate Corrugated Metal Pipe-Arch, 18” Corner Radius

Appendix 8C-41  Inlet Control, Corrugated Metal Arch, $0.3 \leq \text{Rise/Span} < 0.4$

Appendix 8C-42  Inlet Control, Corrugated Metal Arch, $0.4 \leq \text{Rise/Span} < 0.5$

Appendix 8C-43  Inlet Control, Corrugated Metal Arch, $0.5 \leq \text{Rise/Span}$

Appendix 8C-44  Critical Depth, Corrugated Metal Arch

Appendix 8C-45  Outlet Control, Corrugated Metal Arch, Concrete Bottom, $0.3 \leq \text{Rise/Span} < 0.4$

Appendix 8C-46  Outlet Control, Corrugated Metal Arch, Concrete Bottom, $0.4 \leq \text{Rise/Span} < 0.5$

Appendix 8C-47  Outlet Control, Corrugated Metal Arch, Concrete Bottom, $0.5 \leq \text{Rise/Span}$

Appendix 8C-48  Outlet Control, Corrugated Metal Arch, Earth Bottom, $0.3 \leq \text{Rise/Span} < 0.4$

Appendix 8C-49  Outlet Control, Corrugated Metal Arch, Earth Bottom, $0.4 \leq \text{Rise/Span} < 0.5$

Appendix 8C-50  Outlet Control, Corrugated Metal Arch, Earth Bottom, $0.5 \leq \text{Rise/Span}$

Appendix 8C-51  Inlet Control, Structural Plate Corrugated Metal, Circular or Elliptical
Appendix 8C-52  Inlet Control, Structural Plate Corrugated Metal Arch, High and Low Profile
Appendix 8C-53  Critical Depth, Structural Plate Ellipse, Long Axis Horizontal
Appendix 8C-54  Critical Depth, Structural Plate Arch, Low and High Profile
Appendix 8C-55  Throat Control, Circular Section, Side-Tapered
Appendix 8C-56  Face Control, Non-Rectangular Section, Side-Tapered to Circular
Appendix 8C-57  Throat Control, Box Section, Tapered Inlet
Appendix 8C-58  Face Control, Box Section, Side-Tapered
Appendix 8C-59  Face Control, Box Section, Slope-Tapered
Appendix 8C-60  Discharge Coefficients for Roadway Overtopping
Appendix 8C-61  Circular Pipe Flow Chart (Diameter = 12”)
Appendix 8C-62  Circular Pipe Flow Chart (Diameter = 15”)
Appendix 8C-63  Circular Pipe Flow Chart (Diameter = 18”)
Appendix 8C-64  Circular Pipe Flow Chart (Diameter = 21”)
Appendix 8C-65  Circular Pipe Flow Chart (Diameter = 24”)
Appendix 8C-66  Circular Pipe Flow Chart (Diameter = 27”)
Appendix 8C-67  Circular Pipe Flow Chart (Diameter = 30”)
Appendix 8C-68  Circular Pipe Flow Chart (Diameter = 33”)
Appendix 8C-69  Circular Pipe Flow Chart (Diameter = 36”)
Appendix 8C-70  Circular Pipe Flow Chart (Diameter = 42”)
Appendix 8C-71  Circular Pipe Flow Chart (Diameter = 48”)
Appendix 8C-72  Circular Pipe Flow Chart (Diameter = 54”)
Appendix 8C-73  Circular Pipe Flow Chart (Diameter = 60”)
Appendix 8C-74  Circular Pipe Flow Chart (Diameter = 66”)
Appendix 8C-75  Circular Pipe Flow Chart (Diameter = 72”)
Appendix 8C-76  Circular Pipe Flow Chart (Diameter = 84”)
Appendix 8C-77  Circular Pipe Flow Chart (Diameter = 96”)
Appendix 8C-78  Rectangular Channel Flow Chart (B = 2’)
Appendix 8C-79  Rectangular Channel Flow Chart (B = 3’)
Appendix 8C-80  Rectangular Channel Flow Chart (B = 4’)
Appendix 8C-81  Rectangular Channel Flow Chart (B = 5’)
Appendix 8C-82  Rectangular Channel Flow Chart (B = 6’)
Appendix 8C-83  Rectangular Channel Flow Chart (B = 7’)
Appendix 8C-84  Rectangular Channel Flow Chart (B = 8’)
Appendix 8C-85  Rectangular Channel Flow Chart (B = 9’)
Appendix 8C-86  Rectangular Channel Flow Chart (B = 10’)
Appendix 8C-87  Rectangular Channel Flow Chart (B = 12’)
Appendix 8C-88  Rectangular Channel Flow Chart (B = 14’
Appendix 8C-89  Rectangular Channel Flow Chart (B = 16’)
Appendix 8C-90  Rectangular Channel Flow Chart (B = 18’)
Appendix 8C-91  Rectangular Channel Flow Chart (B = 20’)
Appendix 8D-1  Recommended Manning’s n-Values
Appendix 8D-2  Entrance Loss Coefficients (K_e), Outlet Control, Full or Partly Full
Appendix 8E-1  Energy Dissipation
Appendix 8F-1  Handling Weight for Corrugated Steel Pipe (2-⅜” x ⅛” Corrugations)
Appendix 8F-2  Handling Weight for Corrugated Steel Pipe, (3” x 1” or 125 mm x 25 mm Corrugations)
Appendix 8F-3  Dimension and Weight of Minimum Size Counterweight
Appendix 8F-4  Diameter Dimensions and D^{2.5} Values for Structural Plate Corrugated Circular Pipe (9” x 2-⅜” Aluminum Corrugations)
Appendix 8F-5  Geometric Properties and Critical Flow Factors for Circular Conduits Flowing Full and Partly Full
Appendix 8F-6  Velocity Head and Resistance Computation Factors for Circular Conduits Flowing Full and Party Full
Chapter 8 - Culverts

8.1 Overview

8.1.1 Introduction

Culverts are usually defined as short conduits used to convey flow through a highway fill. The flow types that occur in culverts are many and varied. In this chapter, culvert design considerations are presented from the planning stage through the design stage. Design should consider hydraulic and structural capacity, erosion and debris control, environmental impacts, safety concerns, and legal aspects. Design concepts are covered and design procedures are summarized along with sample problems. Sources of additional information on culvert design are provided.

The FHWA web site for specific publications, [http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm](http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm) may be used for additional information on the following topics:

HDS-5, Hydraulic Design of Highway Culverts

Note: An errata sheet for this publication is available at the above web site

HEC-9, Debris Control Structures,

HEC-14, Hydraulic Design of Energy Dissipators for Culverts and Channels,

HEC-20, Stream Stability at Highway Structures,

Note: An errata sheet for this publication is available at the above web site

HDS-6, River Engineering for Highway Encroachments – Highways in the River Environment

8.1.2 Objective

The objective of this chapter is to provide the user with the information needed to select, plan, and design highway culverts based on VDOT methods when a Design Hydraulic Study is required by the Location Hydraulic Study. Using the information provided, the user will be able to design conventional culverts. The chapter also provides references, which will enable the user to apply special culvert designs for unusual circumstances. For open bottom structures, structures with a combined width of 20’ or more, or a base flood discharge of > 500 cfs refer to Chapter 12 unless exempted by VDOT Hydraulics staff.

Due to the magnitude of the changes to this section, shading has been omitted.
8.2 Design Policy

The following Federal and VDOT policies will guide the selection, planning, and design of highway culverts:

- All culverts should be hydraulically designed
- The overtopping flood selected should be consistent with the class of highway and commensurate with the risks at the site
- Survey information should include topographic features, channel characteristics, aquatic life, high-water information, existing structures and other related site-specific information
- Culvert location in both plan and profile should be investigated to consider sediment build-up in the barrels, upstream, or downstream of the culvert
- The cost savings of multiple use culverts (utilities, stock and wildlife passage, land access and fish passage) should be weighed against the advantages of separate facilities
- Culverts should be designed to accommodate debris or proper provisions should be made for debris maintenance
- Material selection should consider service life, which includes abrasion and corrosion
- Culverts should be located and designed to present a minimum (reasonable) hazard to traffic and people
- The detail of documentation for each culvert site should be commensurate with the risk and importance of the structure
- Where practicable, some means should be provided for personnel and equipment access to facilitate maintenance
- Culverts should be regularly inspected and maintained
- The impacts on the base flood (100-year flood event) should be evaluated on all culverts, regardless of the drainage area size

Due to the magnitude of the changes to this section, shading has been omitted.
8.3 Design Criteria

Criteria for the planning and design of culverts are discussed in this section. These criteria should be considered for all culvert designs.

8.3.1 Site Criteria

The following criteria relate to site conditions, which affect the selection of a particular culvert type, geometry, and debris protection.

8.3.1.1 Structure Type Selection

Often, a choice must be made between a culvert and a bridge at a given site. In making that decision, the following criteria should be considered:

Culverts are used:
- Where bridges are not hydraulically required or feasible
- Where debris and ice are tolerable
- Where a culvert is more economical than a bridge
- Where environmentally acceptable

Bridges are used:
- Where culverts cannot be used
- Where a bridge is more economical than a culvert
- To satisfy land use requirements
- To mitigate environmental impacts caused by a culvert
- To avoid floodway encroachments
- To accommodate ice and large debris
- To avoid cost and impact of channel diversions necessary for culvert construction

Because of the numerous types of drainage structures that are available, a general rule would dictate that various types such as box culverts, pipe culverts, standard bridges, etc., be taken into consideration when determining the type of proposed structure.

This design evaluation should consider cost comparisons, construction time, earth movement, maintenance, and service life expectancy.

Design Considerations:
- All non-rigid culverts:
  - are comparatively flexible
  - rely on uniform soil pressure around the entire circumference of the structure to maintain proper and equal load distributions
  - are more sensitive to improper bedding and backfill than rigid structures

*Due to the magnitude of the changes to this section, shading has been omitted.*

Chapter 8-3 of 77
• Structural plate pipe arch culverts:
  o concentrate considerable pressure in the haunch area
  o require near perfect backfill and compaction in haunch area during construction
  o should be avoided wherever alternate structural shapes are feasible, such as:
    1. aluminum or steel box culverts
    2. bottomless arch culverts on footings
    3. circular culverts buried below streambed

8.3.1.2 Topography
The culvert length and slope should be chosen to fit the existing topography, to the degree practicable: the culvert invert should be aligned with the channel bottom and the skew angle of the stream, except in instances where countersinking one or more culvert barrels is needed to satisfy environmental requirements.

8.3.1.3 Debris Control
Debris control should be designed using the FHWA's Hydraulic Engineering Circular No. 9, "Debris-Control Structures" and should consider:

• Where experience or physical evidence indicates the watercourse will transport a heavy volume of debris
• Culverts located in mountainous or steep regions of the state
• Culverts that are under high fills
• Where clean-out access is limited (However, access must be available to clean out and otherwise maintain the debris control device)

8.3.1.4 Soil and Water Data
The pH and resistivity of the soil and water as well as the velocity of flow, where an abrasive bed load is present or anticipated, are major factors in determining service life of metal pipe. An evaluation of the pH, resistivity and abrasive bed load potential must be conducted at each location where metal pipe is an allowable option and where any of the following conditions exist:

• Diameter or span of 36” or greater. For multiple pipe installations, the span is measured between the interiors of the outside walls of the outer most pipes and is measured along a line perpendicular to the barrel of the pipe
• Culvert is to be installed in a live stream environment (perennial or intermittent)
• Culvert is to be installed in an area of documented premature pipe failure

The pH and resistivity analysis of the soil and water are to be requested from the Materials Division for each culvert location meeting the noted criteria. In areas of documented premature pipe failure, the pH and resistivity analysis is to be requested for any type of proposed pipe material.

The locations where pH and resistivity information is needed should be noted on the plans that are used to request culvert subsurface information from the Materials Division.

_Due to the magnitude of the changes to this section, shading has been omitted._

Chapter 8-4 of 77
It is recognized that the pH values of the soil and water could experience seasonal changes during the course of the year. Should the Materials Division feel that the results of their initial pH test are not a true representation of the most severe conditions that the culvert will be exposed to, they should perform additional test and provide their best recommendation for the values to be used in determining the allowable pipe materials.

8.3.1.5 Protective Coating for Structures Exposed to Tidal Water or Corrosive Environment

Treatment of concrete exposed to the normal ebb and flow of tidal water is defined in Section 404 of the VDOT Road and Bridge Specifications. Corrosive environment may be indicated in certain geographic areas by the degradation of concrete culverts, concrete lined ditches or other concrete structures. Proposed concrete items in these areas should have a protective coating or alternative materials should be considered.

The Drainage Designer is responsible for preliminary determination for need and location of protective coating and is to specify in the drainage structure description where protective coating is required.

The final determination for need and location of protective coating should be made by the Materials Division. The request for the final determination should be made either by the use of Form LD-252 or direct contact between the Drainage Designer and the Materials Division.

The Drainage Designer is responsible for ensuring that the following notation is noted in the final drainage structure description on the plans and in the drainage summary:

*Pipe or structure is to have protective coating applied in accordance with Section 404 of the VDOT Road and Bridge Specifications.*

8.3.1.6 Requesting Data and Materials Division Recommendations

The Drainage Designer will determine locations where subsurface investigation and other culvert data/recommendations are required.

Subsurface, pH, resistivity, abrasive bed load data and channel bed material classification and recommendations for bedding, pipe camber and protective coating will be requested by the Roadway Designer, from the Materials Division, on Form LD-252. This request will be made immediately after locations requiring such information have been determined by the Drainage Designer or as soon after Field Inspection as possible.
8.3.1.7 Subsurface Investigation

Subsurface data will be requested for all culvert installations with a diameter or span of 36” or greater. For multiple pipe installations, the span is measured between the interiors of the outside walls of the outer most pipes and is measured along a line perpendicular to the barrel of the pipe. Subsurface data may be requested for culvert installations with smaller diameter or if deemed necessary. Subsurface data should be requested on all pipes of any size 24” or greater in diameter that are to be bored or jacked.

Subsurface data should be requested for all SWM basins in order to determine if:
- The native material will support the dam and provide adequate protection for seepage under the dam
- Excavation from the basin may be used to construct the dam
- Rock may be encountered in the area of excavation
- A high water table is present which may alter the performance of the SWM basin

Borings shall be taken and information provided in accordance with Section 305.06 of the Materials Division Manual of Instructions (MOI). For large basins, more than one boring for the dam and one boring for the area of the basin may be needed. The number and locations of the borings are to be determined and requested by the Drainage Designer.

The existing subsurface soils data is not to be shown on the plans, however, the recommended amount of additional excavation and type of backfill material is to be shown in the drainage description.

At each location where a subsurface investigation is requested for pipe or box culvert installations, one should evaluate and classify the bed material in the outlet channel in close proximity of the downstream end of the proposed culvert. The bed material is to be classified in accordance with the AASHTO Soil Classification System.

This information is needed in order to evaluate the scour potential at the culvert outlet. This information is to be requested by the Drainage Designer along with the other soil and water data for each appropriate culvert installation.

8.3.1.8 Pipe Camber

Construction of longitudinal camber in a pipeline shall be considered when all of the following conditions are present:
- Grade of the pipe is less than 0.5%
- Fills (not height of cover) greater than 20’
- Diameter or span 36” or greater
- Foundation/subsurface is subject to settlement
The Drainage Designer will request that the Materials Division determine the amount of anticipated settlement along the pipeline. This request will accompany the request for culvert subsurface data. The plan description for the structure will then note a camber equal to the amount of anticipated settlement.

8.3.2 Hydraulic Criteria

These criteria relate to the hydraulic design of culverts based on flood flows, upstream and downstream water surface elevations, allowable velocities, and flow routing.

8.3.2.1 Design Storm

For stream crossing and longitudinal encroachments the inundation of the travelway and clearance below the low shoulder dictates the level of traffic services provided by the facility. New construction and projects that increase the level of service of the roadway shall have a minimum 18" clearance from the low shoulder of the crossing to the design storm as determined by the functional classification of roadways presented in Chapter 6, Hydrology. The analysis will document the flood elevations for the base flow, 2, 5, 10, 25, 50 and Base Flood events.

The above requirements are minimum, and design deviation for less than the minimum flood event requires approval from VDOT. Culverts should be designed to pass floods greater than those noted above where warranted by potential risk to adjacent property, loss of human life, injury, or heavy financial loss. Designing to a higher flood event does not require special approval, but may require justification if it results in an increase in cost without documented benefit to adjacent property and the public.

Future development of contributing watersheds and floodplains that have been zoned or delineated in local or regional planning documents (not flood plain zones) should be considered in determining the design flood. For the Interstate System, development during a period 20 years in the future should be considered. Adopted regional plans and approved zoning will be considered in determining the design discharge on all systems.

8.3.2.2 Allowable Headwater

The allowable headwater is the depth of water that can be ponded at the upstream end of the culvert during the design flood, measured from the culvert inlet invert. The headwater depth or elevation may also be limited by giving due consideration to inlet and outlet velocities and the following upstream water surface elevation controls:

- Not higher than an elevation that is 18" below the outer edge of the shoulder at its lowest point in the grade
- Upstream property damage
- Elevations established to delineate NFIP or other floodplain zoning
- HW/D is at least 1.0 and not to exceed 1.5 where HW is the headwater depth from the culvert inlet invert and D is the height of the barrel
- Low point in the road grade which is not necessarily at the culvert location
- Elevation of terrain or ditches that will permit flow to divert around the culvert

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-7 of 77
8.3.2.3 Review Flood

After sizing a drainage facility, it will be necessary to review this proposed facility with a higher discharge. This is done to ensure that there are no unexpected flood hazards inherent in the proposed facilities. The review flood is usually the base flood and in some cases, a flood event larger than the base flood is used for analysis to ensure the safety of the drainage structure and nearby development.

8.3.2.4 Tailwater Relationship – Channel

When the tailwater relationship is developed for the receiving channel, the designer should:

- Evaluate the hydraulic conditions of the downstream channel to determine tailwater depths for a range of discharges which include the check storm (see Chapter 6, Hydrology)
- For minor drainage installations with a 100-year discharge of less than 500 cfs, calculate the tailwater using a single cross section analysis
- For sensitive locations calculate the tailwater depth using step-backwater methods (such as HEC-RAS, etc.) or other step methods as appropriate. (Step-backwater methods yield the most accurate tailwaters)
- When using step-backwater methods to define barrel losses for subcritical flow in the culvert barrel, use critical depth at the culvert outlet if it is greater than the channel depth
- When using full flow nomographs to define barrel losses, use a calculated tailwater based on critical depth \( (d_c) \) and the height of the barrel \( (D) \) when that term \( TW = \frac{(d_c + D)}{2} \) is greater than the depth of flow in the outlet channel
- Use the headwater elevation of a downstream culvert if it is greater than the channel depth

8.3.2.5 Tailwater Relationship - Confluence or Large Water Body

When the tailwater relationship is developed from the confluence of a large body of water, the designer should:

- Use the highwater elevation that has the same frequency as the design flood if events are known to occur concurrently (statistically dependent)
- If events are statistically independent, evaluate the joint probability of flood magnitudes and use a likely combination resulting in the greater tailwater depth. Guidelines are provided in Joint Probability Analysis, Chapter 6, Appendix 6I.
- If tidal conditions are present at the site, use the mean high tide

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-8 of 77
8.3.2.6 Maximum Outlet Velocity

Our culvert outlet protection procedure has emphasis on the existing soil type to: 1) insure protection of the downstream channel or swale where material or lining in the downstream channel or swale may be unstable (erodible) under the anticipated velocities exiting the culvert, and 2) insure protection of the culvert end by providing measures to prevent the formation of a scour hole at the culvert outlet.

The type of material in the swale/channel at culvert outlets will need to be determined based on observations or field borings. The allowable velocity for natural material can be found in the table shown in Appendix 7D-2 of the VDOT Drainage Manual. The guidelines and procedures presented herein shall be implemented on all VDOT projects and those which will ultimately come under Department jurisdiction.

Highlights of these procedures/details are:

1. Maintains current rip rap sizes for outlet velocities 8 fps and greater
2. Establishes new riprap size for outlet velocities up to 8 fps
3. Allows the use of EC-3 Type 3 for velocities less than 6 fps
4. Maintains current apron dimensions for culvert installations with a total hydraulic opening of less than 7 ft².
5. Increases apron length to five times the height of the culvert for culvert installations with a total hydraulic opening of 7 ft² or greater.
6. Evaluates need for outlet protection based on 2-year culvert outlet velocity and allowable velocity of material in outlet channel or swale
7. Evaluates type of outlet protection required based on culvert outlet velocity for design discharge

The objectives of these details/procedures are to:

1. Minimize impacts to right of way of easement areas at smaller culvert sites
2. Minimize length of stream impacts
3. Minimize need for outlet protection where channel/swale material will be stable for culvert outlet velocities
4. Provide alternative to riprap at sites with low outlet velocities
5. Satisfy DCR Minimum Standard 11

Due to the magnitude of the changes to this section, shading has been omitted.
OUTLET PROTECTION DETAILS

- Dimensions Of Outlet Protection Apron:
  - Type A Installation – Minimum 3H Length & Minimum 3S Width
  - Type B Installation – Minimum 5H Length & Minimum 3S Width
    - Where:  
      - S = Span of Culver
      - H = Height of Culvert
    - For a multiple culvert line installations the largest S and H, dimensions of the individual culvert lines should be used in determining the minimum apron length dimensions.

- Outlet Protection Material
  - Standard EC-3 Type 3
  - Class A1 – Class A1 Dry Riprap
  - Class I – Class I Dry Riprap
  - Class II – Class II Dry Riprap

NEW OUTLET PROTECTION PROCEDURE

The following procedure shall be used to analyze the need for outlet protection on:

- All cross drain culverts
- All storm drain outlet pipes
- All entrance and crossover pipes with a diameter of 24” (or equivalent hydraulic opening) or greater

**Step 1 - Determine if Culvert Outlet Protection is required for protection of swale or channel.**

A. Compute culvert outlet velocity for 2-year design flood.

B. Compare 2-year design flood culvert outlet velocity to allowable velocity for outlet swale/channel material or lining.
   - Swale/channel material type based on field borings/observations or proposed lining.
   - Allowable velocity for natural swale/channel material based on VDOT Drainage Manual Chapter 7 - Appendix 7D-2.

C. If two year design storm culvert outlet velocity is equal to or less than allowable velocity for swale/channel material, no Culvert Outlet Protection is required for swale/channel protection.
   - Go to Step 2.

D. If two year design flood culvert outlet velocity is greater than allowable velocity for swale/channel material, Culvert Outlet Protection is required.
   - Go to Step 3.

*Due to the magnitude of the changes to this section, shading has been omitted.*
Step 2 - Determine Culvert Outlet Protection required for culvert end protection

A. Compute culvert outlet velocity for culvert design flood.

B. If culvert outlet velocity for culvert design flood is less than 6 fps, Culvert Outlet Protection is not required for culvert end protection.
   ➢ Stop

C. If culvert outlet velocity for design storm is 6 fps or greater, Culvert Outlet Protection is required for culvert end protection.
   ➢ Go to Step 3.

Step 3 – Determine Class of Culvert Outlet Protection to use.

A. When EC-1 Culvert Outlet Protection is required by either Step 1 or Step 2, EC-3 Type 3 or the Class of EC-1 to be specified shall be based on the culvert design storm outlet velocity with the following velocity limitations.
   • EC-3 Type 3 – maximum outlet velocity is 6 fps.
   • EC-1 Class A1 – maximum outlet velocity is 8 fps.
   • EC-1 Class I – maximum outlet velocity is 14 fps.
   • EC-1 Class II – maximum outlet velocity is 19 fps.
   • Use Special Design Culvert Outlet Protection for outlet velocity greater than 19 fps.

➢ Go to Step 4

Step 4 - Determine Type of EC-1 Installation to use.

A. When Culvert Outlet Protection is required by either Step 1 or Step 2, specify the Type of Installation to use based on the total hydraulic opening of the culvert installation.
   • Use Type A Installation for culvert installations with a total hydraulic opening of less than 7 ft².
   • Use Type B Installation for culvert installations with a total hydraulic opening of 7 ft² or greater.

PLAN DESCRIPTION

• ____ Sq. Yds. (Tons) Standard EC-1 Class ____ Required Type _____ Installation
• ____ Sq. Yds. Standard EC-3 Type 3 Culvert Outlet Protection Required

Road and Bridge Standard drawings 113.01 and 113.04 and Road and Bridge Specification Sections 414 and 606 have been revised to incorporate these protection measure details.

Due to the magnitude of the changes to this section, shading has been omitted.
8.3.2.7 Minimum Velocity
The minimum velocity in a culvert barrel should be adequate to prevent siltation during the design storm flows. When the streambed material size is unknown, use three (3) feet per second.

8.3.2.8 Storage Routing - Temporary or Permanent
It is VDOT practice to design culverts without recognizing or calculating the available upstream floodplain storage. The Department does not permit the consideration of any upstream floodplain storage, with the resultant attenuation of peak discharges, in the design of any culverts, bridges, or other drainage structures for either its own facilities or those that would ultimately come under its jurisdiction.

The Department will permit such consideration where it can be clearly shown that the drainage structure and roadway embankment in question and the upstream floodplain area have been designed to function as an impoundment for such facilities as ponds, lakes, detention/retention basins, etc. Another exception would be where approved FEMA delineated floodplain studies are in effect which indicate that the peak discharges have been reduced due to consideration of upstream flood storage.

VDOT's State Hydraulics and Utilities Engineer must approve any exception to the above.

8.3.2.9 Roadway Overtopping
Roadway overtopping should not be allowed for discharges equal to or less than the design discharge for new culvert installations. Overtopping is permitted to limit impacts to the 100-year flood event.

Roadway overtopping may occur when evaluating existing culvert installations for current design flows.

If roadway overtopping is indicated for the design flood event, it is necessary to consider the risk to highway users of loss of life, injury, and property damage. The highway embankment may be at risk based on:

- The depth of flow across the roadway
- The velocity of flow across the roadway
- The duration of roadway overtopping
- The resistance of the embankment to scour

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-12 of 77
8.3.3 **Geometric Criteria**

Design criteria related to the culvert geometry, including the inlet structure, the barrel, and the outlet structure are summarized in this section.

### 8.3.3.1 Culvert Size and Shape

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

- For the Interstate System, the minimum size of main line culverts will generally be 24” due to maintenance considerations
- For other systems, 15” will generally be the minimum culvert diameter, except that hydraulically adequate 12” diameter culverts may be used if the culvert length is less than 50’ or if it is located under an entrance
- Use arch or elliptical shapes only if required by hydraulic limitations, site characteristics such as cover, structural criteria, or environmental criteria

### 8.3.3.2 Multiple Barrels

Multiple barrel culverts should be designed to utilize the natural dominant channel with minimal or preferably no widening of the channel so as to avoid conveyance loss through sediment deposition in some of the barrels. An example of this concept would be a situation wherein a quadruple 10’x10’ box culvert is to be placed in a 15’ wide channel. The gross waterway opening (i.e. 400 ft²) and/or the configuration of that opening must be reduced for computational purposes commensurate with the amount of over bank area which will be displaced, on the premise that the natural stream channel and overbank configuration will reestablish through the culvert over time. It may even be necessary to place temporary timber/rock weirs to assist this process. Multiple barrels should be avoided where:

- 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)
- Fish passage is required unless special treatment is provided to ensure adequate low flow. When fish passage is required, all barrels are laid 6” below the streambed and a low flow diversion should be used to maintain the necessary depth in the appropriate barrel(s)
- A high potential exists for debris clogging the culvert inlet

### 8.3.3.3 Culvert Skew

The culvert skew should not exceed 45°, as measured from a line perpendicular to the roadway centerline, without the approval of VDOT. Furthermore, the Drainage Designer is to request, from the Structure and Bridge Division, the required details for modification of the standard drawings. This information is to be requested on Form LD-423. Box Culvert skews should be shown to the nearest 5° increment.

*Due to the magnitude of the changes to this section, shading has been omitted.*

*Chapter 8-13 of 77*
8.3.3.4 End Treatment (Inlet or Outlet)
The culvert inlet type should be selected from the following categories based on the considerations given and the inlet entrance loss coefficient, $K_e$. Appendix 8D-2 provides recommended values of $K_e$. Consideration should also be given to safety since some end treatments can be hazardous to errant vehicles. All culverts 48" diameter and larger should employ VDOT's standard headwalls, where available, or a comparable special design end treatment where a standard treatment does not apply.

- End Treatments will be provided, regardless of the highway classification, on:
  - All culverts conveying a live stream
  - All circular culverts with a diameter of 48" or greater
  - All culverts of an arch or elliptical shape with a hydraulic opening of 12 ft$^2$ or greater
  - All multiple line structures with a combined hydraulic opening of 12 ft$^2$ or greater

- Types of End Treatments:
  - Standard endwall
  - Modified endwall or special design endwall
  - Special Design Concrete Slab End Treatment, Special Design Drawing No. isd-2045 and msd-2045
  - Other types of end treatment with a foundation of sufficient width and depth to protect the culvert bedding material from seepage

The Standard ES-2 drawing in the Road and Bridge Standards includes a pay line designation that should not be interpreted as a required length of pipe to be attached to the end section. The connector section length may be whatever length the supplier wishes to attach, but the portion of the culvert included within the limits of the "C" dimension will be considered, for payment purposes, to be included in the price bid for the end section.

The supplier may furnish metal end sections with no connector section or with whatever length of connector section they determine convenient. The supplier and contractor will be responsible for determining what culvert pipe length will be required based on the length of connector sections, if any, that is furnished. Regardless of the length connector furnished as an attachment to the end section, that portion of the culvert designated "C" in the standard drawing will be measured and paid for as a part of the end section.

It is especially important that inspectors and other field personnel be aware of these instructions in order that an end section will not be rejected simply because the length of the connector is not the same as that shown on the Standard drawing. This variance is entirely acceptable provided the contractor has appropriately adjusted the length of the pipe.

The following sections present pros and cons for each type of end treatment.
8.3.3.4.1 Projecting Inlets or Outlets
Projecting inlets or outlets extend beyond the roadway embankment. These structures:

- Are susceptible to damage during roadway maintenance and from errant vehicles
- Have low construction cost
- Have poor hydraulic efficiency for thin materials such as corrugated metal
- Should not be used for culverts 48” diameter and larger
- Are subject to buoyancy

8.3.3.4.2 Prefabricated End Sections
Prefabricated end sections are available for both corrugated metal and concrete pipes. These sections:

- Should not be used for culverts 48” diameter or larger
- Retard embankment erosion and incur less damage from maintenance
- May improve projecting pipe entrances by increasing hydraulic efficiency, reducing the accident hazard, and improving their appearance
- Are hydraulically equivalent to a headwall, but can be equivalent to a beveled or side-tapered entrance if a flared, enclosed transition takes place before the barrel
- Are susceptible to buoyancy and may need concrete anchor blocks to resist hydrostatic uplift forces

8.3.3.4.3 Headwalls with Bevels
Headwalls with bevels are the standard VDOT design. These headwalls:

- Increase culvert efficiency
- Provide embankment stability and embankment erosion protection
- Provide protection from buoyancy
- Shorten the required structure length
- Reduce maintenance damage

8.3.3.4.4 Improved Inlets
Improved inlets are special designs which:

- Should be considered for exceptionally long culverts which will operate in inlet control or widening projects with increased flow to eliminate replacing existing culvert barrel(s)
- Can increase the hydraulic performance of the culvert, but may also increase total culvert cost
- If slope-tapered, should not be considered where fish passage is required
- Can increase outlet velocity

*Due to the magnitude of the changes to this section, shading has been omitted.*

*Chapter 8-15 of 77*
8.3.3.4.5 Wingwalls
Wingwalls are generally used in conjunction with headwalls and:

- Are used to retain the roadway embankment to avoid a projecting culvert barrel
- Are used where the side slopes of the channel are unstable
- Are used where the culvert is skewed to the normal channel flow
- Provide the best hydraulic efficiency if the flare angle is between 30° and 60°
- Are governed by VDOT height of embankment guidelines

8.3.3.4.6 Aprons
Aprons are special designs that can be used at culvert inlets and outlets and:

- Are used to reduce scour from high headwater depths or from high approach velocities in the channel
- Should extend at least one pipe diameter upstream
- Should not protrude above the normal streambed elevation

8.3.3.4.7 Cut-off Walls
Cut-off walls may be used at the entrance or the outlet of a culvert, and:

- Are used to prevent piping along the culvert barrel and undermining at the culvert ends
- Are an integral part of all of VDOT’s standard endwalls
- Should be included (a minimum of 1.5’ in depth) when other than VDOT standard endwalls are employed

8.3.3.4.8 Trash Racks or Debris Deflectors
Trash racks or debris deflectors may be necessary at sites where large amounts of detritus are produced. Such structures:

- May create clogging problems
- Require maintenance
- Should only be used where there is an established need

8.3.4 Safety Considerations

Each site should be inspected periodically to determine if safety problems exist for traffic or for the structural safety of the culvert and embankment.

Culvert headwalls and endwalls should be located outside the clear zone distance of the highway. The clear zone distance from the edge of pavement is a function of the design speed of the roadway. The typical clear zone distance for a high-speed highway is 30’. The designer is referred to the VDOT Road Design Manual for further information and to AASHTO for additional guidance. An exception to this clear zone requirement occurs if traffic is separated from the walls by guardrail that is required due to obstacles other than the walls.

*Due to the magnitude of the changes to this section, shading has been omitted.*

Chapter 8-16 of 77
Where feasible, grate drop inlets or load-carrying grates may be substituted for culvert headwalls or endwalls, and thereby reducing safety hazards. However, in making this substitution, consideration must be given to the possibility of creating a greater safety hazard by increasing the potential for flooding if the grates clog. The drainage designer should continuously coordinate roadway related issues and information with the roadway design team.

Pipe endwalls with load carrying grates (Standards EW-11 and EW-11A) are designed as a safety feature to prevent an errant vehicle from encountering the hazards of a collision with conventional endwalls or end sections. They are intended for use on low height embankments which would be traversable by an out of control vehicle and where guardrail would otherwise not be required or desired.

Standard EW-11 is to be used for cross drain culverts. The grate configuration must be installed perpendicular to the edge of the shoulder line.

Standard EW-11A is designed for use at crossover locations where there is no other alternative to placing a pipe culvert under the crossover.

The Drainage Designer is to carefully study each situation before specifying Standard EW-11 or EW-11A Endwalls on the plans. Guidelines for the use of these structures are as follows:

- Pipe endwalls with load carrying grates are to be used with traversable slopes (3:1 or flatter) on all classes of highways.
- Pipe endwalls with load carrying grates are not to be installed where guardrail is required.
- Pipe endwalls with load carrying grates will not be required on culverts with ends located outside of the normal clear zone width. For clear zone width guidelines, see Section A-2 of the VDOT Road Design Manual.
- Crossover locations should be thoroughly studied to eliminate, if possible, the need for a pipe culvert under the crossover. In the event there is no other alternative, the Standard EW-11A is to be specified.
- When pipe endwalls with load carrying grates are specified, the plans must be reviewed to ensure that all other hazards in the area are treated in an equally safe manner.

8.3.5 Allowable Pipe Materials

Refer to Road and Bridge Standards PC-1, as well as Section 9.4.9.4 of the VDOT Drainage Manual

*Due to the magnitude of the changes to this section, shading has been omitted.*

*Chapter 8-17 of 77*
8.3.6 Other Design Considerations

8.3.6.1 Buoyancy Protection
When water is displaced by embankment material or by a culvert, a buoyant or upward force exists. If the buoyant force is greater than the weight of the object displacing the water, flotation will occur. Pipe flotation (or hydrostatic uplift) can be a problem where the following conditions exist:

- Lightweight pipe is used (i.e., corrugated metal or plastic)
- Pipe is on a steep grade (usually inlet control)
- There is little or no weight on the end of the pipe (i.e., flat embankment slopes, minimal cover and/or no endwalls)
- High headwater depths (HW/D > 1.0)

8.3.6.2 Relief Opening
Where multiple-use culverts or culverts serving as relief openings have their outlet set above the normal stream flow line, special precautions should be provided to prevent headcuts, erosion from undermining the culvert outlet, or damage to downstream properties due to concentrated flow.

8.3.6.3 Land Use Culverts
Land use culverts are installations where storm drainage requirements are combined with other land based uses, such as farm or pedestrian crossings. For such installations:

- The land use is temporarily forfeited during the design flood, but is available during lesser floods
- Two or more barrels may be required, with one situated to be dry during floods less than the selected design flood
- The outlet of the higher land use barrel may need protection from headcutting
- The culvert should be sized so as to ensure that it can serve its intended land use function up to and including during a 2-year flood
- The height and width constraints should satisfy the hydraulic or land use requirements, whichever use requires the larger culvert

8.3.6.4 Erosion and Sediment Control
Temporary erosion and sediment control measures should be included in the construction plans. These measures include the use of the following: sediment basins and traps, silt barriers, dewatering basins, filter cloth, temporary silt fence and rock check dams. These measures should be utilized as necessary during construction to minimize pollution of streams and damage to wetlands. For more information, see Chapter 10, Erosion and Sediment Control.

*Due to the magnitude of the changes to this section, shading has been omitted.*

Chapter 8-18 of 77
8.3.6.5 Environmental Considerations and Fishery Protection

In addition to controlling erosion, siltation and debris at the culvert site, care must be exercised in selecting the location of the culvert. Where compatible with good hydraulic engineering a site should be selected that will permit the culvert to be constructed in the "dry" or that will cause the least impact to the stream or wetlands. This selection must consider the entire site involvement, preferably eliminating or at least minimizing the need for entrance and exit channels.

Where there is a U.S. Army Corps of Engineers jurisdictional stream bed, as determined by the Environmental Division, both up and downstream inverts of the proposed culvert will be set lower than the normal flow line of the stream in order to provide for the re-establishment of the streambed and low flow depth in the culvert that will facilitate fish passage.

Where the culvert is a multiple barrel or multiple cell structure, and* all barrels or cells are to be lowered below stream grade a low flow diversion should be used to maintain low flow in the appropriate barrel(s). The grade of a culvert located to facilitate fish passage should never be steeper than the grade of the natural stream in the site area. Preferably, the culvert barrel should be flattened as necessary to limit the velocity of flow in the culvert. The Corps of Engineers’ culvert countersinking requirements are described in detail in Section 8.3.7.

In areas of known fish habit, highway culverts are to be designed to accommodate the passage of fish. The design criteria for such culverts can be found in the following publications.

- An Analysis of the Impediments to Spawning Migrations of Anadromous Fish in Virginia Culverts (Pages 61 through 66) August 1985, by Mudre, Ney & Neves
- Nonanadromous Fish Passage in Highway Culverts Report No. VTRC 96-R6 October 1995 by Fitch

Summary of General Design Criteria:
- Criteria apply to normal water (ordinary high water) conditions. Set invert elevations of the low flow culvert 6” minimum below the streambed.
- Maintain a depth, width and velocity of flow in the culvert that matches, as nearly as practicable, the depth, width and velocity of flow in the natural channel up and down stream of the culvert.

8.3.6.6 Pipe in High Fills

Concrete pipe with a height of cover exceeding 30’ requires Special Design Pipe, certified in accordance with Section 105 of VDOT’s Road and Bridge Specifications and Method A Bedding in accordance with Standard PB-1.

The drainage description for these pipes should specify:
- **Special Design Concrete Pipe, Method A Bedding**
- **Pipe design to be in accordance with Section 105 of VDOT’s Road and Bridge Specifications**

*Due to the magnitude of the changes to this section, shading has been omitted.*

*Chapter 8-19 of 77*
In order to facilitate inspection and future rehabilitation (if needed) of culverts in fills (not cover) of 20' or greater, the minimum culvert size allowed/specified should be a 60" diameter. On Lower Functional Classification (LFC) roadways, as defined in the Allowable Pipe Type Tables in the Road and Bridge Standard PC-1, the District Construction or Maintenance Engineer and/or the Resident Manager/Engineer may waive the minimum 60" diameter size requirement provided that locations where the hydraulic capacity would require a pipe diameter of less than 60", the minimum pipe diameter shall be that necessary for adequate hydraulic conveyance plus 12" with a 36" minimum diameter and a 60" maximum diameter. The table below shows the minimum pipe diameter to use based on that required for hydraulic capacity.

<table>
<thead>
<tr>
<th>IF THE MINIMUM PIPE DIAMETER REQUIRED TO MEET HYDRAULIC CAPACITY IS:</th>
<th>THEN USE THIS PIPE DIAMETER IN FILLS ≥ 20':</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESIRABLE</td>
<td>MINIMUM</td>
</tr>
<tr>
<td>12” – 24”</td>
<td>60”</td>
</tr>
<tr>
<td>30”</td>
<td>-</td>
</tr>
<tr>
<td>36”</td>
<td>-</td>
</tr>
<tr>
<td>42”</td>
<td>-</td>
</tr>
<tr>
<td>48”</td>
<td>-</td>
</tr>
<tr>
<td>54”</td>
<td>-</td>
</tr>
</tbody>
</table>

It is recognized that it will be potentially more difficult for the inspection, maintenance and future rehabilitation (if necessary) of culverts in high fill areas if a size smaller than a 60" diameter is utilized.

8.3.6.7 Pipe Rehabilitation (Pipe not Replaced)

When existing pipes are damaged or deteriorated such that they are no longer functional or their functionality has been considerably impacted, a decision needs to be made as to what type of retrofit method should be employed. These methods include replacing the existing pipe or rehabilitating it by leaving it in place and lining it with one of several approved materials. The Drainage Designer should refer to the following guidance pertaining to the appropriate pipe rehabilitation methods to be used on each project:

The Special Provision for Pipe Rehabilitation lists three possible methods for accomplishing this work:

- Corrugated steel pipe liner
- Flexible pipe liner
- Smooth wall steel pipe liner

*Due to the magnitude of the changes to this section, shading has been omitted.*

Chapter 8-20 of 77
Some issues to be considered in the initial decision making process of whether to install a new pipe or rehabilitate the existing pipe are as follows:

- What is the condition of the existing pipe and what are the deficiencies that need to be addressed?
- Is the existing pipe located in a “hostile environment”? For example, is the pH of the water and soil and the resistivity beyond the acceptable limits shown in Table C (page 107.21) of Road and Bridge Standard PC-1 for the applicable pipe material?
- What is the height of cover over the existing pipe? If the height of cover is ≤ 5’, an economic evaluation should be performed to determine the feasibility of excavating and replacing, rather than rehabilitating the existing pipe. Where consideration is being given to the utilization of a flexible liner, an economic evaluation should be performed to determine the feasibility of excavating and replacing rather than lining the existing pipe, regardless of the height of cover.
- If considering a liner, what impact will the liner have on the hydraulic capacity of the existing pipe? This condition must be evaluated by a Hydraulic Engineer to determine if the liner reduces hydraulic capacity of the existing pipe to a point that upstream water surface elevations for the design storm event and the 100-year flood would be increased beyond that which is acceptable. If so, one option to consider would be to line the existing pipe and jack another line of pipe beside it to make up for the loss in hydraulic capacity.
- If using a liner, has the outlet velocity of the pipe increased as a result of changed hydraulic properties, i.e. decrease in Manning’s n value, decrease in flow area, etc.? This condition must be evaluated by a Drainage Design Engineer to determine if additional outlet protection (riprap) is required to dissipate outlet velocities.
- Has the deterioration of the pipe resulted in a situation where structural strength needs to be restored as part of the replacement method/material selected? This condition must be evaluated by a Structural and/or Materials Engineer to ensure the resulting repair provides sufficient strength to result in a safe and long lasting repair.

While these are some main points to consider in the initial decision making process for pipe rehabilitation, they are not all inclusive. Other issues relative to site-specific characteristics or limitations must also be taken into account in arriving at a final decision on the method of rehabilitation to use.

When considering a flexible liner, a decision matrix, as shown in Table A, can be useful in selecting the best type of flexible liner to utilize based on the existing pipe material and the noted deficiencies of the existing pipe or site limitations.

When using a Cured-in-Place Pipe (CIPP) liner as the method of rehabilitating an existing pipe, Scheduling and Contract Division’s Form C-9 (CIPP Inspection Checklist) shall be used by the VDOT Inspector to document the contractor’s pre-installation, installation and post-installation activities.

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-21 of 77
<table>
<thead>
<tr>
<th>Pipe Deficiency or Site Limitation</th>
<th>Concrete</th>
<th>Corrugated Metal</th>
<th>Plastic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minor Cracks</td>
<td>A, B, C, D</td>
<td>NA</td>
<td>A, B, C, D</td>
</tr>
<tr>
<td>Major Cracks and/or Spalls</td>
<td>A, B, C, D</td>
<td>NA</td>
<td>A, B, C, D</td>
</tr>
<tr>
<td>Joints Separated &gt;1”</td>
<td>A, B, C, D</td>
<td>A, B, C, D</td>
<td>A, B, C, D</td>
</tr>
<tr>
<td>Coating Removed, NC</td>
<td>NA</td>
<td>A, B, C, D</td>
<td>NA</td>
</tr>
<tr>
<td>Coating Removed, Min.C</td>
<td>NA</td>
<td>A, B, C, D</td>
<td>NA</td>
</tr>
<tr>
<td>Coating Removed, Maj.C</td>
<td>NA</td>
<td>A, B, C, D</td>
<td>NA</td>
</tr>
<tr>
<td>Minor Deformation, &lt;5% of inside diameter</td>
<td>NA</td>
<td>A, B, C, D</td>
<td>A, B, C</td>
</tr>
<tr>
<td>Intermediate Deformation, 5% to 7% of inside diameter</td>
<td>NA</td>
<td>A, B, C, D</td>
<td>A, B, C</td>
</tr>
<tr>
<td>Major Deformation, &gt;7% of inside diameter</td>
<td>NA</td>
<td>A, B, C, D</td>
<td>A, B, C</td>
</tr>
<tr>
<td>Height of cover</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Access (Limited space to end of pipe, accessible by manhole or drop inlet)</td>
<td>A, B, D</td>
<td>A, B, D</td>
<td>A, B, D</td>
</tr>
<tr>
<td>Bends in pipe</td>
<td>A, B, D</td>
<td>A, B, D</td>
<td>A, B, D</td>
</tr>
</tbody>
</table>

* Note: An economic evaluation should be performed to determine the feasibility of excavating and replacing rather than lining the existing pipe.

**LEGEND:**
- Category A – Cured In Place Pipe (CIPP)
- Category B – Fold and Form Flexible Liner
- Category C – HDPE, PVC, or Polypropylene (PP) slip liners
- Category D – Spray-On Liner (only applicable for pipes 36” or larger)
- NA – Not applicable
- NC – No Corrosion
- Min.C – Minor Corrosion
- Maj.C – Major Corrosion

*Due to the magnitude of the changes to this section, shading has been omitted.*

*Chapter 8-22 of 77*
Note:
The VDOT Materials Division Approved Products List No. 38 Pipe Rehabilitation Systems should be consulted for the current products approved for use. This list is available at: [http://www.virginiadot.org/business/resources/bu-mat-MD298-07.pdf](http://www.virginiadot.org/business/resources/bu-mat-MD298-07.pdf)


8.3.6.8 Existing Box Culvert Extensions

When the extension of an existing box culvert is required, the Drainage Designer shall specify Standard BCE-01 as a part of the box culvert description on the plans.

8.3.6.9 Small Box Culverts

Box culverts with heights and widths less than 4’ should be avoided due to concerns with inspection and maintenance. If a box culvert with a height or width less than 4’ is needed (e.g., for extension of an existing structure), the District Drainage Engineer should be consulted to determine if other alternate hydraulic structures are available.

8.3.6.10 Pile Foundation Design for Box Culverts

When the Materials or Structures and Bridge Division recommends pile foundations for box culverts, details are to be requested, by the Road Designer, from the Structure and Bridge Division on Form LD-422.

8.3.6.11 Trenchless Applications (Culvert Replacement or New Pipe)

There are certain cases where it is not feasible to install pipe through the existing embankment by the open-trench method. The alternative is to replace or install the pipe through the embankment by a trenchless construction method. The Drainage Designer is to specify the trenchless construction method of the pipe on the plans, where applicable. The three methods of trenchless applications accepted by the Department are as follows:

- Jack and Bore
- Microtunneling
- Pipe Jacking
The Engineer shall select the appropriate trenchless construction method for the application, and develop the plan accordingly, while taking the following items under consideration:

- Pipe Application
- Pipe Depth
- Pipe Length
- Pipe Diameter
- Pipe Type
- The working space required for the entry and receiving pits, and all appurtenant items, and obtaining any required R/W
- Existing Subsurface Conditions

For any trenchless construction methods to be successful, the Engineer should perform a predesign survey of the existing surface features and subsurface conditions, especially along the proposed pipe alignment.

The predesign survey should include, but not be limited to the following:

- General Site Conditions
- Subsurface Utility Engineering (SUE) located in or near the proposed pipe alignment
- Geotechnical Investigations (including groundwater)
- Environmental Conditions
- Required Drive Lengths
- Pipe Diameters, Site Access/constraints
- Depth
- Grade
- Tolerances
- Potential Impact to Surface Activities, including Maintenance of Traffic (MOT)
- Location of Existing / Abandoned / Proposed Utilities
- Rights-of-Way Requirements

The Engineer shall refer the Manual of Instructions by VDOT’s Materials Division for requirements of geotechnical investigations required prior to trenchless pipe applications and/or construction.

The Engineer shall refer to Chapter 13 of the VDOT Survey Manual to properly locate the existing utilities and underground hazards located at the project site, and within and immediately adjacent to the preferred trenchless construction method.

The Engineer shall refer to Table B, Trenchless Technology Applications, to aid in selection of the appropriate trenchless construction method relative to the existing site conditions.
Based on results of the geotechnical investigation, the Engineer shall determine the most appropriate trenchless technology, based on Table C, Applicability of Trenchless Technologies to Various Soil and Rock Conditions. If it is found that subsurface conditions may be marginal/possible, where difficulties may occur, the Engineer should make their determination based on further exploratory analysis and/or by consultation with the Materials Division and/or specialty contractors.

* Working Space is also required for all appurtenant items, such as control shed, pumps, lines, slurry tanks, etc.

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* Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-25 of 77
Table C - Applicability of Trenchless Technologies to Various Soil and Rock Conditions

<table>
<thead>
<tr>
<th>Soil and Rock Type</th>
<th>Jack and Bore</th>
<th>Micro-tunneling</th>
<th>Pipe Jacking</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft to very soft clays, silts, and organic deposits</td>
<td>Y</td>
<td>Y</td>
<td>M</td>
</tr>
<tr>
<td>Medium to very stiff clays and silts</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
</tr>
<tr>
<td>Hard clays and highly weathered shales</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
</tr>
<tr>
<td>Very loose to loose sands <em>above</em> the water table</td>
<td>M</td>
<td>Y</td>
<td>M</td>
</tr>
<tr>
<td>Medium to dense sands <em>below</em> the water table</td>
<td>N</td>
<td>Y</td>
<td>N</td>
</tr>
<tr>
<td>Medium to dense sands <em>above</em> the water table</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
</tr>
<tr>
<td>Gravel and cobbles with a diameter less than 2-4&quot;</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
</tr>
<tr>
<td>Soils with significant cobbles, boulders, and obstructions with a diameter more than 4-6&quot;</td>
<td>M</td>
<td>M</td>
<td>M</td>
</tr>
<tr>
<td>Weathered rocks, marls, chalks, and firmly cemented soils</td>
<td>Y</td>
<td>Y</td>
<td>M</td>
</tr>
<tr>
<td>Slightly weathered to unweathered rock</td>
<td>Y</td>
<td>M</td>
<td>N</td>
</tr>
</tbody>
</table>

*Source:* Iseley et al. (1999).

Y = generally used; M = possible, but difficulties may occur; N = generally unsuitable.

The following shall be deemed “high-risk” trenchless applications by the Department, and shall be reviewed by the District Materials Engineer prior to application acceptance:

- Proposed pipe diameter (O.D.) 24” and greater; and
- Pipe cover less than three times (3 x D) the pipe diameter; and
- ADT greater than 25,000 vehicles per day (vpd); or
- Proposed pipe diameter (O.D.) 60” and Greater; or
- Any other situation where there is significant risk (as interpreted by the Department).

For Land Use Permit Applications, trenchless construction will be given a conditional approval, as long as the application addresses the criteria listed above, and the applicant’s contractor has the minimum required experience as detailed in the Special Provisions available at VDOT’s Construction Division.

*Due to the magnitude of the changes to this section, shading has been omitted.*

*Chapter 8-26 of 77*
8.3.6.11.1 Jack and Bore
The Jack and Bore method includes the forming of a bore from the launching pit to a receiving pit, by means of a rotating cutting head, attached to the leading end of an auger string. Usually, a steel pipe that serves as a casing for carrier pipes, is installed in the process by jacking. As the boring operation proceeds, spoils are brought back to the launching pit for removal.

This method provides limited tracking and steering capability. With a steering head and water-level grade monitoring system, an accuracy of 1% of the length can be maintained in the vertical grade. Horizontal grade is generally not controlled and obstructions or boulders can cause large deflections.

8.3.6.11.2 Microtunneling
Microtunneling is similar to Jack and Bore, but includes remotely-controlled, laser-guided, steering head, known as a microtunnel boring machine (MTBM). An auger system can be used to remove the excavated soil, but more commonly a hydraulic system with slurry is used. The slurry aids in the drilling and counterbalancing groundwater and earth pressures.

Microtunneling is typically extremely accurate, with positional accuracy within 1 inch along the entire pipe run possible. Because of the level of accuracy, microtunneling is suitable for construction of large-diameter sewers and in congested subsurface environments with limited allowance for alignment deviations.

8.3.6.11.3 Pipe Jacking
Pipe jacking includes directly installing pipes behind a shield machine by hydraulic jacking from a drive shaft. In pipe jacking, personnel are usually required to enter the pipe to perform the excavation, whereby the excavation could be performed mechanically or manually. For the safety of those entering the workspace, a minimum pipe diameter of 42” is recommended.

For pipe jacking, it is recommended to have a minimum cover depth of 6’, or two times (2X) the O.D. of the pipe, whichever is greater. For slurry-installed pipe, it is recommended to have a minimum cover depth of 6’, or three times (3X) the O.D., whichever is greater.

Typical accuracies for pipe jacking include a tolerance of +/-3” for alignment, and +/-2” for grade.

RCP or steel casing pipe is normally employed in a pipe jacking operation. If steel casing pipe is used, a concrete (or occasionally metal or plastic) carrier pipe is installed inside of the steel pipe. The void between the two pipes is to be pressure grouted in accordance with Section 302.03 of the VDOT Road and Bridge Specifications.

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-27 of 77
In some specific circumstances, it has been deemed appropriate to install only the steel pipe to serve as the drainage pipe. THIS IS NOT TO BE CONSIDERED A UNIVERSALLY ACCEPTABLE PRACTICE.

The use of steel pipe as the actual drainage pipe must conform to Notes 1, 2 and 4 for Table A of “Allowable Types of Pipe” as shown in standard PC-1. Any deviation from this policy must go through the Design Waiver process, as per IIM-LD-227.

8.3.7 Counter Sinking and Low Flow Considerations

8.3.7.1 Definitions

8.3.7.1.1 Stream Bed

The stream bed is the substrate along the length of a stream, which lies below the ordinary high water elevation. The substrate may consist of organic matter, bedrock or inorganic particles that range in size from clay to boulders, or a combination of materials. Areas contiguous to the stream bed, but above the ordinary high water elevation, are not considered part of the stream bed.

8.3.7.1.2 Culvert

A culvert is generally defined as an enclosed structure that is used to convey surface waters from one side of an embankment to the other. For the purposes of this Manual there is no distinction between temporary and permanent culvert installations.

8.3.7.2 Policy

The District Environmental staff will determine if the culvert impacts a jurisdictional stream bed (US Army Corps of Engineers) and will notify the appropriate project authority and the Hydraulic Engineer when the below requirements must be incorporated into the design:

- Culverts constructed in jurisdictional stream beds are required to have the upstream and downstream inverts set (countersunk) below the natural stream bed elevation to stimulate natural stream bed establishment within the culvert and to meet the requirements of the environmental permitting process. The countersinking requirement does not apply to floodplain culverts or extensions or maintenance of existing structures where the existing structure will remain in service.

- When performing the hydraulic analysis for any culvert installation that is to be countersunk, the analysis shall either:

Due to the magnitude of the changes to this section, shading has been omitted.
1) Consider the hydraulic opening as being that above the countersunk portion of the culvert, or

2) Determine the required hydraulic opening (size) based on no countersinking; then specify the next larger size structure (3” or 6” greater height as appropriate) with the additional opening installed below the steam bed.

- When performing a hydraulic analysis for any multiple barrel culvert crossing, it is appropriate to consider the natural channel and flood plain configuration as projecting through the crossing, the same as if it were a bridge spanning a flood plain. For the purpose of determining the hydraulic capacity of the crossing, any culvert area that is outside the natural channel area and below the flood plain elevation will be considered obstructed and, therefore, not available for hydraulic conveyance.

- Culverts will be adequately sized to allow for the passage of ordinary high water with the countersinking, invert and flood plain restrictions taken into account.

- If the culvert is greater than 24” (or equivalent) in diameter, or rise in the case of noncircular shapes, the inlet and outlet ends shall be countersunk a minimum of 6” below the natural stream bed. If the culvert is 24” (or equivalent) or less in diameter, or rise in the case of noncircular shapes, the inlet and outlet ends shall be countersunk a minimum of 3” below the natural stream bed.

8.3.7.3 Multiple Barrel Culverts
When multiple barrel culverts are used, the 6” countersink requirement may only be needed for one barrel. The Hydraulic Engineer should determine whether it is appropriate and/or feasible to countersink one barrel or all of the barrels considering the following:

- The width of the culvert barrel(s) receiving the low flow should be approximately the width of the normal stream to avoid accelerating velocities (at normal flow) through the culvert.

- Narrow and constructed floodplains may necessitate all barrels being at the lowest possible elevation. Wide floodplains with significant over bank areas may permit one barrel to be countersunk and the remaining barrels to be either at the floodplain elevation or at an elevation slightly higher than the natural stream bed.

- Pipe Culverts may be designed to have barrels at different invert elevations. However, special provisions are needed to ensure proper bedding and backfill. Special Design Endwalls will be required. These considerations may negate any potential cost savings associated with not countersinking all barrels a like amount.
• Precast box culverts may be designed to have barrels at different invert elevations. In doing so, the installation is usually configured with the top of all barrels at the same elevation. This will require the same special considerations for bedding, backfill and endwall design as noted in Section 3.1.3. Cast in place box culverts usually have all barrels of the same size and elevation in order to construct the box culvert using standard details.

• Multiple barrel culverts that are constructed with all barrels countersunk shall provide measures for directing the low flow through one or more barrels that approximate the width of the normal stream. (See Road and Bridge Standards EC-13).26F

• If the normal stream width is approximately equal to the total span of all barrels, low flow diversion measures normally should not be needed. If the Hydraulic Engineer elects not to utilize a low flow diversion structure, the District Environmental Manager shall be notified of the decision and be provided justification in order to advise the environmental review agencies during the permitting process.

• When low flow diversion measures are needed, they shall be constructed to permit the stream to continue the natural meander or moving process normally associated with flood flows. The low flow diversion structures shall be constructed of rip rap, or other similar material. The rip rap material used should be small enough to allow movement during flood events (i.e., Class I Dry Rip Rap).

• Other methods of achieving the desired low flow conditions may also be employed. These shall be reviewed and approved by the District Environmental Manager.

8.3.7.4 Special Culvert Installations

8.3.7.4.1 Culverts on Bedrock

If the bedrock prevents countersinking, evaluate the use of a three-sided structure to cross the waterway or evaluate alternative locations for the new culvert that will allow for countersinking. If none of these alternative measures are practicable, the Hydraulic Engineer shall submit documentation to the District Environmental Manager, including the cost, engineering factors, and site conditions that prohibit countersinking the culvert, and shall coordinate the evaluation of options to minimize disruption of the movement of aquatic life. Options that must be considered include partial countersinking (such as less than 3” of countersinking, or countersinking of only one end of the culvert), constructing stone step pools and low rock weirs downstream of the culvert, or other measures that provide for the movement of aquatic life.

NOTE: Blasting of bedrock stream bottoms through the use of explosives is not acceptable as a means of providing for countersinking of pipes on bedrock.

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-30 of 77
8.3.7.4.2 Culverts on Steep Terrain

Culverts on steep terrain (slope of 5% or greater) may generate flow velocities that cause excessive scour at the outlet and may prevent the establishment of a natural bed of material through the culvert. Should this situation present itself, the Hydraulic Engineer shall coordinate the evaluation of alternatives to countersinking. These include partial countersinking of the inlet end and implementation of measures to minimize any disruption of the movement of aquatic life, constructing a stone step/pool structure, using river rock/native stone rather than riprap or constructing low rock weirs to create a pool or pools. Stone structures should be designed with sufficient-sized stone to prevent erosion or washout and should include keying-in as appropriate. These structures should be designed both to allow for aquatic life passage and to minimize scour at the outlet. The Hydraulic Engineer shall submit documentation to the District Environmental Manager, including the cost, engineering factors, and site conditions that prohibit countersinking the culvert, and shall coordinate the evaluation of options to minimize disruption of the movement of aquatic life.

8.3.7.4.3 Culverts at the Confluence of Two Streams

The outlet end of culverts that discharge a tributary directly into another stream must be countersunk below the natural stream bed at the discharge point. If this measure is not practicable, the Hydraulic Engineer shall submit documentation to the District Environmental Manager, including the cost, engineering factors, and site conditions that prohibit countersinking the culvert, and shall coordinate the evaluation of options to minimize disruption of the movement of aquatic life.

8.3.7.4.4 Other Situations

Other unusual circumstances that prohibit countersinking shall be evaluated on a case-by-case basis. The Hydraulic Engineer shall submit documentation to the District Environmental Manager, including the cost, engineering factors, and site conditions that prohibit countersinking the culvert, and shall coordinate the evaluation of options to minimize disruption of the movement of aquatic life.

Proposed culverts that do not include countersinking are subject to environmental agency review and approval and may require additional documentation or evaluation of other alternative measures.
8.3.8 Drainage Design at Railroads

On VDOT projects, where there is a need to install a culvert or a storm sewer pipe within railroad right of way, either under or adjacent to the tracks, the Hydraulic Engineer should contact the Department of Rail and Public Transportation to determine the specific design and construction criteria required by the Railroad Company and to initiate the process for obtaining any approvals needed from the Railroad Company. Railroad Companies generally follow engineering practices recommended by the American Railway Engineering and Maintenance-of-Way Association (AREMA) in their Manual of Recommended Practices for Railway Engineering, Volume I, Chapters 4 & 5. Railroad Companies reserve the authority to adopt and use more stringent design requirements, as they deem necessary. Some of the basic criteria for culverts and storm sewers that are to be located on railroad right of way are presented in this memorandum.

Projects that have railroad involvement generally are not advertised for construction until the Rail/Highway Agreement is fully executed. The execution of the Agreement by the Railroad Company is contingent upon their review and acceptance of the project design, especially the drainage design, as it relates to or affects their facilities. It is important that the Railroad Company be provided a complete and current set of plans and drainage computations for their review. The plan review and comment period by the Railroad Company can typically take three months or more for each submittal. Many projects take two or more reviews to address comments or correct plan omissions or errors. The time needed for review and coordination with the Railroad Company should be taken into consideration when establishing project schedules.

8.3.8.1 Criteria

8.3.8.1.1 Hydraulic Design Criteria

Culvert design follows the same FHWA methods used for VDOT highway projects with the following minimum criteria:

- The 25-year discharge shall produce a headwater elevation at the culvert entrance no greater than the top of the pipe (HW/D = 1.0).
- The 100-year discharge shall produce a headwater elevation at the culvert entrance no greater than 1.5 times the height of the culvert (HW/D = 1.5) or 2.0' below the elevation of the bottom of the rail, whichever is less.

Where field conditions do not permit installation of pipes sizes meeting this criterion, “pre and post construction” computations must be provided showing the headwater elevations for the 25-year and 100-year floods and demonstrating that there will be no increase in headwater depth due to the proposed construction. The Engineering Department of the Railroad Company must approve such designs.

*Due to the magnitude of the changes to this section, shading has been omitted.*

Chapter 8-32 of 77
**8.3.8.1.2 Pipe Size and Cover**

The minimum pipe size for use under the track is 36” diameter. A smaller size pipe may be allowed with the approval of the engineering department of the railroad.

The maximum pipe size for use under the track is 72” diameter. A larger size pipe may be allowed with the approval of the engineering department of the railroad.

The minimum pipe cover is to be 5.5’ as measured from the outside top of the pipe (casing pipe if used) to the bottom of the rail. Since survey crews often obtain the elevation of the top of the rail, an assumed rail height of 7 ½” may be used in determining the elevation of the bottom of the rail. Cover may also be determined by using the top of the cross tie elevation if the top of the rail elevation is unknown. In locations where the minimum cover cannot be obtained, a request must be made to the Railroad Company for an exception, with a complete explanation of the need for the exception.

**8.3.8.1.3 Pipe Materials and Installation**

Pipes to be installed under existing tracks will generally require the bore and jack or tunneling method of installation and must be so noted on the construction plans. An exception to this may be granted by the Railroad Company for spur tracks or tracks with infrequent use. Special circumstances, such as minimum cover, or other restrictions may sometimes necessitate that a pipe or box culvert be installed by the open cut method. These sites should be carefully reviewed by VDOT, the Department of Rail and Public Transportation and the Railroad Company to decide the appropriate methods and materials to be specified in the construction plans.

**SMOOTH WALL STEEL PIPE**

The Railroad Company’s standard pipe material for the bore and jack installation method is smooth wall steel pipe capable of supporting the Cooper E-80 loading. A structural analysis that is consistent with the Cooper E-80 loading requirements must be available for the Railroad Company’s review and approval should they desire. Section 105 of the Road and Bridge Specifications outlines the procedures that should be followed for this process.

The smooth wall steel pipe may function as the carrier pipe (i.e., used to convey the stormwater run-off) or function as a casing pipe for the actual carrier pipe. If installed as the carrier pipe, the smooth wall steel pipe must conform to the criteria set forth in the appropriate notes for uncoated galvanized steel pipe shown in Table A & A1 of the “Allowable Pipe Criteria for Culverts and Storm Sewers” in Standard PC-1 of the Road and Bridge Standards. The State Location and Design Engineer and the District Materials Engineer must approve any deviation from the noted criteria.

The drainage description for smooth wall steel pipes installed under the railroad by the bore and jack method should specify:

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-33 of 77
Jacked Smooth Wall Steel Pipe Req’d.

Pipe shall be designed to support Cooper E-80 loading in accordance with Section 105 of the Road and Bridge Specifications and installed by the bore and jack method. Smooth wall steel pipe shall have a minimum wall thickness of (See Table A).

Table A

<table>
<thead>
<tr>
<th>Smooth Wall Steel Casing Pipe</th>
<th>Minimum Wall Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>For Installation Under Railroads</td>
<td></td>
</tr>
<tr>
<td>Pipe Size (Inches)</td>
<td>Minimum Wall Thickness (Inches)</td>
</tr>
<tr>
<td>-------------------</td>
<td>-------------------------------</td>
</tr>
<tr>
<td>24</td>
<td>0.500</td>
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</tr>
<tr>
<td>72</td>
<td>1.000</td>
</tr>
</tbody>
</table>

CONCRETE PIPE

Under certain conditions, CSX Transportation, Inc. will allow concrete pipe Class V to be installed beneath the tracks without a casing pipe. In these cases, Class V concrete pipe may be used up to a cover height of 14’. For cover heights greater than 14’, a Special Design Concrete Pipe must be used. A structural analysis that is consistent with the Cooper E-80 loading requirements must be provided to the Railroad Company for their review and approval. Section 105 of the Road and Bridge Specifications outlines the procedures that should be followed for this process. The drainage description for such pipes should specify:

For cover heights 14’ or less:
Jacked Concrete Pipe Req’d. Class V
Pipe shall be installed by the bore and jack method.

For cover heights greater than 14’:
Special Design Jacked Concrete Pipe Req’d.
Pipe shall be designed to support Cooper E-80 loading in accordance with Section 105 of the Road and Bridge Specifications and installed by the bore and jack method.

The note referencing the Cooper E-80 loading and Section 105 of the Road and Bridge Specifications should also be included on the appropriate Drainage Summary Sheet.

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-34 of 77
CORRUGATED STEEL PIPE
For pipes to be installed under proposed or relocated tracks to be constructed on a new location, the open cut method of installation should be used. The pipe material generally accepted by the Railroad Company for this type of installation is corrugated steel capable of supporting the Cooper E-80 loading requirements. Aluminized Type 2 or Polymer Coated are the standard types of corrugated steel pipe allowed by VDOT. A structural analysis that is consistent with the Cooper E-80 loading requirements must be available for the Railroad Company’s review and approval should they desire. Section 105 of the Road and Bridge Specifications outlines the procedures that should be followed for this process. The drainage description for such pipes should specify:

Corrugated Steel Pipe Req’d.

Pipe shall be designed to support Cooper E-80 loading in accordance with Section 105 of the Road and Bridge Specifications.

The note referencing the Cooper E-80 loading and Section 105 of the Road and Bridge Specifications should also be included on the appropriate Drainage Summary Sheet. For locations where VDOT does not normally allow corrugated steel pipe (see Allowable Pipe Type Tables in Standard PC-1 of the Road and Bridge Standards), concern should be expressed to the Railroad Company about the use of this type of pipe material. Railroad Companies generally require that VDOT own and maintain any drainage structures that VDOT installs on railroad right of way. Therefore, we should endeavor to use the type of material that has proven to provide an appropriate life expectancy for specific site conditions. However, the Railroad Company will have final approval on the type of material and the installation method.

8.3.8.1.4 Drop Inlets
Drop inlets should generally not be located on the railroad right of way. When determined necessary to locate drop inlets on railroad right of way, they should be located no closer than 18’ from the track centerline. Railroads have a responsibility to their employees and customers to provide a hazard free operating corridor and are concerned with the hazard potential presented by grate inlets, especially those located in ditches. Any grate inlet that must be located within 18’ from the track centerline, or in an area where there is concern with a hazard potential due to grate openings, should have the bar spacing of the grates specified as would be required for pedestrian accessible areas. Where a Standard DI-5 or DI-7 inlet is proposed in these areas, a Type III grate shall be specified.

8.3.8.1.5 Ditches
Drainage ditches on railroad right of way that will convey VDOT roadway or bridge deck run off must be analyzed for the effects of the 100-year frequency discharge. This does not necessarily mean that the ditch must contain the 100-year flood but rather the effects of the 100-year flood must be documented. The analysis must be submitted to the Engineering Department of the Railroad Company for their review and approval. The analysis should present a factual scenario that is clear and easily understood.

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-35 of 77
A computer printout that is not clearly presented or explained is not usually acceptable to the Railroad Company.

8.3.8.1.6 Foundations for Signals
The location of proposed drainage structures may conflict with the foundations of proposed Railroad Company installed warning devices at rail crossings. The location of the warning device is prescribed by federal regulations and varies according to the typical section of the roadway and the alignment of the rail crossing. The location of proposed drainage structures in these areas should be reviewed with the Department of Rail and Public Transportation to determine any possible conflicts.

8.3.8.1.7 Endwalls and Other Structures
For construction detail requirements when placing pipe endwalls, manholes and other such structures adjacent to railroads, see Section 2E-24 of the VDOT Road Design Manual.

8.3.8.2 Guidelines
The following general guidelines are presented to assist the Drainage Designer in developing a design that is acceptable to the Railroad Company. These guidelines are representative of the comments received from Railroad Companies on past VDOT projects:

- For projects that are rebuilding an existing crossing, the existing drainage patterns should not be altered and documentation (a narrative with hydrologic and hydraulic computations) should be provided to the Railroad Company that indicates no increase in volume, velocity or flow depth/headwater depth is caused by the project on railroad right of way.
- Railroad Companies do not generally allow new drainage outfalls to discharge onto railroad right of way. Any existing outfall that is to be replaced or altered should be acceptable provided the documentation as previously noted for volume, velocity and flow depth/headwater depth is provided to the Railroad Company.
- When a constructed outfall (ditch or pipe) must be directed into a railroad ditch paralleling the rail bed, the constructed ditch or pipe should intersect the railroad ditch at an angle, in lieu of perpendicular, in order to lessen concerns with potential erosion. The appropriate erosion control measures should be applied at the intersection point to ensure stability of the rail bed and the existing railroad ditch.
- Proposed storm drain pipes paralleling the railroad tracks are not generally permitted to occupy the railroad right of way.
- Proposed roadway culverts and storm drains are not generally permitted to connect to existing railroad culverts. For situations where such a connection is unavoidable, the Railroad Company usually requires that VDOT assume maintenance responsibility for the railroad culvert.

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-36 of 77
• Scuppers, deck drains, drop inlets or other concentrated flow outlets from bridge decks are generally not allowed to drain directly onto the railroad right of way.
• Primary and emergency spillways and outfall structures of stormwater management basins, as well as the basin itself, are generally not allowed to be located on the railroad right of way. Where flow from a stormwater management basin is directed onto railroad right of way, documentation should be provided to the Railroad Company that indicates no increase in volume, velocity or flow depth/headwater depth is caused by the project on railroad right of way.

8.4 Design Concepts
8.4.1 General

The design of a culvert system for a highway crossing should consider: roadway requirements, planning and location, hydrology, ditches and channels, and erosion and sediment control. Each of these chapters should be consulted as appropriate. The discussion in this section is focused on alternative analyses and design methods.

For economy and hydraulic efficiency, culverts should be designed to operate with the inlet submerged during design flood flows. At many sites, either a bridge or a culvert will fulfill the structural and hydraulic requirements; therefore, the structure choice should be based on construction and maintenance costs, risk of failure, risk of property damage, traffic safety, and environmental considerations.

8.4.2 Design Methods

The designer should choose whether to use hand methods (nomographs or equations) or computer software solutions, such as FHWA's HY8.

The FHWA’s Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts," is the primary reference on culvert design.

8.4.2.1 Hydrologic Methods

Hydrologic methods are either steady state (constant discharge over time) or unsteady (flow varies with time, as in a hydrograph). See Chapter 6 for recommended methods.

The constant discharge method:

• Is the typical method used for most culvert designs
• Is usually assumed to be the peak discharge
• Will yield a conservatively sized structure where temporary storage is available but is not considered
8.4.2.2 Computational Methods

Computational methods include manual methods (Appendix 8B-1) and computer solutions. Manual methods usually employ design nomographs, provided in Appendix 8C. However, the design equations may also be applied. Computer solutions are usually employed for larger installations; however, they can be used for all situations.

8.4.2.2.1 Manual Methods

Manual methods using design equations and nomographs through (Appendix 8B-1):

- Require a trial and error solution that is straightforward and easy using design nomographs
- Provide reliable designs for many applications
- Require additional computations for tailwater, outlet velocity, hydrographs, routing and roadway overtopping
- Nomographs for a variety of barrel shapes are included in Appendix 8C

8.4.2.2.2 Computer Solution

One example of culvert analysis software is HY8, FHWA’s Culvert Analysis Microcomputer Program, which:

- Is an interactive program
- Uses the theoretical basis for the nomographs
- Can compute tailwater, improved inlets, road overtopping, hydrographs, routing and multiple independent barrels, and irregular shaped conduits
- Calculates backwater profiles in the culvert barrel(s)
- Develops and plots tailwater rating curves
- Develops and plots performance curves

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-38 of 77
8.4.3  **Culvert Hydraulics**

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

- Analysis of non-uniform flow with regions of both gradually varying and rapidly varying flow
- Determination of how the flow type changes as the flow rate and tailwater elevations change
- Application of backwater and drawdown calculations and energy and momentum balances
- Incorporation of the results of hydraulic model studies
- Determination of whether hydraulic jumps occur and whether they are inside or downstream of the culvert barrel
- Analysis of flows under subatmospheric pressure in the culvert barrel

The design procedures described in this chapter incorporate the following concepts:

8.4.3.1  **Control Section**

- The control section is the location where there is a unique relationship between the flow rate and the upstream water surface elevation
- The control section may be located at or near the culvert inlet (inlet control) or the culvert outlet (outlet control)
- Inlet control is governed by the inlet geometry
- Outlet control is governed by the culvert inlet geometry, as well as the barrel characteristics, and tailwater elevation(s)
- Tailwater control may be located downstream of the culvert

8.4.3.2  **Minimum Performance**

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

8.4.3.3  **Inlet Control**

For inlet control, the control section is at, or near, the upstream end of the barrel (the inlet). The flow passes through critical depth near the inlet and becomes shallow, high velocity (supercritical) flow in the culvert barrel. Depending on the tailwater elevation, a hydraulic jump may occur downstream of the inlet.

*Due to the magnitude of the changes to this section, shading has been omitted.*

*Chapter 8-39 of 77*
### 8.4.3.3.1 Headwater Factors - Inlet Control

The following factors are considered when calculating the inlet control headwater.

- **Headwater depth** is measured from the inlet 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. The inlet face area is the same as the barrel area, except for tapered improved inlets.
- **Inlet edge** configuration describes the entrance geometry. Some typical inlet edge 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 except for some improved inlets. Typical shapes are rectangular, circular, elliptical, and arch. Carefully check for additional control sections for special culvert designs.

### 8.4.3.3.2 Flow Conditions – Inlet Control

Three regions of inlet control flow are shown in Figure 8-1. They are unsubmerged, transition, and submerged. Generally, as the flow rate increases, inlet control flow passes through an unsubmerged condition (water surface below the crown of the control section), transition (between partly full and full flow), and submerged (water surface above the crown of the control section). The transition region is poorly defined and tends to be unstable. Its curve is usually drawn tangent to the unsubmerged and submerged performance curves.
Figure 8-1. Performance Curves - Unsubmerged, Transition, and Submerged
Four types of inlet control flow profiles within culverts are shown in Figure 8-2.

Figure 8-2. Types of Inlet Control Flow
8.4.3.3.2.1 **Unsubmerged - Inlet Control**
For headwaters below the inlet crown, the entrance operates as a weir, as shown in Figure 8-2, diagrams A and B. As shown, the outlet of the culvert may be unsubmerged or submerged.

- A weir is a 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
- Such tests are then used to develop equations. Appendix A of HDS-5 contains the equations, which were developed from model test data.

8.4.3.3.2.2 **Submerged - Inlet Control**
For headwaters above the inlet crown, the culvert operates as an orifice as shown in Figure 8-2, diagram C.

- An orifice is a submerged opening 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. Appendix A of HDS-5 contains flow equations, which were developed from model test data.

8.4.3.3.2.3 **Transition Zone - Inlet Control**
The transition zone is located between the unsubmerged and the submerged flow conditions where the relationship between flow and headwater depth 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.

8.4.3.3.2.4 **Special Condition - Inlet Control**
Figure 8-2, diagram D illustrates a special case of inlet control, where both the entrance and the outlet are submerged. To maintain this condition, a source of air must be supplied to the barrel; otherwise the barrel will tend to surge and alternate between full flow and partly full flow.

8.4.3.3.2.5 **Inlet Control Nomographs**
The inlet control flow versus headwater curves, which are established using the above procedure, are the basis for constructing the inlet control design nomographs in Appendix 8C. Note that in the inlet control nomographs, headwater (HW) is measured from the inlet invert to the total upstream energy grade line, including the approach velocity head.

8.4.3.4 **Outlet Control**

Culverts operating in outlet control have subcritical or full flow in their barrels. The control of the flow is at the downstream end of the culvert (the outlet) or further downstream.

*Due to the magnitude of the changes to this section, shading has been omitted.*

*Chapter 8-43 of 77*
The tailwater depth is assumed to be a function of either critical depth at the culvert outlet or the downstream channel depth, whichever is higher. In outlet control, the type of flow is dependent on the entire culvert, including the inlet configuration, the barrel, and the tailwater.

Five types of outlet control flow profiles within culverts are depicted in Figure 8-3. Note that both the inlet crown and the outlet crown may be submerged or unsubmerged.

Figure 8-3. Types of Outlet Control Flow

*Due to the magnitude of the changes to this section, shading has been omitted.*

Chapter 8-44 of 77
Figure 8-3; diagram A represents full flow throughout the culvert barrel. Both the entrance and the outlet are submerged.

Figure 8-3; diagram B shows the barrel inlet flowing partly full, but the rest of the barrel under full flow. The entrance is unsubmerged due to the inlet contraction, and the outlet is submerged.

Figure 8-3, diagram C represents full flow in the culvert barrel. The entrance is submerged and the outlet is unsubmerged.

Figure 8-3, diagram D represents full flow in the upper section of the barrel and partly full flow (subcritical) in the lower section of the barrel. The entrance is submerged and the outlet is unsubmerged.

Figure 8-3, diagram E depicts partly full flow (subcritical) over the length of the barrel. Both the entrance and the outlet are unsubmerged.

8.4.3.4.1 Headwater Factors - Outlet Control

The following factors are considered when calculating the headwater from outlet control.

- Barrel Roughness is a function of the barrel material and geometry. Typical materials include concrete and corrugated metal. The roughness is represented by a hydraulic resistance coefficient such as Manning's n-value. Typical Manning's n-values are presented in Appendix 8D-1
- Barrel Area is the full flow cross-section measured perpendicular to the flow
- Barrel Length is the total culvert length from the entrance crown to the exit crown of the culvert. Because the design height of the barrel and the embankment 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, and is often the same as the natural stream slope. However, when the culvert inlet or outlet is raised or lowered, the barrel slope is different from the stream slope
- Tailwater Elevation is based on the downstream water surface elevation. Backwater calculations from a downstream control, single section approximation, downstream lake levels, tidal elevations, or field observations are used to define the tailwater elevation. Tailwater elevations are normally calculated for different flood frequencies
8.4.3.4.2 Flow Condition - Outlet Control

Full flow in the culvert barrel is assumed for the analysis 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 outlet control headwater can be computed using the following equations:

8.4.3.4.2.1 Losses

The total headloss through the culvert is defined by Equation 8.1.

\[ H_L = H_e + H_f + H_o + H_b + H_j + H_g + H_v \]  

Where:

\[ H_L \] = Total energy loss, ft.
\[ H_e \] = Entrance loss, ft.
\[ H_f \] = Friction losses, ft.
\[ H_o \] = Exit loss, ft. (equals velocity head if \( K_o = 1.0 \))
\[ H_b \] = Bend losses, ft. (see HDS-5)
\[ H_j \] = Losses at junctions, ft. (see HDS-5)
\[ H_g \] = Losses at grates, ft. (see HDS-5)
\[ H_v \] = Velocity head, ft.

8.4.3.4.2.2 Velocity

Velocity is computed using the continuity equation.

\[ V = \frac{Q}{A} \]  

Where:

\[ V \] = Average full barrel velocity, fps
\[ Q \] = Flow rate, cfs
\[ A \] = Cross sectional area of flow with the barrel full, ft\(^2\)

8.4.3.4.2.3 Velocity Head

The velocity head represents the kinetic energy of full flow in the culvert barrel. It is used in calculating the losses in the culvert (inlet, barrel, outlet, etc.).

\[ H_v = \frac{V^2}{2g} \]  

Where:

\[ H_v \] = Velocity Head, ft.
\[ V \] = Average full barrel velocity, fps
\[ g \] = Acceleration due to gravity, 32.2 ft./s\(^2\)
### 8.4.3.4.2.4 Entrance Loss

The losses at the culvert entrance are a function of the velocity head. The more efficient the inlet, the lower the $K_e$ value.

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

Where:

- $H_e$ = Entrance head loss, ft.
- $K_e$ = Entrance loss coefficient, see Appendix 8D-2

### 8.4.3.4.2.5 Friction Loss

Friction loss in the culvert barrel is due to wall friction. It is a function of barrel roughness, size, shape, and velocity head, and is calculated using Manning's Equation.

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

Where:

- $n$ = Manning's roughness coefficient, see Appendix 8D-1
- $L$ = Length of the culvert barrel, ft.
- $R$ = Hydraulic radius of the full culvert barrel $= \frac{A}{P}$, ft
  - $A$ = Cross section area of pipe, $ft^2$
  - $P$ = Wetted perimeter of the barrel, ft.

### 8.4.3.4.2.6 Exit Loss

The exit loss is a function of the velocity head in the barrel and the velocity head in the downstream channel. The latter is often neglected.

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

Where:

- $V_d$ = Channel velocity downstream of the culvert, fps (if downstream velocity is neglected, use Equation 8.4d).
- $H_o = H_v \left( \frac{V^2}{2g} \right)$

### 8.4.3.4.2.7 Other Losses

Other possible losses in the culvert include junctions, bends, grates, etc. If present, these losses are functions of the velocity head multiplied by a loss coefficient. The loss coefficients are found in HDS-5.

*Due to the magnitude of the changes to this section, shading has been omitted.*
8.4.3.4.2.8 Barrel Losses
The various culvert losses are totaled to obtain the total headloss in the barrel. Losses for bends, junctions, grates, etc., should be added to Equation 8.5.

\[ H = H_E + H_o + H_f \]

\[ H = \left[ 1 + K_e + \left( \frac{29n^2L}{R^{1.33}} \right) \right] \left( \frac{V^2}{2g} \right) \] (8.5)

8.4.3.4.2.9 Energy Grade Line - Outlet Control
The energy grade line represents the total energy at any point along the culvert barrel. Equating the total energy at sections 1 and 2, upstream and downstream of the culvert barrel in Figure 8-4, the following relationship results:

\[ HW_o + \frac{V_u^2}{2g} = TW + \frac{V_d^2}{2g} + H_L \] (8.6)

Where:

- \( HW_o \) = Headwater depth above the outlet invert, ft.
- \( V_u \) = Approach velocity, fps
- \( TW \) = Tailwater depth above the outlet invert, ft.
- \( V_d \) = Downstream velocity, fps
- \( H_L \) = Sum of all losses (Equation 8.1)

Figure 8-4. Full Flow Energy and Hydraulic Grade Lines
8.4.3.4.2.10 Hydraulic Grade Line - Outlet Control

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 straight, parallel lines separated by the velocity head except at the inlet and the outlet.

Figure 8-5. Outlet Control Energy and Hydraulic Grade Lines

Due to the magnitude of the changes to this section, shading has been omitted.
**8.4.3.4.2.11 Outlet Control Nomographs (Full-flow)**

The outlet control nomographs were developed assuming that the culvert barrel is:

- Flowing full (See Figure 8-5, diagrams A and B)
- \(d_c \geq D\), (See Figure 8-5, diagram C)
- \(V_u\) is small and its velocity head can be considered to be a part of the available headwater (HW) used to convey the flow through the culvert
- \(V_d\) is small and its velocity head can be neglected

For these conditions, Equation 8.6 becomes:

\[
HW = TW + H - S_oL
\]

Where

- \(HW\) = Depth from the inlet invert to the energy grade line, ft.
- \(H\) = Headloss read from the nomograph (Equation 8.5), ft.
- \(S_o\) = Slope of culvert barrel, ft./ft.
- \(L\) = Length of culvert barrel, ft.

**8.4.3.4.2.12 Outlet Control (Partly Full-flow)**

Equations 8.1 through 8.7 were developed for full barrel flow. The equations also apply to the flow situations which are effectively full flow conditions, if \(TW < d_c\) (Figure 8-5, diagrams C and D), backwater calculations may be required which begin at the downstream water surface and proceed upstream. If the depth intersects the top of the barrel (Figure 8-5, diagram D), the full flow hydraulic grade line extends from that point upstream to the culvert entrance.

**8.4.3.4.2.13 Outlet Control Nomographs (Partly Full-flow) - Approximate Method**

Based on numerous backwater calculations performed by the FHWA staff, it was found that the full flow hydraulic grade line, extended from the upstream end of the barrel to the outlet, pierces the plane of the culvert outlet at a point about one-half way between critical depth and the top of the barrel, or \((d_c+D)/2\) above the outlet invert. \(TW\) based on the downstream channel depth should be used if it is higher than \((d_c+D)/2\).

The following equation should be used for headwater (HW):

\[
HW = h_o + H - S_oL
\]

Where:

- \(h_o\) = The larger of \(TW\) or \(\left(\frac{d_c+D}{2}\right)\), ft

Adequate results are obtained down to about \(HW = 0.75D\). For lower headwaters, backwater calculations are required.

*Due to the magnitude of the changes to this section, shading has been omitted.*

*Chapter 8-50 of 77*
8.4.3.5 **Outlet Velocity**

Culvert outlet velocities should be calculated to determine the need for erosion protection at the culvert exit. Culverts usually have 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 depth and the Froude number may also be needed.

**8.4.3.5.1 Inlet Control**

The velocity is calculated using Equation 8.2 with the flow area (A) equal to the cross section of the flow prism at the culvert outlet. First, the outlet depth must be determined. Either of the following methods may be used.

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

- Assume normal depth and velocity in the culvert barrel. This approximation may be used since the water surface profile approaches normal depth if the culvert is long enough. This outlet velocity may be slightly higher than the actual velocity at the outlet. Normal depths may be obtained from design aids in Appendix 8C.

![Diagram of Outlet Velocity - Inlet Control](image)

**Figure 8-6. Outlet Velocity - Inlet Control**

*Due to the magnitude of the changes to this section, shading has been omitted.*

*Chapter 8-51 of 77*
8.4.3.5.2 Outlet Control

The cross sectional area of the flow is defined by the geometry of the outlet and either critical depth, tailwater (downstream channel) depth, or the height of the conduit.

- Critical depth is used when the tailwater is less than critical depth
- Tailwater depth is used when tailwater is greater than critical depth, but below the top of the barrel
- The total barrel area is used when the tailwater level exceeds the top of the barrel

![Outlet Control Diagram](image)

Figure 8-7. Outlet Velocity - Outlet Control

8.4.3.6 Roadway Overtopping

Roadway overtopping will begin when the culvert 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 the FHWA's HDS No. 1, Hydraulics of Bridge Waterways. Curves for discharge coefficients are also included in Appendix 8C-60.

8.4.3.6.1 Length of Roadway Crest

The length of the roadway (weir) crest is difficult to determine when the crest is defined by a roadway sag vertical curve. It is recommended that the sag vertical curve be subdivided into a series of segments. The flow over each segment is then calculated for a given headwater. The flows for each segment are then added together to determine the total flow. Alternatively, the entire 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. The computer program HY8 allows input of the actual road surface x and y coordinates.

*Due to the magnitude of the changes to this section, shading has been omitted.*

Chapter 8-52 of 77
8.4.3.6.2 Total Flow

The flow over the roadway is calculated for a given upstream water surface elevation using Equation 8.9.

\[ Q_r = C_d L (H_{Wr})^{1.5} \]  

(8.9)

Where:

- \( Q_r \) = Overtopping flow rate, cfs
- \( C_d \) = Overtopping discharge coefficient (weir coefficient) = \( k_t \ C_r \)
- \( k_t \) = Submergence coefficient
- \( C_r \) = Discharge coefficient
- \( L \) = Length of the roadway crest, ft.
- \( H_{Wr} \) = Headwater depth, measured above the roadway crest, ft.

- Roadway overflow plus culvert flow must equal the 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 for various headwater elevations
- Performance curves for the culvert and the road overflow may be summed to yield an overall performance curve

Computer programs such as HY8 are recommended for design when evaluating roadway overtopping.

8.4.3.6.3 Performance Curves

Performance curves are plots of flow rate versus headwater depth or water surface elevation. The culvert performance curve is made up of the controlling portions of the individual performance curves for each of the following control sections as shown in Figure 8-8:

![Figure 8-8](image)

Figure 8-8. Overall Culvert Performance Curve

*Due to the magnitude of the changes to this section, shading has been omitted.*

Chapter 8-53 of 77
Inlet control performance curve is developed using the inlet control nomographs in Appendix 8C
Outlet control performance curve is developed using Equations 8.1 through 8.7, the outlet control nomographs in Appendix 8C, or backwater calculations
Roadway overtopping performance curve is developed using Equation 8.9
Overall performance curve is the sum of the flow through the culvert and the flow across the roadway and 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 occur. Calculate the upstream water surface depth above the roadway for each selected flow rate. Use these water surface depths and Equation 8.9 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 8-8.

8.4.4 Special Design Considerations

8.4.4.1 General

The following sections describe and discuss special culvert design considerations. References are provided for the detailed design methods.

8.4.4.2 Tapered Inlets

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section. A tapered inlet with additional depression at the upstream end also improves performance by increasing the head applied to the throat section.

- Tapered inlets are not recommended for use on short culverts or culverts flowing in outlet control because the simple beveled edge is of equal hydraulic benefit
- Design criteria and methods have been developed for two basic tapered inlet designs: the side-tapered inlet and the slope-tapered inlet
- Tapered inlet design charts from FHWA's HDS-5 for both rectangular box culverts and circular pipe culverts are included in Appendix 8C.

Due to the magnitude of the changes to this section, shading has been omitted.
Tapered inlets have several possible control sections including the face, the bend (for slope-tapered inlets), and the throat. The headwater depth for each control section is referenced to the invert of that section.

### 8.4.4.2.1 Side-Tapered Inlets

The side-tapered inlet has an enlarged face section with the transition to the culvert barrel accomplished by tapering the sidewalls (Figure 8-9). The face section is about the same height, as the barrel height and the inlet floor is an extension of the barrel floor. The inlet roof may slope upward slightly, provided that the face height does not exceed the barrel height by more than 10% (1.1D). The intersection of the tapered sidewalls and the barrel is defined as the throat section. There are two possible control sections, the face and the throat. $HW_f$, shown in Figure 8-9, is the headwater depth measured from the face section invert and $HW_t$ is the headwater depth measured from the throat section invert. The throat of a side-tapered inlet is a very efficient control section. The flow contraction is nearly eliminated at the throat.

![Figure 8-9. Side-Tapered Inlet](image)

The side-tapered inlet throat should be designed to be the primary control section for the design range of flows and headwaters.

*Due to the magnitude of the changes to this section, shading has been omitted.*

*Chapter 8-55 of 77*
8.4.4.2.2 Slope-Tapered Inlets

The slope-tapered inlet, like the side-tapered inlet, has an enlarged face section with tapered sidewalls meeting the culvert barrel walls at the throat section as shown in Figure 8-10. In addition, a vertical FALL is incorporated into the inlet between the face and throat sections. This FALL concentrates more head on the throat section. At the location where the steeper slope of the inlet intersects the flatter slope of the barrel, a third section, designated the bend section, is formed. Therefore, a slope-tapered inlet has three possible control sections, the face, the bend, and the throat.

![Figure 8-10. Slope-Tapered Inlet](image)

The slope-tapered inlet combines an efficient throat section with additional head exerted on the throat. The face section does not benefit from the FALL between the face and throat; therefore, the face sections of these inlets are larger than the face sections of equivalent depressed side-tapered inlets. The required face size can be reduced by the use of bevels or other favorable edge configurations. The slope-tapered inlet is the most complex inlet improvement recommended in this drainage manual.

*Due to the magnitude of the changes to this section, shading has been omitted.*

*Chapter 8-56 of 77*
Construction difficulties are inherent, but the benefits in increased performance can be significant. With proper design, a slope-tapered inlet passes more flow at a given headwater elevation than any other configuration. Slope-tapered inlets can be applied to both box culverts and circular pipe culverts. The slope-tapered inlet throat should be the primary control section in a slope-tapered inlet design.

HDS-5, Hydraulic Design of Highway Culverts, contains complete design methodology and design charts and forms for culverts with improved inlets. Most of the design charts have been included in Appendix 8C.

8.4.4.3 Buoyancy Protection
The buoyancy of a pipeline depends upon the weight of the pipe, the weight of the volume of water displaced by the pipe, the weight of the liquid load carried by the pipe and the weight of the backfill over the pipe. Lighter weight pipe materials are generally more susceptible to uplift forces than heavier materials.

If the summation of the weight of the pipe, weight of the water in the pipe (based on normal depth) and the weight of the fill over the pipe is less than the hydrostatic uplift (buoyant) forces acting upon the pipe, additional weight must be added to the pipe in order to stabilize it for the design conditions. The normal depth for determining buoyancy protection should be either the $Q_{100}$ headwater depth or the depth of overtopping, whichever is less.

A concrete endwall will usually provide sufficient weight to counteract potential buoyant forces. However, in low fill situations it is usually more desirable and economical to use end sections in lieu of endwalls or, in the case of secondary roadways, pipes are often installed projecting beyond the embankment slopes with no end treatment. In these situations, a concrete anchor block (counterweight) must be designed for each installation where it is determined that flotation may be a potential problem. See Figure 8-17.

In Section 8.5.3, a procedure is outlined (with example) showing how to analyze a pipe installation for flotation potential and, where it is determined that there is a potential problem, how to determine the amount of counterweight needed.

8.4.4.4 Minor Structure Excavation
Quantities for minor structure excavation will be computed for pipes and box culverts with a diameter or span of 48" and larger. For multiple pipe installations, the span is measured between the interiors of the outside walls of the outer most pipes and is measured along a line perpendicular to the barrel of the pipe. Minor structure excavation will be computed to a point 18" outside the periphery of the barrel section, or to a point bound by vertical planes coincident with the bedding limits shown on the Standard PB-1 drawings.
The minor structure excavation quantity for wingwalls and other appurtenances will be based on the “ratio” of the plan area of the wingwalls or appurtenances to the plan area of the barrel.

For single line culverts, the width of the barrel will be the nominal span or opening of the pipe or box culvert; for multiple spans, the barrel width will be the overall distance between inner faces of the outermost barrel openings. This dimension is defined by the S+2D value noted on the standard drawings for endwalls for multiple barrel culverts in the Road and Bridge Standards. The length of all culverts will be from end to end of the culvert. The outside wall thickness and the 18” outside the neatlines of the periphery of the culvert are not to be included in computing the “ratio.”

Once the “ratio” has been determined, it is used to compute the total cubic yards of Minor Structure Excavation for the structure and appurtenances, by using the excavation quantity for the barrel section and increasing this quantity by the “ratio.”

The sketch below denotes the area to compute the typical plan area for determination of box culvert “ratio.” For computation of “ratio” for pipes see Appendix D, Table D-28 through D-31 in the Road Design Manual.

Where End Sections are required and the pipe option of metal or concrete is allowed, use the area of the ES-2 (metal) end section for computing the “ratio.”

Where there is not sufficient survey data to accurately determine minor structure excavation quantities, additional survey must be secured and incorporated before making final quantity determinations.

Minor Structure Excavation will be measured in cubic yards and paid for on a Plan Quantity basis.

Excavation for wingwalls and other appurtenances will be based on the “ratio” of the plan area of the wingwalls or appurtenances to the plan area of the barrel.

A separate entry is to be shown on the Drainage Summary Sheet for cubic yards of Minor Structure Excavation for Pipes and cubic yards of Minor Structure Excavation for Box Culverts.
Figure 8-10(a) Typical Box Culvert

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-59 of 77
8.5 Design Procedures and Examples

8.5.1 Documentation Requirements

The results of the detailed analysis shall be incorporated into the Design Hydraulic Report as documented in Chapter 17 and shall include:

- Existing and Proposed Culvert Information: Inverts, Material, Size, Length, n-value and entrance condition and loss coefficient
- Tailwater Source and data
- Allowable HW/D design headwater elevation and basis for its selection
- Design Storm elevation HW/D and clearance to low shoulder
- Culvert outlet appurtenances and energy dissipation calculations and designs, including riprap proposed as needed for culvert outlet protection.

Sample Documentation provided in Chapter 17 Appendix.

8.5.2 VDOT Culvert Design Procedure

The following design procedure provides a convenient and organized method for designing culverts for a constant discharge, considering inlet and outlet control. The procedure does not address the effect of storage, which is discussed in Chapter 11, Stormwater Management.

- The designer should be familiar with all of the equations in Section 8.4 before using these procedures
- Following the design method without an understanding of culvert hydraulics can result in an inadequate, unsafe, or overly costly structure
- The culvert calculation form has been provided in Appendix 8B-1 to guide the user. It contains blocks for the project description, designer's identification, hydrologic data, culvert dimensions and elevations, roadway controls and property elevations, trial culvert description, inlet and outlet control HW, culvert barrel selected, and comments.

Step 1 Assemble site data and project file

a. The minimum site data are:
   - USGS, site and location map
   - Embankment cross section
   - Roadway profile
   - Photographs
   - Field visit (sediment, debris)
   - Design data at nearby structures
   - Existing utilities
b. Studies by other agencies including:

- Small dams — NRCS, USCOE, TVA, BLM
- Canals — NRCS, USCOE, TVA, USBR
- Floodplain — NRCS, USCOE, TVA, FEMA, USGS, NOAA
- Storm drain - local or private

c. Environmental constraints including:

- Commitments contained in review documents
- Commitments contained in permits or permit applications
- Fish migration
- Wildlife passage
- Wetlands resources

d. Design criteria:

- Review Section 8.3 for applicable criteria
- Prepare risk assessment or analysis, if needed

**Step 2 Determine hydrology**

- See Chapter 6, Hydrology
- Minimum data are drainage area map and a discharge-frequency plot

**Step 3 Design downstream channel**

- See Chapter 7, Ditches and Channels
- Minimum data are geometry and the rating curve for the channel that provides tailwater elevations for various flood frequencies

**Step 4 Summarize data on design form**

- Enter data from steps 1-3

**Step 5 Select design alternative**

- See Section 8.3.3, Geometric Criteria
- Choose culvert material, shape, and entrance type
- Consider flow line, cover, and utilities

**Step 6 Select design discharge \( Q_d \)**

- See Section 8.3.2 Hydraulic Criteria
- Determine flood frequency from criteria
- Determine \( Q \) from discharge-frequency plot (Step 2)
• Divide Q by the number of barrels
• Select trial size

**Step 7 Determine inlet control headwater depth (HWi)**

• Use the appropriate inlet control nomographs in Appendix 8C

**Step 8 Determine outlet control headwater depth at inlet (HWoi)**:

a. Calculate the tailwater depth (TW) using the design flow rate and normal depth (single section), using a water surface profile, or obtain it from other sources

b. Calculate critical depth (dc) using the appropriate chart in Appendix 8C
   • Locate flow rate and read dc
   • dc cannot exceed D
   • If dc > 0.9D, consult Handbook of Hydraulics (King and Brater) for a more accurate dc, if needed, since curves are truncated where they converge

c. Calculate \(\frac{d_c + D}{2}\)

d. Determine ho
   \[h_o = \text{the larger of } TW \text{ or } \frac{d_c + D}{2}\]

e. Determine Ke

f. Entrance loss coefficient from Appendix 8D-2

g. Determine losses through the culvert barrel (H)
   • Use the nomographs in Appendix 8C or Equation 8.5 or 8.6 if outside range of nomograph scales

h. Calculate outlet control headwater (HWoi)
   • Use Equation 8.8, if Vu and Vd are neglected:
     \[HW_{oi} = h_o + H - S_o L\]
   • Add other losses (bends, grates, etc.) to right side of equation.
   • Use Equation 8.1, 8.4c and 8.6 to include Vu and Vd.
   • If HWoi is less than 1.2D and control is outlet control:
     ➢ The barrel may flow partly full

*Due to the magnitude of the changes to this section, shading has been omitted.*

Chapter 8-62 of 77
The approximate method of using the greater of tailwater or \( \frac{d_c + D}{2} \) may not be applicable.

Backwater calculations should be used to check the result.

If the headwater depth falls below 0.75D, the approximate nomograph should not be used.

Step 9  Determine controlling headwater (HWc)

a. Compare \( HW_i \) and \( HW_{oi} \), and use the higher

b. Compare HW to allowable HW criteria (cover, \( \frac{HW}{D} \), shoulder)

Step 10  Compute discharge over the roadway (\( Q_r \)) if applicable (See Section 8.4.3.6)

Step 11  Compute total discharge (\( Q_t \))

\[
Q_t = Q_d + Q_r
\]

Step 12  Calculate outlet velocity (\( V_o \)) and normal depth (\( d_n \))

If **inlet control** is the controlling headwater:

a. Calculate flow depth at culvert exit

- Use normal depth (\( d_n \)), or
- Use water surface profile

b. Calculate flow area (A)

c. Calculate exit velocity, \( V_o = \frac{Q}{A} \)

If **outlet control** is the controlling headwater:

a. Calculate flow depth at culvert exit

- Use \( d_c \) if \( d_c > TW \)
- Use \( TW \) if \( d_c < TW < D \)
- Use \( D \) if \( D < TW \)

b. Calculate flow area (A)

c. Calculate exit velocity, \( V_o = \frac{Q}{A} \)
Step 13  Review results

Compare alternative design with constraints and assumptions. If any of the following are exceeded, repeat steps 5 through 12:

- The barrel must have adequate cover
- The length should be close to the approximate length
- The headwalls and wingwalls must fit the site
- The allowable headwater should not be exceeded and \( \frac{HW}{D} \) should be at least 1.0 and not exceed 1.5
- The design storm should provide 18” of clearance to the low shoulder of the crossing

Step 14  Analyze base flood discharge

Step 15  Related designs

Consider the following options (See Sections 8.3.6 and 8.4.4 and Chapter 11, Stormwater Management):

- Tapered inlets if culvert is extremely long, in inlet control, and has limited available headwater
- Flood routing if a large upstream headwater pool exits
- Energy dissipators or standard EC-1, as needed. Special design energy dissipators may be required. Appendix 8E-1 contains procedures and discussion for a riprap basin
- Weirs, if needed to maintain low flow through multiple barrel culverts

8.5.3 Culvert Design Sample Problems

The following example problem follows the Design Procedure Steps described in Section 8.5.2.

Step 1.  Assemble Site Data and Project File

a. Site survey project file contains:

- USGS, site, and location maps
- Roadway profile, and
- Embankment cross-section

b. Site visit notes indicate:

- No sediment or debris problems
- No nearby structures
- Studies by other agencies – none

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-64 of 77
c. Environmental, risk assessment shows:
   - No buildings near floodplain
   - No sensitive floodplain values
   - No FEMA involvement
   - Convenient detours exist

d. Design criteria:
   - 50-year frequency for design, and
   - Base flood frequency for impact
   - Allowable headwater depth for design flood = 8.5’
   - 100-year floodplain depth = 10.0’

**Step 2. Determine Hydrology**

USGS Regression equations yield:

\[ Q_{50} = 400 \text{ cfs} \]
\[ Q_{100} = 500 \text{ cfs} \]

**Step 3. Account for tailwater**

Slope = 0.05 ft./ft.
Length = 100’

The predetermined depths and velocities for the downstream channel are:

<table>
<thead>
<tr>
<th>Q (cfs)</th>
<th>TW (ft.)</th>
<th>V (ft./s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>400</td>
<td>2.8</td>
<td>18</td>
</tr>
<tr>
<td>500</td>
<td>3.1</td>
<td>19</td>
</tr>
</tbody>
</table>

**Step 4. Summarize data on design form**

(See Figure 8-11)

**Step 5. Select trial design structure**

Shape: Box
Size: 7.0’ (B) by 6.0’ (D)
Material: Concrete
Entrance: Wingwalls with 30°-75° flare

*Due to the magnitude of the changes to this section, shading has been omitted.*

Chapter 8-65 of 77
Step 6. Select design discharge
\[ Q_d = Q_{50} = 400 \text{ cfs} \]

Step 7. Determine inlet control headwater depth (HW\(_i\))

Use inlet control nomograph - Appendix 8C-8

a. \( D = 6.0' \)

b. \( \frac{Q}{B} = \frac{400}{7} = 57 \text{ cfs} \)

c. \( \frac{HW}{D} = 1.30 \)

d. \( HW_i = \left(\frac{HW}{D}\right) D = (1.30)6.0 = 7.80 \text{ ft.} \) (Neglect the approach velocity.)

Step 8. Determine outlet control headwater depth at inlet (HW\(_{oi}\)):

a. \( TW = 2.8' \) for \( Q_{50} = 400 \text{ cfs} \)

b. \( d_c = 4.6' \) from Appendix 8C-14

c. \( \left(\frac{d_c+D}{2}\right) = \left(\frac{4.6+6.0}{2}\right) = 5.3 \text{ ft.} \)

d. \( h_o = \) the larger of \( TW \) or \( \left(\frac{d_c+D}{2}\right) = 5.3 \text{ ft.} \)

e. \( K_e = 0.4 \) from Appendix 8D-2 (for 30°-75° wingwalls)

f. Determine \( (H) \) - use Appendix 8C-15
   - \( K_e \) scale = 0.4
   - Culvert length, \( L = 100 \text{ ft.} \)
   - \( n = 0.012 \) (same as on Appendix 8C-15)
   - Area = 42 ft\(^2\)
   - \( H = 2.3 \text{ ft.} \)

g. \( HW_{oi} = h_o + H - S_0L \)
\[ HW_{oi} = 2.3 + 5.3 - 100(0.05) \]
\[ HW_{oi} = 2.6 \text{ ft.} \]
Step 9. **Determine controlling headwater (HW_c)**

\[
HW_i = 7.80' \\
HW_{oi} = 2.6' \\
HW_c = \text{The greater of } HW_i \text{ or } HW_{oi} \\
HW_c = HW_i = 7.80'
\]

The culvert is in **inlet control**

Step 10. **Compute discharge over the roadway (Q_r)**

Not applicable

Step 11. **Compute total discharge (Q_t)**

\[Q_t = 400 \text{ cfs}\]

Step 12. **Determine outlet velocity (V_o)**

- Use Appendix 8C-83
- Enter 4.8 (400 x 0.012) on the horizontal, “Qn” scale
- Read vertically to the “Slope” curve of 0.05
- Read horizontally to the “Vn” scale and find a value of 0.37. Then divide this by the “n” value (0.012) and find a velocity of 30.8 fps.

Step 13. **Repeat steps 5-10 for check flood (100-yr.):**

- Compare design with constraints and assumptions. If any of the following are exceeded, repeat steps 5 through 12:
  - 100-year floodplain depth = 10.0' > 9.6'
  - Overtopping flood frequency > 50-year

Step 14. **Design Considerations (None)**

Step 15. **Complete any additional necessary documentation**
Figure 8-11. Completed Culvert Design Form, Sample Problem

* Special Design Energy Dissipator Required

Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-68 of 77
8.5.4 Buoyancy Protection Procedure

8.5.4.1 Hydrostatic Uplift and Resistance

- Resistance = Weight of pipe + Weight of water (in pipe) + Weight of fill (over pipe), lbs/ft.
- Hydrostatic Uplift (Buoyant) Force = Weight of water displaced by the pipe, lbs/ft.
- The following average values can be used in the analysis:

  Weight of Pipe - See manufacturer's weight tables for type and size of pipe specified.
  Weight of Fill (Dry) - 100 lbs/ft³
  Weight of Fill (Saturated) - 37.6 lbs/ft³
  Weight of Water - 62.4 lbs/ft³

![Figure 8-12. Hydrostatic Uplift Forces and Effects on Pipe](image)

If Resistance < Hydrostatic Uplift:

Increase weight on end of pipe by adding concrete endwall or concrete anchor blocks.
8.5.4.2 Buoyancy Protection Sample Problem

Given:

- 48" Corrugated Metal Pipe, 12 gage
- Fully Coated with Paved Invert
- Std. ES-2 End Section
- \( Q = 96 \text{ cfs} \)

Computed Values:

- \( \text{HW/D} = 1.25 \)
- \( \text{HW} = 5' \)
- \( d_n = 2.5' \)
- \( d_c = 2.8' \)

Assumed Values:

- Weight of Fill (Dry) = 100 lbs/ft\(^3\)
- Weight of Fill (Saturated) = 37.6 lbs/ft\(^3\)
- Weight of Water = 62.4 lbs/ft\(^3\)
- Weight of Pipe = 84 lbs/ft.

...Handling weight of corrugated steel pipe available in Appendix 8F-1 and 8F-2...

---

**Figure 8-13. Buoyant Forces Acting on Pipe**

**Step 1:** Compute buoyant force acting on pipe

(At any section along length of pipe)

Buoyant force (lbs/ft.) = Weight of water displaced by pipe (lbs/ft\(^3\))

Buoyant force = \( L(A)(\gamma) \)

Where:

*Due to the magnitude of the changes to this section, shading has been omitted.*
Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-71 of 77

\[ L = \text{Unit length of pipe, ft.} \]
\[ A = \text{Cross sectional area of pipe, ft}^2 \]
\[ \gamma = \text{Unit weight of water} = 62.4 \text{ lbs/ft}^3 \]
\[
\begin{align*}
1 \left( \frac{\pi D^2}{4} \right) (62.4) \\
= 1 \left( \frac{\pi (4)^2}{4} \right) (62.4) \\
= 784 \text{ lbs/ft.}
\end{align*}
\]

Step 2: Compute total surcharge at section 1

(Located at inlet end of pipe)

Surcharge (lbs./ft.) = Wt. of Fill + Wt. of Water + Wt. of Pipe

Compute weight of fill material (Hatched Area, Figure 8-14)

Weight of Fill = Area x 1 ft. x 37.6 lbs/ft^3. (saturated fill)

Area = \left[ \frac{D^2 - \frac{\pi D^2}{4}}{2} \right] = \left[ \frac{(4)^2 - \frac{\pi (4)^2}{4}}{2} \right] = 1.7 \text{ sq. ft.}

Weight of Fill = 1.7 \left(1\right)(37.6)

Weight of Fill = 64 \text{ lbs/ft.}

Compute weight of water (inside pipe)

Weight of Water = Area of flow x 1 ft. x 62.4 lbs/ft^3

Assume depth of water, \(d = d_n = 2.5\)'

\[
\frac{d}{D} = \frac{2.5}{4} = 0.625
\]

\[
\frac{\text{Area}}{D^2} = 0.516 \text{ (From Appendix 8F-5)}
\]

Area of flow = 0.516 x 4 ft^2 = 8.26 ft^2
Weight of Water = 8.26 ft^2 x 1 ft. x 62.4 lbs/ft^3.
Weight of Water = 515 lbs/ft.
- Determine weight of pipe
  
  84 lbs/ft.

- Compute total surcharge at section 1
  
  Surcharge = Wt. of Fill + Wt. of Water + Wt. of Pipe
  Surcharge = 64 + 515 + 84
  = 663 lbs/ft.

- Section 1 Summary
  
  Surcharge (663 lbs./ft.) < Buoyant Force (784 lbs./ft.)
  Therefore, pipe is unstable at Section 1

**Step 3:** Compute total surcharge at Section 2

(Section 2 is located where headwater elevation intercepts the fill slope 6’ from the inlet end of the pipe)
Surcharge (lbs./ft.) = Wt. of Fill + Wt. of Water + Wt. of Pipe

- Compute weight of fill material (Hatched Area, Figure 8-15)
  
  \[
  \text{Area} = \left( \frac{D^2 - \pi D^2}{4} \right) + 1D = \left( \frac{4^2 - \pi (4)^2}{4} \right) + 1(4) = 5.7 \text{ sq. ft.}
  \]

  Weight of Fill = 5.7 ft.² x 1 ft. x 37.6 lbs/ft³ (saturated fill)
  Weight of Fill = 214 lbs/ft.

- Compute weight of water (Inside Pipe)
  
  Assume depth of water, \(d = d_n = 2.5’\)
  Weight of Water = 515 lbs/ft.
  (Same as Section 1)
- **Determine weight of pipe**
  
  84 lbs/ft. (Same as Section 1)

- **Compute total surcharge at Section 1**

  Surcharge (lbs./ft.) = Wt. Fill + Wt. Water + Wt. Pipe
  
  Surcharge = 214 + 515 + 84 = 813 lbs/ft.

- **Section 2 Summary**

  Surcharge (813 lbs/ft.) > Buoyant Force (784 lbs/ft.)
  
  Therefore, pipe is stable at Section 2

**Step 4:** *Determine minimum weight required to counteract buoyant force*

a. Plot a graph of length along the pipe (from inlet end) versus total surcharge buoyancy (weight). Let the horizontal axis represent the length along the pipe (ft.) and the vertical axis represent the surcharge/buoyancy (lbs/ft.) as shown in Figure 8-16.

![Figure 8-16. Surcharge/Buoyancy along Length of Pipe](image)

b. Plot the values of length along pipe and total surcharge for Section 1 (from Step 2) and Section 2 (from Step 2) on the graph (Points A and B) and connect them with a straight line (Line A).

c. Plot horizontal line (Line B) on the graph representing the buoyant force computed in Step 1.

*Due to the magnitude of the changes to this section, shading has been omitted.*

*Chapter 8-73 of 77*
d. The area of the triangle formed by Line A, Line B and the vertical axis of the graph (hatched area) represents the minimum weight required to balance the uplift (buoyant) force.

e. Determine minimum required weight (area of triangle).

1) Using ratio and proportion analysis, determine length along horizontal axis where Line A and Line B intersect (Point C).

   Find intersection (Point C) at 4.88' 

2) Weight (Area) = (Vertical side x Horizontal side)/2.

3) Weight = (122 lbs./ft. x 4.88 ft.) / 2 = 298 lbs.

f. Determine minimum weight of required anchor block. Set minimum weight of anchor block equal to the greater of:

1) The required additional weight (Step 4e) plus 100 lbs. or 

2) 1.5 times the required additional weight (Step 4e).

g. Determine size of required anchor block.

1) Use minimum size anchor block if its weight is equal to or greater than minimum weight required (Step f).

2) If minimum weight required (Step f) is greater than weight of minimum size anchor block, increase dimensions of minimum size anchor block to provide weight equal to or greater than minimum required weight (Step f).

Typical counterweight details are shown in Figure 8-17. Dimensions for the weight of minimum size counterweight can be found in Appendix 8F-3.
Figure 8-17. Counterweight Details for Pipes Subject to Uplift Forces

*1/2-inch diameter steel rod to be field bent as necessary and embedded in fresh concrete
** Class A-3 concrete

Dimensions:
A=Variable as needed, 6” minimum
B=D/2+12”
C=Variable as needed, D+12” minimum
D=Pipe diameter
8.6 References


Due to the magnitude of the changes to this section, shading has been omitted.

Chapter 8-76 of 77


Policy Memorandum 3-78, "Flood Plain Management Program for State Agencies."


VDOT Road and Bridge Specifications - Latest Edition.

VDOT Survey Instructions Manual.
<table>
<thead>
<tr>
<th>HYDROLOGICAL DATA:</th>
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</thead>
<tbody>
<tr>
<td>D.A. = AC.</td>
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<table>
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<tbody>
<tr>
<td>Q etc.</td>
<td>ADT etc.</td>
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<tr>
<td>Q etc.</td>
<td>Detours Available</td>
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<td>Overtopping Stage</td>
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<td>Flood Plain Management</td>
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<td>Q etc.</td>
<td>Criteria and Significant Impact</td>
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<table>
<thead>
<tr>
<th>CULVERT TYPE &amp; SIZE</th>
<th>HEADWATER COMPUTATIONS</th>
</tr>
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<tbody>
<tr>
<td>Q etc.</td>
<td>Q/B</td>
</tr>
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<td></td>
<td>INLET CONTROL</td>
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<tr>
<td></td>
<td>CONT.</td>
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<tr>
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<td>OUTLET VELOCITY</td>
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<td>End Treat.</td>
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<td>COMMENTS</td>
</tr>
<tr>
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</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>SUMMARY &amp; RECOMMENDATIONS:</th>
</tr>
</thead>
</table>

|                     |                     |
| Base Flood 1% Exceed. Prob. | Elev. |
Appendix 8C-1  Inlet Control, Circular Concrete

Source: HDS -5
Appendix 8C-2  Inlet Control, Circular Corrugated Metal

CHART 2

EXAMPLE

\[ D = 36 \text{ inches} \times (3.0 \text{ feet}) \]
\[ Q = 66 \text{ cfs} \]

\[ \frac{H}{D} \quad \text{HW} \]

(1) \( 1.0 \quad 3.6 \)
(2) \( 2.1 \quad 5.3 \)
(3) \( 3.2 \quad 6.6 \)

*0 in feet

HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

Source:  HDS-5
Appendix 8C-4  Critical Depth, Circular

CHART 4

BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
CIRCULAR PIPE

Source:  HDS-5
Appendix 8C-5  Outlet Control, Circular Concrete

CHART 5

HEAD FOR CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

Source: HDS-5
Appendix 8C-6  Outlet Control,
Circular Corrugated Metal

Source: HDS-5
Appendix 8C-7  Outlet Control, Circular Structural Plate Corrugated Metal

For outlet crown not submerged, compute HW by methods described in the design procedure.

Source: HDS-5
Appendix 8C-8  Inlet Control, Concrete Box

Source: HDS-5
Appendix 8C-9
Inlet Control, Concrete Box,
Flared Wingwalls at 18° to 33.7° and 45°,
Beveled Top Edge

Source: HDS-5
Appendix 8C-10

Inlet Control, Concrete Box, 90° Headwall, Chamfered or Beveled Edges

Source: HDS-5
Appendix 8C-11
Inlet Control,
Single Barrel Concrete Box,
Skewed Headwalls Chamfered or Beveled Edges

CHART 11

HEADWATER DEPTH FOR INLET CONTROL
SINGLE BARREL BOX CULVERTS
SKewed HEADWALLS
CHAMFERED OR BEVELED INLET EDGES

FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

Source: HDS-5
Appendix 8C-12  Inlet Control, Concrete Box, Flared Wingwalls, Normal and Skewed Inlets, Chamfered Top Edge

Example

B = 7 FT, D = 5 FT, Q = 500 CFS
Q / B = 71.5

Inlet WW
H
W
Ft
Normal
13.9
45° WW
2.27
18.4° WW
Skewed 15° - 40°
18.4 or More
WW
2.20

Wingwall Flare
45°
18.4°

Headwater Depth in Terms of Height (Hw/D)

<table>
<thead>
<tr>
<th>Hw/D</th>
<th>Normal Inlets</th>
<th>30° Skew</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
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</tr>
<tr>
<td>4</td>
<td>4</td>
<td>4</td>
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<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

Height of Barrel (H) in Feet

Discharge per Foot of Barrel Width (WB) in CFS

Note:

Headwater scale for skewed inlets is constructed for 30° skew and 3.1 wingwall flare (18.4°). Also a good approximation for any skew angle from 15° to 45° and for greater flare angles of wingwalls.

Headwater depth for inlet control rectangular box Culverts Flared Wingwalls Normal and Skewed Inlets 3/4" Chamfer at Top of Opening

Source: HDS-5
Appendix 8C-13  Inlet Control, Concrete Box with Offset Flared Wingwalls, Beveled Top Edge

CHART 13

EXAMPLE
\[ B = 7 \text{ FT} \quad D = 5 \text{ FT} \quad Q = 600 \text{ CFS} \]
\[ \frac{B}{D} = 1.4 \]

WINGWALL TOP EDGE HW HW
FLARE ANGLE BEVEL "D" FT

<table>
<thead>
<tr>
<th>D</th>
<th>45°</th>
<th>33.7°</th>
<th>18.4°</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>2.06</td>
<td>1.90</td>
<td>1.82</td>
</tr>
<tr>
<td>11</td>
<td>2.06</td>
<td>1.90</td>
<td>1.82</td>
</tr>
<tr>
<td>10</td>
<td>2.06</td>
<td>1.90</td>
<td>1.82</td>
</tr>
<tr>
<td>9</td>
<td>2.06</td>
<td>1.90</td>
<td>1.82</td>
</tr>
<tr>
<td>8</td>
<td>2.06</td>
<td>1.90</td>
<td>1.82</td>
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<td>7</td>
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<tr>
<td>6</td>
<td>2.06</td>
<td>1.90</td>
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</tr>
<tr>
<td>5</td>
<td>2.06</td>
<td>1.90</td>
<td>1.82</td>
</tr>
<tr>
<td>4</td>
<td>2.06</td>
<td>1.90</td>
<td>1.82</td>
</tr>
<tr>
<td>3</td>
<td>2.06</td>
<td>1.90</td>
<td>1.82</td>
</tr>
</tbody>
</table>

TOP EDGE BEVEL ANGLE REQUIRED

ANGLE
0.042
45°
0.063
33.7°

HEIGHT IN TERMS OF
(FT/\(Q^2\))

1.5
3
6
12
24
48

HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
OFFSET FLARED WINGWALLS
AND BEVELED EDGE AT TOP OF INLET

WINGWALLS
FLARE ANGLE MIN OFFSET
1:1 45° 3/4" x B (FT)
1:1.5 33.7° 1" x B
1:2 26.6° 1-1/4" x B
1:3 18.4° 1-1/2" x B

* USE 33.7° x 0.0083 D TOP EDGE BEVEL AND READ HW ON SCALE FOR 18.4° WW

HEADWATER DEPTH IN TERMS OF
HEIGHT HW (FT)

1.0
0.9
0.8
0.7
0.6
0.5

BUREAU OF PUBLIC ROADS
OFFICE OF R9 D AUGUST 1968

Source: HDS-5
Appendix 8C-14  Critical Depth, Concrete Box

Source: HDS-5
For outlet crown not submerged, compute HW by methods described in the design procedure.
Appendix 8C-16
Inlet Control, Corrugated Metal Box, Rise/Span <0.3

Source: HDS-5
Appendix 8C-17
Inlet Control,
Corrugated Metal Box,
0.3 ≤ Rise/Span < 0.4

Source: HDS-5
Appendix 8C-18
Inlet Control, Corrugated Metal Box, 0.4<Rise/Span<0.5

Example:
D = 9.87 ft
Q = 1520 cfs

Entrance Type

H  W

HW

H  W

(2)
0.88
8.51

(3)
0.90
8.70

(5)
0.97
9.38

Source: HDS-5
Appendix 8C-19
Inlet Control, Corrugated Metal Box, 0.5\leq \text{Rise/Span}

CHART 19

<table>
<thead>
<tr>
<th>Entrance Conditions</th>
<th>(2) 90° headwell.</th>
<th>(3) Thick well projecting.</th>
<th>(5) Thin wall projecting.</th>
</tr>
</thead>
<tbody>
<tr>
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<td></td>
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</tbody>
</table>

Example:
D=8.0 ft
Q=1004 cfs

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<thead>
<tr>
<th>Entrance Type</th>
<th>HW</th>
<th>HW</th>
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<tr>
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<td></td>
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<tr>
<td>(2) 1.04</td>
<td>8.32</td>
<td></td>
</tr>
<tr>
<td>(3) 1.07</td>
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<td>(5) 1.15</td>
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</table>

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation

Source: HDS-5
Appendix 8C-20  Critical Depth, Corrugated Metal Box

**Example:**

RISE (D) = 6 ft 6 in
SPAN (B) = 22 ft 1 in
AREA (A) = 118.4 ft²
FLOW (Q) = 1050 ft³/s
RISE / SPAN = 6.5 / 22.06 = 0.29

\[
\frac{Q}{A D^{0.5}} = \frac{1050}{118.4 (6.5)^{0.5}} = 3.48
\]

\[
d_c = 0.65
\]

\[
d_c = 0.65 (6.5) = 4.1 \text{ ft}
\]
Appendix 8C-21  Outlet Control, Corrugated Metal Box, Concrete Bottom  Rise/Span <0.3

CHART 21

Area ($ft^2$)  $\bar{n}$
20 - 30  0.025
31 - 150  0.024

Source: HDS-3
Appendix 8C-22   Outlet Control, Corrugated Metal Box, Concrete Bottom
0.3 ≤ Rise/Span < 0.4

Source: HDS-5
Appendix 8C-23  Outlet Control, Corrugated Metal Box, Concrete Bottom, 0.4 ≤ Rise/Span < 0.5

Source: HDS-5
Appendix 8C-24
Outlet Control,
Corrugated Metal Box, Concrete Bottom
0.5 ≤ Rise/Span

Source: HDS-5
Appendix 8C-25  Outlet Control, Corrugated Metal Box, Corrugated Metal Bottom, Rise/Span <0.3

CHART 25

<table>
<thead>
<tr>
<th>Area (ft²)</th>
<th>k₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 - 28</td>
<td>0.035</td>
</tr>
<tr>
<td>29 - 63</td>
<td>0.034</td>
</tr>
<tr>
<td>64 - 150</td>
<td>0.033</td>
</tr>
</tbody>
</table>

Source: HDS-5
Appendix 8C-26  Outlet Control, Corrugated Metal Box,
Corrugated Metal Box,
0.3 ≤ Rise/Span ≤ 0.4

Source: HDS-5
Appendix 8C-27  Outlet Control, Corrugated Metal Box, Corrugated Metal Bottom, 0.4 ≤ Rise/Span < 0.5

CHART 27

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation

Duplication of this nomograph may distort scale

Source: HDS-5
Appendix 8C-28  Outlet Control, Corrugated Metal Box,
Corrugated Metal Bottom,
0.5 ≤ Rise/Span

Source: HDS-5
Appendix 8C-29

Inlet Control, Oval Concrete,
Long Axis Horizontal

EXAMPLE
Size 76" x 48" Q=300 cfs

| (1) | 2.8 | 11.2 |
| (2) | 2.2 | 6.6  |
| (3) | 2.3 | 9.2  |

* D in feet

To use scale (2) or (3) draw a straight line through known values of size and discharge to intersect scale (1). From point on scale (1) project horizontally to solution on either scale (2) or (3).

HEADWATER DEPTH IN TERMS OF RISE (HW/D)

HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL
WITH INLET CONTROL

Source: HDS-5
Appendix 8C-31  Critical Depth, Oval Concrete,  
Long Axis Horizontal

Source: HDS-5
Appendix 8C-32  Critical Depth, Oval Concrete, Long Axis Vertical

Source: HDS-5
Appendix 8C-33  Outlet Control, Oval Concrete, Long Axis Horizontal or Vertical

CHART 33

For outlet crown not submerged, compute HW by methods described in the design procedure.

SIZE = \frac{Q}{0.02} \times 100

NOTE
Dimensions on size scale are ordered for long axis horizontal installation. They should be reversed for long axis vertical.

HEAD FOR OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL OR VERTICAL
FLOWING FULL
n = 0.012

Source: HDS-5

BUREAU OF PUBLIC ROADS JAN, 1963

1 of 1 VDOT Drainage Manual
Appendix 8C-34  
Inlet Control,  
Corrugated Metal Pipe-Arch

CHART 34

**EXAMPLE**
Size: 36" x 22'
D = 20' 6"

<table>
<thead>
<tr>
<th>HW D (feet)</th>
<th>(1)</th>
<th>(2)</th>
<th>(3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.10</td>
<td>1</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>1.15</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.22</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*On in feet

**EXAMPLE**
Headwall
Mitered to conform to slope

**ENTRANCE TYPE**

<table>
<thead>
<tr>
<th>(1)</th>
<th>(2)</th>
<th>(3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>.6</td>
<td>.6</td>
<td>.6</td>
</tr>
<tr>
<td>.6</td>
<td>.6</td>
<td>.6</td>
</tr>
<tr>
<td>.5</td>
<td>.5</td>
<td>.5</td>
</tr>
<tr>
<td>.35</td>
<td>.35</td>
<td>.35</td>
</tr>
</tbody>
</table>

**HEADWATER DEPTH FOR C. M. PIPE-ARCH CULVERTS WITH INLET CONTROL**

*ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG*

Source: HDS-5
Chapter 8 - Culverts

Appendix 8C-35
Inlet Control,
Structural Plate Pipe-Arch,
18" Corner Radius

CHART 35

EXAMPLE
SIZE 12.9 x 8.3 Q=1000 CFS

PROJECT Headwall

NO BEVEL

Type of inlet
90° Headwall
33.7° x 0.100 Bevel
No Bevel
Projecting

HEADWATER DEPTH FOR INLET CONTROL
STRUCTURAL PLATE PIPE-ARCH CULVERTS
18-IN. RADIUS CORNER PLATE
PROJECTING OR HEADWALL INLET
HEADWALL WITH OR WITHOUT EDGE BEVEL

Source: HDS-5
Chapter 8 - Culverts

Appendix 8C-36
Inlet Control,
Structural Plate Pipe-Arch,
31" Corner Radius

CHART 36

EXAMPLE
SIZE 17'4" x 11'5" Q = 2500 CFS

PROJECT HEADWALL NO BEVEL BEVEL

HW FT 18.9 16.7 15.2

HEADWALL DEPTH IN TERMS OF ARCH HW/AD

DISCHARGE (CFS)

HEADWATER DEPTH FOR INLET CONTROL
STRUCTURAL PLATE PIPE-ARCH CULVERTS
31-IN. RADIUS CORNER PLATE
PROJECTING OR HEADWALL INLET
HEADWALL WITH OR WITHOUT EDGE BEVEL

Source: HDS-5
Appendix 8C-37  
Critical Depth,  
Standard Corrugated Metal Pipe-Arch  

CHART 37

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JAN. 1964

CRITICAL DEPTH  
STANDARD C.M. PIPE-ARCH

Source: HDS-5
Chapter 8 - Culverts

Appendix 8C-38 Critical Depth, Structural Plate
Corrugated Metal Pipe-Arch,
18" Corner Radius

CHART 38

![Chart 38: Critical Depth vs Discharge for Structural Plate Corrugated Metal Pipe-Arch with 18" Corner Radius](chart.png)

Source: HDS-5
Appendix 8C-39  Outlet Control, Standard Corrugated Metal Pipe-Arch

CHART 39

For outlet crown not submerged, compute HW by methods described in the design procedure.

HEAD FOR STANDARD C.M. PIPE-ARCH CULVERTS FLOWING FULL
n=0.024

Source: HDS-5
Appendix 8C-40  
Outlet Control,  
Structural Plate Corrugated Metal  
Pipe-Arch, 18” Corner Radius

CHART 40

HEAD FOR  
STRUCTURAL PLATE  
CORRUGATED METAL  
PIPE ARCH CULVERTS  
18 IN. CORNER RADIUS  
FLOWING FULL  
\( n = 0.0327 \) TO 0.0306

BUREAU OF PUBLIC ROADS JAN. 1963

Source: HDS-5
Appendix 8C-41  Inlet Control, Corrugated Metal Arch,
0.3 ≤ Rise/Span < 0.4

CHART 41

Entrance Conditions
(2) 90° headwall,
(4) Mitered to embankment,
(5) Thin wall projecting corrugated metal.

Example
A = 122.2 ft
Q = 1014 cfs

<table>
<thead>
<tr>
<th>Entrance Type</th>
<th>HW</th>
<th>D</th>
<th>HW</th>
<th>Lf</th>
</tr>
</thead>
<tbody>
<tr>
<td>(2)</td>
<td>0.93</td>
<td>7.37</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(4)</td>
<td>0.95</td>
<td>7.52</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(5)</td>
<td>1.03</td>
<td>8.16</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Discharge (Q) in cfs

Arch Area in square feet

Headwater Depth to Rise (Hw / D)

Source: HDS-5
Appendix 8C-42  Inlet Control, Corrugated Metal Arch,  
0.4 ≤ Rise/Span < 0.5

CHART 42

Entrance Conditions
(2) 90° headwall.
(4) Mitered to embankment.
(5) Thin wall projecting corrugated metal.

Discharge (Q) in cfs

Entrance Type HW D HW IRI
(2) 2.03 26.74
(4) 2.40 31.64
(5) 2.33 30.69

Headwater Depth to Rise (H/W / D)

EXAMPLE
A = 2775 ft²
Q = 6000 cfs

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation

Headwater Depth for C. M. Arch Culverts
0.4 ≤ RISE / SPAN < 0.5
With Inlet Control

Source: HDS-5

 Duplication of this nomograph may distort scale.
Appendix 8C-43 Inlet Control, Corrugated Metal Arch, 0.5 ≤ Rise/Span

CHART 43

Entrance Conditions
(2) 90° headwall.
(4) Mitered to embankment.
(5) Thin wall projecting corrugated metal.

Example
A = 1.05 ft²
Q = 1400 cfs

Discharge (Q) in cfs

Arch Area in square feet

Headwater Depth to Rise (H /W / D)

Entrance Type

Element

1.50
1.75
1.63

HW
D
W

12.38
14.44
13.45

HEADWATER DEPTH
FOR C.M. ARCH CULVERTS
0.5 ≤ RISE/Span
WITH INLET CONTROL

Source: HDS-5
Appendix 8C-44  Critical Depth, Corrugated Metal Arch

Example:
RISE (D) = 5 ft 9 in
SPAN (B) = 16 ft
AREA (A) = 66.8 ft²
FLOW (Q) = 400 ft³/s
RISE/SPAN = 5.75/16 = 0.36
Q/AD⁰.⁵ = 400/(66.8)(5.75)⁰.⁵
2.5
\[ \frac{D_c}{D} = 0.47 \]
\[ D_c = (0.47)(5.75) = 2.7 \text{ ft} \]

Source: HDS-5
Appendix 8C-45  Outlet Control, Corrugated Metal Arch, Concrete Bottom, 0.3 ≤ Rise/Span < 0.4

CHART 45

Discharge (q) in cfs.

Head (h) in feet

Area of Culvert

Discharge (q) in cfs.

Area (ft²)

<table>
<thead>
<tr>
<th>Area (ft²)</th>
<th>h</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 - 60</td>
<td>0.025</td>
</tr>
<tr>
<td>61 - 155</td>
<td>0.024</td>
</tr>
<tr>
<td>156 - 260</td>
<td>0.023</td>
</tr>
</tbody>
</table>

Example:
Q = 600 cfs, H = 100 ft

HEAD FOR C.M. ARCH CULVERTS
FLOWING FULL
CONCRETE BOTTOM
0.3 ≤ RISE / SPAN < 0.4

Source: HDS-5
Appendix 8C-46  Outlet Control, Corrugated Metal Arch, Concrete Bottom, 0.4 ≤ Rise/Span < 0.5

CHART 46

Discharge (Q) in cfs.  Area of Culvert

5000  4000  3000  2000  1500  1000  800  700  600  500  400  300  200  100

Rise

Head (H) in feet

SPAN

Length (L) in feet

Ke

0.35  0.50  0.80

50  100  200  300  400  500

Area [ft²]  H

20 - 150  0.025
151 - 360  0.024

Example

HEAD FOR C.M. ARCH CULVERTS
FLOWING FULL CONCRETE BOTTOM
0.4 ≤ RISE/SPAN < 0.5

Source: HDS-5
Appendix 8C-47  Outlet Control, Corrugated Metal Arch, Concrete Bottom,
0.5 ≤ Rise/Span

CHART 47

HEAD FOR C.M. ARCH CULVERTS
FLOWING FULL
CONCRETE BOTTOM
0.5 ≤ RISE / SPAN

Source: HDS-5
Appendix 8C-48  Outlet Control, Corrugated Metal Arch, Earth Bottom, 0.3 ≤ Rise/Span < 0.4

Source: HDS-5
Appendix 8C-49  Outlet Control, Corrugated Metal Arch,
Earth Bottom,
0.4≤ Rise/Span <0.5

Chart 49

Area of Culvert

Discharge (Q) in cfs.

5000
4000
3000
2000
1000
100
50
25
100
360
340
320
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
10
0

500
400
300
200
100
0

Length (L) in feet

0.25
0.50
0.75
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
7.0
7.5
8.0
8.5
9.0
9.5
10.0

Head (H) in feet

Area (ft²)

20 - 90
91 - 360

H

 Turning Line

Nomographs adapted from material furnished by
Kaiser Aluminum and Chemical Corporation

Source:
HDS-5
Appendix 8C-50  Outlet Control, Corrugated Metal Arch,
Earth Bottom,
0.5 ≤ Rise/Span

CHART 50

Discharge (Q) in cfs.

Area of Culvert

5000
4000
3000
2000
1000
Area (ft²)

200
100

20
10

50
40

Length (L) in feet

50
100
200
300
400
500

(Note: Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may entail severe penalties.)

Source: HDS-5
Appendix 8C-52  Inlet Control, Structural Plate Corrugated Metal Arch, High and Low Profile

Source: HDS-5
Appendix 8C-53  Critical Depth, Structural Plate Ellipse, Long Axis Horizontal

**CHART 53**

**EXAMPLE:**
Rise (D) = 13 ft
Span (B) = 20 ft 1 in
Area (A) = 201.8 ft²
Flow (Q) = 2100 ft³/s

\[
Q/AD^{0.5} = 2100/(201.8)(13)^{0.5} = 2.9
\]

\[
d_c/D = 0.65
\]

\[
d_c = (0.65)(13) = 8.5 ft
\]

Source: HDS-5
Appendix 8C-54  Critical Depth, Structural Plate Arch, Low and High Profile

Source: HDS-5
Chart 8C-55  Throat Control, Circular Section, Side-Tapered

Source: HDS-5
Chapter 8 - Culverts

Chart 8C-56  Face Control, Non-Rectangular Section, Side-Tapered to Circular

CHART 56

Scale

- Entrance Type
  (1) Beveled Edge
  (2) Square Edge
  (3) Thin Edge Projecting
      0.0428 or 0.0838

Example

FACE SECTION

THROAT SECTION

ELEVATION

TAPER

PLAN

TAPER MAY VARY
FROM 4:1 TO 6:1

Example

E = 72 INCHES (6.0 FEET)
Q = 600 CFS

Inlet

Type

HW
Q
Bf
in
feet

(1) 20
66
91

(2) 2.26
66
91

(3) 0.65
66
91

HEADWATER DEPTH AT FACE IN TERMS OF BARREL HEIGHT (HW/E) IN FT PER FT

RATIO OF DISCHARGE TO WIDTH OF FACE (Q/Bf) IN CF/FT

HEIGHT OF FACE (E) IN INCHES

Example

FACE CONTROL FOR SIDE-TAPERED INLETS TO PIPE CULVERTS
(NON-RECTANGULAR SECTIONS ONLY)

Source: HDS-5
Chart 8C-57  Throat Control, Box Section, Tapered Inlet

THROAT CONTROL FOR BOX CULVERTS WITH TAPERED INLETS

Source: HDS-5
Chapter 8 - Culverts

Appendix 8C-58  Face Control, Box Section, Side-Tapered

CHART 58

SCALE  ENTRANCE  TYPE

(1)  15° TO 25°  WINDWALL FLARES
     WITH TOP EDGE BEVELED
     OR
     25° TO 30°  WINDWALL FLARES
     WITH NO BEVELS (SQUARE EDGES)

(2)  25° TO 45°  WINDWALL FLARES
     WITH TOP EDGE BEVELED
     OR
     45° TO 90°  WINDWALL FLARES
     WITH BEVELS ON TOP AND SIDES

EXAMPLE

D = 8 FEET  Q = 1200 CFM

INLET  HW  Cx  B1

TYPE    (FEET)

(1)  19  109  10
(2)  169  109  10

RATIO OF DISCHARGE TO WIDTH OF THE FACE (Q/B) IN CFM OF WATER

HEIGHT OF BOX (H) IN FEET

HEADWATER DEPTH AT THE FACE IN TERMS OF HEIGHT (HW/HD) IN FT PER FT

SOURCE: HDS-5
Appendix 8C-59 Face Control, Box Section, Slope-Tapered

CHART 59

Scale
Entrance Type
(1) 15° to 26° wing wall flares with top edge beveled or 26° to 90° wingwall flares with no bevels
(2) 26° to 45° wing wall flares with top edge beveled or 45° to 90° wing wall flares with bevels on top and sides

Vertical Face

Mitered Face

Face control for box culverts with slope tapered inlets

Source: HDS-5
Chart 8C-60  Discharge Coefficients for Roadway Overtopping

Source: HDS-5
Appendix 8C-61  Circular Pipe Flow Chart (Diameter = 12"")

PIPE FLOW CHART
12-INCH DIAMETER

Source: HDS-3
Appendix 8C-62  Circular Pipe Flow Chart (Diameter = 15"")

Source: HDS-3
Appendix 8C-63  Circular Pipe Flow Chart (Diameter = 18")

Source: HDS-3
Appendix 8C-64  Circular Pipe Flow Chart (Diameter = 21")

Source: HDS-3
Appendix 8C-65  Circular Pipe Flow Chart (Diameter = 24")

Source: HDS-3
Appendix 8C-66  Circular Pipe Flow Chart (Diameter = 27"

Source: HDS-3
Appendix 8C-67  Circular Pipe Flow Chart (Diameter = 30"")

Source: HDS-3
Appendix 8C-72  Circular Pipe Flow Chart (Diameter = 54")

Source: HDS-3
Appendix 8C-73  Circular Pipe Flow Chart (Diameter = 60")

Source: HDS-3
Appendix 8C-74  Circular Pipe Flow Chart (Diameter = 66")

Source: HDS-3
Appendix 8C-75  Circular Pipe Flow Chart (Diameter = 72")

Source: HDS-3
Appendix 8C-77  Circular Pipe Flow Chart (Diameter = 96")

Source: HDS-3
Appendix 8C-78  Rectangular Channel Flow Chart (B=2')

Source:  HDS-3
Appendix 8C-80  Rectangular Channel Flow Chart (B=4')

Source: HDS-3
Appendix 8C-81  Rectangular Channel Flow Chart (B=5')

Source: HDS-3
Appendix 8C-82  Rectangular Channel Flow Chart (B=6')

Source: HDS-3
Appendix 8C-83  Rectangular Channel Flow Chart (B=7')

Source: HDS-3
Appendix 8C-84  Rectangular Channel Flow Chart (B=8')

Source: HDS-3
Appendix 8C-85 Rectangular Channel Flow Chart (B=9')

Source: HDS-3
Appendix 8C-86  Rectangular Channel Flow Chart (B=10'')

Source: HDS-3
Appendix 8C-87  Rectangular Channel Flow Chart (B=12”)

Source: HDS-3
Appendix 8C-88  Rectangular Channel Flow Chart (B=14")

Source: HDS-3
Appendix 8C-89  Rectangular Channel Flow Chart (B=16'')

Source: HDS-3
Appendix 8C-90  Rectangular Channel Flow Chart (B=18")

Source: HDS-3
Appendix 8C-91 Rectangular Channel Flow Chart (B=20')

Source: HDS-3
### Appendix 8D-1  Recommended Manning’s n-Values

<table>
<thead>
<tr>
<th>Type of Conduit</th>
<th>Wall Description</th>
<th>Manning’s n</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Pipe</td>
<td>Smooth walls</td>
<td>0.010-0.013</td>
</tr>
<tr>
<td>Concrete Boxes</td>
<td>Smooth walls</td>
<td>0.012-0.015</td>
</tr>
<tr>
<td>Corrugated Metal</td>
<td>2 2/3 by 1/2 inch corrugations</td>
<td>0.022-0.027</td>
</tr>
<tr>
<td>Pipes and Boxes, Annular or Helical Pipe (n varies Barrel size) See HDS5</td>
<td>6 by 1 inch corrugations</td>
<td>0.022-0.025</td>
</tr>
<tr>
<td></td>
<td>5 by 1 inch corrugations</td>
<td>0.025-0.026</td>
</tr>
<tr>
<td></td>
<td>3 by 1 inch corrugations</td>
<td>0.027-0.028</td>
</tr>
<tr>
<td></td>
<td>6 by 2 inch structural plate</td>
<td>0.033-0.035</td>
</tr>
<tr>
<td></td>
<td>9 by 2 1/2 inch structural plate</td>
<td>0.033-0.037</td>
</tr>
<tr>
<td>Corrugated Metal</td>
<td>2 2/3 by 1/2 inch corrugations</td>
<td>0.012-0.024</td>
</tr>
<tr>
<td>Pipes, Helical Corrugations, Full Circular Flow</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spiral Rib Metal</td>
<td>Smooth walls</td>
<td>0.011-0.012</td>
</tr>
</tbody>
</table>

*Note 1:* The Values indicated in this table are recommended Manning’s “n” design values. Actual Field values for older existing pipelines may vary depending on the effects of abrasion, corrosion, deflection and joint conditions. Concrete pipe with poor joints and deteriorated walls may have “n” values of 0.014 to 0.018. Corrugated metal pipe with joint and wall problems may also have higher “n” values, and in addition, may experience shape changes which could adversely effect the general hydraulic characteristics of the culvert.

*Note 2:* For further information concerning Manning n values for selected conduits consult Hydraulic Design of Highway Culverts, Federal Highway Administration, HDS No. 5, Table 4.

Source: HDS-5
## Appendix 8D-2 Entrance Loss Coefficients ($K_e$), Outlet Control, Full or Partly Full

<table>
<thead>
<tr>
<th>Type of Structure and Design of Entrance</th>
<th>Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pipe, Concrete</strong></td>
<td></td>
</tr>
<tr>
<td>Mitered to conform to fill slope</td>
<td>0.7</td>
</tr>
<tr>
<td>*End-Section conforming to fill slope</td>
<td>0.5</td>
</tr>
<tr>
<td>Projecting from fill, sq. cut end</td>
<td>0.5</td>
</tr>
<tr>
<td>Headwall or headwall and wingwalls</td>
<td></td>
</tr>
<tr>
<td>Square-edge</td>
<td>0.5</td>
</tr>
<tr>
<td>Rounded (radius = D/12)</td>
<td>0.2</td>
</tr>
<tr>
<td>Socket end of pipe (groove-end)</td>
<td>0.2</td>
</tr>
<tr>
<td>Projecting from fill, socket end (groove-end)</td>
<td>0.2</td>
</tr>
<tr>
<td>Beveled edges, 33.7° or 45° bevels</td>
<td>0.2</td>
</tr>
<tr>
<td>Side-or slope-tapered inlet</td>
<td>0.2</td>
</tr>
<tr>
<td><strong>Pipe, or Pipe-Arch, Corrugated Metal</strong></td>
<td></td>
</tr>
<tr>
<td>Projecting from fill (no headwall)</td>
<td>0.9</td>
</tr>
<tr>
<td>Mitered to conform to fill slope, paved or unpaved slope</td>
<td>0.7</td>
</tr>
<tr>
<td>Headwall or headwall and wingwalls square-edge</td>
<td>0.5</td>
</tr>
<tr>
<td>*End-Section conforming to fill slope</td>
<td>0.5</td>
</tr>
<tr>
<td>Beveled edges, 33.7° or 45° bevels</td>
<td>0.2</td>
</tr>
<tr>
<td>Side-or slope-tapered inlet</td>
<td>0.2</td>
</tr>
<tr>
<td><strong>Box, Reinforced Concrete</strong></td>
<td></td>
</tr>
<tr>
<td>Headwall parallel to embankment (no wingwalls)</td>
<td></td>
</tr>
<tr>
<td>Square-edged on 3 edges</td>
<td>0.5</td>
</tr>
<tr>
<td>Rounded on 3 edges to radius of D/12 or B/12</td>
<td></td>
</tr>
<tr>
<td>or beveled edges on 3 sides</td>
<td>0.2</td>
</tr>
<tr>
<td>Wingwalls parallel (extension of sides)</td>
<td></td>
</tr>
<tr>
<td>Square-edged at crown</td>
<td>0.7</td>
</tr>
<tr>
<td>Wingwalls at 10° to 25° to barrel</td>
<td></td>
</tr>
<tr>
<td>Square-edged at crown</td>
<td>0.5</td>
</tr>
<tr>
<td>Wingwalls at 30° to 75° to barrel</td>
<td></td>
</tr>
<tr>
<td>Crown edge rounded to radius of D/12 or beveled top edge</td>
<td>0.2</td>
</tr>
<tr>
<td>Square Edge at crown</td>
<td>0.4</td>
</tr>
<tr>
<td>Side-or slope-tapered inlet</td>
<td>0.2</td>
</tr>
</tbody>
</table>

*Note : “End Sections conforming to fill slope,” made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

Source HDS-5
8E.1 Riprap Basin

Riprap basins are used for energy dissipation at the outlets of high velocity culverts.

Riprap basin design is based on laboratory data obtained from full-scale prototypical installations. The principal features of riprap basins are as follows:

1. Pre-shaping and lining with riprap of median size, $d_{50}$.
2. Constructing the floor at a depth of $h_s$ below the invert, where $h_s$ is the depth of scour that would occur in a pad of riprap of size $d_{50}$.
3. Sizing $d_{50}$ so that $2 < h_s/d_{50} < 4$.
4. Sizing the length of the dissipating pool to be $10(h_s)$ or $3(W_o)$, whichever is larger for a single barrel. The overall length of the basin is $15(h_s)$ or $4W_o$ whichever is larger.
5. Angular rock results are approximately the same as the results of rounded material.
6. Layout details and dimensions are shown on Figure 8E-1.

For high tailwater ($\frac{TW}{d_o} > 0.75$), the following applies:

1. The high velocity core of water emerging from the culvert retains its jet-like character as it passes through the basin.
2. The scour hole is not as deep as with low tailwater and is generally longer.
3. Riprap may be required for the channel downstream of the rock-lined basin.
8E.2 Design Procedures and Sample Problems

The procedure shown below should be used to determine the dimension for a riprap basin energy dissipator for culvert and pipe installations with pipe velocities greater than or equal to 19 feet per second as classified in Section 8.3.2.6. Maximum Outlet Velocity within the Chapter 8 text.

**Step 1:** Determine input flow parameters: $D_e$ or $d_E$, $V_o$, $F_r$ at the culvert outlet

Where:

$d_E = \text{Equivalent depth at the brink} = \sqrt{\frac{A}{2}}$

Note: $d_E = y_e$ in Figure 8E-2

**Step 2:** Check $TW$

Determine if $\frac{TW}{d_o} \leq 0.75$

Note: $d_o = d_E$ in Figure 8E-2 for rectangular sections

**Step 3** Determine $d_{50}$

a. Use Figure 8E-2.
b. Select $d_{50}/d_E$. Satisfactory results will be obtained if $0.25 < d_{50}/d_E < 0.45$.
c. Obtain $h_s/d_E$ using Froude number ($F_r$) and Figure 8E-2.
d. Check if $2 < h_s/d_{50} < 4$ and repeat until a $d_{50}$ is found within the range.

**Step 4:** Size basin

a. As shown in Figure 8E-1.
b. Determine length of the dissipating pool, $L_s = 10h_s$ or $3W_o$ minimum.
c. Determine length of basin, $L_B = 15h_s$ or $4W_o$ minimum.

Thickness of riprap: Approach = $3d_{50}$ or $1.5d_{\text{max}}$

Remainder = $2W_o$ or $1.5d_{\text{max}}$
Appendix 8E-1  Energy Dissipation

Step 5: Determine exit velocity at brink ($V_b$)

a. Basin exit depth, $d_B = \text{critical depth at basin exit}$

b. Basin exit velocity, $V_B = \frac{Q}{W_b d_B}$

c. Compare $V_B$ with the average normal flow velocity in the natural channel ($V_d$)

Step 6: High tailwater design

a. Design a basin for low tailwater conditions, Steps 1-5.

b. Compute equivalent circular diameter ($D_E$) for brink area from:
   \[ A = \frac{\pi D_E^2}{4} = d_0 (W_0) \]

   c. Estimate centerline velocity at a series of downstream cross sections using Figure 8E-4.

   Size riprap using HEC -11 "Use of Riprap for Bank Protection."¹

Step 7: Design Filter

The design filter is necessary unless the streambed material is sufficiently well graded. To design a filter for riprap, use the procedures in Section 4.4 of HEC-11.

Dissipator geometry can also be computed using the "Energy Dissipator" module that is available in the microcomputer program HY8, Culvert Analysis.
Figure 8E-1. Details of Riprap Basin Energy Dissipator

NOTE A - IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS-SECTIONAL AREA AT SECTION A-A SUCH THAT \( Q_{exit}/(\text{CROSS SECTION AREA AT SEC. A-A}) = \) SPECIFIED EXIT VELOCITY.

NOTE B - WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A-A.
Figure 8E-2. Riprap Basin Depth of Scour

NOTE: $2 \leq h_s/d_{50} \leq 4$

IF $TW/Y_0 > 0.75$

RIPRAP MAY BE REQUIRED ON BANKS AND CHANNEL BOTTOM DOWNSTREAM FROM BASIN. SEE DESIGN EXAMPLE IN TEXT.
### Riprap Basin Design Checklist

<table>
<thead>
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<th>DESIGN VALUES (Figure 8E-2)</th>
<th>TRIAL 1</th>
<th>FINAL TRIAL</th>
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<tr>
<td>Equivalent Depth, $d_e$</td>
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<td>$d_e$</td>
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<td>$h_f/d_e$</td>
<td>$h_f/d_e$</td>
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<td>$h_f/D_w$</td>
<td>$h_f/D_w$</td>
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<td>$2 &lt; h_f/D_w &lt; 4$</td>
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#### Basin Dimensions

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<td>Pool length is the larger of:</td>
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<tr>
<td></td>
<td>3$W_s$</td>
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<td>Basin length is the larger of:</td>
<td>15$b_s$</td>
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<td>4$W_s$</td>
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<td>Approach Thickness</td>
<td>3$D_{90}$</td>
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<tr>
<td>Basin Thickness</td>
<td>2$D_{90}$</td>
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#### Tailwater Check

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<tr>
<th>Tailwater, $TW$</th>
<th>Equivalent depth, $d_e$</th>
<th>TW/$d_e$</th>
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<td>$TW/d_e &gt; 0.75$</td>
<td>$d_e$</td>
<td>$TW/d_e$</td>
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</table>

$D_e = (4A_e/w)^{1/3}$

#### Downstream Riprap (Figure 8E-4)

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<th>L/$D_e$</th>
<th>$L$</th>
<th>$V_L/V_s$</th>
<th>$V_L$</th>
<th>$D_{90}$</th>
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</table>

Figure 8E-3. Riprap Basin Design Checklist
8E.2.1 Riprap Design for Low Tailwater Condition-Sample Problem

Given: Box culvert: 8.0 ft by 6.0 ft.
Design discharge $Q = 800$ cfs
Supercritical flow in culvert
Normal flow depth $d_o = brink depth d_E = 4.0$ ft
Tailwater depth, $TW = 2.8$ ft
Downstream channel velocity = 18 fps

Step 1: Determine input flow parameters: $D_o$ or $d_E$, $V_o$, $F_r$ at the culvert outlet

- $d_o = d_E$ for rectangular section
- $d_o = d_E = 4.0$ ft.
- $V_o = \frac{Q}{A} = \frac{800}{4.0(8.0)} = 25$ fps
- $F_r = \frac{V_o}{\sqrt{gd_E}} = \frac{25}{\sqrt{32.2(4.0)}} = 2.2 < 3.0$

Step 2: Check $TW$:

Determine if $\frac{TW}{d_E} < 0.75$

$\frac{2.8}{4.0} = 0.70 < 0.75$

Therefore, $\frac{TW}{d_E} < 0.75$, O.K.

Step 3: Determine $d_{50}$:

a. Use Figure 8E-2

b. Try $d_{50}/d_E = 0.45$

- $d_{50} = \left(\frac{d_{50}}{d_E}\right) d_E = 0.45(4.0) = 1.8$ ft.

c. Obtain $h_S/d_E$ using $F_r = 2.2$ and line $0.41 \leq d_{50}/d_E \leq 0.50$

- $h_S/d_E = 1.6$
d. Check if $2 < \frac{h_s}{d_{50}} < 4$:

$$h_s = \left( \frac{h_s}{d_E} \right) d_E = 1.6(4.0) = 6.4 \text{ ft.}$$

$$\frac{h_s}{d_{50}} = \frac{6.4}{1.8} = 3.55 \text{ ft.}$$

$2 < 3.55 < 4$, O.K.

Step 4: Size the basin:

a. As shown in Figure 8E-1
b. Determine length of dissipating pool, $L_S$:
   $$L_S = 10h_s = 10(6.4) = 64 \text{ ft.}$$
   $$L_S \text{ min.} = 3W_0 = 3(8) = 24 \text{ ft}$$
   Therefore, use $L_S = 64 \text{ ft}$

c. Determine length of basin, $L_B$:
   $$L_B = 15h_s = 15(6.4) = 96 \text{ ft}$$
   $$L_B \text{ min.} = 4W_0 = 4(8) = 32 \text{ ft}$$
   Therefore, use $L_B = 96 \text{ ft}$

d. Thickness of riprap:
   Approach = $3d_{50} = 3(1.80) = 5.4 \text{ ft}$
   Remainder = $2d_{50} = 2(1.80) = 3.6 \text{ ft}$

Step 5: Determine $V_B$:

a. $d_B$ = Critical depth at basin exit = 3.30 ft. (assuming a rectangular cross section with width $W_B = 24 \text{ ft}$.)
   $$Q = \frac{800}{W_B d_B} = 10 \text{ fps}$$

b. $V_B = \frac{800}{W_B d_B} = 10 \text{ fps}$

c. $V_B = 10 \text{ fps} < V_d = 18 \text{ fps}$
8E.2.2 Riprap Design for High Tailwater Condition-Sample Problem

Given: Data on the channel and the culvert are the same as Sample Problem 1, except that the new tailwater depth,

\[
TW = 4.2 \text{ ft.}
\]

\[
\frac{TW}{d_o} = \frac{4.2}{4.0} = 1.05 > 0.75
\]

Downstream channel can tolerate only 7.0 fps

Steps 1 through 5 are the same as Sample Problem 8E.2.1.

Step 6: High tailwater design:

a. Design a basin for low tailwater conditions, Steps 1-5 as above:
   \[D_{50} = 1.8 \text{ ft}, \ h_S = 6.4 \text{ ft}\]
   \[L_S = 64 \text{ ft}, \ L_B = 96 \text{ ft}\]

b. Compute equivalent circular diameter, \(D_E\), for brink area from:
   \[A = \frac{\pi D_E^2}{4} = d_o(W_0) = 4.0(8.0) = 32 \text{ ft}^2\]
   \[D_E = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4(32)}{\pi}} = 6.4 \text{ ft.}\]
   \[V_0 = 25 \text{ fps (Sample Problem 8E.2.1).}\]

c. Estimate centerline velocity at a series of downstream cross sections using Figure 8E-5.

<table>
<thead>
<tr>
<th>(\frac{L}{D_E})</th>
<th>L</th>
<th>(\frac{V_L}{V_0})</th>
<th>(V_L)</th>
<th>(D_{50})</th>
</tr>
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<tbody>
<tr>
<td>10</td>
<td>64</td>
<td>0.59</td>
<td>14.7</td>
<td>1.4</td>
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<td>96</td>
<td>0.36</td>
<td>9.0</td>
<td>0.6</td>
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<td>20</td>
<td>128</td>
<td>0.30</td>
<td>7.5</td>
<td>0.4</td>
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<tr>
<td>21</td>
<td>135</td>
<td>0.28</td>
<td>7.0</td>
<td>0.4</td>
</tr>
</tbody>
</table>

\(^1\) Use \(W_0 = D_E\) in Figure 8E-5.
\(^2\) From Figure 8E-6.
\(^3\) Is on a logarithmic scale so interpolations must be performed logarithmically.

d. Size riprap using HEC 11. The channel can be lined with the same size rock used for the basin. Protection should extend at least 135 ft downstream.

This information is summarized in the worksheet for riprap basin design, Figure 8E-4.
Figure 8E-4. Riprap Basin Design Worksheet, Sample Problem
Figure 8E-5. Distribution of Centerline Velocity for Flow from Submerged Outlets
Figure 8E-6. Riprap Size Versus Exit Velocity
8E.2.3 Computer Output

The dissipator geometry can be computed using the “Energy Dissipator” module, which is available in FHWA’s HY8, Culvert Analysis microcomputer program. The output of the culvert data, channel input data, and computed geometry using this module are shown below.

<table>
<thead>
<tr>
<th>FHWA CULVERT ANALYSIS, HY-8, VERSION 6.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>CURRENT DATE</td>
</tr>
<tr>
<td>06-02-1997</td>
</tr>
</tbody>
</table>

### CULVERT AND CHANNEL DATA

- **CULVERT NO. 1**
  - **CULVERT TYPE:** 8.0 ft X 6.0 ft, BOX
  - **CULVERT LENGTH:** 300 ft
  - **NO. OF BARRELS:** 1.0
  - **FLOW PER BARREL:** 400 cfs
  - **INVERT ELEVATION:** 172.5 ft
  - **OUTLET VELOCITY:** 25 fps

- **DOWNSTREAM CHANNEL**
  - **CHANNEL TYPE:** IRREGULAR
  - **BOTTOM WIDTH:** 8.0 ft
  - **TAILWATER DEPTH:** 2.8 ft
  - **TOTAL DESIGN FLOW:** 400 cfs
  - **BOTTOM ELEVATION:** 172.5 ft
  - **NORMAL VELOCITY:** 32 fps

- **OUTLET DEPTH:** 4.0 ft

### RIPRAP STILLING BASIN – FINAL DESIGN

- **THE LENGTH OF THE BASIN** = 96.3 ft
- **THE LENGTH OF THE POOL** = 64.2 ft
- **THE LENGTH OF THE APRON** = 32 ft
- **THE WIDTH OF THE BASIN AT THE OUTLET** = 8.0 ft
- **THE DEPTH OF POOL BELOW CULVERT INVERT** = 6.4 ft
- **THE THICKNESS OF THE RIPRAP ON THE APRON** = 6.6 ft
- **THE THICKNESS OF THE RIPRAP ON THE REST OF THE BASIN** = 5.0 ft
- **THE BASIN OUTLET VELOCITY** = 17 fps
- **THE DEPTH OF FLOW AT BASIN OUTLET** = 6.0 ft
Appendix 8F-1  Handling Weight for Corrugated Steel Pipe
(2$\frac{3}{4}$"x$\frac{1}{2}$" Corrugations)

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<th>Inside Diameter</th>
<th>Specified Thickness</th>
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* Lock seam construction only; weights will vary with other fabrication practices.

** For other coatings or linings the weights may be interpolated.

Note: Pipe arch weights will be the same as the equivalent round pipe.
For example: for 42 x 29, 2$\frac{3}{4}$ x $\frac{1}{2}$ in Pipe Arch, refer to 36 in diameter pipe weight.
Smooth steel lined CSP weighs approximately 5% more than single wall galvanized.

Source:
## Appendix 8F-2  Handling Weight for Corrugated Steel Pipe
(3"x1" or 125 mm x 25 mm Corrugations)

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<th>Specified Thickness in Inches</th>
<th>Approximate Pounds per Lineal Foot**</th>
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* Lack seam construction only; weights will vary with other fabrication practices.

** For other coatings or linings, the weights may be interpolated.

*** 125 x 25mm may be referred to as 5 x 1 in.
and weights approximately 12% less than 3 x 1 in.

Note: Pipe arch weights will be the same as the equivalent round pipe.
For example, for 42 x 20, 27 x 1 in. Pipe Arch, refer to 36 in. diameter pipe weight.
Smooth steel lined CSP weights approximately 5% more than single wall galvanized.

Source:
Appendix 8F-3  Dimension and Weight of Minimum Size Counterweight

**DIMENSIONS AND WEIGHT OF MINIMUM SIZE COUNTERWEIGHT**

\[ A = 6'' \]
\[ B = D / 2 + 12'' \]
\[ C = D + 12'' \]
\[ D = \text{PIPE DIAMETER} \]

*WEIGHT OF CONCRETE @ 150 LBS. PER CU. FT.*

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<tr>
<td>15</td>
<td>6 19.5 27</td>
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**Source:**
### Diameter Dimensions and $D^{2.5}$ Values for Structural Plate Corrugated Circular Pipe
(9" x 2 1/2" Aluminum Corrugations)

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Source: [Link to Source]
### Geometric Properties and Critical Flow Factors for Circular Conduits Flowing Full and Partly Full

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*Source: VDOT Drainage Manual*
## Appendix 8F-6

**Velocity Head and Resistance Computations Factors**
for Circular Conduits Flowing Full and Partly Full

### Table 3. -- Velocity head and resistance computation factors for circular conduits flowing full and partly full

<table>
<thead>
<tr>
<th>(A) Relative depth $d/D$</th>
<th>(B) Relative velocity head $aV^2/2gD$</th>
<th>(C) Manning Eq. resistance computation factor $K_r$</th>
<th>(D) Darcy Eq. resistance computation factor $K_r$</th>
<th>(A) Relative depth $d/D$</th>
<th>(B) Relative velocity head $aV^2/2gD$</th>
<th>(C) Manning Eq. resistance computation factor $K_r$</th>
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Source:
DEPARTMENT OF TRANSPORTATION
LOCATION AND DESIGN
HYDRAULIC COMMENTARY FOR ENVIRONMENTAL PERMIT FOR CULVERTS

LOCATION
Project:
Route:
PPMS:
Station:
City/County:
Waterway:

PREPARED BY
Name:
Organization:
Date:

1. Type and size of structure ___________________________ Length ________________
   Invert in_______ out_________ Height of cover_______ Drainage Area________
   Design Discharge_______ Design Frequency_______ Design Headwater Elev.________
   100-yr Discharge _______ 100-yr Headwater Elev. _________
   OHW elevation________
   Outlet Protection __________________________

2. Temporary structures for construction __________________________

_________________________________________________________

3. Applicable flood plain management criteria:
   Note: Use ONLY the one statement that is applicable and erase all the rest, including this
   instruction and the FEMA delineation description information.

For project within a FEMA delineated floodplain:

FEMA regulates flood level, flood velocity, and flow distribution and this project is within
FEMA community panel number ________ and Zone ____. This project complies with
FEMA requirements because there will be no increase in flood levels, velocities or flow
distribution. A copy of an excerpt from the aforementioned map panel showing the crossing
site has been included.

FEMA regulates flood level, flood velocity, and flow distribution and this project is within
FEMA community panel number ________ and Zone ____. This project complies with
FEMA requirements because a bridge/culvert will be replaced with a hydraulically equivalent
replacement structure. A copy of an excerpt from the aforementioned map panel showing the
crossing site has been included.
For project permits in a FEMA floodplain carrying a Zone A (or Zone X) designation that does not have base flood elevations. In such instances, an increase in 100-year flood level not exceeding one foot is acceptable.

FEMA regulates flood level, flood velocity, and flow distribution and this project is within FEMA community panel number: __________ and Zone A (or X). This project complies with FEMA requirements because there will be no more than a one foot increase in flood levels, velocities and flow distribution will not be changed significantly. A copy of an excerpt from the aforementioned map panel showing the crossing site has been included.

For projects not within a FEMA floodplain, include the following statement:

FEMA regulates flood level, flood velocity and flood distributions and this project is not within a designated or delineated FEMA floodplain. The project complies because there are no FEMA requirements applicable within the project area.

4. EROSION AND SEDIMENT CONTROL
   An erosion and sediment control plan will be prepared and implemented in compliance with the Erosion and Sediment Control Law, the Erosion and Sediment Control Regulations, and VDOT's Annual Erosion and Sediment Control Standards and Specifications approved by the Department of Conservation and Recreation.

5. STORMWATER MANAGEMENT
   Design of this project will be in compliance with the Stormwater Management Act, the Stormwater Management Regulations, and VDOT's Annual Stormwater Management Standards and Specifications approved by the Department of Conservation and Recreation.

6. COUNTERSINKING AND MULTIPLE BARRELL CULVERTS

Note: Use ONLY the statements that are applicable and erase all the rest.

The upstream and downstream inverts of culverts with diameters greater than 24” (or equivalent) will be countersunk a minimum of 6” below the stream bed.

The upstream and downstream inverts of culverts with diameters equal to or less than 24” (or equivalent) will be countersunk a minimum of 3” below the stream bed.

At least one barrel of a multiple barrel culvert structure will be countersunk a minimum of 6” for a diameter greater than 24” (or equivalent) or a minimum of 3” for a diameter equal to or less than 24” (or equivalent).

The width of the countersunk culvert barrel(s) receiving the low flow is approximately the width of the normal stream bed.
Low flow design measures have been implemented for multiple barrel culverts in which all barrels will be countersunk.

Culverts on bedrock will be countersunk a minimum of 3” below the stream bed.

Culverts on bedrock will be countersunk at the upstream end a minimum of 3” and at the downstream end stone step pools, low rock weirs or other measures will be constructed.

Countersinking of the culverts is not practicable due to __________________________ (See IIM-214.2 Section 4). See attached supporting documentation

7. IMPACT STATEMENT

________________________________________________________________________

________________________________________________________________________
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<td>9.4.8.6</td>
<td>Minimum Grades</td>
<td>9-37</td>
</tr>
<tr>
<td>9.4.8.7</td>
<td>Maximum Grades</td>
<td>9-37</td>
</tr>
<tr>
<td>9.4.8.8</td>
<td>Pipe on Radius</td>
<td>9-38</td>
</tr>
<tr>
<td>9.4.8.9</td>
<td>Minor Structure Excavation</td>
<td>9-39</td>
</tr>
<tr>
<td>9.4.8.10</td>
<td>Trenchless Applications</td>
<td>9-39</td>
</tr>
<tr>
<td>9.4.8.11</td>
<td>Extension of Existing Pipes</td>
<td>9-40</td>
</tr>
<tr>
<td>9.4.9</td>
<td>Hydraulic Grade Line</td>
<td>9-40</td>
</tr>
<tr>
<td>9.4.9.1</td>
<td>Introduction</td>
<td>9-40</td>
</tr>
<tr>
<td>9.4.9.2</td>
<td>Tailwater and Outfall Considerations</td>
<td>9-41</td>
</tr>
<tr>
<td>9.4.9.3</td>
<td>Conservation of Energy and Energy Losses</td>
<td>9-42</td>
</tr>
<tr>
<td>9.4.9.3.1</td>
<td>Conduit Friction Losses</td>
<td>9-44</td>
</tr>
<tr>
<td>9.4.9.3.2</td>
<td>Junction Losses</td>
<td>9-44</td>
</tr>
<tr>
<td>9.4.9.3.3</td>
<td>Plunging Losses</td>
<td>9-47</td>
</tr>
<tr>
<td>9.4.9.3.4</td>
<td>Inlet Shaping (IS-1)</td>
<td>9-47</td>
</tr>
<tr>
<td>9.4.9.3.5</td>
<td>Total Headlosses</td>
<td>9-47</td>
</tr>
<tr>
<td>9.4.9.4</td>
<td>Use of Alternate Pipe Materials</td>
<td>9-48</td>
</tr>
</tbody>
</table>
9.5 Design Procedures and Sample Problems ............................................................... 9-50
  9.5.1 Design Documentation ...................................................................................... 9-50
  9.5.2 Spread Calculations .......................................................................................... 9-50
    9.5.2.1 Uniform Cross Slope Procedure ....................................................... 9-50
    9.5.2.2 Composite Gutter Sections Procedure ............................................. 9-52
      9.5.2.2.1 Composite Gutter Sample Problem ........................................... 9-52
    9.5.2.3 Temporary Barrier Wall ..................................................................... 9-53
  9.5.3 Inlet Spacing Procedure .................................................................................... 9-53
    9.5.3.1 Curb Inlet on Grade Sizing Procedure .............................................. 9-54
      9.5.3.1.1 Curb Inlet on Grade Sample Problem ........................................... 9-56
    9.5.3.2 Curb Inlet in Sag Sizing Procedure .................................................. 9-58
      9.5.3.2.1 Curb Inlet in Sag Sizing Sample Problem ..................................... 9-59
  9.5.4 Grate in Sag Procedure .................................................................................... 9-61
    9.5.4.1 Grate in Sag Sample Problem .......................................................... 9-61
  9.5.5 Storm Drain Conduit Design Procedure ........................................................... 9-62
  9.5.6 Hydraulic Grade Line Procedure ....................................................................... 9-64
    9.5.6.1 Storm Drain Conduit Design and Hydraulic Grade Line Sample
             Problem ............................................................................................. 9-67

9.6 References .................................................................................................................... 9-70

List of Tables
Table 9-1. Criteria for Inlet Design .................................................................................... 9-3
Table 9-2. Design Frequencies for Storm Drain Conduit .................................................... 9-3
Table 9-3. Access Hole Spacing ....................................................................................... 9-7
Table 9-4. Joint Probability Analyses ................................................................................ 9-41

List of Figures
Figure 9-1. Uniform Cross Section .................................................................................... 9-18
Figure 9-2. Composite Cross Section ............................................................................. 9-19
Figure 9-3. Curb Inlets ................................................................................................. 9-20
Figure 9-4. Slotted Drain Inlets ..................................................................................... 9-21
Figure 9-5. Grate Inlets ................................................................................................. 9-22
Figure 9-6. Curb Opening Inlets (Operating as an Orifice) .......................................... 9-30
Figure 9-7. Use of Energy Losses in Developing a Storm Drain System ......................... 9-43
Figure 9-8. Angle Between Inflow and Outflow Pipes ................................................. 9-45
Figure 9-9. Losses in Junction Due to Change in Direction of Flow Lateral .................... 9-46
Figure 9-10. Storm Drain Layout Sample Problem ....................................................... 9-67
Figure 9-11. Storm Drain Design Form LD 229, Sample Problem .................................. 9-68
Figure 9-12. Hydraulic Grade Line Design Form LD 347, Sample Problem ................... 9-69
List of Appendices

Appendix 9B-1  LD-204 Stormwater Inlet Computations
Appendix 9B-2  LD-229 Storm Drain Design Computations
Appendix 9B-3  LD-347 Hydraulic Grade Line Computations
Appendix 9C-1  Flow in Triangular Gutter Sections
Appendix 9C-2  Flow Characteristic Curves (Straight Cross Slope with Curb)
Appendix 9C-3  Flow Characteristic Curves (24" Gutter) - VDOT Standard
Appendix 9C-4  Flow Characteristic Curves (Straight Cross Slope, 18" Gutter)
Appendix 9C-5  Flow Characteristic Curves (Straight Cross Slope 12" Gutter)
Appendix 9C-6  Flow Characteristic Curve (Roll Type Gutter)
Appendix 9C-7  Flow in Composite Gutter Sections
Appendix 9C-8  Ratio of Frontal Flow to Total Gutter Flow
Appendix 9C-9  Velocity in Triangular Gutter Sections
Appendix 9C-10 Grate Inlet Frontal Flow Interception Efficiency
Appendix 9C-11 Grate Inlet Side Flow Interception Efficiency
Appendix 9C-12 Grate Inlet Capacity in Sump Conditions (VDOT Version)
Appendix 9C-13 Performance Curve DI-1 in a Sump
Appendix 9C-14 Performance Curve DI-7 in a Sump
Appendix 9C-15 Performance Curve DI-12 in a Sump (Side Slope 3:1)
Appendix 9C-16 Performance Curve DI-12 in a Sump (Side Slope 6:1)
Appendix 9C-17 Curb-Opening and Slotted Drain Inlet Length for Total Interception
Appendix 9C-18 Curb-Opening and Slotted Drain Inlet Interception Efficiency
Appendix 9C-19 Depressed Curb-Opening Inlet Capacity in Sump Locations
Appendix 9C-20 Curb-Opening Inlet Capacity in Sump Locations
Appendix 9C-21 Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats
Appendix 9C-22 Ratio of Frontal Flow to Total Flow in a Trapezoidal Channel
Appendix 9C-26 Values of Hydraulic Elements of Circular Section for Various Depths of Flow
Appendix 9D-1  P-1-7/8 and P-1-7/8-4 Grates - FHWA Classification
Appendix 9D-2  P-1-1/8 Grate - FHWA Classification
Chapter 9 - Storm Drains

9.1 Introduction

9.1.1 Objective

This chapter provides guidance on storm drain design and analysis. The quality of the final in-place system usually reflects the attention given to every aspect of the design as well as that accorded to the construction and maintenance of the facility. Most aspects of storm drain design such as system planning, pavement drainage, gutter flow calculations, inlet spacing, pipe sizing, and hydraulic grade line calculations are included in this chapter.

The design of a drainage system must address the needs of the traveling public as well as those of the local community through which it passes. The drainage system for a roadway traversing an urbanized region is more complex than for roadways traversing sparsely settled rural areas. This is often due to:

- The wide roadway sections, flat grades, shallow water courses, absence of side channels
- The potential for more costly property damage which may occur from ponding of water or from flow of water through built-up areas
- The fact that the roadway section must carry traffic, but also act as a channel to convey the water to a disposal point. Unless proper precautions are taken, this flow of water along the roadway could interfere with or possibly halt the passage of highway traffic
- The potential weakening of roadway base and subgrade due to saturation from extensive ponding

The primary goal of storm drain design is to limit the amount of water flowing on the travelway or ponding at sag points in the roadway grade to quantities that will not interfere with the passage of traffic for the design frequency storm. This is accomplished by:

- Placing inlets at such points and at such intervals to intercept flows and control spread
- Providing adequately sized storm drain conduit to convey flow from the inlets to a suitable outfall location
- Providing outfall conditions that do not cause excessive backwater throughout the storm drain system
9.2 Design Policy

9.2.1 Definition

For purposes of interpretation of the policies and procedures of VDOT, a storm drain or storm sewer system is defined as follows:

A storm sewer system is a drainage system (existing and/or proposed) consisting of a series of at least two interconnecting pipes and two structures (drop inlets, manholes, junction boxes, etc) designed to intercept and convey stormwater runoff from specific storm event without surcharge. An exception to this general rule is: one or more cross drain pipes connected by one or more drop inlets, “hydraulically designed” to function as a culvert(s) and not connected to a storm drain system.

9.2.2 General Policies

Refer to Chapter 2 for general Department policies.

Storm drain systems should be designed for all urban sections in accordance with the criteria and guidelines provided herein. The design of the storm drain system should consider local stormwater management criteria and plans where applicable.

Rev. 9/11
9.3 Design Criteria

9.3.1 Design Frequency and Spread

Table 9-1 provides recommended inlet design frequencies and allowable spreads for various roadway classifications. Table 9-2 provides design frequencies for storm drain conduit.

Table 9-1 Criteria for Inlet Design

<table>
<thead>
<tr>
<th>Roadway Classification</th>
<th>Design Speed (mph)</th>
<th>Design Storm Frequency (year$^{1, 2}$)</th>
<th>Design Storm Intensity (in./hr.)</th>
<th>Maximum Design Spread Width$^3$ (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interstate</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Shoulder On Grade</td>
<td>All</td>
<td>10</td>
<td>Actual</td>
<td>Sh. Width$^5$</td>
</tr>
<tr>
<td>Sag Location$^5$</td>
<td>All</td>
<td>50</td>
<td>Actual</td>
<td>Sh. Width$^5$</td>
</tr>
<tr>
<td>Principal Arterial</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Shoulder On Grade</td>
<td>≤ 50</td>
<td>10</td>
<td>Actual</td>
<td>Sh. Width + 3</td>
</tr>
<tr>
<td></td>
<td>&gt; 50</td>
<td>10</td>
<td>Actual</td>
<td>Sh. Width</td>
</tr>
<tr>
<td></td>
<td>All</td>
<td>10</td>
<td>Actual</td>
<td>Sh. Width + 3</td>
</tr>
<tr>
<td></td>
<td>Sag Location$^5$</td>
<td>≤ 50</td>
<td>N/A$^4$</td>
<td>½ Driving Lane + Gutter Width (If Any)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt; 50</td>
<td>10</td>
<td>Sh. Width + 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>≤ 50</td>
<td>N/A$^4$</td>
<td>½ Driving Lane + Gutter Width (If Any)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt; 50</td>
<td>50</td>
<td>Sh. Width + 3</td>
</tr>
<tr>
<td></td>
<td>Minor Arterial, Collector, Local</td>
<td>≤ 50</td>
<td>N/A$^4$</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt; 50</td>
<td>N/A$^4$</td>
<td>Sh. Width</td>
</tr>
<tr>
<td></td>
<td></td>
<td>All</td>
<td>N/A$^4$</td>
<td>Sh. Width + 3</td>
</tr>
<tr>
<td></td>
<td>Sag Location$^5$</td>
<td>≤ 50</td>
<td>N/A$^4$</td>
<td>½ Driving Lane + Gutter Width (If Any)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt; 50</td>
<td>50</td>
<td>Sh. Width + 3</td>
</tr>
<tr>
<td></td>
<td>Minor Arterial, Collector, Local</td>
<td>≤ 50</td>
<td>N/A$^4$</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&gt; 50</td>
<td>N/A$^4$</td>
<td>Sh. Width</td>
</tr>
<tr>
<td></td>
<td></td>
<td>All</td>
<td>N/A$^4$</td>
<td>Sh. Width + 3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sag Location$^5$</td>
<td>N/A$^4$</td>
<td>½ Driving Lane + Gutter Width (If Any)</td>
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</tr>
</tbody>
</table>

Table 9-2 Design Frequencies for Storm Drain Conduit

<table>
<thead>
<tr>
<th>Roadway Classification</th>
<th>Design Speed (mph)</th>
<th>Design Storm Frequency (year$^{1, 2}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Principal Arterial</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With Shoulder</td>
<td>All</td>
<td>25</td>
</tr>
<tr>
<td>Without Shoulder</td>
<td>≤ 50</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>&gt; 50</td>
<td>25</td>
</tr>
<tr>
<td>Minor Arterial, Collector, Local</td>
<td>All</td>
<td>10</td>
</tr>
</tbody>
</table>

$^1$ Rev. 3/19
The following notes apply to the superscripts in Table 9-1 and Table 9-2:

Notes 1 through 3 are General Notes and should be applied to any functional classification roadway where the site conditions are comparable to the conditions described in each note.

1. At locations where the vertical alignment of the roadway creates a sag condition in either a depressed roadway section or a roadway section utilizing concrete barriers, and ponded water on the roadway can only be removed through the storm drain system, a 50-year storm frequency and the actual time of concentration should be used as the design criteria for both the drop inlet and the pipe system.

2. Federal Flood Insurance criteria dictate that the effects of the 100-year storm event (using the actual time of concentration) on buildings insured under the Flood Insurance Program must be investigated. Such cases should only be encountered where the roadway traverses a designated floodplain area containing insured buildings and the depth of water on the pavement is sufficient to overtop the curb and flow to the buildings.

3a. The maximum design spread width may not be obtainable due to the pavement/shoulder slope and the height of the curb. In locations where the curb would be overtopped and water would escape the roadway section prior to achieving the maximum design spread width, the maximum depth of ponded water allowed adjacent to the curb for the design storm shall be curb height minus 1”.

b. For those locations that show a maximum spread width of “1/2 Driving Lane Width + Gutter width (If Any)”, the table assumes that the driving lane is adjacent to the curb/curb and gutter section. If the driving lane is not adjacent to the curb/curb and gutter section (e.g., there is a parking and/or bicycle lane between the curb/curb and gutter section and the driving lane), then the maximum spread width shall be 10’, except in no case shall the spread of the water be allowed to encroach beyond the center of the Bicycle lane.

c. For those locations that show a maximum spread width of “Shoulder Width” (not “Shoulder Width + 3’), the table assumes that the shoulder width will be a minimum of 6’. Where the shoulder width is less than 6’, the maximum spread width shall be 6’, except in no case shall the spread of the water be allowed to encroach more than 3’ into the driving lane adjacent to the shoulder.

Rev. 3/19
Notes 4 through 5 should normally be applied to the specific locations as noted in the criteria table.

4. At locations where it may be reasonably anticipated that the runoff from storm events with rainfall intensities greater than 4 in/hr will overtax the drop inlet system to the point that excess flow will escape the roadway section and result in potential damage to the adjacent property and/or roadway right of way, the drop inlet system shall be analyzed for a check storm event with a rainfall intensity of 6.5 in/hr.

If all of the runoff from the check storm event is found to be contained within the roadway section, both at the site and down grade, or if runoff escaping the roadway section is found to not be damaging to adjacent property, the drop inlet system may be used as originally designed under the general criteria. If the drop inlet system fails to meet the check storm criteria, it must be re-designed to accommodate the runoff from the check storm event.

5. Drop inlets in these locations are prone to clogging and are often located in areas where maintenance is difficult. To compensate for partial clogging, the computed slot length value should be adjusted by multiplying by a factor of two (2). The adjusted computed slot length value should then be used to determine the slot length specified on the plans.

6. There shall be no encroachment into the driving lane.

9.3.2 Hydrology

The Rational Method is the recommended method for the design of storm drain systems. Drainage systems involving detention storage, pumping stations, and large or complex storm systems require the development of a runoff hydrograph. The Rational Method is described in Chapter 6, Hydrology.

9.3.3 Pavement Drainage

The desirable gutter profile grade for curbed pavements should not be less than 0.5%. The minimum gutter profile grade is 0.2%. The minimum pavement cross slope should not be less than 2% except during the occurrence of superelevation transition. The coincident occurrence of superelevation transitions and sag points or zero grades should be avoided.

Rev. 3/19
9.3.4 Inlet Design

Drainage inlets should be sized and located to limit the spread of water on travel lanes in accordance with the design criteria specified in Section 9.3.1.

Grate inlets and local depression at curb opening inlets should be located outside the through travel lanes to minimize the shifting of vehicles attempting to avoid these areas. All inlet grates should be bicycle safe when used at locations where bicycle travel is anticipated.

Curb inlets are preferred to grate inlets because of their debris handling capabilities.

To properly drain sag vertical curves, it is recommended practice to place flanking inlets on each side of the inlet located at the low point in the gutter grade. See section 9.4.6.7 for specific recommendations. In addition to determining the spread of water resulting from the inlet in the low point of the gutter grade, the spread on the approach roadway just upgrade of the sag point should also be determined. A longitudinal slope of 0.1% should be used in determining the spread on the approach roadway. There are cases where special treatment of the gutter gradient is provided. In those instances, the flattest grade that will actually occur on the approach gradient should be used in lieu of 0.1%.

9.3.5 Conduit Design

Storm drains should have adequate capacity to accommodate runoff that will enter the system. They should be designed considering anticipated future development based on local land use plans. The minimum recommended conduit size for storm drainage pipe is 15" diameter or its equivalent for non-circular shapes. Where necessary, it will be permissible to use a 12" diameter pipe for laterals or initial pipe runs of 50’ or less.

Where feasible, the storm drains should be designed to avoid existing utilities. A minimum velocity of 3 fps for the design storm is desirable in the storm drain in order to prevent sedimentation from occurring. Attention should be given to the storm drain outfalls to ensure that potential erosion is minimized.

The proposed storm drain system design should be coordinated with the proposed sequence of construction and maintenance of traffic plans on large construction projects in order to prevent unsafe ponding of water and to maintain an outlet throughout the construction of the project.

Rev. 9/09
9.3.6 Access Hole Spacing

The maximum spacing of access structures whether manholes, junction boxes, or inlets should be as identified in Table 9-3 below.

<table>
<thead>
<tr>
<th>Pipe Diameter (in)</th>
<th>Maximum Distance (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>50</td>
</tr>
<tr>
<td>15 - 42</td>
<td>300'</td>
</tr>
<tr>
<td>≥ 48</td>
<td>800</td>
</tr>
</tbody>
</table>

Note 1: This distance may be increased to 400' if the flow velocity for the design storm exceeds 5 fps and the flow depth for the design storm is at least 25% of the pipe diameter.

9.3.7 Hydraulic Grade Line

The hydraulic grade line should be checked for all storm drain systems using the VDOT method described in Section 9.5.6. For the design storm, the storm drain should be designed such that the hydraulic grade line does not exceed any critical elevation. A critical elevation is defined as a level above which there would be unacceptable inundation of the travel way or adjoining property. This includes the tops of manholes, junctions, and inlets. Because the inlet design is predicated on free-fall conditions, their hydraulic grade line should not exceed an elevation that interferes with the operational conditions of any inlet because the inlet design is predicated on free-fall conditions. Refer to Table 9-2 for design and check storm frequencies.

9.3.8 Unique Conditions

There may be unique situations that do not permit the application of the criteria provided herein. In such cases, the designer should develop and document site-specific criteria indicating the rationale and factors used to determine such criteria.

9.3.9 Drainage Design at Railroads

(See Section 8.3.8 of the VDOT Drainage Manual for applicable criteria to Storm Drains.)

Rev. 7/14
9.4 Design Concepts

9.4.1 System Planning

The design of a storm drain system is generally a process that evolves as a project develops. The primary ingredients to this process are listed below in a general sequence by which they may be accomplished:

1. Data collection (Section 9.4.1.1)
2. Coordination with other agencies and adjacent projects
3. Preliminary layout of project with respect to surrounding area
4. Plan layout of storm drain system
   • Locate main outfall(s)
   • Determine direction of flow
   • Determine contributing drainage areas
   • Determine inlet type, spacing, and capacity (Sections 9.4.4.5, 9.4.5, 9.4.6, and 9.4.7)
   • Determine location of existing utilities
   • Determine location of existing storm drain systems
   • Locate additional access holes
5. Size the conduit (Section 9.4.8)
6. Perform hydraulic grade line analysis (Section 9.4.9)
7. Prepare the plan
8. Documentation of design (Section 9.5.1)

9.4.1.1 Required Data

The designer should be familiar with land use patterns and local comprehensive land use plans, the nature of the physical development of the area(s) to be served by the storm drainage system, the stormwater management plans for the area and the ultimate pattern of drainage (both by overland flow and by enclosed storm drains) to existing outfall locations. Furthermore, there should be an understanding of the characteristics of the outfall since it usually has a significant influence on the design of the storm drainage system. In environmentally sensitive areas, there may be water quality requirements to consider as well.

Actual surveys are the most reliable means of gathering the required data. Photogrammetric mapping has become one of the most important methods of obtaining the large amounts of data required for drainage design. Existing topographic maps are available from the U. S. Geological Survey and the National Resources Conservation Service. Many municipalities and some county governments and even private developers are also valuable sources for the kind of data needed to perform a proper storm drainage design. Governmental planning agencies should be consulted regarding development plans for the area in question. Often, in rapidly growing urban areas, the physical characteristics of an area to be served by a storm drainage system may change drastically in a very short time. In such cases, the designer is to anticipate these changes and consider them in the storm drainage design.
Local comprehensive stormwater management plans and floodplain ordinances should also be considered in the storm drainage design process.

For detailed information of survey requirements, refer to the Virginia Department of Transportation Survey Instruction Manual.

When an existing storm drain is to be used, the designer should secure the following information:

- Invert elevations for all significant system components including conduits, drop inlets, catch basins, manholes, junctions, etc.
- Type and size of conduit

This information should extend beyond the limits of the proposed project, at least to the next access structure.

9.4.1.2 Preliminary Layout

Preliminary or working layouts, featuring the basic components of the intended design, are invaluable in the design development. After design completion, the layout facilitates documentation of the overall plan.

The following items may be included in the preliminary layout:

- General roadway layout (plan and profile)
- Basic hydrologic data
- Pertinent physical features
- Characteristics of flow diversion (if applicable)
- Detention features (if applicable)
- Outfall location and characteristics
- Surface features (topography)
- Utilities
- Proposed or existing foundations and structures

The layout should be used to develop a logical storm drain system that identifies and minimizes utility conflicts, avoids conflicts with structures and conforms to the proposed construction sequencing and maintenance of traffic plans. Additionally, the layout can be used to identify locations for necessary soil borings.

9.4.1.3 Special Considerations

Primary consideration in the planning of the storm drainage system should be directed toward avoidance of utilities and deep excavations. In many cases, traffic must be maintained on existing roadways or temporary bypasses may be constructed with temporary drainage provided during the construction phase. Consideration should be given to the actual trunk line layout and its constructability with regards to the maintenance of traffic plan. Some instances may dictate a trunk line on both sides of the roadway with very few cross laterals while other instances may dictate a single trunk line. Such decisions are usually based on economics but may be controlled by existing utilities or other physical features.
The designer should accommodate all natural drainage areas contributing to the system and minimize interference to natural drainage patterns. Except in unusual circumstances, a storm drain system should discharge to a single outfall.

Generally, storm drainage pipes should not decrease in size in a downstream direction regardless of the available pipe slope. However, if found necessary, any decrease in pipe size should not exceed 6”.

9.4.2 Hydrology

9.4.2.1 Applicable Methods

Refer to Chapter 6, Hydrology, for detailed description of hydrologic methods. The recommended method used for storm drain design is the Rational Method. The subsequent text in this chapter assumes use of the Rational Method for estimating peak discharge rates.

9.4.2.2 Runoff Coefficients

Recommended runoff coefficients for various types of land surfaces are provided in Chapter 6, Appendix 6E-1.

9.4.2.3 Time of Concentration

When determining the discharge for inlet size and spacing, use the estimated time of concentration for the drainage area to the location of the inlet unless otherwise indicated by the criteria identified in Table 9-1. When determining the discharge for conduit sizing, use the longest travel time to the upstream end of the conduit under consideration.

9.4.2.4 Rainfall Intensity

The rainfall intensity should be based on the time of concentration identified in Section 9.4.2.3 or the limiting value identified in Table 9-1. Refer to Chapter 6 for determining the appropriate rainfall intensity when using the actual time of concentration.

9.4.3 Pavement Drainage

9.4.3.1 Introduction

A chief objective in the design of a storm drain system is to move any accumulated water off the travelway as quickly and efficiently as possible. Where the flow is concentrated, the design objective should be to minimize the depth and horizontal extent of that flow. Appropriate longitudinal and transverse slopes can serve to move water off the travel way to minimize the depth of sheet flow and thus minimize the potential for hydroplaning. An objective of the design should be to establish efficient drainage in conjunction with the geometric and pavement design.
9.4.3.2 Hydroplaning


NCHRP research project I-29, "Improved Surface Drainage of Pavements," suggests that hydroplaning conditions can develop for relatively low vehicle speeds and at low rainfall intensities for storms that frequently occur each year. Analysis methods developed through this research effort provide guidance in identifying potential hydroplaning conditions. Unfortunately, it is virtually impossible to prevent water from exceeding a depth that would be identified through this analysis procedure as a potential hydroplaning condition for a wide pavement section during high intensity rainfall. Some of the primary controlling factors for hydroplaning are:

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

Speed appears as a significant factor in the occurrence of hydroplaning; therefore, it is considered to be the driver's responsibility to exercise prudence and caution when driving during wet conditions (AASHTO* Highway Drainage Guidelines*, 2007, Chapter 9, Storm Drain Systems and AASHTO *Model Drainage Manual*, 2014, Chapter 13 Storm Drainage Systems). In many respects hydroplaning conditions are analogous to ice or snow on the roadway.

Designers do not have control over all of the factors involved in hydroplaning. However, many remedial measures can be included in development of a project to reduce hydroplaning potential. The following is provided as guidance for the designer as practical measures to consider in accordance with the AASHTO Policy on Geometric Design of Highways:

Pavement Sheet Flow

- Maximize transverse slope

Gutter Flow

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

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*Rev. 1/17*
9.4.3.3 Longitudinal Slope

A minimum longitudinal slope is more important for a curbed pavement section than for an uncurbed pavement section since a curbed pavement section is susceptible to the spread of stormwater adjacent to the curb. Flat slopes on uncurbed pavements can also lead to a spread problem if vegetation is allowed to build up along the pavement edge.

Desirable gutter grades should not be less than 0.3% for curbed pavements with an absolute minimum of 0.2%. Minimum grades can be maintained in very flat terrain by use of a rolling profile. To provide adequate drainage in sag vertical curves, a minimum slope of 0.3% should be maintained within 50’ of the low point in the curve. This is accomplished where the length of the curve divided by the algebraic difference in the grades is equal to or less than 165 ft/%. Although spread is not usually a problem at crest vertical curves, on extremely flat curves a similar minimum slope should be provided to facilitate drainage.

9.4.3.4 Cross Slope

The current AASHTO Policy on Geometric Design is standard practice and should be consulted prior to any deviation from the recommendations contained herein.

“Pavement cross slope is often a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort. It has been found that cross slopes of 2% have little effect on driver effort in steering, especially with power steering, or on friction demand for vehicle stability.” HEC-12 (archived) thus, the minimum recommended cross slope is 2%.

A careful check should be made of designs to minimize the number and length of flat pavement sections in cross slope transition areas. Consideration should be given to increasing cross slopes in sag vertical curves, crest vertical curves, and in sections of flat longitudinal grades where needed. Where curbs are used, depressed gutter sections can be effective at increasing gutter capacity and reducing spread on the travelway. Where curbs or raised barriers are used at superelevation transitions, inlets should be located at the upstream side of the transition where the cross slope is at 1%.

Generally, shoulders should be sloped to drain away from the travelway except in areas of narrow raised medians.

9.4.3.5 Curb and Gutter

Curbing at the outside edge of pavements is normal practice for low-speed, urban highway facilities. It serves several purposes, including containing the surface runoff within the roadway section and directing it away from adjacent properties, preventing erosion, providing pavement delineation and enabling the orderly development of property adjacent to the roadway. Curbs may be either barrier or mountable type.

*Rev. 9/09
A curb and gutter forms a triangular channel that can be an efficient hydraulic conveyance facility, which can convey runoff of a lesser magnitude than the design flow without impact of the traffic. When a design storm flow occurs, there is a spread or widening of the conveyed water surface. This spread of the water surface includes not only the gutter width, but also parking lanes or shoulders, and portions of the travelway. The designer, as discussed in Section 9.4.4.5, must limit this spread.

Where practicable, it is desirable to intercept runoff from cut slopes and other areas draining toward the roadway before it reaches the curb and gutter section. This minimizes the deposition of sediment and other debris on the roadway and reduces the amount of water that must be carried in the gutter section.

9.4.3.6 Shoulder Curbs

Shoulder curbs may be appropriate to protect fill slopes from erosion caused by water from the roadway pavement. See VDOT Road Design Manual, Appendix A" for details.

Shoulder curbs may be appropriate at bridge ends where concentrated flow from the bridge deck would otherwise run down the fill slope. This section of curb should be long enough to include any pavement transitions. Shoulder curbs are not required on the high side of superelevated sections or adjacent to barrier walls on high fills. Drop inlets are the preferred means of intercepting flow along these sections. Drop inlets should be located in accordance with the criteria in Table 9-1 for spread and frequency. A limiting factor that sometimes dictates the location of shoulder curb drop inlets is the requirement that the depth of the design flow at the curb should be limited to 1” below the top of the curb. Shoulder cuts are not recommended on primary and interstate highways because of the difficulty of maintenance.

9.4.3.7 Depressed Median/Median Barrier

Depressed 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 travel way. Where median barriers are used particularly at horizontal curve locations with associated superelevations, it is necessary to provide inlets and connecting storm drain pipes to collect the water that accumulates against the barrier. Slotted drains adjacent to the median barrier and in some cases weep holes in the barrier can also be used for collection of the water.

9.4.3.8 Impact Attenuators

The location of impact attenuator systems should be reviewed to determine the need for drainage structures in these areas. With impact attenuator systems such as BRAKEMASTER or CAT systems, it is necessary to have a clear or unobstructed open area as traffic approaches the point of impact in order to allow a vehicle to impact the

\[Rev. 1/17\]
If the impact attenuator is placed in an area where superelevation or other grade separation occurs, grate inlets and/or slotted drains may need to be placed to prevent water from running through the clear open area and crossing the travelway. Curb, curb-type structures or swales cannot be used to direct water across this clear open area as these types of structures could cause vehicle vaulting in the area of the impact attenuator system.

9.4.3.9 Underdrains

Underdrains are important in assisting the removal of standing water like stormwater runoff, and preventing the pavement section from becoming saturated, and ultimately failing.

1. The Drainage Engineer should take the following guidelines into account when locating underdrains:

- When a Standard Underdrain UD-3, UD-4 or UD-7 passes through a commercial entrance, "non-perforated" pipe is required between the limits of the curb returns. This "non-perforated" pipe is to be summarized with the applicable underdrain. (See Standards UD-3, UD-4, and UD-7 and Sample Summary)
- Standard underdrains will provide drainage for pavement structures, as recommended by the Materials Division.
- Standard EW-12 shall be used at outlet ends of all underdrains which do not tie to other drainage structures (inlets, manholes, etc.).
- When ramp gore areas are above and sloping toward rigid pavement, abutted by asphalt shoulders, UD’s will be provided at the gore to collect and drain water under the pavement.
- Designers are cautioned that special attention must be given to superelevated curves and transitions to assure that the underdrain is properly located to provide drainage for subbase material.

2. The Roadway Designer and Drainage Engineer should adhere to the following design procedures when locating underdrains:

- The Roadway Designer will submit Form LD-252 to the Materials Division, requesting preliminary pavement design and underdrain type and location recommendations. Form LD-252 will be submitted during the early stages of project development so that the requested information will be available to the Drainage Designer during the drainage design phase prior to the Field Inspection.
- The Materials Division will provide the Roadway Designer with recommendations for the preliminary pavement design and the type and location of underdrains for the project. Underdrain recommendations will include Standard UD-2, UD-4, UD-5, UD-6 and/or UD-7 underdrains, as appropriate. Recommendations will include Standard UD-1 underdrains when sufficient data exists to determine locations.

Rev. 7/14
The Drainage Designer will depict the underdrains on the drainage layer of the electronic files and/or hard copy of the plans at the locations recommended by the Materials Division.

The Drainage Designer will depict Standard UD-3 Sidewalk Underdrains on the drainage layer of the electronic files and/or hard copy of the plans at the locations recommended by the District Construction Engineer.

The Drainage Designer will:
- Determine the locations for CD-1 or CD-2's at:
  - Down grade end of cut to fill transitions
  - Sag points in roadway grade
  - Bridge approach slabs
- Determine outlet pipe locations for all parallel underdrain systems. Unless otherwise approved by the State Materials and the State Hydraulics and Utilities Engineer, the following criteria will apply to spacing of outlet pipes:
  - UD-1 – Variable spacing
  - UD-2 – 500’ maximum spacing
  - UD-3 – 1000’ maximum spacing
  - UD-4 – 350’ maximum spacing
  - UD-5 – 350’ maximum spacing
  - UD-7 – 350’ maximum spacing
- For Rural (shoulder/ditch design) projects:
  - Determine the modifications required (if any) to the ditch typical section in order to provide a minimum 12” of freeboard (vertical clearance) between invert of outlet pipe and invert of receiving ditch; or
  - Design a storm sewer system under the ditch line for the connection of underdrain outlet pipes that provide for the minimum 12” of freeboard between the invert of the outlet pipe and the invert of the receiving structure.
- For Urban (curb and gutter/storm sewer design) projects:
  - Design the storm sewer system to provide the minimum 12” of freeboard between the invert of the outlet pipe connection and the invert of the receiving structure.
- Specify EW-12 Endwall at end of outlet pipe or specify connection to another structure (manhole, drop inlet, etc.)
- Depict the required underdrains and/or outfall systems on the drainage layer of the electronic files or on redline prints of the plans. The information will be transmitted to the Roadway Designer along with the normal drainage design for the project.
3. The following is a comprehensive listing of the typical underdrains as specified in the VDOT Road & Bridge Standards:

- **Drainage for Pavement Subbase:**

<table>
<thead>
<tr>
<th>STANDARD</th>
<th>USAGE AND PURPOSES</th>
</tr>
</thead>
<tbody>
<tr>
<td>UD-1</td>
<td>As recommended by materials division to lower ground water table in cuts</td>
</tr>
<tr>
<td>UD-2</td>
<td>Drains raised grass median strips as recommended by Materials Division</td>
</tr>
<tr>
<td>CD-1 &amp; 2</td>
<td>Drains subsurface water from cuts and fills according to road and bridge standards and as recommended by Materials Division</td>
</tr>
<tr>
<td>UD-3</td>
<td>Drains area under sidewalk</td>
</tr>
<tr>
<td>UD-4</td>
<td>Provides drainage for pavement structure as recommended by Materials Division</td>
</tr>
<tr>
<td>UD-5</td>
<td>Same as UD-4; more easily added to previously constructed projects</td>
</tr>
<tr>
<td>UD-7</td>
<td>Provides pavement structure drainage as recommended by Material Division for existing pavements</td>
</tr>
<tr>
<td>EW-12</td>
<td>Used at outlet ends of all underdrains which do not tie to other drainage structures (inlets, manholes, etc.)</td>
</tr>
</tbody>
</table>

- **Underdrains in Gore Areas**
  - Ramp gore areas on down grades are prone to retaining water that may spill over the pavement. This may result in slippery pavement and icing if the pavement structure is not adequately drained. See Standard UD-4 for method of installation.

*Rev. 7/14*
4. The following is an example of what is required of plan details of standard underdrains:

- When showing EW-12’s on plans, label as follows showing appropriate slope:
  - 1 – St’d. EW-12 Req’d. (4:1 Slope)

- Following is a typical method of summarizing underdrains:

<table>
<thead>
<tr>
<th>STA. to STA.</th>
<th>UD-1</th>
<th>UD-4</th>
<th>CD-1</th>
<th>OUTLET PIPE</th>
<th>EW-12</th>
</tr>
</thead>
<tbody>
<tr>
<td>STA. to STA</td>
<td>Perforated</td>
<td>Non-Perforated</td>
<td>L.F.</td>
<td>L.F.</td>
<td>L.F.</td>
</tr>
<tr>
<td>20+00 To 31+00 Rt.</td>
<td>1100</td>
<td>500</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>25+00 To 51+00 Rt.</td>
<td>2350</td>
<td>400</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>31+50 Lt.</td>
<td>200</td>
<td>250</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>TOTALS</td>
<td>1100</td>
<td>2350</td>
<td>250</td>
<td>200</td>
<td>1150</td>
</tr>
</tbody>
</table>

**9.4.4 Gutter Flow**

**9.4.4.1 Introduction**

Gutter flow calculations are necessary in order to relate the quantity of flow in the curbed channel to the spread of water on the shoulder, parking lane, or travel lane. Gutter flow calculations can be performed using equations in the following sections or using nomographs provided in Appendices 9C-1 through 9C-7. Computer programs, such as the FHWA HEC-12 or HY25, are often used for this computational process.

**9.4.4.2 Manning’s “n” for Pavement and Gutter Flow**

It is recommended that an n-value of 0.015 be used in the computational analysis for pavement and gutter flow.

*Rev. 7/14*
9.4.4.3 Flow in Gutters

Flow in a gutter operates under the principles of open channel flow. Gutter capacity is a function of the geometric shape of the gutter, the roughness of the pavement surface, the longitudinal slope, and the allowable spread.

The gutter capacity for a uniform cross slope (as shown in Figure 9-1) may be computed using Equation 9.1.

\[
Q = \frac{0.56}{n} S_x^{0.5} S^{1.1} T^{0.8}
\]  

(9.1)

Where:

- \( Q \) = Gutter flow rate, cfs
- \( n \) = Manning’s roughness coefficient
- \( S \) = Longitudinal (gutter) slope, ft/ft
- \( S_x \) = pavement cross slope, ft/ft
- \( T \) = Spread width, ft
9.4.4.4 Composite Gutter Sections

The designer may choose to use a composite section. A composite section may be one in which the concrete gutter maintains a steeper cross slope than the travel lanes as shown in Figure 9-2.

![Figure 9-2 Composite Cross Section](image)

The steeper gutter slope for the width of the gutter pan (W) increases the gutter capacity. The capacity of composite gutter sections for varying hydraulic controls, such as spread, depth, cross slope, and longitudinal slope can be found using Appendices 9C-1, and 9C-3 through 9C-8 and is used in the inlet spacing procedure.

9.4.4.5 Spread

The spread is a function of discharge, cross section geometry, section roughness, and longitudinal slope. The spread increases with the length of pavement and/or increase in contributing drainage area.

Spread calculations are used to determine curb inlet spacing on roadways. The calculated spread for the design discharge should not exceed the allowable spread. Refer to Table 9-1 for allowable spread criteria. The spread in a uniform gutter section may be calculated using Equation 9.2.

\[
T = 1.243(Qn)^{3/5} S_x^{5/8} S^{3/16}
\]

(9.2)

The spread in various types of composite sections may be determined by using the appropriate chart in Appendices 9C-3 through 9C-8.

9.4.5 Inlets and Structures

9.4.5.1 Inlet Types

Standard details for various inlet types can be found in the VDOT Road and Bridge Standards, Volume 1. Where practicable, the designer should select an appropriate standard detail that accommodates the hydraulic and geometric needs.

Rev. 7/14
Inlets used for the drainage of highway surfaces can be divided into four major classes as follows:

- Curb-Opening inlets
- Combination inlets
- Slotted drain inlets
- Grate inlets

**9.4.5.1.1 Curb-Opening Inlets**

These inlets are vertical openings in the curb covered by a top slab. They can convey large quantities of water and debris. They are preferable to grates for pavement drainage especially at locations where grate inlets would be hazardous for pedestrians or bicyclists. (See VDOT DI-3 & 4 series inlets)

**DI-13 Shoulder Slot Inlets**

The DI-13 was specifically designed to:

- Collect water running along the bituminous curbing used under a guardrail system in high embankment areas.
- Discharge collected water through a 15” corrugated pipe exiting the back of the structure and traversing down the slope to the toe of the embankment.
- Be an economical structure to pre-cast because of its standardized dimensions.

Any modification to the standard details for this structure, or use in areas not consistent with the above guidelines, voids the original intent of the structure’s design. The details for the DI-13 are not to be altered in any manner from those noted on the standard drawings.

**9.4.5.1.2 Combination Inlets**

Inlets with curb opening and grate combinations are common. The designer should ignore the interception capacity of the grate when computing the capacity of a combination inlet. Combination inlets are sometimes used in order to place the inlet chamber and storm drain trunk line under the gutter pan and away from the sidewalk or utility space. (See VDOT DI-2 series inlets)

**DI-2 Inlets**

If a structure is needed to both intercept the water collected along the bituminous curbing under a guardrail system and to accommodate pipe sizes or locations other than those shown in the Standard DI-13 details, a Standard DI-2 structure may be considered for use.

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Rev. 7/14

Chapter 9-20 of 70
The structure should utilize a Type A Nose Detail (in order to match the Standard MC-3B curb configuration) and the concrete gutter and grate should employ one inch of additional (local) depression (in lieu of the standard 2") below the normal shoulder elevation.

In order to satisfy the guardrail alignment and block out requirements in the areas where the DI-2’s are utilized, a cast-in-place only structure must be specified. No DI-2’s should be placed within 25’ of a bridge terminal wall in order to avoid conflict with the Guardrail Fixed Object Attachment.

The following notes should be included with the structure description for DI-2’s utilized along bituminous curbing under guardrail:

- Type A Nose Required
- Concrete gutter and grate elevation at curb line to be one inch below normal shoulder elevation
- Structure to be cast-in-place only

9.4.5.1.3 Slotted Drain Inlets and Trench Inlets

Slotted drain inlets consist of a slotted opening with bars perpendicular to the opening. Slotted inlets function as weirs with flow entering from the side. 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. They can also be used to intercept flow in areas of limited space such as a retrofit in a problem area. The designer should ensure that maintenance access is provided with the design for this type of inlet. The two types of slotted inlets in general use are the vertical riser type and the vane type. VDOT does not have a standard for this type of inlet.

Trench drains are usually comprised of a long narrow grate built on a tray or preformed trench. They have uses similar to slotted drains.

9.4.5.1.4 Grate Inlets

Typically, grate inlets are used in depressed medians, graded areas, ditches, at the toe of fill in urban areas and other areas not subject to traffic. Grates should be bicycle safe where bike traffic is anticipated and structurally designed to handle the appropriate loads when they do need to be located in areas subject to traffic. (See VDOT DI-1, 5, 7 & 9 series inlets)

Rev. 7/14
When grate drop inlets, such as DI-5’s, DI-7’s and DI-12’s are specified, it is necessary to note on the plans and in the Drainage Summary the type of grate that is required. A general guideline for selecting grate type is:

<table>
<thead>
<tr>
<th>DI-5</th>
<th>DI-7</th>
<th>DI-12</th>
</tr>
</thead>
</table>
| Type I | Type I | Type I Ltd. Access and Rural Unlimited Access-
         | Type II   | Urban Areas-
         | Pedestrian Access Unlikely | Pedestrian Accessible Area |

The grate is to be installed so that the bars are parallel to the flow line of the ditch or swale.

When it is necessary to locate a grate drop inlet in an area subject to occasional traffic (e.g., shoulders, parking areas, etc.) a DI-1 or a DI-7 with the load carrying Grate B shall be specified. The DI-5, DI-7 Grate A, and DI-12 are not load carrying grates and should not be located in areas normally subject to traffic.

Where DI-7 or 12 series inlets are utilized to intercept concentrated flow (e.g., roadside, median, berm or toe ditches) the type of inlet that requires the concrete gutter should be specified (e.g., DI-7A, DI-7B, DI-12A or DI-12C).

DI-12 Multigrate Drop Inlets

The DI-12 Multigrate Drop Inlet is intended to provide one (1) standard grate configuration to handle the various traversable and non-traversable ditch slopes. The DI-12 Drop Inlet is to be located only in areas not normally subject to traffic. The narrow width of the DI-12 grate makes it more adaptable to narrow medians where difficulty retaining a traversable slope has been experienced with the DI-7 Drop Inlet’s width. The DI-7 is still the preferred structure to be used in locations where a traversable slope can be maintained.

- To provide the most economical design, all locations should first be checked to see if the smaller chambered DI-12B or DI-12C drop inlet can be used. The size of the pipes entering and exiting the chamber will generally dictate whether or not a Standard DI-12B or DI-12C drop inlet can be used.
- The Standard DI-12 and DI-12A drop inlets are to be specified at locations where the DI-12B and DI-12C drop inlets cannot be used.

Figure 9-5 Grate Inlets

Rev. 7/14
• Toe of fill and top of cut ditches with 2:1 slopes may use the St'd. DI-12 series drop inlet as well as those in the St'd. DI-5 and St'd. DI-7 series.

**DI-5 Drop Inlets**

• DI-5 Drop Inlets shall specify the type of cover (St'd. PG-2A Type) which most closely matches the ditch configuration at the inlet location. This data shall be shown both in the structure description on the plans and in the Drainage Summary.

### 9.4.5.1.5 Inlet Locations

Inlets are required at locations to collect runoff within the design controls specified in Table 9-1. In addition, there are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations should be marked on the plans prior to any computations regarding discharge, spread, inlet capacity, or bypass. Examples of such locations are as follows:

- Sag points in the gutter grade
- Either side of sag point inlet (flanking inlets)
- Upstream of median breaks, entrance/exit ramp gores, crosswalks and street intersections
- Immediately upstream and downstream of bridges
- At 1% cross slope upstream of cross slope reversals
- On side streets at intersections, where flow is approaching the main line
- Behind curbs, shoulders, or sidewalks to drain low areas or to intercept concentrated flow
- Where necessary to collect snow melt

Inlets should not be located in the path where pedestrians are likely to walk.

### 9.4.5.2 Structures

#### 9.4.5.2.1 Structure Heights

All drop inlets (both curb and median), catch basins, junction boxes and other structures that require a frame and cover or grate at finished grade elevation, shall show the height dimension (H) on the plans and on the Drainage Summary. This dimension is to be measured from the invert elevation to the top of structure and is to be shown in the drainage description. Manholes will be shown as the number of linear feet required, as measured from the invert to the top of the concrete or masonry structure, not including the frame and cover.

#### 9.4.5.2.2 Safety Slabs

Structures requiring safety slabs are to be determined by the Drainage Designer. Safety Slabs (Standard SL-1) shall be considered for use in deep drainage structures in

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Rev. 7/14
order to reduce the hazard potential for persons accidentally falling into or within the structure.

Standard SL-1 Safety Slabs shall be required as part of the drainage design for manholes, junction boxes and drop inlets with heights greater than 12 feet. The spacing of the slabs should be 8 feet to 12 feet with no slab located within 6 feet of the top or bottom of the structure. The slabs should be located so as to not interfere with the flow into or through the structure. On tall structures, where pipes inflow at various locations vertically, the safety slabs should not be placed below any 30 inch diameter or larger pipe opening.

Safety Slabs should not be considered for use where both the interior length and width of the structure’s chamber are less than 4 feet or the interior diameter is less than 4 feet. This condition generally occurs with some of the smaller cast-in-place inlet structures (e.g., DI-1A, DI-3AA, DI-3BB, DI-3CC, DI-7, DI-7A, DI-7B, etc.) However, if the contractor installs the precast option for these structures (which is often the case), the precast option would have a chamber dimension of 4 feet or greater and, therefore, safety slabs could be utilized. The Drainage Designer should assume that precast units in lieu of cast-in-place will be used and specify safety slabs accordingly. The following General Note should be included on the General Note Sheet:

D-18  St’d. SL-1 Safety Slab locations are based on the assumed use of precast structures. If cast-in-place structures are utilized, and the interior chamber dimensions (length and width, or diameter) are less than 4 feet, the safety slabs shall not be installed.

On structures whose vertical height is 12’ or greater and Safety Slabs are not specified, the use of bolt down or lock down covers or grates should be considered.

The cost of the SL-1 is included in the bid price for the structure. The drainage descriptions should specify how many safety slabs are needed for each structure and the quantity should be noted in the remarks column on the Drainage Summary.

9.4.6 Inlet Capacity

9.4.6.1 General

Inlets should first be located on the preliminary layout. The designer should locate inlets starting from the crest of the gutter grade and working down grade to the sag point. The location of the first inlet from the crest can be found by determining the length of pavement and the area back of the curb sloping toward the roadway that will generate the design runoff. The design runoff can be computed as the maximum allowable flow in the curbed channel that will meet the design criteria for spread of water on the travelway as specified in Table 9-1.

Rev. 7/14
Where the contributing drainage area consists of a strip of land parallel to and including a portion of the highway, the first inlet location can be calculated as follows:

\[ L = \frac{43,560Q_i}{CiW} \]  

(9.3)

Where:

- \( L \) = Distance from the crest, ft
- \( Q_i \) = Maximum allowable flow, cfs, as determined by allowable spread
- \( C \) = Composite runoff coefficient for contributing drainage area
- \( W \) = Width of contributing drainage area, ft
- \( i \) = Rainfall intensity for design frequency, in/hr

If the drainage area contributing to the first inlet from the crest is irregular in shape, trial and error will be necessary to match a design flow with the maximum allowable spread.

### 9.4.6.2 Curb Inlets on Grade and Bypass Flow

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. The throat of a typical curb inlet may be depressed below the normal gutter line to improve efficiency. This additional depression is referred to as “local depression.”

Curb inlets on grade should be designed either to intercept all of the approach flow or most of the approach flow, allowing only a small portion to bypass the inlet and carry on downgrade to the next inlet. Generally, allowing for bypass flow maximizes the use of the inlet opening and is acceptable if the resultant bypass flow does not cause the allowable spread to be exceeded downstream. Department practice is not to allow bypass flow immediately up gradient of the following locations:

- Intersections
- Superelevation transitions
- Ramps
- Bridges

To space successive down grade inlets, it is necessary to compute the amount of flow which will be intercepted by the inlet \( (Q_i) \) and subtract it from the total gutter flow to compute the bypass flow. The bypass flow from the first inlet is added to the computed flow to the second inlet. The second inlet should then be located and sized to meet the spread criteria defined in Table 9-1 using the combined flow.
9.4.6.3 Curb Inlets on Grade – Design Equations

Computer programs are often used for the following calculations. The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

\[
L_T = \frac{0.6Q^{0.42}S^{0.3}}{(nS_e)^{0.8}}
\]  

(9.4)

Where:

- \(L_T\) = Curb opening length required to intercept 100% of the gutter flow, ft
- \(Q\) = Gutter flow rate, cfs
- \(n\) = Manning’s roughness coefficient (0.015 for VDOT applications)
- \(S\) = Longitudinal slope, ft/ft (along flowline)
- \(S_e\) = Equivalent cross slope, ft/ft

Appendix 9C-17 provides a graphical solution of Equation 9.4.

If no local depression is applied, the equivalent cross slope \((S_e)\) is equal to the pavement cross slope \((S_x)\). If a local depression is applied the effective cross slope \((S_e)\) is determined using Equation 9.5.

\[
S_e = S_x + S_wE_o
\]  

(9.5)

Where:

- \(S_e\) = Equivalent cross slope, ft/ft
- \(S_x\) = Cross slope of the pavement, ft/ft
- \(S_w\) = Cross slope of gutter measured from cross slope of the pavement, ft/ft
  \[
  = \frac{a}{12W}
  \]
- \(W\) = Width of local depression, ft
- \(a\) = Depth of total inlet depression, inches [measured from point where cross slope \((S_x)\) intercepts face of curb]
  \[
  = 12W(S_w - S_x) + \text{Local Depression}
  \]
- \(S_w\) = Normal cross slope of area defined by “W”, ft/ft
- \(E_o\) = Ratio of flow in the depressed section to the total gutter flow

VDOT standard inlets (DI-2, DI-3 and DI-4) used in curb and gutter sections apply 1” of local depression for each 1’ width of the concrete gutter pan. For inlets used in curb only sections, a local depression of 1” is applied over a horizontal distance of 1’ from the face of the curb. For applications of local depression for other types of inlets, see the standard drawings in the VDOT Road and Bridge Standards.
The efficiency ratio \((E_o)\) may be determined using Equation 9.6 or by using Appendix 9C-8.

\[
E_o = \frac{K_w}{K_w + K_o}
\]  

\((9.6)\)

\(S_w = \) Normal cross slope of area defined by "W", ft/ft

Where:

\(E_o = \) Ratio of depression flow to total gutter flow  
\(K_w = \) Conveyance of the depressed gutter section, cfs  
\(K_o = \) Conveyance of the gutter section beyond the depression, cfs

The conveyance, \(K\), of any portion of the gutter section may be computed using Equation 9.7. The curb height is ignored when considering the wetted perimeter.

\[
K = 1.486 \frac{A^{\frac{5}{3}}}{nP^{\frac{2}{3}}}
\]  

\((9.7)\)

Where:

\(K = \) Conveyance of cross section, cfs  
\(A = \) Cross section flow area, sq. ft  
\(n = \) Manning’s roughness coefficient  
\(P = \) Wetted perimeter, ft

The designer may select a standard curb opening length that is equal to or longer than the required length, \(L_T\). If the provided length is longer than that required length using Equation 9.4, there will be no bypass flow for the design discharge and the designer should proceed to the next inlet down grade.

If the designer selects an inlet that is shorter than the required length computed by Equation 9.4, there will be bypass flow at the inlet location. The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by:

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

\((9.8)\)

Where:

\(E = \) Curb opening efficiency  
\(L = \) Curb opening length provided, ft  
\(L_T = \) Curb opening length required for 100% interception, ft
Appendix 9C-18 provides the graphical solution of Equation 9.8.

The intercepted flow is then computed as:

\[ Q_i = EQ \]  \hspace{1cm} (9.9a)

Where:

\[ Q_i = \text{Intercepted flow, cfs} \]
\[ E = \text{Curb opening efficiency} \]
\[ Q = \text{Total flow approaching inlet, cfs} \]

The bypass flow \((Q_b)\) is the difference between the total approach flow and the intercepted flow.

\[ Q_i = Q - Q_b \]  \hspace{1cm} (9.9b)

If bypass flow occurs, the designer must ensure that the computed bypass flow is included in the spread calculation and the inlet capacity calculation for the next inlet down grade.

### 9.4.6.4 Slotted Inlets on Grade

Slotted inlets can be used on curbed or uncurbed sections and are usually located in areas of limited space. They should be placed longitudinally in the gutter. Slotted inlets should generally be connected into inlet structures or manholes so they will be accessible to maintenance personnel in case of clogging or freezing.

The determination of required length of slotted drain for total interception is the same as outlined in Section 9.4.6.3 for curb inlets on grade except that no gutter depression is applied to slotted drain inlets (Equation 9.4). Similarly, if the provided length of slotted drain is shorter than the required length for total interception, the calculation of intercepted flow is determined using Equations 9.8, 9.9a, and 9.9b.

### 9.4.6.5 Curb Inlets in Sag

The capacity of a curb-opening inlet in sag depends on water depth at the curb, the curb opening length, the height of the curb opening and the local depression. 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.
The equation for the interception capacity of a depressed curb-opening inlet operating as a weir is:

\[ Q_i = C_w(L+1.8W)d^{1.5} \]  

(9.10)

Where:

- \( Q_i \) = Intercepted flow, cfs
- \( C_w \) = Weir coefficient, use 2.3
- \( L \) = Length of curb opening, ft
- \( W \) = Width of local depression, ft
- \( d \) = Depth of water at curb measured from a point where the normal pavement cross slope \( (S_x) \) would intercept the curb face, ft

The weir coefficient and 1.8-multiplier reflect the effective flow conditions at the lip of the transition to the inlet depression. Thus, the effective depth is considered from the water level to the gutter line, not the depressed throat of the inlet.

The weir equation for curb-opening inlets without depression is

\[ Q_i = C_wLd^{1.5} \]  

(9.11)

Where:

- \( Q_i \) = Intercepted flow, cfs
- \( C_w \) = Weir coefficient, use 3.0
- \( L \) = Length of curb opening, ft
- \( d \) = Depth of water at curb measured from a point where the normal pavement cross slope \( (S_x) \) intercepts the curb face, ft

The depth limitation for operation as a weir is less than or equal to 1.2\( h \), where \( h \) is the height of the curb opening. The weir coefficient applies to the inlet throat.
Curb-opening inlets operate an orifice at depths greater than approximately 1.4h. Typical curb opening inlet throat configurations are shown in Figure 9-6. Throat configuration (b) is typical of VDOT curb inlets. The interception capacity can be computed by:

$$Q_i = C_0 A [2gd_o]^{0.5}$$  \hspace{1cm} (9.12)

Where:

- $C_0$ = Orifice coefficient, use 0.67
- $h$ = Height of curb-opening orifice, ft
- $A$ = Clear area of opening, $ft^2$
- $d_o$ = Effective head on the center of the orifice throat, ft
- $g$ = Acceleration due to gravity =32.2, $ft/s^2$

*Rev. 9/09*
For VDOT standard curb inlets, \[ d_o = \left( \theta - \frac{h}{2} \right) \sin \theta \]

Where:
- \( d_i \) = Depth of flow at the curb including inlet depression (if present), ft
- \( h \) = Throat opening measured normal to the throat opening, ft
- \( \theta \) = Inlet throat angle (see figure 9-6 (b))

Equation 9.12 is applicable to depressed and undepressed curb-opening inlets.

Appendices 9C-19, 9C-20, and 9C-21 provide graphical solutions for the capacities of curb inlets in sags.

**9.4.6.6 Combination Inlets on Grade or in a Sag**

If combination curb opening and grate inlet or a slotted drain and grate inlet is used, it should be designed as an on grade curb or slotted drain inlet without consideration of the grate due to its propensity to clog.

**9.4.6.7 Flanking Inlets**

At major sag points significant ponding may occur. It is recommended practice to place a minimum of one flanking inlet on each side of the inlet at the sag point. The flanking inlets should be placed to limit ponding in the flatter slope approaches to the sag inlet and to act in relief of the sag inlet should it become clogged. The recommended location for flanking inlets is at points upgrade from the sag where the edge of pavement elevation is no higher than 0.3’ above the edge of pavement elevation at the sag point. This is typically 50’-75’ upgrade of the sag.

**9.4.7 Grate Inlets**

**9.4.7.1 Grate Inlets on Grade (Depressed Sections)**

Grate inlets on grade are used in depressed medians and ditches. VDOT standard inlets 5, 7A, 12A and 12C are typically used for this purpose. It is preferable to use a small backup berm or dike (as shown in the Road and Bridge Standards, Volume 1) located just downstream of the inlet. The berm ensures the interception of the on grade flow and causes the inlet to function as a sag inlet. The designer needs to indicate on the plans that a back-up berm is required and provide details for the berm, including the height of the berm. For grate inlets on-grade on roadways or in depressed medians and ditches without the use of a back-up berm, use the procedures presented in FHWA publication, HEC-12 or HEC-22.

The computation methodology for grate inlets in sag is presented in Section 9.4.7.2. These computations are often done with a computer program, such as FHWA HY-22 or Visual Urban.

*Rev. 9/09*
9.4.7.2 Grate Inlets in Sag (Depressed Sections)

When grates are used in a sag, assume that the efficiency of the grate will be reduced by 50% due to clogging with debris. This is accomplished by dividing the effective perimeter and open area of the grate by two and using the resulting values in the computational process.

A grate inlet in sag operates as a weir up to a depth of about 5" and as an orifice for depths greater than about 17". Between these depths, a transition from weir to orifice flow occurs. The capacity of a grate inlet operating as a weir is:

\[ Q_w = C_w P d^{1.5} \]  

(9.13)

Where:

- \( C_w \) = Weir coefficient, use 3.0
- \( P \) = Effective perimeter of grate, ft

The effective perimeter of the grate in sag is \( 2(L+W) \), when the grate is used in a depressed median or ditch and \( 2W +L \) when adjacent to a curb, where (L) is the length of grate and (W) is the width of grate.
- \( d \) = Depth of water at curb measured from the normal cross slope gutter flow line, ft

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

\[ Q_o = C_o A (2gd)^{0.5} \]  

(9.14)

Where:

- \( C_o \) = Orifice coefficient, use 0.67
- \( A \) = Effective clear open area of grate, ft²
- \( g \) = Gravitational acceleration, 32.2 ft/s²

Appendix 9C-12 provides a nomographic solutions of Equations 9.13 and 9.14 for various, generic grate sizes. Appendices 9C-13 through 9C-16 provide nomograph solutions based on actual physical model testing for VDOT standard grate type inlets. The effects of grate size on the depth at which a grate operates as an orifice are apparent from the charts. Transition from weir to orifice flow results in interception capacity less than that computed by either weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used.

9.4.7.3 Grate Inlets in Curb and Gutter Sections

To determine the efficiency of grate inlets in curb and gutter sections, use the procedures presented in FHWA publications, HEC-12 or HEC-22.
9.4.8 Storm Drain Conduit

9.4.8.1 Introduction

This section describes the methodology for computing conduit sizes. Section 9.5.5 presents the VDOT recommended method of calculation, which may be performed using the design form LD-229 provided in Appendix 9B-2.

After the preliminary locations of inlets, connecting pipes, and outfalls are determined, the next step is the computation of the rate of discharge to be carried by each reach of the storm drain, and the determination of the size and slope of pipe required to convey this discharge. This is done by starting at the upstream reach, calculating the discharge and sizing the pipe, then proceeding downstream, reach by reach, to the point where the storm drain connects with other drains or the outfall. If possible, the conduit depth should be set based on either the minimum depth of the inlet for the pipe size or the minimum cover for the pipe. The grade of the storm drain pipe should approximate the road grade if the conduit is a trunkline paralleling the roadway.

When the primary trunk line passes through a junction (structure) or when two or more secondary trunk lines converge and are carried forward in a single primary trunk line, it is preferable to match invert elevations of inflow pipes, with the invert elevation of the outflow pipe set at least 0.1' lower than the lowest inflow pipe invert elevation. This is applicable both for pipes of the same size or different sizes. The designer is cautioned to ensure that, when using the matching invert concept, the minimum cover requirements for the particular pipe sizes are met. The invert elevations of lateral pipes entering the junction that are significantly smaller in size than the trunk line can be established based on that required to provide the required flow capacity in the lateral pipe or to meet minimum cover requirements.

Matching crown line elevations in a junction may at times provide a slightly improved hydraulic grade line performance for one specified design storm. However, the preferred method of matching inverts provides a more efficient flow transition over a wide range of discharges. In areas where the grade of the storm sewer conduit is steeper than the finished grade profile, matching invert elevations, in lieu of matching crown line elevations, can also reduce the depth to which the pipe must be laid.

The rate of discharge at any point in the storm drain is not necessarily the sum of the inlet flow rates of all inlets above that section of storm drain. It is generally less than this total. The time of concentration is most influential and as the time of concentration grows larger, the rainfall intensity to be used for the design decreases. In some cases, where a relatively large drainage area with a short time of concentration is added to the system, the peak flow may be larger using the shorter time even though the entirety of all drainage areas are not contributing. In a long run of trunk main, without added areas, the same flow rate should be maintained throughout the entire length of the pipe.

*Rev. 7/13*
Occasionally, anomalies may occur where 1) there are highly impervious sections at the lower end of the watershed, and the total upstream area flows through the lower impervious area or, 2) a smaller less pervious area is a tributary to the primary watershed. These situations are analyzed by considering two anomalies:

**Anomaly 1** – Identify the discharge of any single previous pipe or inlet, based on its contributing CA and rainfall intensity (I) for corresponding flow time. Then compare these discharges with that calculated using the normal procedure, and adopt the larger for design.

**Anomaly 2** – Calculate the runoff to consider how much discharge each incoming pipe is contributing at the same time as the peak from the other incoming pipe/s. In this calculation, the intensity associated with the time of concentration from one pipe (x) is used and the associated discharge calculated for that pipe. Discharges from the other pipes (1 to n-1) are determined by the following equation:

\[
Q_x = S \{CA (1 \text{ to } n-1) x (T_{cx} / T_{cmax}) X I_x \} \quad (9.32)
\]

Where:
- \(Q_x\) is contributing discharge from incoming pipe, x,
- \(CA (1 \text{ to } n-1)\) is the product of the Area and the Runoff Coefficient for other incoming pipes (1 to n-1),
- \(T_{cx}\) is the flow time for the pipe, x,
- \(T_{cmax}\) is the maximum flow time for the pipe, and
- \(I_x\) is the rainfall intensity corresponding to \(T_{cx}\) for the pipe, x.

Add these contributing discharges to the discharge from the first pipe. Compare the largest value of these discharges with the discharge using normal procedure and adopt the larger discharge for design.

The prudent designer will be alert for unusual conditions and determine which time of concentration controls for each pipe segment.

For ordinary conditions, storm drains should be sized on the assumption that they will flow full or practically full under the design discharge but will not flow under pressure head. Actual tailwater conditions may cause the system to flow full, especially in low-lying areas. The Manning's formula is recommended for determining the initial size or capacity of the conduit. Hydraulic grade line calculations are then made to check the effects of tailwater conditions and energy losses through the system.

Refer to Table 9-2 for design frequencies for storm drain conduit. In locations such as depressed sections and underpasses where ponded water can be removed only through the storm drain system, a 100-year frequency storm should be considered to design the storm drain that drains the sag point.

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*Rev. 7/13*
9.4.8.2 Accumulation of Time in Conduit System

The Rational Method is used to determine peak discharges through the storm drain conduit network assuming the limitations of the method are reasonable. It is necessary to compute the incremental travel time through the system and accumulate this time to adjust the time of concentration that is used to compute the peak discharge for each consecutive segment of the conduit. The design rainfall intensity is based on the estimated accumulated time to the upstream node of the conduit run to be sized. Refer to Chapter 6, Hydrology, for the rainfall intensity equation for the Rational Method. Travel through each length of conduit is computed using the uniform flow velocity in the conduit. The velocity in the storm drain should be based on partial flow or full flow, whichever is applicable. The designer should also check for conditions that would create full flow in the system; such as, the hydraulic grade line.

9.4.8.3 100-Year Pipe at Sag Point

Where a storm drain system drains a major sag point in a depressed roadway section or a roadway section with concrete barriers, and ponded water on the roadway can only be removed through the storm drain, it shall be sized to accommodate the runoff from a 50-year frequency rainfall using the actual time of concentration.

At these locations and many others where excessive ponded water on the pavement could be reasonably expected to cause personal injury or significant property damage, the storm drain system shall be analyzed for a check storm event with a 100-year frequency, using the actual time of concentration. If the ponded depths of water on the pavement from the check storm event are determined to cause insignificant risk, the storm drain system may be used as originally designed. If the storm drain system fails to meet the check storm criteria, it must be re-designed to accommodate the runoff from the check storm event. New subdivision streets shall not have sag inlets where backwater conditions and the check storm event together exceed the rim by 0.25’ to protect the public.

This can be done by computing the bypass occurring at each upstream inlet during a 100-year rainfall and accumulating it at the sag point. The inlet at the sag point as well as the storm drain conduit leading from the sag point must be sized to accommodate this additional bypass within the criteria established. To design the conduit leading from the sag point, it may be helpful to convert the additional bypass created by the 100-year rainfall into an equivalent CA, which can be added to the design CA. This equivalent CA can be approximated by dividing the 100-year bypass by $I_{10}$ in the conduit at the sag point.

Some designers may want to design separate systems in order to prevent the above ground system from draining into a depressed area. This concept may be more costly but in some cases may be justified. Each case must be evaluated on its own merits and the impacts and risk of flooding a sag point assessed.

Rev. 7/16
9.4.8.4 Conduit Material Selection

Refer to Table A1 on sheet 18 of the Road & Bridge Standards PC-1 for a list of allowable materials for the storm drain pipe. The type of material is allowable dependent upon the functional classification of the roadway.

VDOT’s standard minimum finished height of cover for all pipes, except for those under entrances, shall be 2.0’ or \( \frac{1}{2} D \), whichever is greater. In cases in which these cover heights cannot be achieved, an absolute minimum height of 1.0’ will be allowed, only after all possible means has been exhausted by the designer and agreed to by the Department. The minimum finished height of cover for pipes under entrances is 9” (Sheet 1 of Road and Bridge Standards PC-1).

9.4.8.5 Hydraulic Capacity

The most widely used formula for determining the hydraulic capacity of storm drains for gravity and pressure flows is the Manning's formula and it is expressed by the following equation:

\[
V = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}
\]  
(9.15)

Where:

- \( V \) = Mean velocity of flow, fps
- \( n \) = Manning's roughness coefficient
- \( R \) = Hydraulic radius, ft = \( \frac{A}{P} \)
- \( A \) = Flow area, ft²
- \( P \) = Wetted perimeter, ft
- \( S \) = Slope of the energy grade line, ft/ft

In terms of discharge, by using the Continuity Equation \( Q = AV \), the above formula becomes:

\[
Q = VA = \frac{1.486}{n} AR^{\frac{2}{3}} S^{\frac{1}{2}}
\]  
(9.16)

Where:

- \( Q \) = Rate of flow, cfs
- \( A \) = Flow area, ft²

For storm drains flowing full, the above equations become:

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Rev. 7/14
\[ V = \frac{0.590}{n} D^{\frac{2}{3}} S^{\frac{1}{2}} \]
\[ Q = \frac{0.463}{n} D^{\frac{8}{3}} S^{\frac{1}{2}} \]  \hspace{1cm} (9.17)

Where:

\( D \) = Diameter of pipe, ft

Appendices 9C-23, 24, and 25 provide nomographs for solution of Manning's formula for full flow in circular storm drains. Appendix 9C-26 can be used to determine partial flow depth in storm drains through the ratio of various hydraulic elements, such as velocity \((V/V_{\text{full}})\) and discharge \((Q/Q_{\text{full}})\). The typical design process will use either a computer program or “Field’s Hydraulics Calculator” (circular slide rule) to determine the pipe size and grade.

### 9.4.8.6 Minimum Grades

All storm drains should be designed such that velocities of flow will not be less than 3 fps at design flow. For very flat grades the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. The storm drainage system should be checked to be sure there is sufficient velocity in all of the drains to deter settling of particles. Minimum slopes required for a velocity \((V)\) can be calculated using Equation 9.18 (Manning’s formula). A slope of 0.2% is considered the minimum slope for constructibility.

\[ S = \frac{0.453(nV)^2}{R^{\frac{4}{3}}} \]  \hspace{1cm} (9.18)

### 9.4.8.7 Maximum Grades

Slopes that incur uniform flow velocities in excess of 10 fps should be avoided because of the potential for abrasion. Slopes in excess of 16% 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. (See Special Design Drawing No. A-73 and MA-73 for Anchor Details for Concrete Pipe)

Corrugated pipe may be used on steep slopes in situations similar to those where shoulder slot inlets are proposed. Corrugated 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 the Allowable Pipe Type Table C in Standard PC-1 of the VDOT Road and Bridge Standards. (See VDOT’s Road and Bridge Standard PI-1, for Anchor Details for Corrugated Pipe)

In steeper terrain, large elevation differences can be accommodated using drop structures, otherwise known as “step down” manholes, to reduce the pipe gradient.

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Rev. 7/14
Where “step down” manholes are used, the Drainage Designer should provide any needed protection to prevent deterioration of the bottom of the manhole. This protection can be provided by the addition of a ½ inch steel plate in the bottom of the manhole, additional concrete, or other design approved by VDOT. This protection should be considered for use if the vertical difference between the inverts of the inlet pipe and outlet pipe is 4’ 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
- The size of the main pipes is 48” or greater in diameter (for circular pipe) or the hydraulic opening is 12 ft² or greater (for shapes other than circular)

For Step Down Manholes located in the VDOT right of way that require manhole bottom protection (e.g., additional concrete, steel plate, etc.), bottom protection designs shall be submitted for VDOT approval prior to fabrication.

Velocity dissipation is usually needed at the outlet of pipes on steep grades and the Drainage Designer should provide the type of dissipation appropriate for velocity, pipe size, discharge and site constraints.

9.4.8.8 Pipe on Radius
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 by one of the following methods:

- Open Joint Method - relatively long radius - using standard pipe and open joints a maximum of 25% of the spigot length.
- Bevel Method - mid range radius - using modified pipe with one side shorter than the other.
- Bevel and Open Joint Method - for shortest radius - a combination of the two methods above.

Bevel pipe is expensive to manufacture and somewhat difficult to install. It is generally more economical to use bend joints 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

Revised 7/19
The following table is a guideline for the minimum radius that should be obtainable using pipe from any manufacturer. A longer radius may be used as needed with the plan description denoting the method of obtaining the required radius. Certain manufactures may produce standard pipe joint lengths shorter than 8 feet. If so, a radius shorter than that shown in the table may be obtainable.

<table>
<thead>
<tr>
<th>Pipe Diameter</th>
<th>MINIMUM RADIUS BASED ON 8 foot PIPE JOINT LENGTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inch</td>
<td>Open Joint*</td>
</tr>
<tr>
<td>12</td>
<td>240</td>
</tr>
<tr>
<td>15</td>
<td>280</td>
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<td>96</td>
<td>730</td>
</tr>
<tr>
<td>108</td>
<td>615</td>
</tr>
</tbody>
</table>

* Maximum of 25% of spigot length

### 9.4.8.9 Minor Structure Excavation

(See Section 8.4.4 of the VDOT Drainage Manual for applicable criteria to Storm Drains)

### 9.4.8.10 Trenchless Applications

(See Section 8.3.6.11 of the VDOT Drainage Manual for applicable criteria to Storm Drains)

*Rev. 7/16
9.4.8.11 Extension of Existing Pipes

Existing pipes are to be extended with the same size and type of pipe that is in place. If end sections are required, then only the appropriate end section for the type of pipe (Standard ES-1, ES-2, or ES-3) is to be specified. Pipes for extension are to be so noted in the "Remarks" column of the Drainage Summary.

9.4.9 Hydraulic Grade Line

9.4.9.1 Introduction

This section describes the methodology for computing the hydraulic grade line. Section 9.5.6 presents the VDOT recommended method of calculation, which may be performed with design form LD-347 provided in Appendix 9B-3.

The hydraulic grade line (HGL) is the last important feature to be established relating to the hydraulic design of storm drains. This grade line aids the designer in determining the acceptability of the proposed system by establishing the elevations to which water will rise in the structures (inlets, manholes, etc.) along the system when the system is operating under for the recommended design frequency storm.

In general, if the HGL is above the crown of the pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the HGL is below the crown of the pipe, open channel flow calculations are appropriate. A special concern with storm drains designed to operate under pressure flow conditions is that inlet surcharging and possible access hole lid displacement can occur if the hydraulic grade line rises above the surface elevation. A design based on open channel conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As hydraulic calculations are performed, frequent verification of the existence of the desired flow condition should be made. Storm drain systems can often alternate between pressure and open channel flow conditions from one section to another.

The detailed methodology employed in calculating the HGL through the system begins at the system outfall with the tailwater elevation. If the outfall is an existing storm drain system, the HGL calculation must begin at the outlet end of the existing system and proceed upstream through the existing system, then upstream through the proposed system to the initial inlet. The same considerations apply to the outlet of a storm drain as to the outlet of a culvert.
9.4.9.2 Tailwater and Outfall Considerations

For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth. In determining the HGL, begin with the actual tailwater elevation or an elevation equal to 0.8 times the diameter of the outlet pipe (0.8D), whichever is higher.

When estimating tailwater depth on the receiving stream, the designer should consider the joint or coincidental probability of two events occurring at the same time. For the case of a tributary stream or a storm drain, its relative independence may be qualitatively evaluated by a comparison of its drainage area with that of the receiving stream. A short duration storm, which causes peak discharges on a small watershed, may not be critical for a larger watershed. Also, it may safely be assumed that if the same storm causes peak discharges on both watersheds, the peaks will be out of phase. To aid in the evaluation of joint probabilities, refer to Table 9-4 Joint Probability Analyses.

### Table 9-4 Joint Probability Analyses

<table>
<thead>
<tr>
<th>Watershed Area Ratio</th>
<th>Frequencies For Coincidental Occurrence</th>
<th>10-Year Design</th>
<th>100-Year Design</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Main Stream</td>
<td>Tributary</td>
<td>Main Stream</td>
</tr>
<tr>
<td>10 000 TO 1</td>
<td>1</td>
<td>10</td>
<td>2</td>
</tr>
<tr>
<td>100 TO 1</td>
<td>2</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>10 TO 1</td>
<td>5</td>
<td>10</td>
<td>25</td>
</tr>
<tr>
<td>1 TO 1</td>
<td>10</td>
<td>10</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>10</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>10</td>
<td>100</td>
</tr>
</tbody>
</table>

For a storm drain system, in the table above, the area ratio refers to the size ratio of the drainage area of the outfall channel (mainstream) to the drainage area of the storm drain system (tributary). Using this approach suggests that two possible conditions should be checked. For example, for an area ratio of 100:1, a 10-year design could be considered as the higher of a 10-year storm on the storm drain system with a 5-year tailwater or a 5-year storm on the storm drain system with a 10-year tailwater. There is an ongoing research project, NCHRP project 15-36, for joint probability analysis. This report is due in October 2009.

*Rev. 9/09*
9.4.9.3 Conservation of Energy and Energy Losses

When computing the hydraulic grade line, the calculations proceed from the system outfall upstream to each structure on the system. The calculation of the hydraulic grade line is based on conservation of energy as shown in Equation 9.19, which includes major and minor energy losses within the system.

\[ HGL_{us} = HGL_{ds} + h_f + h_m \]  

(9.19)

Where:

- \( HGL_{us} \) = Elevation of the hydraulic grade line at upstream structure, ft
- \( h_m \) = Summation of minor head losses such as junctions, bends etc., ft
- \( h_f \) = Friction head loss, ft
- \( HGL_{ds} \) = Elevation of hydraulic grade line at downstream structure, ft

Major head losses result from friction within the pipe. Minor head losses include those attributed to the following:

- Junctions
- Exits
- Entrances
- Bends in pipes
- Access holes
- Conflict pipes
- Plunging flow
- Expansions and contractions
- Appurtenances such as weirs, diverters, valves and meters

When computing the hydraulic grade line, the design discharge and the effective conduit velocity should be used in computing the minor head losses. If the HGL is below the crown line of the conduit, partial-flow or normal velocity of the conduit (based on the design discharge) should be used in computing the losses. If the HGL is above the crown line of the conduit, the full flow velocity (the design discharge divided by the cross sectional area of the conduit) should be used in computing the losses. Since it is not known where the HGL will fall (above or below the crown line of the conduit) the designer should first calculate the HGL assuming partial-flow or normal velocity in the conduits. If the computed HGL is below the crown line of the conduits, the assumption of normal velocity and the computed HGL is verified. If the computed HGL is above the crown line of the conduits, then full flow velocity should be assumed and the HGL recalculated.

Energy losses used in analyzing a storm drain system are indicated in Figure 9-7.
Figure 9-7 Use of Energy Losses in Analyzing a Storm Drain System
9.4.9.3.1  Conduit Friction Losses

The friction slope is the energy slope in feet per foot for that run. The friction loss is simply the energy gradient multiplied by the length of the run. Energy losses from pipe friction may be determined by rewriting the Manning’s equation with terms as previously defined:

\[ S_{fo} = 0.453 \frac{Q^2n^2}{A^2R^3} \]  \hspace{1cm} (9.20)

Then the head losses due to friction may be determined by the formula:

\[ H_f = S_{fo}L \]  \hspace{1cm} (9.21)

Where:

- \( H_f \) = Friction head loss, ft
- \( S_{fo} \) = Friction slope, ft/ft
- \( L \) = Length of outflow pipe, ft

9.4.9.3.2  Junction Losses

Junction losses are the sum of entrance, exit and bend losses. The total junction losses are given in Equation 9.22.

\[ H_t = H_i + H_o + H_\Lambda \]  \hspace{1cm} (9.22)

Where:

- \( H_t \) = Total junction losses, ft
- \( H_i, H_o, H_\Lambda \) = Entrance, exit, and bend losses, respectively, ft.

9.4.9.3.2.1  Entrance (Expansion) Losses

Equation 9.23 represents the entrance loss at a junction.

\[ H_i = K_e \frac{V_i^2}{2g} \]  \hspace{1cm} (9.23)

Where:

- \( H_i \) = Entrance head loss, ft
- \( V_i \) = Velocity in the inlet conduit, fps Where more than one inlet pipe is present, use the velocity of the one with the greatest momentum (Q*V).
- \( g \) = Gravitational acceleration, 32.2 ft/s^2
- \( K_e \) = Entrance loss coefficient (VDOT \( K_e = 0.35 \)).
### 9.4.9.3.2.2 Exit (Contraction) Losses

The exit loss, \( H_o \), is a function of the change in velocity in the outlet of the pipe as shown in Equation 9.24.

\[
H_o = K_o \frac{V_o^2}{2g} \tag{9.24}
\]

Where:

- \( H_o \) = Exit loss, ft
- \( V_o \) = Velocity in the outlet conduit, fps
- \( K_o \) = Exit loss coefficient (VDOT \( K_o = 0.25 \)) (A \( K_o \) value of 0.3 should be used when computing the loss at the initial inlet of the system)

### 9.4.9.3.2.3 Bend Losses

The loss at bends in the conduit system is shown in Figure 9-8 and is computed with Equation 9.25. Bend losses are applied to a junction in which the outgoing conduit is at an angle greater than 0° to the incoming conduit. The sharper the bend (approaching 90°) the more severe the energy loss becomes. Conduits should not be designed to have bend angles greater than 90°. Where more than one culvert enters a junction at an angle, the \( H_\Delta \) should be figured on all bends and the largest one used as a bend loss.

\[
H_\Delta = K \frac{V_i^2}{2g} \tag{9.25}
\]

Where:

- \( H_\Delta \) = Headloss at a bend, ft
- \( K \) = Bend loss coefficient
- \( V_i \) = Velocity in the inlet conduit, fps

---

Figure 9-8 **Deflection Angle Between Inflow and Outflow Pipes**

---

Rev. 9/09
VDOT recommended values of K for change in direction of flow in laterals can be found on design form LD-347, Appendix B-3. Figure 9-9 shows a graphical representation of the bend loss coefficient (K) to change in direction of flow lateral.

\[ H_L = K \cdot \frac{V_1^2}{2g} \]

- \( V_1 \) = Velocity of flow in lateral in ft.p.s.
- \( g \) = Acceleration due to gravity, 32 ft/sec/sec
- \( H_L \) = Feet of head lost in Jct. due to change in direction of lateral flow
- \( K \) = Factor from graph

Figure 9-9 Losses in Junction Due to Change in Direction of Flow Lateral

Rev. 9/09
9.4.9.3.3 Plunging Losses

Plunging losses are applied if the surface inlet inflow is 20% or more of the total flow through the junction or a lateral conduit enters a junction with its invert elevation above the crown line elevation of the outgoing trunkline conduit and the flow from the lateral is 20% or more of the total flow through the junction. Plunging flow losses increase the total junction loss by 30% as defined by Equation 9.26.

When comparing discharges with significant differences in TC, use CA values for comparison.

\[ H_t = 1.30(H_o + H_t + H_\Delta) \]  
\[ (9.26) \]

9.4.9.3.4 Inlet Shaping (IS-1)

Inlet shaping refers to how the invert is shaped within the access hole to provide smooth flow through the structure. Applying VDOT Standard IS-1, inlet shaping, reduces the total junction losses by 50% as defined by Equation 9.27 if there are no plunging losses or Equation 9.28 if there are plunging losses.

\[ H_t = 0.50(H_o + H_t + H_\Delta) \] (Where no plunging losses occur)  
\[ (9.27) \]

or

\[ H_t = (0.50)(1.30)(H_o + H_t + H_\Delta) \] (Where plunging losses occur)  
\[ (9.28) \]

VDOT Standard IS-1, inlet shaping, should be specified in all structures where a change of flow direction occurs, intersecting flows occur and any other location where there is concern with continuity of flow through a structure.

9.4.9.3.5 Total Headlosses

The total headlosses are computed by adding the conduit friction loss to the total junction losses as represented by Equation 9.29.

\[ H = H_f + H_t \]  
\[ (9.29) \]

Where:

\[ H = \text{Total headloss, ft} \]
9.4.9.4 Use of Alternate Pipe Materials

Where alternative pipe materials are allowed for the storm sewer system, the highest VDOT approved Manning’s “n” value for the allowable pipe materials (typically 0.013) shall be used in the design process for determining the required pipe size and the initial hydraulic grade line elevations. Where the initial hydraulic grade line analysis determines the elevations of the hydraulic grade line to be of a critical nature (e.g., near, at or exceeding the top or throat elevation of the manholes, junction boxes, drop inlets and other such structures), a “check” hydraulic grade line for the storm sewer system shall be computed using the lowest VDOT approved Manning’s “n” value for the allowable pipe materials in order to ensure the adequacy of the storm sewer system. For example, if Concrete, PVC (Polyvinylchloride), Polyethylene (PE) Corrugated Type S and Polymer Coated Corrugated Steel Double Wall are allowable pipe materials for the storm sewer system and, using a Manning’s “n” value of 0.013 (the highest for the allowable pipe materials), the hydraulic grade line elevations are near the top or throat elevation of the structures, a check hydraulic grade line using a Manning’s “n” value of 0.011 (the lowest for the allowable pipe materials) would be developed to determine if the hydraulic grade line elevations remain below the top or throat elevations of the structures. If the check hydraulic grade line analysis indicates a concern for the adequacy of the storm sewer system, then appropriate changes to the storm sewer system shall be made to ensure its adequacy with regards to the hydraulic grade line elevations.

The roughness coefficient for each pipe material represents the value for newly installed pipe and has been determined by laboratory tests with an adjustment factor to compensate for the additional losses experienced in actual field installations. Values may be higher for existing pipe installations that have experienced some deterioration.*

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>ROUGHNESS COEFFICIENT (n)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Pipe</td>
<td>0.013</td>
</tr>
<tr>
<td>PVC (Polyvinylchloride)</td>
<td>0.011</td>
</tr>
<tr>
<td>Storm Drain Pipe (Smooth Interior)</td>
<td></td>
</tr>
<tr>
<td>Polyethylene Double Wall (Type S) (Smooth Interior)</td>
<td>0.012</td>
</tr>
<tr>
<td>Steel or Aluminum Spiral Rib Pipe</td>
<td>0.014</td>
</tr>
<tr>
<td>Polymer Coated Corrugated Steel Double Wall (Smooth Interior)</td>
<td>0.013</td>
</tr>
</tbody>
</table>

* Rev. 7/14
The allowable pipe type criteria for culverts and storm sewers are presented in Standard PC-1 in the Road and Bridge Standards. The allowable pipe types are those that provide for a 75-year service life for pipes under the roadways and facilities that are constructed, funded or will ultimately be maintained by the Department.

A project specific Allowable Pipe Type Table for both culvert pipe and storm sewer pipe (as appropriate) is to be shown at the end of the Drainage Summary for every project (C, M, and N).

The types of allowable pipe for each project will vary with classification of roadway and geographic location within the State. Numerous combinations of pipe types may be used on a particular project.

It will be necessary to formulate a table(s) to specifically fit each project based upon the various roadway classifications involved and location of the project.

The Contractor has the option to install any of the allowable materials noted in the project specific Allowable Pipe Type Tables, unless otherwise noted on the plans.

Chapter 3 includes an example of Allowable Pipe Type Tables for a culvert and a storm drain scenario, relative to a Route 64 project in York County.

*Rev. 7/14*

‡ Represents general value. May vary with size and shape of corrugations.
9.5 Design Procedures and Sample Problems

The typical design process would perform all of the calculations in Section 9.5 by the use of computer programs with the possible exception of the hydraulic grade line procedure.

9.5.1 Design Documentation

The following items should be included in the drainage documentation.

- Computation forms for inlets (LD-204), conduits (LD-229), hydraulic grade lines (LD-347)
- Drainage area map
- Information concerning outfalls and tailwaters, existing storm drains, and other design considerations
- A storm drain schematic
- Output from acceptable computer programs.

9.5.2 Spread Calculations

9.5.2.1 Uniform Cross Slope Procedure

**Condition 1:** Find spread, given gutter flow, Q.

**Step 1:** Determine the following parameters:

- Longitudinal slope (S)
- Cross slope ($S_x$)
- Manning's roughness coefficient (n)

**Step 2:** Compute spread ($T$), using Equation 9.2.

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

Alternatively, use the chart contained in Appendix 9C-2. The procedure for using this chart is provided below.

**Step 3:** Draw a perpendicular line from gutter flow (Q) to longitudinal or gutter slope (S).

**Step 4:** Draw a perpendicular line from longitudinal or gutter slope (S) to cross slope ($S_x$).

**Step 5:** Draw a perpendicular line from cross slope ($S_x$) to intersect with Width (x). Spread ($T$) will be the value at the intersection with Width (x).

* Rev. 9/09
Condition 2: Find gutter flow \( (Q) \), given spread \( (T) \).

This procedure is similar to Condition 1. Solve Equation 9.1 for discharge, \( Q \), or work the procedure for Condition 1 in reverse.

\[
Q = \frac{0.56}{n} S_x^{1/2} S^{5/3} T^{8/3}
\]

**9.5.2.1.1 Uniform Cross Slope Sample Problem**

**Condition 1:** Find spread given the following:

**Step 1:** Determine the following parameters.

Gutter flow, \( Q = 5 \text{ cfs} \)
Longitudinal slope, \( S = 0.0035 \text{ ft/ft} \)
Cross slope, \( S_x = 0.0208 \text{ ft/ft} \)
Manning's roughness coefficient, \( n = 0.015 \)

**Step 2:** Compute Spread \( (T) \), using Equation 9.2.

\[
T = 1.243(Qn)^{\frac{3}{8}} S_x^{-\frac{5}{8}} S^{-\frac{3}{16}}
\]

\[
= 1.243 \times (5 \times 0.015)^{\frac{3}{8}} \times 0.0208^{-\frac{5}{8}} \times 0.0035^{-\frac{3}{16}}
\]

\[
= 15.3 \text{ ft}
\]

Refer to the chart in Appendix 9C-2 to view the graphical solution.

**Condition 2:** Find the gutter capacity given the following:

**Step 1:** Determine the following parameters.

Allowable spread, \( T = 14' \)
Longitudinal slope, \( S = 0.0035 \text{ ft/ft} \)
Cross slope, \( S_x = 0.0208 \text{ ft/ft} \)
Manning's roughness coefficient, \( n = 0.015 \)

**Step 2:** Compute gutter flow \( (Q) \), using Equation 9.1.

\[
Q = \frac{0.56}{n} S_x^{1/2} S^{5/3} T^{8/3}
\]

\[
= \frac{0.56}{0.015} \times 0.0208^{\frac{5}{3}} \times 0.0035^{\frac{1}{2}} \times 14^{\frac{8}{3}}
\]

\[
= 3.96 \text{ cfs} \text{ (Say 4.0 cfs)}
\]
9.5.2.2 Composite Gutter Sections Procedure

The capacity of a composite section at an allowable spread can be calculated using Equation 9.1 by breaking the problem into three triangular sections; however, it may be more expedient to use the appropriate nomograph contained in Appendices 9C-2 through 9C-6.

**Condition 1:** Find spread, given gutter flow.

**Step 1:** Determine input parameters, including longitudinal slope ($S$), cross slope ($S_x$), gutter pan width ($W$), Manning's $n$, and gutter flow ($Q$).

**Step 2:** Draw a line from gutter flow ($Q$) to longitudinal or gutter slope ($S$).

**Step 3:** Draw a line from longitudinal or gutter slope ($S$) to roadway cross slope ($S_x$).

**Step 4:** Draw a perpendicular line from cross slope ($S_x$) to intersect with Width ($x$). Spread ($T$) will be the value at the intersection with Width ($x$).

**Condition 2:** Find gutter flow, given spread.

**Step 1** Determine input parameters, including spread ($T$), cross slope ($S_x$), longitudinal slope ($S$), gutter pan width ($W$), and Manning's $n$.

**Step 2** Perform the procedure given in Condition 1 in reverse.

**Step 3** The gutter flow is the point at which the line crosses the “Discharge” axis.

Note: The chart contained in Appendix 9C-7 can also be used to calculate the spread in a composite gutter section.

9.5.2.2.1 Composite Gutter Sample Problem

**Condition 1** Using the chart in Appendix 9C-3, determine Spread ($T$).

**Step 1** Determine input parameters.

- Longitudinal slope ($S$) = 0.04 ft/ft
- Cross slope ($S_x$) = 0.0208 ft/ft
- Depressed section width ($W$) = ft
- Manning's $n$ = 0.015
- Gutter flow ($Q$) = 3.8 cfs

Note: Appendix 9C-3 is only applicable for gutter pan width, $W = 2'$.
Steps 2 and 3: Using Appendix 9C-3, draw perpendicular lines using the information contained in Step 1 and using the procedure for Condition 1 in Section 9.5.2.2.

Step 4: Determine the Spread (T).

\[ T = 7.5' \]

9.5.2.3 Temporary Barrier Wall

During construction of a roadway project, it is sometimes necessary to account for the drainage at a temporary barrier wall. The most common barrier walls recognized by the Department are the Standard MB-7D PC, and the MB-11A, whereby the concrete barrier is configured in 10’ or 20’ lengths, with a 31” slot.

The Engineer should perform spread calculations based on a 4 in/hr rainfall event, assuming that the slot flow be reduced to 75% of the equation value, to account for a theoretical 25% blockage.

9.5.3 Inlet Spacing Procedure

In order to design the location of the inlets for a given project, information such as a layout or plan sheet suitable for outlining drainage areas, road profiles, typical cross sections, grading cross sections, superelevation diagrams and contour maps are necessary. The inlet computation sheet, LD-204, Appendix 9B-1, should be used to document the computations. The procedure follows:

Step 1: Locate high points (crests) and low points (sags) and mark on the plans the location of inlets, which are necessary even without considering any specific drainage area. These would include sags with flankers, curb returns from roads draining onto an intersection, and superelevation transitions prior to cross slope reversal when the cross slope is 1%.

Step 2: Starting at the high point, work towards the low point.

Step 3: From the drainage map, select a trial drainage area approximately 300’ to 500’ below the high point and delineate the area including any area that may come over the curb (offsite area). Where practical, large offsite areas should be intercepted before reaching the roadway.

Step 4: Indicate the proposed inlet number in Col. 1 and in Col. 2 the type of inlet. Col. 3 will be filled in after the inlet is sized. In Col. 4 show the station and reference the baseline.

Step 5: Compute the drainage area in acres and enter in Col 5.

*Rev. 7/16*
Step 6: Determine the C-value for each land use as described in Chapter 6 and enter in Col. 6.

Step 7: Calculate the product C and A for each land use and enter in Col. 7.

Step 8: Sum the CA products and enter in Col. 8.

Step 9: Depending on the classification of roadway and type of inlet, determine time of concentration ($t_c$) based on the criteria defined in Table 9-1. Determine the rainfall intensity ($i$) based on the classification of roadway as defined in Table 9-1 and enter in Col. 9.

Step 10: Calculate discharge ($Q$) by multiplying Col. 8 and Col. 9 and enter in Col. 10. The discharge ($Q$) in Col. 10 is also entered in the total discharge ($Q_T$) Col. 12 for the first inlet.

Step 11: Determine gutter slope or longitudinal slope ($S$) and cross slope ($S_x$) and enter in Col. 13 and 14, respectively.

Step 12: Using the appropriate Appendix 9C-2 through 9C-7, or Equation 9.2, determine spread ($T$) and enter in Col. 15.

If spread ($T$) exceeds the allowable spread, based on the functional classification of roadway, the designer should consider reducing the drainage area to the inlet. This eliminates the need to account for carryover discharge when designing the next downstream inlet unless the designer allows carryover discharge and/or the designer is required to evaluate the check storm. In that instance, the designer should proceed to the curb inlet on grade sizing procedure in Section 9.5.3.1.

If the designer is sizing a sag inlet, refer to the curb inlet in sag sizing procedure in Section 9.5.3.2.

9.5.3.1 Curb Inlet on Grade Sizing Procedure

This procedure uses the same computations as described under Section 9.5.2, Inlet Spacing Procedure. The results can be entered in LD-204, Appendix 9B-1.

Step 1: Repeat Steps 5 through 12 from the inlet spacing procedure presented in Section 9.5.3.

Step 2: Determine the gutter pan width ($W$) and enter in Col. 16.

Step 3: Compute the ratio of flow in the depressed section ($W$) to the spread ($T$), $\frac{W}{T}$ and enter in Col. 17.

Step 4: Determine the gutter pan cross slope ($S_w$) and enter in Col. 18. For VDOT standard gutter pan, this is 1 in/ft (0.083 ft/ft). Compute the ratio of $S_w/S_x$ and enter in Col. 19.
Step 5: Determine the ratio of frontal flow to total gutter flow \((E_o)\) using Appendix 9C-8 and enter in Col. 20.

Step 6: Compute the total inlet depression \((a)\) and enter in Col. 21.
\[ a = (S_w - S_x)12W + \text{Local Depression} \]

Step 7: Compute the cross slope of the gutter pan including local depression \((S'_w)\) and enter in Col. 22.
\[ S'_w = \frac{a}{12W} \]

Step 8: Compute the equivalent cross slope \((S_e)\), using Equation 9.5, and enter in Col. 23.
\[ S_e = S_x + S'_w (E_o) \]

Step 9: Compute the required inlet length \((L_T)\) for total interception using Appendix 9C-17 or Equation 9.4 and enter into Col. 24.

   If no bypass flow is allowed, round the required inlet length \((L_T)\) up to a nominal dimension of at least \(L_R\). Refer to Road and Bridge Standards to determine nominal lengths available for curb opening inlets. The inlet sizing is complete and the designer can proceed to the next inlet by repeating Steps 1 through 9. If bypass flow is to be considered, proceed to Step 10.

Step 10: Determine the inlet length to be specified \((L)\) to be used and enter in Col. 25.

Step 11: Compute \(\left(\frac{L}{L_T}\right)\) and enter in Col. 26.

Step 12: Determine capture efficiency \((E)\) using Appendix 9C-18 or Equation 9.8 and enter in Col. 27.

Step 13: Compute the flow intercepted \((Q_i)\), using Equation 9.9a, by multiplying Col. 12 and Col. 27 and enter in Col. 28.

Step 14: Calculate bypass flow or carryover flow \((Q_b)\), using Equation 9.9b, by subtracting Col. 28 from Col. 12 and enter in Col. 29.

Step 15: The carryover flow \((Q_b)\) from the first inlet is entered in Col. 11 for the next downstream inlet.

Step 16: Repeat Steps 1 through 16 for each successive inlet until analyzing the sag inlet. Note: When computing the total gutter flow \((Q_T)\), add the carryover flow \((Q_b)\) from the previous upstream inlet.
9.5.3.1.1 Curb Inlet on Grade Sample Problem

Find: The curb inlet length required for 100 % interception and what the bypass flow would be if a 6’ slot were used.

Step 1: Repeat Steps 5 through 15 from the inlet spacing procedure presented in Section 9.5.3.

Given: \( Q = 2 \text{ cfs}, n = 0.015, S = 0.01 \text{ ft/ft}, S_x = 0.0208 \text{ ft/ft}, W = 2’, S_w = 0.0833 \text{ ft/ft}, \text{ local inlet depression} = 2” \)

Use Appendix 9C-3 to find Spread (T) = to 7.6’

Step 2: Determine the ratio of flow in the depressed section (W) to the Spread (T), \( \frac{W}{T} \).

\[ \frac{W}{T} = \frac{2}{7.6} = 0.26 \]

Step 3: Determine the ratio of frontal flow to total gutter flow (\( E_o \)) using Appendix 9C-8 and enter in Col. 21.

\[ \frac{S_w}{S_x} = \frac{0.0833}{0.0208} = 4 \]

Using Appendix 9C-8, \( E_o = 0.69 \)

Step 4: Compute the total inlet depression (a) and enter in Col. 22.

\[ a = (S_w - S_x)12W + \text{Local Depression} \]
\[ a = (0.0833-0.0208)(12)(2)+2 = 3.5” \]

Step 5: Compute the cross slope of the gutter pan including local depression (\( S’_w \)) and enter in Col. 23.

\[ S’_w = \frac{a}{12W} = \frac{3.5}{12(2)} = 0.146 \]

Step 6: Compute the equivalent cross slope (\( S_e \)), using Equation 9.5, and enter in Col. 24.

\[ S_e = S_x + S’_w (E_o) \]
\[ S_e = 0.0208 + (0.146)(0.69)= 0.121 \text{ ft/ft} \]

Step 7: Compute the required inlet length (\( L_T \)) for total interception using Appendix 9C-17 or Equation 9.4 and enter into Col. 25.
If no bypass flow is allowed, round the required inlet length (L_T) up to a nominal dimension of at least L_R. Refer to Road and Bridge Standards to determine nominal lengths available for curb opening inlets. The inlet sizing is complete and the designer can proceed to the next inlet by repeating Steps 1 through 7. If bypass flow is to be considered, proceed to Step 8.

$$L_T = \frac{0.6Q^{0.42}S^{0.3}}{(nS_e)^{0.6}}$$

$$L_T = \frac{0.6(0.6)^{0.42}(0.01)^{0.3}}{[0.015(0.121)]^{0.8}} = 8.9\text{ ft}$$

The minimum length for 100% interception would be $L_T = 8.9'$

**Step 8:** Determine the inlet length to be specified (L) to be used and enter in Col. 26. In this instance the design would round up to the nearest nominal inlet length as provided by the Road and Bridge Standards.

If no bypass flow were allowed, a standard length of 10' would be appropriate.

Using an inlet length of 6' would require proceeding to Step 9.

**Step 9:** Compute \( L \) and enter in Col. 26.

**Step 10:** Determine capture efficiency \( (E) \) using Appendix 9C-18 or Equation 9.8 and enter in Col. 27.

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

$$E = 1 - \left(1 - \frac{6}{8.9}\right)^{1.8} = 0.87$$

**Step 11:** Compute the flow intercepted \( (Q_i) \) by multiplying Col. 12 and Col. 27 and enter in Col. 28.

$$Q_i = EQ_T$$

$$Q_i = 0.87 \times 2 = 1.74\text{ cfs}$$

**Step 12:** Calculate bypass flow or carryover flow \( (Q_b) \), using Equation 9.9b, by subtracting Col. 28 from Col. 12 and enter in Col. 29.

$$Q_b = QT - Q_i$$

$$Q_b = 2 - 1.74 = 0.26\text{ cfs}$$

**Step 13:** The carryover flow \( (Q_b) \) from the first inlet is entered in Col. 11 for the next downstream inlet.

*Chapter 9-57 of 70*
9.5.3.2 Curb Inlet in Sag Sizing Procedure

Step 1: Determine the allowable depth of ponding (d) and enter in Col. 30. This is the depth above the undepressed gutter line to the water surface associated with the allowable spread and should be at least 1" below the top of curb.

Step 2: Determine the height of the curb inlet opening (h) and enter in Col. 31. Calculate the ratio of d/h and enter in Col. 32.

If \( \frac{d}{h} < 1.2 \), the inlet is in weir control.

If \( \frac{d}{h} > 1.2 \), the inlet is transitioning to orifice control and design would proceed to Step 4.

Step 3: Compute the required length \( (L_R) \) when the inlet is in weir flow by rearranging Equation 9.10 as follows and enter in Col. 25, then proceed to Step 5.

\[
L_R = \frac{Q}{C_w d^{1.5}} - 1.8W
\]

\[
Q_i = C_w (L+1.8 W) d^{1.5}
\]

(9.30)

Where:

- \( Q \) = Total flow reaching inlet, cfs
- \( C_w \) = Weir coefficient, use 2.3.
- \( d \) = Allowable ponding depth, ft
- \( W \) = Inlet depression width, ft

Step 4: Compute the required length \( (L_R) \) when the inlet is in orifice flow by rearranging Equation 9.12 as follows and enter in Col. 25, then proceed to Step 5.

\[
L_R = \frac{Q}{C_o d \sqrt{2gd_o}}
\]

\[
Q_i = C_o A [2qd_o]^5
\]

\[
= C_o (d L_R) [2qd_o]^5
\]

(9.31)

* Rev 9/09
Where:

- $L_R$: Required length of opening
- $Q$: Total flow reaching inlet, cfs
- $C_o$: Orifice coefficient = 0.67
- $d$: Depth of opening, ft. The depth will vary slightly with the inlet detail used.
- $g$: Acceleration due to gravity = 32.2, ft/s²
- $d_o$: Effective head at the centroid of the inlet opening, ft

Step 5: Select a standard inlet length ($L$) that is greater than the required length ($L_R$).

Step 6: If the area behind the inlet is prone to flooding or there is the potential for property damage, a check storm intensity of $i = 6.5$ in/hr will be used to evaluate all inlets down to the sag inlet in question. If the inlet can handle the check storm without flooding then the previous design need not be changed. However, if there is flooding, it may be necessary to double the inlet size. Refer to Table 9-1.

### 9.5.3.2.1 Curb Inlet in Sag Sizing Sample Problem

Find: The required inlet length assuming a factor of safety of 2.

Given: $Q = 3$ cfs, allowable spread = 8', $S_x = 0.0208$ ft/ft, Inlet depression = 2”, Standard length increment =2', $W = 2'$, Curb height = 6”, slot height = 5”

Step 1: Determine the allowable depth of ponding ($d$) and enter in Col. 30. This is the depth above the undepressed gutter line to the water surface associated with the allowable spread and should be at least 1” below the top of curb.

The depth to 1” below the top of curb = 6 - 1 = 5” (0.42’).

The depth of allowable ponding = $T(S_x)$

$T(S_x) = 8(0.0208) = 0.17$ ft

Depth of ponding is less than 1” below the top of curb (0.34<0.42)

---

Rev. 9/09
Step 2:  Determine the height of the curb inlet opening \((h)\) and enter in Col. 31. Calculate the ratio of \(d/h\) and enter in Col. 32.

If \(\frac{d}{h} < 1.2\), the inlet is in weir control

If \(\frac{d}{h} > 1.2\), the inlet is transitioning to orifice control and design would proceed to Step 4

\[
\frac{d}{h} < 1.2, \quad \frac{4}{5} = 0.80
\]

0.80 < 1.2, therefore proceed to Step 3

Step 3:  Compute the required length \(\left(L_R\right)\) when the inlet is in weir flow by rearranging Equation 9.10 as follows and enter in Col. 25, then proceed to Step 5.

\[
L_R = \frac{Q}{C_w d^{1.5}} \cdot 1.8W
\]

\[
= \frac{3}{2.3(0.34)^{1.5}} \cdot 1.8(2)
\]

\[= 2.97 \text{ ft}\]

Step 4:  Compute the required length \(\left(L_R\right)\) when the inlet is in orifice flow by rearranging Equation 9.12 as follows and enter in Col. 25, then proceed to Step 5.

\[
L_R = \frac{Q}{C_o d \sqrt{2gh}}
\]

Step 5:  Select a standard inlet length \((L)\) that is greater than the required length \(\left(L_R\right)\).

Using a factor of safety of 2, the required length \(\left(L_R\right)\) is 5.95". Use an actual inlet length \((L)\) of 6".

Step 6:  If the area behind the inlet is prone to flooding or there is the potential for property damage, a check storm intensity of \(i = 6.5 \text{ in/hr}\) will be used to evaluate all inlets down to the sag inlet in question. If the inlet can handle the check storm without flooding then the previous design need not be changed. However, if there is flooding, it may be necessary to double the inlet size. Refer to Table 9-1.
9.5.4 Grate in Sag Procedure

Step 1: Choose a grate and determine standard dimensions to use as a basis for calculations. These dimensions usually include open area and perimeter.

Step 2: Determine an allowable ponding depth (d) for the inlet location. If used in a median ditch, the depth should be the medium depth minus a freeboard or the height of the backup berm. The designer should consider the available depth when evaluating median ditches for roads in superelevation.

Step 3: Determine the capacity of a grate inlet operating in weir control using Appendices 9C-12 through 9C-16 or Equation 9.13. Under weir conditions, the grate perimeter controls the capacity. To account for clogging, assume one-half of the perimeter of the inlet is available.

Step 4: Determine the capacity of a grate inlet operating under orifice control using Appendices 9C-12 through 9C-16 or Equation 9.14. Under orifice conditions, the grate area controls the capacity. To account for clogging, assume one-half of the grate opening area is available.

Step 5: Compare the calculated capacities from Steps 3 and 4 and choose the lower value as the design capacity.

9.5.4.1 Grate in Sag Sample Problem

Determine the capacity of a DI-7 inlet with a Type I grate.

Given: Allowable depth of ponding above grate = 2'

Step 1: Choose a grate and determine standard dimensions to use as a basis for calculations. These dimensions usually include open area and perimeter.

Using Appendix 9C-14, determine:

Grate open area = 6 ft²
Grate perimeter = 12.8'

Step 2: Determine an allowable ponding depth (d) for the inlet location. If used in a median ditch, the depth should be the medium depth minus a freeboard or the height of the backup berm. The designer should consider the available depth when evaluating median ditches for roads in superelevation

Allowable depth of flow above grate = 2'

Step 3: Determine the capacity of a grate inlet operating in weir control using the Appendices 9C-12 through 9C-16 or Equation 9.13. Under weir conditions, the grate perimeter controls the capacity. To account for clogging, assume one-half of the perimeter of the inlet is available.

---

Rev. 9/09
Step 4: Determine the capacity of a grate inlet operating under orifice control using the Appendices 9C-12 through 9C-16 or Equation 9.14. Under orifice conditions, the grate area controls the capacity. To account for clogging, assume one-half of the grate opening area is available.

\[ Q_i = C_o A (2gd)^{0.5} \]

\[ = 0.67(6)(0.5)[2(32.3)(2)]^{0.5} \]

\[ = 23 \text{ cfs} \]

Step 5: Compare the calculated capacities from Steps 3 and 4 and choose the lower value as the design capacity.

Inlet capacity for the DI-7 is 23 cfs

Compare these results using the Appendices 9C-12 through 9C-16. Note that Appendix 9C-14 is specifically used for a DI-7, with a Type I grate. This inlet type is generally used in depressed roadway medians.

9.5.5 Storm Drain Conduit Design Procedure

The design process must begin at the most upstream conduit and proceed downstream to the outfall. The sizes of conduits for all branches upstream of a conduit run must be evaluated before proceeding downstream.

The following procedure refers to the tabulated form LD-229 “Storm Drain Design Computations” in Appendix 9B-2. Note that when the Engineer utilizes other software forms to show storm drainage calculations for VDOT review, the forms shall contain similar data to that of LD-229, and be landscape oriented with legible font and page size.

Step 1: Identify the upstream and downstream structures (inlets, manholes, etc.) in Col. 1 and 2.

Step 2: Enter the drainage area for the inlet at the upstream end in Col. 3.

Step 3: Enter the runoff coefficient in Col. 4 for the drainage area identified in Step 2.

Step 4: Multiply the runoff coefficient from Col. 4 with the drainage area from Col. 3 to determine the incremental CA value and place in Col. 5.

Rev. 7/16
Step 5: If the conduit is to convey flow from a source in addition to that identified in Step 2, add the additional CA value to what was determined in Step 4 to yield the accumulated CA and enter in Col. 6.

Step 6: Determine the longest travel time by using the inlet time of concentration from a previous upstream inlet plus the intervening pipe flow time (Step 13) or the time of concentration for the localized inflow intercepted by the inlet at the upstream end of a run of pipe. Enter this time in Col. 7.

Step 7: Determine the rainfall intensity \( (i) \) based on the longest time identified in Step 6 and place in Col. 8. Refer to Chapter 6 for Intensity-Duration-Frequency information.

Step 8: Multiply the rainfall intensity \( (i) \) established in Col. 8 with the accumulated CA in Col. 6 to determine the design discharge \( (Q) \) in Col. 9. Verify if any of the following conditions exist:

a. Is the design discharge of any single previous pipe or inlet, based on its contributing CA and rainfall intensity \( (i) \) for corresponding flow time, more than the design discharge calculated above?

b. Is the sum of (1) discharge from any one pipe and (2) the discharges for the other incoming pipes corresponding to the flow time of this pipe more than the design discharge calculated above?

Enter the largest value of above discharges in Col. 9.

Step 9: Determine the minimum conduit slope and diameter and enter in Col. 13 and 14. Compute the invert elevations of the upstream and downstream ends of the conduit. If the designer finds it more convenient to work in percent (ft/100 ft.) as opposed to ft/ft, the unit designation for Col. 13 should reflect percent.

Step 10: Determine pipe length by measuring the out-to-out distance between structures from the plan sheet and enter in Col. 12.

Step 11: With diameter and slope determined, invert elevations for the upstream and downstream ends of a pipe segment are entered in Col. 10 and 11. If possible, the invert elevations should be based on either the minimum depth of the inlet or the minimum cover for the conduit. The minimum slope of the conduit should approximate the slope of the road grade if the conduit is a trunk line or parallel to the highway.

Step 12: Determine the capacity of the conduit using Manning’s Equation or Appendix 9C-23, 9C-24, or 9C-25, and enter in Col. 15. The calculated pipe capacity should exceed the design discharge (Col. 9) identified in Step 8. If the capacity is too low, choose a larger conduit diameter or increase the slope and recompute the capacity.

Rev. 7/13
Step 13: Determine the velocity of flow in the pipe based on the design discharge and actual pipe slope and enter in Col. 16. Partial flow velocity should be used if pipe is not flowing full.

Step 14: Determine the flow time through the conduit by dividing the conduit length Col. 17 with the velocity (Col. 16) and enter in Col. 17. Be careful to ensure consistent time units.

Step 15: Add the travel time through the pipe to the inlet time used in Col. 7 and note this value for possible use in Step 6 for the next conduit run downstream. Determine the time of concentration for the next downstream inlet.

Step 16: Repeat Steps 1 to 13 for subsequent conduit runs downstream.

9.5.6 Hydraulic Grade Line Procedure

All head losses in a storm drainage system should be considered in computing the hydraulic grade line to determine the water surface elevations under design conditions in the various inlets, catch basins, manholes, junction boxes, etc. The hydraulic grade line should be computed for all storm drain systems using the design frequency discharges. At underpasses and roadway sections, where the only relief for ponded water is through the storm drain system, the hydraulic grade line should be checked for the 100-year storm event.

The general assumption for hydraulic grade line is that of outlet control. That is, subcritical flow conditions exist and the head losses are determined from downstream to upstream. Hydraulic control is a set water surface elevation from which the hydraulic calculations begin. The head losses are calculated beginning from the control point to the first junction and the procedure is repeated for the next junction. The VDOT method of computation is recommended and the computations may be tabulated on VDOT Form LD-347, Appendix 9B-3, using the following procedure (Note that when the Engineer utilizes other software forms to show the hydraulic gradient calculations for VDOT review, the forms shall contain similar data to that of LD-347, and be landscape oriented and with legible font and page size):

Step 1: Enter in Col. 1 the station for the junction immediately upstream of the outflow pipe. Hydraulic grade line computations begin at the outfall and are worked upstream taking each junction into consideration.

Step 2: Enter in Col. 2 the outlet water surface elevation, tailwater, if the outlet will be submerged during the design storm or 0.8 times diameter (0.8D) plus the invert out elevation of the outflow pipe whichever is greater.

Step 3: Enter in Col. 3 the diameter ($D_o$) of the outflow pipe.

Step 4: Enter in Col. 4 the design discharge ($Q_o$) for the outflow pipe.

\[ \text{Rev. 7/16} \]
Step 5: Enter in Col. 5 the length ($L_o$) of the outflow pipe.

Step 6: Enter in Col. 6 the friction slope ($S_{fo}$) in ft/ft of the outflow pipe. This can be determined by using Equation 9, pipe capacity charts in Chapter 8, or from the “Field’s Wheel”.

Step 7: Multiply the friction slope ($S_{fo}$) in Col 6 by the length ($L_o$) in Col. 5 and enter the friction loss ($H_f$) in Col. 7.

Step 8: Enter in Col. 8 the velocity ($V_o$) of the flow from the outlet pipe. Velocity should be based upon whether the pipe flowing partially full or full, as applicable.

Step 9: Enter in Col. 9 the contraction loss ($H_d$).

Step 10: Enter in Col. 10 the design discharge ($Q_i$) for each pipe flowing into the junction, except lateral pipes with inflow of 10% or less of the total flow through the junction.

Step 11: Enter in Col. 11 the velocity of flow ($V_i$) for each pipe flowing into the junction (for exception see Step 10). Velocity should be based upon whether the pipe flowing partially full or full, as applicable.

Step 12: Enter in Col 12 the product of $Q_i$ and $V_i$ for each inflowing pipe. When several pipes flow into a junction, the line producing the greatest $Q_iV_i$ (momentum) product is the line that would produce the greatest expansion loss ($H_i$). (For exception, see Step 10).

Step 13: Enter in Col. 13 the controlling expansion loss ($H_i$).

Step 14: Enter in Col. 14 the angle of skew of each inflowing pipe to the outflowing pipe (for exception, see Step 10).

Step 15: Enter in Col. 15 the greatest bend loss ($H_{\Delta}$). Typical coefficients of $K$ can be found on form LD-347.

Step 16: Enter in Col. 16 the total junction losses ($H_t$) by summing the values in Col. 9 ($H_d$), Col. 13 ($H_i$), and Col. 15($H_{\Delta}$).

Step 17: If the junction incorporates surface inflow, such as from drop inlets, and this flow accounts for 20% or more of the total flow through the junction if a lateral pipe enters a junction with its invert elevation above the crown line elevation of the outgoing trunkline pipe and this flow accounts for 20% or more of the total flow through the junction, increase $H_t$ by 30%. Enter the adjusted $H_t$ in Col. 17.

* Rev. 9/09
Step 18: If the junction incorporates VDOT Standard IS-1, reduce the value of \( H_t \) (column 16 or 17, whichever is greater) by 50% and enter the adjusted value in Col. 18.

Step 19: Enter in Col. 19 the total headloss \( (H) \), the sum of \( H_f \) and \( H_t \), where \( H_t \) is the final adjusted value of the \( H_t \) (the greater of column 16, 17 or 18).

Step 20: Enter in Col. 20 the sum of the elevation in Col. 2 and the total headloss \( (H) \) in Col. 19. This elevation is the potential water surface elevation for the junction under design conditions.

Step 21: Enter in Col. 21 the rim elevation or the gutter flow line, whichever is lowest, of the junction under consideration in Col. 1. If the potential water surface elevation exceeds the rim elevation or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the \( H.G.L. \).

Step 22: Once the HGL elevation for the junction under consideration has been established, repeat the procedure starting with Step 1 for the next junction upstream.
9.5.6.1 Storm Drain Conduit Design and Hydraulic Grade Line Sample Problem

Design a storm sewer system for a site in the Richmond area based on the layout shown in Figure 9-10. Use a 10-year design storm and use concrete pipe (n = 0.013).

Design Data:

<table>
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<th>Inlet #</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
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<tbody>
<tr>
<td>CA</td>
<td>0.9</td>
<td>0.5</td>
<td>1.25</td>
<td>1.98</td>
</tr>
<tr>
<td>t c (minutes)</td>
<td>15</td>
<td>12</td>
<td>17</td>
<td>16</td>
</tr>
<tr>
<td>Top Elev.</td>
<td>103.25'</td>
<td>101.25’</td>
<td>101.75’</td>
<td>98.75’</td>
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</table>

Figure 9-10 Storm Drain Layout Sample Problem

Rev. 9/09
<table>
<thead>
<tr>
<th>FROM POINT</th>
<th>TO POINT</th>
<th>AREA DRAIN &quot;A&quot;</th>
<th>RUN-OFF COEFF.</th>
<th>CA</th>
<th>INLET TIME</th>
<th>RAIN FALL</th>
<th>RUN-OFF Q</th>
<th>INVERT ELEVATIONS</th>
<th>LENGTH</th>
<th>SLOPE</th>
<th>DIA.</th>
<th>CAPACITY</th>
<th>VEL.</th>
<th>FLOW TIME</th>
<th>REMARKS</th>
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<td>0.9</td>
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<td>30.4</td>
<td>10.5</td>
<td>0.2</td>
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Discharge is increased from 21.3 cfs to 21.7 cfs due to Anomaly.
<table>
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<tr>
<th>INLET STATION</th>
<th>Outlet Water Surface Elev.</th>
<th>D</th>
<th>Q</th>
<th>L</th>
<th>S_t</th>
<th>H_t</th>
<th>V_o</th>
<th>H_o</th>
<th>Q_i</th>
<th>V_i</th>
<th>QV_i</th>
<th>V_i^2</th>
<th>H_i</th>
<th>Angle</th>
<th>H_o</th>
<th>H_h</th>
<th>H_f</th>
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<th>0.5 H</th>
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<th>Rim Elev.</th>
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</tr>
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</table>

**FINAL H = H_t + H_h**

\[ H_i = 0.35 \frac{V_i^2}{2g}; \quad H_o = 0.25 \frac{V_o^2}{2g}; \quad H_h = K \frac{V_i^2}{2g}; \quad H_f = H_o + H_i + H_h \]

\[ 90^\circ, K = 0.70; \quad 50^\circ, K = 0.50; \quad 20^\circ, K = 0.25 \]

\[ 80^\circ, K = 0.65; \quad 40^\circ, K = 0.43; \quad 15^\circ, K = 0.19 \]

\[ 70^\circ, K = 0.61; \quad 30^\circ, K = 0.35; \quad 10^\circ, K = 0.13 \]

\[ 60^\circ, K = 0.56; \quad 25^\circ, K = 0.30; \quad 5^\circ, K = 0.06 \]
9.6 References


Bridge Deck Drainage Guidelines. FHWA Report No. RD-014, December 1986

Federal Highway Administration. Design of bridge Deck Drainage, Hydraulic Engineering Circular No. 21, 1993


Rev.1/17
# Appendix 9B-1  LD-204 Stormwater Inlet Computations

## Stormwater Inlet Computations Sheet

<table>
<thead>
<tr>
<th>Inlet</th>
<th>Number</th>
<th>Type</th>
<th>Length (ft)</th>
<th>Station</th>
<th>Drainage Area (ac)</th>
<th>C</th>
<th>CA</th>
<th>sum CA</th>
<th>I Dirs</th>
<th>Q Carrying (CFS)</th>
<th>Cтр Gutter Flow</th>
<th>Стр Cross Slope (ft/ft)</th>
<th>T Spread</th>
<th>W (ft)</th>
<th>VIT</th>
<th>S_w (ft/ft)</th>
<th>S_c/3</th>
<th>E_s</th>
<th>E_l (App. 90-6)</th>
<th>S_e= S_L/12</th>
<th>S_e = S_L/12</th>
<th>Computed Length</th>
<th>L Specified Length (ft)</th>
<th>L.T.</th>
<th>D</th>
<th>E (App. 90-16)</th>
<th>Q Carrying (CFS)</th>
<th>d(IN)</th>
<th>L_s</th>
<th>T Spread @ Sag Ft</th>
</tr>
</thead>
</table>
# Appendix 9B-2  LD-229 Storm Drain Design Computations

<table>
<thead>
<tr>
<th>FROM POINT</th>
<th>TO POINT</th>
<th>AREA DRAIN &quot;A&quot;</th>
<th>RUN-OFF COEF</th>
<th>CA</th>
<th>INLET TIME</th>
<th>RAIN FALL</th>
<th>SUR-OFF D</th>
<th>INVERT ELEVATIONS</th>
<th>LENGTH</th>
<th>SLOPE</th>
<th>DIA</th>
<th>CAPA</th>
<th>VEL</th>
<th>FLOW TIME</th>
<th>REMARKS</th>
</tr>
</thead>
</table>

May 2016

STORM SEWER DESIGN

COMPUTATIONS

ROUTE:       PROJ:  
COUNTY:  DISTRICT:  
DESCRIPTION:  

SHEET OF  

LD-229
## Appendix 9B-3  LD-347 Hydraulic Grade Line Computations

### HYDRAULIC GRADE LINE (HGL)

<table>
<thead>
<tr>
<th>Outlet</th>
<th>JUNCTION LOSS</th>
<th>Inlet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Station</td>
<td>Inlet Water</td>
<td>Outlet Surface Elev</td>
</tr>
<tr>
<td>STATION</td>
<td>D₀</td>
<td>Q₀</td>
</tr>
</tbody>
</table>

### Equations:

- \( H_i = 0.35 \frac{V_i^2}{2g} \)
- \( H_o = 0.25 \frac{V_o^2}{2g} \)
- \( H_o = 0.25 \frac{V_o^2}{2g} \)
- \( H_o = K \frac{V_o^2}{2g} \)
- \( H_i = V_i + H + H_o \)

### Constants:

- 90°: \( K = 0.70 \)
- 60°: \( K = 0.60 \)
- 20°: \( K = 0.25 \)
- 80°: \( K = 0.66 \)
- 40°: \( K = 0.43 \)
- 15°: \( K = 0.19 \)
- 70°: \( K = 0.61 \)
- 30°: \( K = 0.35 \)
- 10°: \( K = 0.13 \)
- 60°: \( K = 0.56 \)
- 25°: \( K = 0.30 \)
- 5°: \( K = 0.06 \)
Appendix 9C-1  Flow in Triangular Gutter Sections

\[ Q = \frac{0.56}{n} S_x^{1.67} S^{0.05} T^{-2.67} \]

**EXAMPLE:**

**GIVEN:**
- \( n = 0.016 \)
- \( S_x = 0.03 \)
- \( S = 0.04 \)
- \( T = 6 \text{ FT} \)

**FIND:**
- \( Q = 2.4 \text{ FT}^3/\text{S} \)
- \( Q_n = 0.038 \text{ FT}^3/\text{S} \)

1. For V-Shape, use the nomograph with \( S_x = S_{x1} S_{x2}/(S_{x1} + S_{x2}) \)
2. To determine discharge in gutter with composit cross slopes, find \( Q_s \) using \( T_s \) and \( S_x \). Then, use App. 9C-8 to find \( E_o \). The total discharge is \( Q = Q_s/(1-E_o) \), and \( Q_w = Q - Q_s \).

Source: HEC No. 22, FHWA
Appendix 9C-4 Flow Characteristic Curves (Straight Cross Slope, 18" Gutter)

Source: VDOT
Comment: REV 6/85
Appendix 9C-5  Flow Characteristic Curves
(Straight Cross Slope 12” Gutter)

Source:  VDOT
Comment:  REV 6/85
Appendix 9C-6  Flow Characteristic Curve
(Roll Type Gutter)

Flow Characteristic Curve

Source: VDOT
Comment: REV 6/85
Appendix 9C-7  Flow in Composite Gutter Sections

Source: HEC No. 12, FHWA
For values of W/T greater than 0.4, use the chart on page 2 of this appendix.

Source: HEC No. 12, FHWA
If $W/T$ is greater than 1.0, use $W/T$ equal to 1.0.

Source: HEC No. 22, FHWA
Appendix 9C-9  Velocity in Triangular Gutter Sections

\[ V = \frac{1.12}{n} S^{0.5} S_x^{0.67} T^{0.67} \]

### Example
**Given**
- \( S = 0.02 \)
- \( S_x = 0.015 \)
- \( T = 6 \) FT
- \( n = 0.018 \)

**Find**
- \( V_n = 0.032 \) FT/S
- \( V = 1.95 \) FT/S

Source: HEC No. 22, FHWA
Appendix 9C-10
Grate Inlet Frontal Flow Interception Efficiency

Example:
Given: Reticuline Grate L = 3 ft V = 8 ft/s
Find: Rf = 0.81

Source: HEC No. 22, FHWA
Appendix 9C-11  Grate Inlet Side Flow
Interception Efficiency

Example:
Given:
\( S_x = 0.025 \)
\( L = 2 \text{ FT} \)
\( V = 4 \text{ FT/S} \)

Find: \( R_s = 0.063 \)

Source: HEC No. 22, FHWA
Grate Inlet Capacity in Sump Conditions

Note: See nomographs qc-13 thru 9c-16 for VDOT St'd. grate inlets

Source: HEC No. 22 FHWA
Appendix 9C-13 Performance Curve DI-1 in a Sump

Source: VDOT Transportation Research Council publication “HYDRAULIC EFFICIENCY OF GRATE INLET”, 1988
Appendix 9C-14 Performance Curve DI-7 in a Sump

Source: VDOT Transportation Research Council publication “HYDRAULIC EFFICIENCY OF GRATE INLET”, 1988
Appendix 9C-15 Performance Curve DI-12 in a Sump (Side Slope 3:1)

Source: VDOT Transportation Research Council publication “HYDRAULIC EFFICIENCY OF GRATE INLET”, 1988
Appendix 9C-16  Performance Curve DI-12 in a Sump
(Side Slope 6:1)

Source: VDOT Transportation Research Council publication “HYDRAULIC EFFICIENCY OF GRATE INLET”, 1988
Appendix 9C-17  Curb-Opening and Slotted Drain
Inlet Length for Total Interception

\[ L_T = 0.6 Q^{0.42} S^{0.3}(1/nS_x)^{0.6} \]

For composite cross slopes, use \( S_e \) for \( S_x \).

\[ S_e = S_x + S_w E_0 \quad ; \quad S_w = d/w \]

Example:
Given: \( n=0.016 \); \( S=0.01 \)
\( S_x=0.02 \); \( Q=4 \, \text{ft}^3/\text{s} \)

Find: \( L_T = 34 \, \text{ft} \)

Source: HEC No. 22, FHWA
Appendix 9C-18  Curb-Opening and Slotted Drain Inlet

Interception Efficiency

\[ E = 1 - (1 - L/L_T)^{1.8} \]

Source: HEC No. 22, FHWA
Appendix 9C-19  Depressed Curb-Opening Inlet Capacity in Sump Locations

Source:  HEC No. 22, FHWA
Appendix 9C-20 Undepressed Curb-Opening Inlet Capacity in Sump Locations

Source: HEC No. 22, FHWA
Appendix 9C-21  Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats

\[ Q = 0.67 \cdot h \cdot L \cdot \sqrt{2g \cdot d_o} \]

- \( h \) = WIDTH OF ORIFICE
- \( L \) = LENGTH OF ORIFICE
- \( d_o \) = WATER DEPTH TO THE CENTER OF ORIFICE

Source: HEC No. 22, FHWA
Appendix 9C-22  Ratio of Frontal Flow to Total Flow in a Trapezoidal Channel

Source: HEC No. 22, FHWA
Appendix 9C-26  Values of Hydraulic Elements of Circular Section for Various Depths of Flow

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>Cross-sectional area of waterway</td>
</tr>
<tr>
<td>(p)</td>
<td>Wetted perimeter</td>
</tr>
<tr>
<td>(R)</td>
<td>Hydraulic radius</td>
</tr>
</tbody>
</table>

For pipes full or half full
\[ R = \frac{D}{4} \]

SECTION OF ANY CHANNEL
\[ V = \text{Average or mean velocity in feet per second} \]
\[ Q = a \cdot V = \text{Discharge of pipe or channel in cubic feet per second (cfs)} \]
\[ n = \text{Coefficient of roughness of pipe or channel surface} \]
\[ S = \text{Slope of Hydraulic Gradient (water surface in open channels or pipes not under pressure, same as slope of channel or pipe invert only when flow is uniform in constant section)} \]

SECTION OF CIRCULAR PIPE

HYDRAULIC ELEMENTS OF CHANNEL SECTIONS

Source: HEC No. 22
Appendix 9D-1

P-1-7/8 and P-1-7/8 4 Grates –
FHWA Classification

Source: HEC-12
Chapter 9 – Storm Drains

Appendix 9D-2 P-1-1/8 Grate – FHWA Classification

Source: HEC-12
Chapter 10 - Erosion and Sediment Control

TABLE OF CONTENTS

CHAPTER 10 - EROSION AND SEDIMENT CONTROL ................................................. 10-1

10.1 Introduction ................................................................................................................. 10-1
10.1.1 Objective .............................................................................................................. 10-1
10.1.2 Principal Factors Influencing Erosion ................................................................. 10-1
10.1.2.1 Soil Characteristics ....................................................................................... 10-1
10.1.2.2 Vegetative Cover .......................................................................................... 10-2
10.1.2.3 Topography .................................................................................................. 10-2
10.1.2.4 Climate .......................................................................................................... 10-2

10.2 Design Policy ............................................................................................................. 10-3
10.2.1 Federal Policy ....................................................................................................... 10-3
10.2.2 State Policy .......................................................................................................... 10-3
10.2.2.1 DEQ Certifications ....................................................................................... 10-5
10.2.2.2 VDOT Training/Certifications ....................................................................... 10-7
10.2.2.3 Policy/General Guidelines ............................................................................ 10-8
10.2.2.4 More Stringent ESC Criteria ......................................................................... 10-9
10.2.2.5 Chesapeake Bay Preservation Areas ............................................................. 10-9

10.3 Documentation .......................................................................................................... 10-10
10.3.1 Minimum Requirements for All ESC Plans ......................................................... 10-10
10.3.1.1 Sequence of Construction ............................................................................ 10-10
10.3.1.2 Limits of Disturbance .................................................................................. 10-11
10.3.1.3 Contents of ESC Plan .................................................................................. 10-13
10.3.2 ESC Plan Development Process ........................................................................ 10-13
10.3.2.1 Concurrent Engineering Process for Plan Development (CEP) .................... 10-13
10.3.2.2 Plan Development Process for “No Plan” Projects and Special Advertisement and Award Process (SAAP) Projects .................................................. 10-14
10.3.2.3 Plan Development Process for State Force Construction Projects ............. 10-16
10.3.2.4 Plan Development Process for Minimum Plan and Standard Plan Construction Projects ........................................................................................................ 10-17
10.3.3 SWPPP Applicability and Requirements ............................................................... 10-20
10.3.4 SWPPP Certification ............................................................................................ 10-21
10.3.5 SWPPP General Information Sheets ................................................................. 10-22
10.3.6 SWPPP Documents .............................................................................................. 10-24
10.3.7 SWPPP Components ......................................................................................... 10-25
10.3.8 Computations ........................................................................................................ 10-30
10.3.9 Field Revisions and Evaluations ........................................................................ 10-30
10.3.10 Maintenance ....................................................................................................... 10-31
10.3.11 Standard Forms ................................................................................................. 10-33

10.4 References .................................................................................................................. 10-34

List of Appendices

Appendix 10B-1 Erosion and Sediment Control Plan Details
Appendix 10C-1 Erosion and Sediment Control Plan – Example No Plan Project
Chapter 10 - Erosion and Sediment Control

10.1 Introduction
Erosion and sedimentation are natural or geologic processes whereby soil materials are detached and transported from one location and deposited in another, primarily due to rainfall and runoff. Accelerated erosion and sedimentation can occur at times in conjunction with highway and transportation facility construction. This accelerated process can result in significant impacts such as safety hazards, expensive maintenance problems, unsightly conditions, instability of slopes, and disruption of ecosystems. For this reason, the total design process must be done with consideration given to minimization of erosion and sedimentation.

10.1.1 Objective
The purpose of erosion and sediment control is to provide an effective plan to control soil erosion and prevent sediment from leaving the construction site. The Department’s DEQ\(^*\) approved erosion and sediment control (ESC) and stormwater management (SWM) standards and specifications should be implemented on all regulated land-disturbing activities. Additional information can be found in the Virginia Erosion and Sediment Control Handbook and the Virginia Erosion and Sediment Control Regulations. This Handbook can be ordered online or found at the following website: http://www.deq.virginia.gov/Programs/Water/StormwaterManagement/Publications/ESCHandbook.aspx.

The Virginia Erosion and Sediment Control Regulations can be accessed from the following website: https://law.lis.virginia.gov/admincode/title9/agency25/chapter840/.

10.1.2 Principal Factors Influencing Erosion

10.1.2.1 Soil Characteristics
The properties of soil that influence erosion by rainfall and runoff are ones affecting the infiltration capacity of a soil and the resistance of soil particles to detachment and movement by water or wind. Soils containing high percentages of fine sands and silt are normally the most erodible. As the clay and organic matter content of these soils increases, the potential for erosion decreases. Clays act as a binder to soil particles, thus reducing the potential for erosion. However, while clays have a tendency to resist erosion, once eroded they are easily transported by water. Soils high in organic matter have a more stable structure which improves their permeability. Such soils resist raindrop detachment and infiltrate more rainwater. Clear, well-drained, and well-graded gravels and gravel-sand mixtures are usually the least erodible soils. Soils with high infiltration rates and permeabilities reduce the amount of runoff.

\(^*\)Rev. 7/19
10.1.2.2 Vegetative Cover  
Vegetative cover plays an important role in controlling erosion in the following ways:

- Shields the soil surface from the impact of falling rain
- Holds soil particles in place
- Maintains the soil's capacity to absorb water
- Slows the velocity of runoff
- Removes subsurface water between rainfalls through the process of evapotranspiration

By limiting and staging the removal of existing vegetation and by decreasing the area and duration of exposure, soil erosion, and sedimentation can be significantly reduced. Special consideration should be given to the maintenance of existing vegetative cover on areas of high erosion potential such as erodible soils, steep slopes, drainage ways, and the banks of streams.

10.1.2.3 Topography  
The size, shape, and slope characteristics of a watershed influence the amount and rate of runoff. As both slope length and gradient increase, the rate of runoff increases and the potential for erosion is increased. Slope orientation can also be a factor in determining erosion potential.

10.1.2.4 Climate  
The frequency, intensity, and duration of rainfall are fundamental factors in determining the amounts of runoff produced in a given area. As both the volume and velocity of runoff increase, the capacity of runoff to detach and transport soil particles also increases. Where storms are frequent, intense, or of long duration, erosion risks are high. Seasonal changes in temperature, as well as variations in rainfall, help to define the high erosion risk period of the year. When precipitation falls as snow, no erosion will take place. However, in the spring the melting snow adds to the runoff and erosion hazards are high. Because the ground is still partially frozen, its ability to absorb runoff is reduced. Frozen soils are relatively erosion-resistant. However, soils with high moisture content are subject to uplift by freezing action, and are usually very easily eroded upon thawing. However, Virginia experiences the most intense rainfall events in the warmer summer months, which corresponds with the busiest road construction period of the year. The intense summer rainfalls combined with exposed soils can result in higher rates of erosion on a construction site.

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10.2 **Design Policy**

A policy for erosion and sediment control is stated in the American Association of State Highway Transportation Officials’ publication, "A Policy on Geometric Design of Rural Highways," as follows:

"Erosion prevention is one of the major factors in the design, construction, and maintenance of highways. Erosion can be controlled to a considerable degree by geometric design particularly relating to the cross section. In some respects the control is directly associated with proper provision for drainage and fitting landscape development. Effect on erosion should be considered in the location and design stages."

"Erosion and maintenance are minimized largely by the use of flat side slopes, rounded and blended with natural terrain; drainage channels designed with due regard to width, depth, slopes, alignment and protective treatment; located and spaced facilities for ground water interception; dikes, berms and other protective devices; and protective ground covers and planting."

10.2.1 **Federal Policy**

As a result of the National Environmental Policy Act of 1969, the Federal Water Pollution Control Act (also known as the Clean Water Act) and the Federal Chesapeake Bay Protection Act, much attention has been directed to the control of erosion and sedimentation. As a result of this concern, numerous state and federal regulations and controls governing land-disturbing activities have been developed and published. Federal control requirements are enforced by numerous agencies such as the U.S. Environmental Protection Agency (EPA), U.S. Army Corps of Engineers (COE), Virginia Department of Environmental Quality (DEQ), Fish and Wildlife Service (FWS), Virginia Resources Commission (VMRC), etc., through their administration of various permitting requirements (Section 401, 402 and 404 of the Federal Water Pollution Control Act, and Sections 9 and 10 of the River and Harbor Act).

10.2.2 **State Policy**

The Department of Environmental Quality annually reviews and approves VDOT’s Erosion and Sediment Control Program. This Annual review includes all of VDOT’s erosion and sediment control standards, specifications, policies, and design guidelines as outlined in the Road and Bridge Standards, Road and Bridge Specifications, Drainage Manual, Road Design Manual, Instructional and Informational Memoranda, and other associated directives.

VDOT receives an annual approval of its ESC Standards and Specifications from DEQ. By its annual approval of VDOT’s ESC Standards and Specifications, DEQ authorizes VDOT to administer its ESC Program in accordance with the Approved ESC Standards and Specifications, on all regulated land-disturbing activities (RLDA) undertaken by the Department.

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1Rev. 7/19
VDOT’s approved ESC Standards and Specifications shall apply to all plan design, construction and maintenance activities undertaken by VDOT, either by its internal workforce or contracted to external entities, where such activities are regulated by the Virginia ESC Law and Regulations. During these regulated land-disturbing activities, compliance with the VDOT’s Approved ESC Standards and Specifications (and all parts thereof) will be expected. A standard, specification or product not contained or referenced in VDOT’s Approved ESC Standards and Specifications cannot be used unless it is submitted to and approved by DEQ either as a revision to the Approved ESC Standards and Specifications or a project specific variance.

Statewide use of standards, specifications or products not contained in VDOT’s DEQ Approved ESC Standards and Specifications will require a revision or deviation to the Approved ESC Standards and Specifications. Any revision or deviation to the Approved ESC Standards and Specifications shall be reviewed and approved by DEQ prior to implementation by VDOT. Such review and approval shall be coordinated by the VDOT State MS4/Stormwater Management Engineer in the VDOT Central Office with the DEQ Regulatory Programs Manager in the DEQ Central Office Stormwater Management Division.

Where determined necessary to meet an individual project need, VDOT may request DEQ to grant a project specific variance, waiver, or deviation to the Approved ESC Standards and Specifications:

- All requests for project specific variances and deviations for VDOT projects shall be coordinated by the VDOT State MS4/Stormwater Management Engineer with the DEQ Regulatory Programs Manager. All variance requests shall be accompanied by complete details and documentation, including justification for the requested variance. Copies of any variance requests, approvals and related correspondence are to be sent to the VDOT State MS4/Stormwater Management Engineer.

- All requested variances and deviations are to be considered unapproved until written approval from DEQ is received.

- All approved variances and deviations for Erosion and Sediment Control shall be listed in Note 19 in Section II of the SWPPP General Information Sheets in the construction plans (or other such documents) for the land-disturbing activity.

- All documentation for and approval of requested variances and deviations shall be retained in the appropriate (i.e. design, construction, etc.) files of the proposed activity.

- The VDOT State MS4/Stormwater Management Engineer shall maintain a file of all requested and approved variances and deviations.

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Rev. 7/19
Nonlinear projects, such as those administered by the VDOT’s Capital Outlay Program, are encouraged to utilize VDOT’s Approved ESC Standards and Specifications in the development of the ESC Plan for such projects. Where deemed impractical to use VDOT’s Approved ESC Standards and Specifications and when approved by the VDOT State MS4/Stormwater Management Engineer, DEQ’s ESC Standards and Specifications, as outlined in the Virginia Erosion and Sediment Control Regulations and Handbook, may be utilized in combination with VDOT’s Approved ESC Standards and Specifications to develop ESC Plans for nonlinear projects. Such projects include, but are not limited to, new and/or additions/modifications to Rest Areas, District or Residency Office complexes, Area Maintenance Headquarters/Repair Shops and buildings on the right of way or associated with bridges/piers/tunnels, spreader/tailgate/wash rack sites, holding ponds or containment pads, fuel dispensing facilities, security facilities and drainage improvements to building/parking sites and structures.

Any regulated land-disturbing activity, including* maintenance and construction activities, that disturb more than 10,000 square feet, or 2,500 square feet in areas defined as Tidewater Virginia in the Virginia Chesapeake Bay Preservation Act, must have a specific erosion and sediment control plan developed and implemented in accordance with VDOT’s Erosion and Sediment Control Program. The requirements of the Virginia Erosion and Sediment Control Regulations (VESCR), https://law.lis.virginia.gov/admincode/title9/agency25/chapter840/ and the VDOT Erosion and Sediment Control Standards and Specifications will be incorporated into every design and will be enforced on all VDOT operations.

Refer to Appendix 10B-1 for additional policy and design guidelines.

10.2.2.1 DEQ Certifications

The Virginia ESC Law and Regulations require that the ESC Program administration and the ESC Plan design, implementation and inspection activities be conducted by DEQ certified personnel for all Regulated Land Disturbance Activities (RLDA).

VDOT’s ESC Program will be administrated by a DEQ Certified Program Administrator:

- The Program Administrator shall be the person within the Department who has been designated to have overall responsibility for administration of VDOT’s ESC Program.

- The DEQ Program Administrator Certification is acquired by satisfying the DEQ eligibility/training requirements and passing the DEQ Program Administrator Exam or by possessing a DEQ Combined Administrator Certification or a DEQ Dual Combined Administrator Certification.

- The VDOT State MS4/Stormwater Management Engineer in the Central Office Location and Design Division is currently designated as VDOT’s ESC Program Administrator.

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The Virginia ESC Regulations require that each RLDA be overseen by a DEQ certified Responsible Land Disturber RLD:

- The DEQ RLD Certification is required for the VDOT person who has general oversight of the construction phase of a specific RLDA.

- The RLD for a specific RLDA must be identified prior to beginning any regulated land-disturbing activity (see note 11 in Section I of the SWPPP General Information Sheets). The DEQ RLD Certification is acquired by passing the DEQ RLD Exam; by possessing a DEQ Combined Administrator, Program Administrator, Plan Reviewer or Inspector Certification; by possessing a Professional Engineer, Land Surveyor, Landscape Architect or Architect License pursuant to Chapter 4, Title 54.1, of the Code of Virginia; or as a professional soil scientist as defined in Chapter 22 of Title 54.1 of the Code of Virginia.

The proposed ESC Plan for each RLDA must be reviewed and certified by a DEQ Certified ESC Plan Reviewer to ensure that the ESC Plan has been developed in accordance with VDOT’s Approved ESC Standards and Specifications or variances authorized thereto.

- The DEQ Plan Reviewer Certification is required for any person that has responsibility for reviewing and certifying the proposed erosion and sediment control plan for a specific RLDA. See VDOT Form LD-445C for additional information pertaining to plan review certification for VDOT regulated land-disturbing activities.

- The DEQ Plan Reviewer Certification is acquired by satisfying the DEQ eligibility/training requirements and passing the DEQ Plan Reviewer Exam, by possessing a DEQ Combined Administrator Certification, or possessing a Professional Engineer, Land Surveyor, Landscape Architect or Architect License pursuant to Chapter 4, Title 54.1, of the Code of Virginia.

A DEQ ESC Inspector Certification is required for those persons having responsibility for ensuring the proper implementation of, or compliance with, the proposed ESC Plan and VDOT’s Approved ESC Standards and Specifications, or variances authorized thereto, throughout the construction phase of the RLDA. The ESC Law and Regulations also require that inspections of ESC facilities be conducted by a DEQ certified ESC Inspector.

- The DEQ Inspector Certification is acquired by satisfying the DEQ eligibility/training requirements and passing the DEQ Inspector Certification Exam or by possessing a DEQ Combined Administrator Certification; or by possessing a DEQ Dual Inspector Certification.

*Rev. 7/19
It shall be the responsibility of the Project Authority to ensure that those staff with the appropriate DEQ Certifications (RLD, Plan Reviewer or Inspector) performs the functions required by the ESC Law and Regulations and noted in Section 10.2.2.1 of this document.

- For the purposes of this document, the Project Authority is defined as that person with overall responsibility for a land-disturbing activity or a specific phase of a land-disturbing activity.

- The Project Authority for preconstruction (design) activities is typically the PM, Residency CA, RA or other such person responsible for the preconstruction phase of the land-disturbing activity. This person shall ensure that the proposed ESC Plan has been reviewed and certified by a DEQ Certified Plan Reviewer.

- The Project Authority for actual land disturbance (construction) activities is typically the ACE, RA or other such person responsible for the construction phase of the land-disturbing activity. This person shall ensure that the RLDA has an assigned DEQ Certified RLD and that the implementation of the ESC Plan, including inspection requirements, is being overseen/conducted by a DEQ Certified Inspector.

10.2.2.2 VDOT Training/Certifications

Where land disturbing activities occurring within VDOT right of way are regulated under the Virginia ESC Law and Regulations, Section 107.16(a) of the 2016 VDOT R&B Specifications requires that all contractors performing such land-disturbing activities have a person certified by the VDOT in erosion and sediment control within the project limits. This certification requirement is mandatory for all contractors performing land-disturbing activities under contracts managed by VDOT, including PPTA and Design Build agreements. For contractors performing land-disturbing activities on VDOT right of way under a Land Use Permit, the certification requirements of Section 107.16(a) shall apply if the area of land disturbance within the VDOT right of way exceeds that noted in Sections 10.2.2 and 10.2.2.3 of this document. However, contractors performing maintenance related land-disturbing activities under a hired equipment contract whose work is directly supervised by VDOT personnel are not required to be VDOT certified.

- Successful completion of the Department’s “Erosion and Sediment Control Contractor Certification” (ESCCC) course satisfies the certification requirements of Section 107.16 (a) of the 2016 VDOT R&B Specifications.

- The ESCCC is a joint training effort between the VDOT and the transportation construction industry in Virginia. The VDOT develops the course material and members of the transportation construction industry in Virginia administer the training, testing and issuance of certifications.

*Rev. 7/19*
10.2.2.3 Policy/General Guidelines

Requirements of the Virginia ESC Regulations and the VDOT ESC Standards and Specifications, as approved by the DEQ, shall be incorporated into all erosion and sediment control plans and shall be enforced on all Regulated Land Disturbance Activities managed by VDOT.

The Virginia ESC Law defines land disturbance as any land change which may result in soil erosion from water or wind and the movement of sediments into state waters or onto lands of the Commonwealth, including, but not limited to, clearing, grading, excavating, transporting and filling of land.

Any maintenance or construction activity disturbing 10,000 square feet or greater in areas other than those within Tidewater, Virginia (see below for more discussion) must have a project specific ESC Plan developed and implemented in accordance with VDOT’s Approved ESC Standards and Specifications.

The blading/dragging/grading associated with the maintenance of the travel surface of a dirt roadway is considered a land disturbance for erosion and sediment control, but not for stormwater management. The blading/dragging/grading associated with the maintenance of the travel surface of a gravel or aggregate stabilized roadway is not considered a land disturbance for erosion and sediment control or stormwater management.

VDOT shall be responsible for ensuring compliance with its approved ESC Standards and Specifications by private entities (i.e., agents, contractors, subcontractors, consultants) conducting regulated land disturbance activities on projects managed by VDOT, including those constructed under the Public/Private Transportation Act (PPTA), the Design/Build process and the Capital Outlay Program.

When not included in the proposed ESC Plan for the RLDA, the contractor must provide an ESC Plan in accordance with Section 106 of the VDOT R&B Specifications for borrow pit sites and disposal area sites utilized exclusively to obtain or dispose of project materials, as well as other regulated offsite Support Activities, in accordance with Section 107.16 of the VDOT R&B Specifications. Any such ESC Plan provided by the contractor must comply with VDOT’s Approved ESC Standards and Specifications. Where required, the contractor must design, construct and maintain sediment traps and/or basins at these sites. The contractor shall supply supporting calculations for sediment trap and/or basin design and calculations demonstrating compliance with the Virginia ESC Regulations, VSMP Regulation, and Construction General Permit. All information provided by the contractor should be reviewed by the Engineer or other DEQ certified plan reviewer to ensure accuracy, the use of appropriate methodology and compliance with VDOT’s Approved ESC Standards and Specifications, Virginia ESC Law and Regulations, and the General VPDES Permit for Discharges of Stormwater from Construction Activities (Construction General Permit or CGP), where applicable.

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*Rev. 7/19
10.2.2.4 More Stringent ESC Criteria

When requested by DEQ, and where deemed practical by VDOT, projects located in jurisdictions with more stringent ESC technical criteria than that contained in the Virginia ESC Law and Regulations shall be designed to meet the more stringent criteria. The local criteria may be part of a locally adopted State approved program or may be part of a watershed initiative related to the protection of a water supply, or a TMDL implementation plan. It will be the responsibility of the ESC Plan Designer to demonstrate, through appropriate analysis and documentation, that the local requirements are not practical for the project under consideration. Early coordination should occur between the ESC Plan Designer and the local ESC program authority in order to identify any such requirements.

10.2.2.5 Chesapeake Bay Preservation Areas

Any maintenance or construction activity disturbing 2,500 square feet or greater within the area of Tidewater, Virginia, as defined in the Virginia Chesapeake Bay Preservation Act, must have a project specific ESC Plan developed and implemented in accordance with the VDOT's Approved ESC Standards and Specifications. Tidewater, Virginia is defined as the Counties of Accomack, Arlington, Caroline, Charles City, Chesterfield, Essex, Fairfax, Gloucester, Hanover, Henrico, Isle of Wight, James City, King George, King and Queen, King William, Lancaster, Matthews, Middlesex, New Kent, Northampton, Northumberland, Prince George, Prince William, Richmond, Spotsylvania, Stafford, Surry, Westmoreland and York and the Cities of Alexandria, Chesapeake, Colonial Heights, Fairfax, Falls Church, Fredericksburg, Hampton, Hopewell, Newport News, Norfolk, Petersburg, Poquoson, Portsmouth, Richmond, Suffolk, Virginia Beach and Williamsburg.
10.3 Documentation

Drainage designers should use guidelines and checklists such as those provided in the Virginia Erosion and Sediment Control Handbook (VESCH), the DEQ Website*, or VDOT Form LD-436 to verify that critical design issues have been accounted for with each design phase of the project.

The design of sediment traps, sediment basins and other major erosion and sediment control measures is to be supported by engineering calculations which are to be included as a part of the project’s drainage report. Instruction for designing erosion and sediment control measures can be found in Appendix 10B-1 and the VESCH.

10.3.1 Minimum Requirements for All ESC Plans

The ESC Plan shall include a plan view depicting (using appropriate plan symbols and notes) locations where specific measures are needed in order to control erosion and sediment deposition within the RLDA limits. Specific erosion and sediment control measures include, but are not limited to, protective linings for ditches and steep slopes, pipe outlet protection, silt fence, check dams, sediment traps, sediment basins, diversion berms and ditches, etc. The ESC Plan should be based on the existing field conditions at the time of design, the anticipated sequence of construction, and the site conditions expected as the RLDA is brought to final grade.

Erosion and Sediment Control Plan Information: General information related to the ESC Plan is to be documented utilizing the notes in Section I and II of the SWPPP General Information Sheets. Information required to complete the SWPPP notes will be developed by the ESC Plan Designer with assistance from District Hydraulics or Residency staff as needed.

10.3.1.1 Sequence of Construction

The proposed ESC Plan shall be developed in conjunction with the proposed Sequence of Construction Plan and should denote the required erosion and sediment controls for the intended sequence of major construction activities. In planning the sequence of construction, consideration should be given to elimination or minimization of the need for major erosion and sediment control facilities, such as sediment basins, by strategic planning of the construction timing and location of erosion and sediment control measures, grading operations, temporary and permanent channels and drainage facilities. Any changes to the proposed sequence of construction plan that could potentially cause a significant change to the proposed ESC or related Drainage Plan shall be submitted to the ESC Plan Designer/Hydraulics Engineer for evaluation of impacts.

*Rev. 7/19
10.3.1.2 Limits of Disturbance

The regulatory Limits of Disturbance (LOD) include any land change which may result in soil erosion from water or wind and the movement of sediments into waters or onto lands of the Commonwealth. This includes, but is not limited to, clearing, grubbing, grading, excavating, filling, stockpiling, surcharging, transporting, open trenching, and other activities that expose soils to potential erosion and sedimentation.

The LOD should include the activities noted above, as well as the following (where applicable for a project):

1. Areas required to install and maintain ESC and SWM facilities, both temporary and permanent
2. Temporary easements secured for land-disturbing and construction activities
3. Areas permitted for unavoidable impacts to waters of the United States (U.S.)
4. Areas used for regulated onsite support activities, including stockpiling, laydown, mobilization, equipment storage and maintenance, etc.
5. Construction access routes where vehicle and equipment travel could expose soils
6. Utility easements and installations where soils could be exposed, such as open trenching; these ancillary activities can be difficult to capture during design and planning and often require separate approved plans and permit coverage or modification of existing plans and permit coverage
7. Temporary stream crossings for vehicles, equipment, or utility installations
8. Boring and receiving pits required for jack and bore activities or directional drilling (but not the entire length of the jack and bore or drilling where soils are not exposed)
9. Other areas/activities where regulated land-disturbing and construction activities occur

Some items the designer should consider when developing the LOD:

1. The right-of-way and easements (temporary and permanent) available within the project area
2. Waters of the US within the project limits and permitted impacts to waters of the US
3. The proposed cut and fill limits for grading required by the project (is sufficient area provided to tie proposed grades to existing grades with required ESC in-place)
4. Access to the toe of fill slopes and head of cut slopes for construction and maintenance activities
5. The minimum setbacks required for proper ESC installation, maintenance, and removal

The “simplest” way to develop the LOD would be to include all right of way and easements included in the project area, but that approach is very conservative and could create its own issues to consider:

Rev. 7/19
1. If the entire project area, including all right-of-way and easements, is included in the LOD, then the SWM computations must reflect that area; expanding the LOD could result in the need for additional SWM controls on a project.

2. The contractor may require additional area outside of the right of way and easements for construction access or to properly install and maintain temporary ESC; what legal arrangements must be made to secure the additional right of way, easement, access agreement, etc.

3. If unavoidable impacts to waters of the US were not considered for the entire right of way and easements, including temporary impacts for ESC, then expanding the LOD to use all of the right of way or easement could require additional 401/404 permitting.

4. Expanding the LOD may also require a CGP modification, or trigger the need for permit coverage when the original LOD was < 1.00 acre and the expanded LOD is now ≥ 1.00 acre.

When developing the LOD and delineating it on an ESC Plan, the designer should balance project constructability with minimization of land disturbance. During the concurrent engineering process, the LOD is likely to change and will continue to change as the project moves from preliminary engineering through right-of-way and construction. It is difficult to predict how much area will be needed to construct the project without detailed knowledge of all construction means and methods, which are generally unavailable during the planning stage. The LOD on the plan should be reviewed carefully during the constructability review and at the project pre-construction conference to ensure that everyone is familiar with the LOD and determine if additional LOD is necessary. Potential revisions to the LOD during the construction phase should be discussed at the pre-construction meeting, including the process for review and approval.

The LOD must be clearly delineated in the ESC plan to show where regulated land-disturbing and construction activities are permitted. Conducting regulated land-disturbing activities outside of the permitted LOD shown on the ESC plan is not allowed. When regulated land-disturbing occurs outside of the permitted area, revisions must be made to the LOD shown on the ESC plan. ESC and SWM design and computations may also require revisions to address the LOD, and the CGP coverage may require modification to include the additional project or disturbed area not included in the permitted LOD.

The VDOT CADD Standards include a new line type and weight for drawing the LOD in a project model. The LOD should be placed in a separate layer in MicroStation, so it can be turned on for ESC plan sheets and environmental commitments plan sheets. See the CADD manual for more details. Also, the LOD can be used as the basis for project site map required for securing permit coverage for a project under the CGP.

*Rev. 7/19
10.3.1.3 Contents of ESC Plan
Details of the RLDA’S ESC Plan may be shown on, but is not limited to, the plan, profile, typical section and detail sheets of the construction plan set or other such documents. The ESC Plan shall, at a minimum, contain the following information:

- Section I and II notes of the SWPPP General Information Sheets.

- Limits of Disturbance (LOD) for regulated land-disturbing and construction activities (plan view).

- Location of temporary and permanent erosion and sediment control and related permanent stormwater management features (plan view).

- Construction details for any temporary or permanent erosion and sediment control or related permanent stormwater management features if different from the VDOT R&B Standards and Specifications.

- Location of any surface waters, wetland features, or other environmentally sensitive/critical areas within or immediately adjacent to the RLDA area. (Such features located within close proximity of the project, yet outside the limits of the construction plans or other such documents, shall be described in Notes 6 to 9 in Section I of the SWPPP General Information Sheets.

- Appropriate existing and proposed topographic features.

10.3.2 ESC Plan Development Process

10.3.2.1 Concurrent Engineering Process for Plan Development (CEP)
The CEP for plan development incorporates the principles of teamwork, flexibility, and milestones. The development, review, and approval of the project specific erosion and sediment control plan are included in the CEP milestones as follows:

- Scoping Stage: The ESC Plan Designer/Hydraulics Engineer shall identify any local ESC or related SWM technical criteria or watershed initiatives that may influence the ESC or related post construction SWM design of the project. This should include early coordination with the local ESC/SWM program authority to assess any potential impacts on the project design.

- Preliminary Field Investigation (PFI), Public Hearing (PH), Stages: The ESC Plan Designer/Hydraulics Engineer shall develop preliminary ESC (and associated post construction SWM Plans; (see the latest version of IIM-LD-195 for information on the technical criteria and requirements for permanent SWM facilities) and show the limits of disturbance (LOD) and locations of all major erosion and sediment control, permanent stormwater management, and/or drainage facilities on the plans that may affect the required right of way. Members of the project team shall provide comments, as appropriate, to the ESC Plan Designer/Hydraulics Engineer regarding the preliminary plan, including any pertinent information that might affect the final design of the ESC or post construction SWM Plan.

Revised 7/19
• **Field Investigation** (FI) Stage: Prior to the FI, the ESC Plan Designer/Hydraulics Engineer shall develop final ESC and associated post construction SWM plans and show final design locations, sizes, and other plan details as necessary to accurately determine the right-of-way and/or easement requirements, and to determine whether the selected ESC Plan Concept (Single or Multiple Phase) is appropriate. The ESC and related post construction SWM Plan design shall address any comments or recommendations from the Public Hearing process as accepted/incorporated by the Project Manager (or other such project authority). This phase of the ESC and related post construction SWM Plan design process provides all the necessary information needed to conduct a thorough Field Inspection. Members of the project team shall provide comments, as appropriate, to the ESC Plan Designer/Hydraulics Engineer regarding the proposed ESC and post construction SWM Plan.

• ESC Plan Design Completion: After FI and prior to the Right of Way stage, the ESC Plan Designer/Hydraulics Engineer shall incorporate all changes, deletions, and/or additions into the ESC and related post construction SWM Plan resulting from any FI and/or Quality Control Review comments or plan revisions. The ESC and post construction SWM Plan shall be carefully reviewed for compliance with the approved VDOT ESC Standards and Specifications and the VPDES Construction General Permit (where applicable) including, but not limited to, the limits of disturbance (LOD), the types of proposed measures, means of access for maintenance, and required right of way and/or easements for regulated land-disturbing activities.

• ESC & SWM Plan Design Certification: Prior to the Pre-Advertisement Conference (or similar project meeting), the ESC Plan Designer/Hydraulics Engineer shall have the ESC and related post construction SWM Plan reviewed by a DEQ Certified ESC Plan Reviewer. The ESC Plan Reviewer shall verify that the ESC and related post construction SWM Plan for the project is in compliance with the VDOT Approved ESC and SWM Standards and Specifications. Any comments by the Plan Reviewer shall be addressed with the ESC Plan Designer/Hydraulics Engineer. Once all comments have been reconciled, the ESC Plan Reviewer completes, signs and forwards the ESC & SWM Plan Design Certification Form (LD-445C) to the ESC Plan Designer/Hydraulics Engineer. The ESC Plan Designer/Hydraulics Engineer provides the completed LD-445C form to the Project Manager (or other such project authority) for use in the Construction General Permit Application Process (see the latest version of IIM-LD-242), if applicable. A copy of the completed LD-445C form is to be retained with the other documentation for the proposed ESC Plan.

**10.3.2.2 Plan Development Process for “No Plan” Projects and Special Advertisement and Award Process (SAAP) Projects**

A “No Plan” project is defined as an assembly of letter size sketches and narratives depicting the project’s location, typical cross section, estimated quantities and any other specific details necessary (i.e., ESC and/or post construction SWM plans) for the construction of the project. Any “No Plan” project that disturbs 2,500 square feet or greater in Tidewater, Virginia or 10,000 square feet or greater elsewhere within the
State must have a project specific ESC Plan. A project developed under the “No Plan” concept is one that generally requires little or no survey, engineering or hydraulic analysis in order to produce the necessary contract documents. Any required right of way is generally acquired through donations in lieu of the purchase/condemnation process. See Appendix A of the VDOT Road Design Manual for additional information on the “No Plan” concept.

“SAAP” Projects are defined as those advertised under the Special Advertisement and Award Process. The “No Plan” concept is generally used to produce the required contract documents. “SAAP” projects generally have one or more of the following characteristics:

- They require little or no preliminary engineering.
- They are standard maintenance repair contracts (e.g., bridge, guardrail or concrete pavement repairs).
- They are standard incidental construction and/or improvement projects of limited scope.
- The work being performed involves a singular function or specialty work (e.g., bridge painting, pavement markings or pipe installation).

Any “SAAP” project that disturbs 2,500 square feet or greater in Tidewater, Virginia or 10,000 square feet or greater elsewhere within the State must have a project specific ESC Plan.

- During the early stages of the preparation of the contract assembly for any “SAAP” or “No Plan” Project, the Contract Administrator (CA) (or other such project authority) should conduct a Scoping Meeting to determine what is needed on the project in order to comply with the VDOT Approved ESC and SWM Standards and Specifications.

- The Scoping Meeting should include the CA, the District L&D Engineer and/or Hydraulics Engineer, and the appropriate District Environmental Section personnel in order to accurately determine the project requirements.

- The CA, with the assistance of the District Hydraulics Engineer, or other appropriately qualified personnel, shall prepare a preliminary Straight Line Sketch (SLS). An example is shown in Appendix 10C-1.

- Upon completion of the Preliminary SLS, the CA shall coordinate with the appropriate personnel in the District Hydraulics Section and other appropriate District/Residency sections to schedule a Field Review. The following data should be made available to all Field Review participants:
  - A Vicinity Map – United States Geological Survey (USGS) Topographical Map and County Road Map showing the location and limits of the proposed project.
  - A SLS of the project showing the project limits and the approximate location of proposed drainage items and erosion and sediment control items.
• If during the Field Review it is found that such items as permanent stormwater management facilities, drainage improvements, temporary sediment basins or temporary sediment traps are required, the District Hydraulics Section will determine and request the necessary survey data, and provide engineering support in the development of the SLS to ensure consistency with the VDOT Approved ESC and SWM Standards and Specifications.

• Upon completion of the design of any required permanent stormwater management facilities, drainage improvements, or sediment trapping facilities, the District Hydraulics Section will provide the CA with final comments, recommendations and plan details.

• Final approval of the SLS:
  o Upon incorporation of all the required revisions, a DEQ Certified ESC Plan Reviewer shall make a final review of the ESC and post construction SWM Plan (if applicable). Once any Plan Reviewer comments have been reconciled with the ESC Plan Designer/Hydraulics Engineer, the Plan Reviewer shall complete and sign the LD-445C Erosion and Sediment Control and Stormwater Management Certification form and forward it to the CA for use in the VPDES Construction General Permit Application Process (see the latest version of IIM-LD-242), if applicable. A copy of the completed LD-445C form is to be retained with the other documentation for the proposed ESC Plan.
  o The CA will incorporate the final SLS into the contract assembly.

• Thereafter, any significant change to the project that may impact the ESC, post construction SWM, or Drainage Plan will require resubmission of the revised SLS to the ESC Plan Designer and/or District Hydraulics Engineer for review and approval prior to implementation.

• The final version of the SLS, the SWPPP General Information Sheets and any Construction Notes will serve as the ESC and post construction SWM Plan for the project. During the construction phase of the project, a copy of the ESC and post construction SWM Plan (Record Set) and all other SWPPP documents shall be kept on the project site and in the project file at the appropriate District/Residency Office as documentation that all policies and procedures have been addressed with regards to the post construction SWM, ESC and SWPPP requirements of the project. During construction, any authorized changes to the proposed ESC Plan necessitated by unforeseen conditions or other circumstances shall be documented on the Record Set in accordance with Section 107.16(e) of the 2016 VDOT R&B Specifications.

10.3.2.3 Plan Development Process for State Force Construction Projects
• State Force Construction Projects include land-disturbing activities that are performed with state force equipment and/or hired equipment.
• Residency personnel are to contact the Residency Environmental Specialist and/or
the District Hydraulics Engineer to review any State Force Construction Projects to
determine if the proposed work is of a magnitude that may require drainage
improvements, an ESC Plan, a post construction SWM Plan, and/or a SWPPP. If it is
determined that any of these items are needed, the same procedures outlined in this
document shall be followed.

10.3.2.4 Plan Development Process for Minimum Plan and Standard Plan
Construction Projects
• Minimum Plan projects are those that require a limited amount of survey information
in order to perform the necessary engineering studies and to provide the information
required to secure the necessary rights of way. The minimum amounts of detail
needed to address environmental requirements and to construct the project are
provided in a standard plan assembly format. See Appendix A of the VDOT Road
Design Manual for additional information on the Minimum Plan concept.

• Standard Plan Projects are those that require complete survey information in order
to perform the necessary detailed engineering studies and to develop a complete
and detailed construction plan assembly.

• Projects developed under the Minimum and Standard Plan concepts must have an
ESC plan and a SWPPP if they exceed the land disturbance threshold amounts
noted in Section 10.2.2.3 of this document. In addition, such projects may also
require a post construction SWM Plan (see the latest version of IIM-LD-195 for
applicability and technical criteria and requirements). These plan assemblies should
be developed consistent with the steps identified under the Concurrent Engineering
Plan Development process described in Section 10.3. 2 of this document.

• The ESC Plan shall be developed utilizing either a single phase or a multiple phase
concept. The decision as to which concept to use in the development of the ESC
Plan for each specific RLDA shall be determined by the ESC Plan
Designer/Hydraulics Engineer and the Project Manager (or other such project
authority) during the initial stages of plan development.

  • Single Phase ESC Plan Concept

    • The Single Phase ESC Plan concept may be used on minor construction
     projects where all of the erosion and sediment control measures can be
     clearly depicted on the construction plan sheet (e.g., rural secondary
     project, minor urban widening project, bridge and approach project, etc.)

    • The ESC Plan shall address both those items requiring installation prior to
     the beginning of grubbing operations or the installation of major drainage
     structures and those items to be installed as grading operations and
     installation of minor drainage facilities progress. The ESC Plan shall
     contain or be accompanied by, at a minimum, all those items identified in
     Section 10.3.1 of this document (Contents of an ESC Plan).
In addition to standard plan symbols, supplemental notes/narratives may be used to clearly define the intent and purpose of the proposed erosion and sediment control measures and to define their sequence of installation. Some standard construction notes and symbols have been developed and are included as a part of the VDOT CADD Cell and Custom Line Style Library and the Geopak Road Plan View Labeler.

- Multiple Phase ESC Plan Concept

  - The Multiple Phase ESC Plan concept shall be used on construction projects where additional plan sheet(s) are needed in order to clearly depict the erosion and sediment control measures required at the various stages of construction (e.g., rural multi-lane roadway projects, major urban roadway projects, roadway projects on new locations, roadway projects through environmentally sensitive areas, etc.).

  - Projects may be developed using the Multiple Phase concept on only those portions of the project that require greater detail and clarity than that provided by the Single Phase concept (e.g., construction in environmentally sensitive areas or major waterway areas, areas where plan clutter reduces the ability to clearly show the erosion and sediment control items, and where grading operations are required prior to installation of major temporary ESC measures or permanent drainage improvements).

  - At a minimum, the multiple phase ESC Plan should be developed in two phases:
    - Phase I for those items that need to be installed prior to the beginning of grubbing operations or the installation of major drainage structures.
    - Phase II for those items that need to be installed as grading operations and installation of minor drainage facilities progress.

- Projects with complex grading operations and/or sequence of construction plans may warrant additional ESC Plan Phases to clearly identify all required ESC items.
Generally, the Phase I and the Phase II plan details (including associated narratives or notes) should each be depicted on a separate plan sheet following the applicable construction plan sheet (e.g., Construction Plan Sheet 5, Profile Sheet 5A, ESC Phase I Plan Sheet 5B, ESC Phase II Plan Sheet 5C).

When found appropriate, the Phase I and Phase II plan details may be depicted on a single plan sheet following the applicable construction plan sheet (e.g., Construction Plan Sheet 5, Profile Sheet 5A, ESC Phase I & II Plan Sheet 5B).

In general, when utilizing a separate plan sheet for the Phase I and the Phase II plan details, erosion and sediment control items (including protective linings in permanent ditches and channel relocations) depicted on the Phase I Plan Sheet should not be duplicated on the Phase II Plan Sheet. Temporary erosion and sediment control items depicted on the Phase I & II Plan Sheets should not be duplicated on the Construction Plan Sheet. Permanent drainage improvements identified for completion in Phase I, such as culverts, channels, etc, should also be shown on the Phase II plan.

The ESC Phase I Plan Sheet shall, at a minimum, depict the following:

- Existing contours and appropriate existing hydraulic and topographic features as referenced in the Survey File.
- Proposed centerline, edges of pavement, construction limits, and limits of disturbance.
- Permanent drainage culverts, temporary diversion channels and permanent channel relocations (including any protective linings required) involving natural drainage ways that would be constructed or installed prior to the start of grading operations.
- Temporary Sediment Basins (including grading contours, if applicable) that are to be constructed in the initial phases of the grading operations.
- Permanent stormwater management basins (including grading contours, if applicable) that will be utilized as temporary sediment basins and that are to be constructed in the initial phases of the grading operations.
- Diversion dikes, berm ditches and other perimeter ditches (including any required protective linings) that need to be installed prior to the start of grubbing or other earth moving operations.
- Temporary sediment traps, silt fences, rock check dams, turbidity curtains and any other perimeter controls that need to be installed prior to the start of grubbing or other earth moving operations.
- Inlet protection for existing inlets that require sediment control prior to initiation of land-disturbing activities in the contributing drainage area.
- Any necessary construction notes/narratives (to include the need/location for items not typically shown on the plan view such as temporary slope drains, construction entrances, etc.)

*Rev. 7/19*
The Phase II Plan Sheet shall, at a minimum, depict the following:

- Proposed centerline, edges of pavement, limits of disturbance* and construction limits.
- Any permanent drainage culverts and channel relocations involving natural drainage ways installed under the Phase I Plan.
- Temporary sediment basins and permanent stormwater management facilities installed under the Phase I Plan.
- All culverts, storm sewer pipe, drop inlets and associated drainage structures that will be installed as grading operations progress.
- All required protective ditch linings (e.g., Standard Rolled Erosion Control Product (RECP) Temporary or Rolled Erosion Control Product (RECP) Permanent, concrete, riprap, etc.), paved flumes and associated structures that will be installed as grading operations progress.
- Temporary sediment traps, silt fences, rock check dams, drop inlet silt traps, inlet protection, outlet protection, and any other erosion and sediment control measures needed to be installed as grading operations progress.
- Any necessary construction notes/narratives (to include the need/location for items not typically shown on the plan view such as temporary slope drains, construction entrances, etc.).

The following drainage items from the Phase I and II Plan Sheets shall be depicted on the Construction Plan Sheet:

- Permanent drainage culverts, storm sewer systems, drop inlets and associated structures.
- Permanent channel relocations involving natural waterways.
- Permanent stormwater management facilities.
- Protection ditch linings.
- Outlet protection.
- Rock checkdams that will be left in place after construction to serve as a permanent stormwater management structure.

### 10.3.3 SWPPP Applicability and Requirements

A SWPPP identifies potential sources of pollutants that may reasonably be expected to affect the stormwater discharges from the RLDA, and any support areas included in the VPDES Construction General Permit coverage for the RLDA, and describes and ensures the implementation of practices to minimize pollutants in such discharges. For the purposes of this document, the RLDA is defined as the proposed construction or maintenance related land-disturbing project or activity that generates the need for acquiring coverage under the Construction General Permit and/or requires an ESC Plan.

*Rev. 7/19
The required contents of a SWPPP for those land disturbance activities requiring coverage under the Construction General Permit are found in Part II of the Construction General Permit Regulation (9VAC25-880-70).

Except for the items dealing with the post construction stormwater management requirements, some of the items that must be addressed in the SWPPP (Sections I and II) for land disturbance activities requiring Construction General Permit coverage must also be addressed for those land disturbance activities that do not require Construction General Permit coverage but do require an ESC Plan in accordance with the requirements of the Virginia ESC Law and Regulations.

When the land-disturbing activity requires coverage under the Construction General Permit, the SWPPP must also include a copy of the Construction General Permit, the Construction General Permit Registration Information form LD-445, the Construction General Permit Contact Information form LD-445A, and the Construction General Permit coverage letter received from DEQ showing a project specific permit number.

The SWPPP for the RLDA is to include any onsite support facilities used exclusively for the RLDA (e.g., borrow and disposal sites, the contractor’s storage and fueling areas, etc.) (See the current version of IIM-LD-242 for guidance related to SWPPP information for support facilities).

For those RLDA requiring coverage under the Construction General Permit, Part II A.8 of the Construction General Permit Regulation (9VAC25-880-70) requires the SWPPP to be signed by a person so identified in Part III K.2 of that same document. For a State Agency, that person is the principal executive officer or designee. For VDOT projects, that authority has been delegated for each specific RLDA as listed on the SWPPP General Information Sheet I, Note 11, and acknowledge on Form LD-445H.

Many of the items required in the SWPPP are inherently contained in the construction plans (or other such documents) by means of the erosion and sediment control plans and the post construction stormwater management plans and in other VDOT documents such as the R&B Standards and Specifications, which can be incorporated into the SWPPP by reference.

10.3.4 SWPPP Certification

For those land-disturbing activities requiring coverage under the Construction General Permit, the Construction Permit requires that the SWPPP for any support facilities be included in the permit coverage for the RLDA be developed and included with the SWPPP for the RLDA prior to issuance of permit coverage.

On most VDOT land-disturbing activities, it is the responsibility of the contractor or other such person performing the land-disturbing activity to identify the location of the support facilities and provide the SWPPP for such to the RLD, District Drainage Engineer and District VPDES Coordinator for review and approval (see the current version of IIM-LD-242 for further discussion on support facilities).
Since the Construction General Permit coverage for VDOT RLDAs is normally obtained prior to the identification of the support areas, a mechanism is required whereby the project files can be documented, and DEQ can be assured, that all of the information for the support facilities, as well as other required information not available at the time the Construction General Permit coverage for the RLDA is applied for, has been, or will be, included in the SWPPP for the RLDA. The mechanism to be used for this purpose will be SWPPP Certification on the SWPPP General Information Sheet*.

The DEQ has approved the signature of the delegated authority on the SWPPP General Information Sheet, Section I Note 11 as acknowledged on form LD-445H as meeting the SWPPP signatory requirements contained in Part II A.8 of the Construction General Permit Regulation (9VAC25-880-70).

10.3.5 SWPPP General Information Sheets

In order to provide a clear understanding of what is required in a SWPPP and to provide a reference as to where those items are located within the contract/construction documents, a set of SWPPP General Information Sheets has been developed. The SWPPP General Information Sheets provide a summary of the information required in Part II of the Construction General Permit Regulation (9VAC25-880-70) and, where not included on the General Information Sheets, provide a reference to where that information can be found within the contract/construction documents for the RLDA (e.g., the construction plans or other such documents, the VDOT R&B Standards and/or Specifications, contractor supplied documents, etc.).

The SWPPP General Information Sheets incorporate many of the notes previously included in the ESC General Notes as well as those necessary to identify and describe the post construction stormwater management plan for the RLDA (if applicable).

The SWPPP General Information Sheets are to be included in the construction plan set (or other such documents) for all land disturbance activities requiring Construction General Permit coverage and/or an erosion and sediment control plan. Completion and inclusion of the SWPPP General Information Sheets in the contract documents satisfies one of the many requirements contained in the Construction General Permit. Those persons who oversee or perform activities covered by the Construction General Permit must review and understand all of the conditions and requirements contained within that permit.

The SWPPP General Information Sheets are updated from time to time to clarify and/or include additional requirements as a result of changes to the VSMP Regulation, the Construction General Permit or VDOT’s Approved ESC and SWM Standards and Specifications. Prior to finalization of the construction plans or other such documents for a proposed land disturbance activity, the Project Manager or other such project authority is to verify that the most recent SWPPP General Information Sheets are included.

*Rev. 7/19
The SWPPP General Information Sheets have been developed in two formats as follows:

- Available in the CADD sheet2015.cel cell library (referenced as SWPPP1, SWPPP2 SWPPP3 & SWPPP4) for use with those land disturbance activities that have a formal set of construction plans, such as those developed under a Minimum (M) Plan or Complete (C) Plan Process.

- Available in ProjectWise Central Office folder under the Engineering Services’ eng-ser directory (No Plan sub-directory) as an 8.5” X 11” letter size word document.

The SWPPP General Information Sheets are to be completed by the ESC Plan Designer, the Hydraulic Engineer or other such person who has the responsibility for developing the ESC and post construction SWM Plan (if applicable) for the RLDA.

Information required by those notes on the SWPPP General Information Sheets designated with an asterisk (*) is to be developed / provided by the VDOT RLD. Information required by those notes on the SWPPP General Information Sheets designated with a double asterisk (**) is to be provided / completed by the contractor.

All information/notes in Sections I through VI of the SWPPP General Information Sheets are applicable to land disturbance activities requiring coverage under the VPDES Construction General Permit.

For land disturbance activities requiring an ESC Plan but exempt from the VSMP Regulation or the need for coverage under the Construction General Permit, the information noted on the SWPPP General Information Sheets in Sections III, IV, V and VI, is typically not applicable. Those sections, as well as any other notes/information in other sections of the SWPPP General Information Sheets not applicable to a specific land disturbance activity should be deleted, struck through or noted as “NA” (i.e., not applicable to the land disturbance activity).

The permanent stormwater management facility (SWMF) or best management practice (BMP)* information (when applicable) in Section VI is to be completed by the Hydraulic Engineer (or other such person developing the post construction SWM Plan) and is to be based on the pre-construction design. This information is to be updated when any changes to the post construction SWM Plan are authorized during the construction phase of the activity. Such changes are to be made as a formal revision to the plans. When submitting a request for termination of the Construction General Permit coverage, the RLD is to use the information in the Permanent SWMF/BMP table(s) in completing the post construction SWM control information section on form LD-445D.

Some of the notes on the General Information Sheets (GIS) require project specific user input including input in the field by the RLD and the contractor as specified on the SWPPP GIS.

*Rev. 7/19
10.3.6 SWPPP Documents

For VDOT RLDAs, the required documents for a SWPPP shall include, but are not limited to, the following:

- The construction plans/documents.
- The SWPPP General Information Sheets (with all notes completed with appropriate information *as applicable*).
- The ESC Plan.
- The post construction SWM Plan (if applicable).
- Pollution Prevention (P2) Plan.
- A copy of the VPDES General Permit for Discharges of Stormwater from Construction Activities (the Construction General Permit) (when applicable).
- A copy of the Construction General Permit coverage letter received from DEQ (when applicable).
- A copy of the Construction General Permit Registration Information forms LD-445 and LD-445C (when applicable).
- Documents required to be developed/provided by the contractor for erosion and sediment control, stormwater management and pollution prevention associated with any support facilities to be included in the Construction General Permit coverage for the RLDA.
- All ESC and SWPPP inspection reports.
- All ESC and SWM design computations and supporting data.
- A Record Set of Plans (see Section 10.3.7 of this document for more information)

All documents related to the SWPPP for a RLDA (except for the ESC and SWM design computations and supporting data) shall be maintained at the activity site and shall be readily available for use by those with SWPPP implementation responsibilities. All documents related to the SWPPP for a RLDA shall be readily available for review by others upon request during normal working business hours. SWPPP related information not included in the construction plans/documents, the VDOT R&B Standards, Specifications, Supplemental Specifications, Special Provisions or Special Provision Copied Notes and the ESC and SWM design computation files is to be kept in a designated separate paper and/or electronic file. Where no facilities are available at the activity site to maintain the SWPPP documents, they are to be kept at a central location convenient to the activity site where they will be readily available for use by those with SWPPP implementation responsibilities and would be available for review by others upon request during normal business working hours.

*Rev. 7/19*
Where the SWPPP documents are not stored on site, a copy of such documents, except for the ESC and SWM engineering calculations and documentation, shall be in the possession of those with day to day operational control over the implementation of the SWPPP (e.g. the VDOT RLD, the VDOT ESC Inspector, the contractor’s ESCCC person, etc.) whenever they are on site.

10.3.7 SWPPP Components

The following list outlines the major components of a SWPPP, the person(s) responsible for ensuring that the component is addressed in the SWPPP for a RLDA and how that component is addressed in the construction plans or other such documents for a VDOT land-disturbing activity.

- A copy of the Construction General Permit coverage letter and form LD-445A are to be posted at the construction site.
  - The RLD ensures that a copy of the Construction General Permit Registration Information forms (LD-445 and LD-445C, and the Construction General Permit coverage letter received from DEQ are maintained in the SWPPP file for the RLDA.

- A copy of the Construction General Permit (when applicable).

- A narrative description of the nature of the construction activity, including the function of the project.
  - The ESC Plan Designer incorporates project specific information into the appropriate note(s) on the SWPPP General Information Sheets for the RLDA.

- The intended sequence and timing of activities that disturb soils at the site (e.g., grubbing, excavation, grading, utilities and infrastructure installation).
  - The Contractor or other such person develops/provides project specific information. The RLD ensures that the information is maintained in the SWPPP file for the RLDA.

- A record of the dates when major grading activities occur, when construction activities temporarily or permanently cease on a portion of the site, and when stabilization measures are initiated.
  - The Contractor or other such person develops/provides project specific information. The RLD ensures that the information is maintained in the SWPPP file for the RLDA.

- Estimates of the total area expected to be disturbed by excavation, grading, or other construction activities.
  - The ESC Plan Designer obtains the information and incorporates it into the appropriate note on the SWPPP General Information Sheets for the RLDA.

'Rev. 7/19
• A description of any other potential pollutant sources, such as vehicle fueling, storage of fertilizers or chemicals, sanitary waste facilities, etc.
  o The Contractor or other such person develops/provides project specific information. The RLD ensures that the information is maintained in the SWPPP file for the RLDA.

• Identification of the nearest receiving waters at or near the construction site that will receive discharges from disturbed areas of the RLDA.
  o The ESC Plan Designer determines the information and incorporates it into the appropriate note on the SWPPP General Information Sheets for the RLDA.

• The location and description of any discharge associated with industrial activity other than construction at the site. This includes stormwater discharges from dedicated asphalt plants and dedicated concrete plants that are covered by the Construction General Permit for the RLDA.
  o This information is covered by a standard note on the SWPPP General Information Sheets.

• A legible site map/plan identifying the following items:
  o Directions of stormwater flow and approximate slopes anticipated after major grading activities.

• The ESC Plan Designer ensures that the appropriate information (e.g., grading contours, typical sections, profiles and/or cross sections) is included in the construction plans or other such documents for the RLDA.
  o Areas of soil disturbance and areas of the site which will not be disturbed.
    ▪ The ESC Plan Designer ensures that the appropriate information (e.g., plan view construction limits and/or typical sections/cross sections) is included in the construction plans or other such documents for the RLDA.
  o Locations of major structural and nonstructural control measures identified in the SWPPP, including those that will be permanent after construction activities have been completed.
    ▪ The ESC Plan Designer ensures that the appropriate information is included in the construction plans or other such documents for the RLDA.
  o Locations where stabilization practices are expected to occur.
    ▪ The ESC Plan Designer ensures that the appropriate information (e.g., plan view construction limits and/or typical sections/cross sections) is included in the construction plans or other such documents for the RLDA.
  o Locations of surface waters.
    ▪ The ESC Plan Designer ensures that the appropriate information is included in the construction plans or other such documents for the RLDA.
  o Locations where concentrated stormwater discharges from the construction site.
    ▪ The ESC Plan Designer ensures that the appropriate information is included in the construction plans or other such documents for the RLDA.
o Locations of any support areas (e.g., material, waste, borrow or equipment storage areas) that are to be included in the permit coverage and the SWPPP for the RLDA.
  ▪ The Contractor or other such person provides project specific information. The designated RLD ensures that the information is maintained in the SWPPP file for the RLDA.

o Locations of other potential pollutant sources, such as vehicle fueling, storage of chemicals, concrete wash-out areas, sanitary waste facilities, including those temporarily placed on the construction site, etc.
  ▪ The Contractor or other such person provides project specific information. The designated RLD ensures that the information is maintained in the SWPPP file for the RLDA.

o Areas where final stabilization has been accomplished.
  ▪ The Contractor or other such person provides project specific information. The designated RLD ensures that the information is maintained in the SWPPP file for the RLDA.

• The SWPPP shall include a description of all control measures that will be implemented as part of the construction activity to minimize pollutants in stormwater discharges. For each major construction activity identified, the SWPPP shall clearly describe appropriate control measures, the general sequencing during the construction process in which the control measures will be implemented, and which operator (i.e., contractor) is responsible for implementation of the control measure.

o The ESC Plan Designer/Hydraulics Engineer develops the ESC Plan and the SWPPP for inclusion in the construction plans/documents for the RLDA. The Contractor or other such person develops/provides proposed revisions to the ESC Plan and the SWPPP as necessary to meet differing field conditions or construction sequencing. The VDOT ESC Inspector reviews and the VDOT RLD approves any changes to the ESC Plan and the SWPPP. The RLD ensures that all required information is maintained in the SWPPP file and/or documented on the Record Set of Plans for the RLDA in accordance with Section 107.16(e) of the 2016 Road and Bridge Specifications.

• The SWPPP shall include a description of all erosion and sediment control measures (including supporting calculations) that will be installed during the construction process to control any potential pollutants in stormwater discharges from the construction site.

o The ESC Plan Designer develops the ESC Plan and required calculations for the RLDA. The ESC Plan is incorporated into the construction plans/documents for the RLDA. The ESC calculations are maintained in the project hydraulic files and the location of such files is documented by the ESC Plan Designer in the appropriate note on the SWPPP General Information Sheets for the RLDA.

• The SWPPP shall describe measures to prevent the discharge of solid materials, including building materials, garbage, and debris to state waters, except as authorized by a Clean Water Act §404 permit.

o This information is covered by a standard note on the SWPPP General Information Sheets.
The SWPPP shall describe control measures used to comply with applicable state or local waste disposal, sanitary sewer or septic system regulations.

- This information is covered by a standard note on the SWPPP General Information Sheets.

The SWPPP shall include a description of construction and waste materials expected to be stored on site, with updates as appropriate. The SWPPP shall also include a description of controls, including storage practices, to minimize exposure of the materials to stormwater and for spill prevention and response.

- The Contractor or other such person develops/provides project specific information. The designated RLD reviews and approves the information and ensures that copies of such are maintained in the SWPPP file for the RLDA.

The SWPPP shall include a description of, and all necessary calculations supporting, all post-construction stormwater management facilities (BMPs) that will be installed prior to the completion of the construction process to control pollutants in stormwater discharges after construction operations have been completed.

- The Hydraulic Engineer develops the post construction SWM Plan and required calculations. The post construction SWM Plan is incorporated into the construction plans/documents. The post construction SWM calculations are maintained in the project hydraulic files and the location of such files is documented by the Hydraulic Engineer in the appropriate note on the SWPPP General Information Sheets for the RLDA.

The SWPPP shall include a description of pollutant sources from any applicable support areas and a description of the control measures that will be implemented at those sites to minimize pollutant discharges.

- The Contractor or other such person develops/provides project specific information. The designated RLD reviews and approves the information and ensures that copies of such are maintained in the SWPPP file for the RLDA.

The name and phone number of qualified personnel conducting the ESC inspections shall be included in the SWPPP.

- The VDOT RLD provides the appropriate information on the SWPPP General Information Sheets or SWPPP Certification and ensures a copy is maintained in the SWPPP file for the RLDA.

A report summarizing the scope of the ESC and SWPPP self inspections, names and qualifications of personnel making the inspections, the dates of the inspections, major observations relating to the implementation of the SWPPP, and any corrective actions taken.

- The Contractor’s Erosion and Sediment Control Contractor Certified (ESCCC) person conducts initial inspections and completes the Construction Runoff Control Inspection Form C-107 Part I. The VDOT Certified ESC Inspector verifies inspection information on Form C-107 Part I and the RLD ensures that all of the C-107 Part I forms are maintained in the SWPPP file for the RLDA.

*Rev. 7/19*
The Project Manager, Project Engineer, Area Construction Engineer, or designated individual is responsible for completing and documenting VSMP authority periodic inspections on Form C-107 Part II. The individual signing the form on behalf of VDOT must be certified as a DEQ Stormwater Management Inspector. Copies of the Form C-107 Part II and associated Corrective Action Plans are maintained in the SWPPP file for the RLDA.

Where the RLDA discharges to a surface water with an approved (as of the effective date of the VPDES Construction General Permit) Total Maximum Daily Load (TMDL), the pollutant identified in any Waste Load Allocation (WLA) assigned to a construction activity must be identified in the SWPPP. The SWPPP shall include strategies and control measures to ensure consistency with the assumptions and requirements of any TMDL WLA that applies to the operator's discharge including the following items:

- Permanent or temporary soil stabilization shall be applied to denuded areas within seven days after final grade is reached on any portion of the site.
- Nutrients shall be applied in accordance with manufacturer's recommendations or an approved nutrient management plan and shall not be applied during rainfall events.
- SWPPP inspection requirements shall be amended as follows:
  - Inspections shall be conducted at a frequency of (i) at least once every four business days or (ii) at least once every five business days and no later than 24 hours following a measurable storm event. In the event that a measurable storm event occurs when there are more than 24 hours between business days, the inspection shall be conducted on the next business day; and
  - Representative inspections used by utility line installation, pipeline construction, or other similar linear construction activities shall inspect all outfalls discharging to surface waters identified as impaired or for which a TMDL wasteload allocation has been established and approved prior to the term of this general permit.

Information contained in the SWPPP shall be updated as necessary by the RLD or the RLD's designee to reflect changes required due to differing field conditions and/or construction sequencing. Such changes as well as other information requiring documentation as construction activities are initiated or completed is to be maintained on or with a Record Set of Plans (the Record Plan Set).

The Record Set of Plans is a paper or electronic copy of the construction plans that is used to document/record the following information:

- Approved changes/modifications to the proposed ESC Plan.
- Approved changes/modifications to other components of the SWPPP.

*Rev. 7/19*
• Required SWPPP information such as:
  o Dates of beginning and end of major grading operations.
  o Dates of initiation and completion of temporary/permanent stabilization practices.
  o Locations of material, waste, borrow or equipment storage areas included in the project’s Construction General Permit coverage.
  o Locations of other potential pollutant sources, such as vehicle fueling, storage of chemicals, concrete wash-out areas, sanitary waste facilities, etc., placed on the construction site.
  o Areas where final stabilization has been accomplished.
• The Record Plan Set shall be kept current and shall reflect up to date conditions of the RLDA.
• The Record Plan Set must be maintained at the project site and be available for review upon request (see Section 10.3.6 of this document for exceptions).

10.3.8 Computations

All computations to support the ESC and related post construction SWM Plan, and the drainage design plan, including the drainage area map, shall be developed in accordance with the instructions contained in the VDOT Drainage Manual, Hydraulic Design Advisories, and related Informational and Instructional Memoranda and shall be made part of the project file and the SWPPP for the land disturbance activity.

10.3.9 Field Revisions and Evaluations

The ESC Plan must be fully and effectively implemented throughout the entire construction phase of the project.

During the construction phase of the project, the Project Engineer, the Project ESC Inspector, District NPDES Coordinator, * and the contractor shall continuously evaluate the project for areas that may require the deletion/addition/modification of the proposed erosion and sediment control measures/plan in order for the project to remain in compliance with the approved VDOT ESC Standards and Specifications, the Virginia ESC Law and Regulations, and the VPDES Construction General Permit conditions (where applicable). Changes in the proposed ESC Plan may be needed due to unforeseen site conditions, contractor scheduling, changes in the proposed sequence of construction or other factors unknown at the time of the development of the proposed ESC Plan. See Figure 10-1 for a flow chart showing the plan revision process.

• Minor changes to the proposed ESC Plan (e.g., deletion/addition/modification to non-engineered items such as silt fence, check dams, inlet protection, etc.) may be approved/authorized by the VDOT DEQ Certified Inspector and/or the designated RLD for the activity.

*Rev. 7/19
• When changes to the proposed ESC Plan require detailed hydrologic/hydraulic engineering analysis/calculations (e.g., deletion/addition/modification to engineered items such as sediment traps, sediment basins, etc.), the Project Engineer and/or the Project ESC Inspector shall coordinate a site inspection with the District Hydraulics Engineer and/or the ESC Plan Designer/Hydraulics Engineer. The site inspection should be used to assemble detailed notes, sketches, and photographs to formally document the need for ESC Plan changes. The ESC Plan Designer and/or Hydraulics Engineer will provide the appropriate engineering analysis to document the required changes and to ensure the ESC Plan’s continued compliance with the approved VDOT ESC Standards and Specifications, Virginia ESC Law and Regulations, and Construction General Permit conditions (where applicable).

• Any authorized changes to the proposed ESC Plan must be noted on a designated plan set (Record Set) which shall be retained on the project site and made available upon request (see Section 107.16(e) of the 2016 VDOT R&B Specifications).

During the construction phase of the project, the Project Engineer Project ESC Inspector, and/or District NPDES Coordinator will periodically, upon request, provide the ESC Plan Designer and/or Hydraulics Engineer with a detailed evaluation report that notes the success or failure of the proposed erosion and sediment control measures depicted in the construction plans (or other such documents) and/or the implementation of different measures as a result of new technologies/products. The VDOT MS4/Stormwater Engineer is to be provided a copy of all such reports.

**10.3.10 Maintenance**

Maintenance of the erosion and sediment control items must be continually provided during the duration of the land disturbance activity. Maintenance and corrective actions taken under the CGP shall be noted in the SWPPP documentation maintained onsite during land-disturbing.

The inspection and maintenance of all temporary and permanent erosion and sediment controls shall be conducted in accordance with Sections 107.16 and 303.03 of the 2016 VDOT R&B Specifications.

Accumulated sediment shall, at a minimum, be removed from erosion and sediment control facilities in accordance with Section 303.03 of the 2016 VDOT R&B Specifications.

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*Rev. 7/19*
Figure 10-1 Plan Modification Process Flow Chart
10.3.11 Standard Forms

LD-445A VPDES CONSTRUCTION PERMIT CONTACT INFORMATION AND SWPPP AVAILABILITY FORM

LD-445 GENERAL VPDES PERMIT FOR DISCHARGES OF STORMWATER FROM CONSTRUCTION ACTIVITIES TRACKING FORM

LD-445C EROSION AND SEDIMENT CONTROL (ESC) AND STORMWATER MANAGEMENT (SWM) PLAN CERTIFICATION FORM

LD-445D VPDES CONSTRUCTION GENERAL PERMIT COVERAGE NOTICE OF TERMINATION AND CHESAPEAKE BAY PRESERVATION AREA REPORTING NOTICE

LD-445H DELEGATION OF AUTHORITY

C-107 Part I Construction Runoff Control Inspection Form, Contractor Inspection Sheet

C-107 Part II Construction Runoff Control Inspection Form, VDOT Inspection Sheet

For the current version of these forms, see the VDOT site at: http://vdotforms.vdot.virginia.gov/.

See Section I Note #15 of the SWPPP General Information Sheets for a list of LD-445 forms and the use on different project types.
10.4 References


*Rev. 7/19
Appendix 10B-1 Erosion and Sediment Control Plan Details

10.1 DESIGN GUIDELINES

10.1.1 References

In addition to the information contained herein, the following references contain design and/or construction guidelines and details:

- VDOT Road and Bridge Standards (R&B Standards)
- VDOT Road and Bridge Specifications (R&B Specifications)
- DCR/DEQ Virginia Erosion and Sediment Control Handbook (VESCH)
- VDOT Drainage Manual (VDM)
- L&D Instructional and Informational Memoranda (IIM-LD)
- State Hydraulic Engineer Directives (SHED)

10.1.2 Diversion of Off-Site Stormwater Runoff

- Stormwater runoff from areas outside the project limits shall, where practical, be diverted around the disturbed areas of the project.

- Erosion and sediment control measures such as temporary diversion channels (R&B Standard EC-12), diversion dikes (R&B Standard EC-9), stabilized channels, etc., shall be used to limit the stormwater runoff flowing across the disturbed areas of the project.

- Where diversion of runoff from offsite areas is impractical, the flow can be conveyed through the disturbed area in a culvert or a stabilized channel or ditch (R&B Standards EC-12). Erosion and sediment control measures, such as temporary silt fence, shall be provided along the sides of the ditch or channel to prevent sediment from adjacent disturbed areas from entering the ditch or channel.

10.1.3 Culvert Outlet Protection - R&B Standard EC-1 & EC-3 Rolled Erosion Control Product Permanent (Reference VESCH Standards 3.18, 3.19, and 3.36)

- Erosion control protection shall be provided at the outlet of each culvert where required in accordance with the guidelines set forth in Chapter 8 of this Manual.

Due to the magnitude of the changes to this section, shading has been omitted.
• The placement of the outlet protection shall be in accordance with Standard Drawing EC-1 or EC-3 Rolled Erosion Control Product (RECP) Permanent.

• The Project Engineer, District NPDES Coordinator, and/or the District Environmental Staff shall inspect the outlet ends of all culverts during the construction phase of the project. Where not specified on the plans, but warranted by field conditions, additional outlet protection shall be added in order to ensure the stability of the area adjacent to the culvert outlet.

10.1.4 Rolled Erosion Control Product (RECP) – R&B Standards EC-2 and EC-3 (Reference VESCH Standard 3.36)

• Rolled Erosion Control Product (RECP) consists of a material used to provide temporary soil stabilization (EC-2 Protective Covering), or permanent soil stabilization and outlet protection (EC-3 Soil Stabilization Mat).

• EC-2 Protective Covering is a temporary product used to stabilize a ditch, slope, or other disturbed area until permanent vegetation is established. It is specified in cases where the permanent vegetation can resist erosion from expected tractive forces from flowing water without the need of additional lining.

• EC-3 Soil Stabilization Mat is a permanent product used in conjunction with permanent vegetation to stabilize a ditch, culvert outlet, slope, or other disturbed area where the expected tractive force from flowing water exceeds the ability of permanent vegetation to resist erosion on its own.

• See the VDOT Drainage Manual Chapters 7 and 8 for determining how to design and specify RECPs for stabilization on regulated land disturbing activities, including channel linings and culvert outlet protection.

10.1.5 Rock Check Dams - R&B Standard EC-4 (Reference VESCH Standard 3.20)

• Type I Rock Check Dams are to be used where the total contributing drainage area to the device is 2.0 acres or greater.

• Type II Rock Check Dams are to be used when the total contributing drainage area to the device is less than 2.0 acres.

• Rock Check Dams may be designated as permanent SWM structures that are to be left in place after completion of the project in order to function as a part of the overall SWM Plan for the project. Rock Check Dams designated as permanent structures, and located within the clear zone adjacent to a travelway, shall be designed so as not to present a hazard to traffic (see R&B Standard Drawing EC-4).
Temporary check dams that are not constructed of rock and are not intended to be permanent are discussed below under EC-16 Temporary Check Dams. Temporary check dams should be specified in the plans during design and not used as a substitute during construction for Rock Check Dams specified on the plan.

Rock check dams shall not be used in a live watercourse unless specifically permitted by the U.S. Army Corp of Engineers, Virginia DEQ, or VRMC.

Temporary rock check dams are not considered a substitute for perimeter controls, sediment traps, or sediment basins on a regulated land disturbing activity.

10.1.6 Temporary Silt Fence - R&B Standard EC-5 (Reference VESCH Standard 3.05)

Temporary Silt Fence is to be used to control sediment in non-concentrated (sheet) flow areas.

Temporary Silt Fence is to be used at the toe of embankments (see R&B Standards).

Additional erosion and sediment control measures must be provided to supplement Temporary Silt Fences located along the toe of embankments where the area draining to Temporary Silt Fence exceeds 10,000 square feet per 100 linear feet of Silt Fence.

The slope above a Silt Fence shall be no greater than 50% or 2H:1V. The slope length behind Silt Fence shall be no greater than 100 feet.

Type A Silt Fence shall consist of VDOT approved woven geotextile with wooden posts on 4 foot centers.

Type B Silt Fence shall consist of VDOT approved woven geotextile with steel posts on 4 foot centers.

10.1.7 Brush Barriers - R&B Standard EC-5 (Reference VESCH Standard 3.06)

Brush Barriers may be used to control sediment in non-concentrated (sheet) flow areas.

Additional erosion and sediment control measures must be provided to supplement Brush Barriers located parallel along the toe of embankments if the area draining to the Brush Barrier exceeds 10,000 square feet per 100 linear feet of Brush Barrier.
• It is desirable, where feasible, that Brush Barriers remain in place after completion of the project in order to provide an area for wildlife habit. Any Brush Barriers left in place must have the geotextile fabric removed.

10.1.8 Inlet Protection - R&B Standard EC-6 (Reference VESCH Standards 3.07 and 3.08)

• Provide Inlet Protection Type A at:
  o Grate inlets in graded median and roadside ditches.
  o Grate inlets in sump areas.
  o Grate inlets in other ditch locations or areas of concentrated flow.

• Provide Inlet Protection Type B at curb opening inlets as needed.

• Provide Inlet Protection Type C at culvert and pipe inlets as needed.

• Sediment forebays shall be utilized at drop inlet locations where increased efficiency of sediment removal is desired. The need for sediment forebays may be determined during the design phase of the project by the designer or by the District Hydraulics Engineer; or during the construction phase of the project by the Area Construction Engineer or District NPDES Coordinator.

10.1.9 Sediment Traps – R&B Standard EC-7 (Reference VESCH Standard 3.13)

• Temporary Sediment Traps should be used to detain sediment-laden runoff from small disturbed areas. Use of Temporary Sediment Traps should be limited to those locations where the total contributing drainage area is less than 3 acres.

• Temporary Sediment Traps are normally located in areas of concentrated flow. The outflow from Temporary Sediment Traps is normally controlled by the use of a rock weir (similar to an EC-4 Type I rock check dam). The length of the rock weir outlet structure shall be 6.0 linear feet per acre of drainage (e.g., a sediment trap serving a drainage area of 2.5 acres would be 2.5 x 6.0 = 15.0 ft). Discharges from a sediment trap should be directed to a stabilized channel or stabilized area.

• Temporary Sediment Traps shall not be constructed in live streams.

• The storage volume for Temporary Sediment Traps shall be 134 cubic yards per acre of the total contributing drainage area and shall consist of 50% in the form of wet storage and 50% in the form of dry storage.
The need and location for Temporary Sediment Traps is to be determined by the designer based on the anticipated sequence of construction and total contributing drainage area. The general design for Temporary Sediment Traps is to be in accordance with the details shown on R&B Standard Drawing EC-7 of the R&B Standards. Specific dimensions for each Temporary Sediment Trap are to be determined by the designer and summarized on the Temporary Sediment Trap Detail Sheet.

The Project Engineer, in conjunction with the District NPDES Coordinator, shall determine the time schedule for the removal of the Temporary Sediment Traps.

10.1.10 Temporary Sediment Basins (Reference VESCH Standard 3.14)

Temporary Sediment Basins should be used to detain sediment laden runoff from disturbed areas where the total contributing drainage area is 3 acres or greater. The maximum drainage area controlled by a Temporary Sediment Basin should not exceed 100 acres, unless specifically designed by a qualified professional licensed to practice in the Commonwealth of Virginia.

The storage volume for Temporary Sediment Basins shall be 134 cubic yards per acre of the total contributing drainage area. The storage volume shall consist of 50% in the form of wet storage (permanent wet pool) and 50% in the form of dry storage. The hydraulic performance of the Temporary Sediment Basin shall be predicated on the runoff from the contributing drainage area, not just the contributing disturbed area.

The minimum drawdown time for the dry storage volume shall be 6-hours, and the outlet structures shall be designed to convey the 25-year storm event without overtopping an earthen embankment associated with the basin.

The need and location for Temporary Sediment Basins is to be determined by the designer based on the anticipated sequence of construction.

Specific details and dimensions for each Temporary Sediment Basin are to be determined by the designer and the design details (including wet and dry storage volumes) are to be included in the construction plans.

Concentrated stormwater discharges from Temporary Sediment Basins shall be discharged to a receiving channel and energy dissipators shall be placed at the outfall to provide a stabilized transition from the facility to the receiving channel per Minimum Standard 19 (MS-19) of the Virginia Erosion and Sediment Control Regulations.

*Due to the magnitude of the changes to this section, shading has been omitted.*

5 of 21 VDOT Drainage Manual
The designer is referred to the Virginia Erosion and Sediment Control Handbook Standard 3.14 for further design parameters and construction details.

The Project Engineer, in conjunction with the District Hydraulic Engineer and District NPDES Coordinator, shall determine the time schedule for removal of Temporary Sediment Basins.

Permanent Stormwater Management (SWM) basins may be used as temporary sediment basins during the construction phase of the project by modifying the outflow control structure in order to provide the required wet and dry storage volumes. Typical details for modifying a standard riser structure are shown on R&B Standard Drawing SWM-DR.

10.1.11 Dewatering Basins - R&B Standard EC-8 (Reference VESCH Standard 3.26)

Dewatering Basins are provided to receive sediment-laden water pumped from a construction site in order to allow for filtration before the water reenters a natural watercourse or storm sewer system.

Accumulated sediment in the Dewatering Basin shall be removed and disposed of in an approved disposal area outside of the 100-year flood plain, unless otherwise noted on the plans.

Surface water flow shall be diverted around the Dewatering Basin.

A stabilized conveyance shall be provided from the outlet of the Dewatering Basin to the receiving channel when concentrated flows are expected from the basin.

The need for Dewatering Basins is to be determined by the Hydraulics Engineer during the design phase of the project.

The field location of Dewatering Basins is to be determined by the Contractor during the construction phase of the project.

During the construction phase of the project, the Project Engineer and/or the District NPDES Coordinator may approve the use of a synthetic dewatering basin in lieu of the dewatering basin shown on Standard Drawing EC-8 of the R&B Standards provided that there is no additional cost to the Department regardless of the number of synthetic dewatering basins required for each site.
10.1.12 Temporary Diversion Dike – R&B Standard EC-9 (Reference VESCH Standard 3.)

- Temporary Diversion Dikes divert storm runoff from upslope areas away from disturbed areas and slopes. They can also be used to divert sediment-laden runoff from a disturbed area to a sediment-trapping facility such as a sediment trap or basin.

- Temporary Diversion Dikes must be stabilized immediately following installation to prevent erosion of the dike itself.

- The maximum allowable drainage area is 5 acres, unless designed by a licensed professional.

- Diverted runoff free of sediment must be released through a stabilized outlet or channel.

- Sediment-laden runoff must be diverted and released through a sediment-trapping facility such as a Sediment Trap or Sediment Basin.

10.1.13 Slope Drains - VDOT R&B Standard EC-10 (Reference VESCH Standard 3.15)

- Slope Drains are to be used in high (8’ or greater), long fill situations to control slope erosion. Exceptions would be where the length of fill is less than 100’ or at bridge locations where run-off is being handled by other means.

- The need for Slope Drains is to be determined by the designer.

- During the construction phase of the project, the Project Engineer and/or the District Hydraulic Engineer may require additional slope drains as dictated by field conditions.

10.1.14 Stabilized Construction Entrances - R&B Standard EC-11 (Reference VESCH Standard 3.02)

Wherever construction traffic will enter or cross a public road, a stabilized construction entrance is required to minimize the transporting of sediment onto the adjoining surface. This entrance is to be constructed in accordance with the details shown on Standard Drawing EC-11 of the R&B Standards.

In areas where clay or other soils that can be easily tracked onto a public roadway are encountered, a wash rack may be necessary to facilitate removal of sediment from vehicles using the entrance. Sediment laden wash water and runoff from a wash rack shall be directed to an approved sediment trapping device.

*Due to the magnitude of the changes to this section, shading has been omitted.*
• Surface water shall be piped under the construction entrance. If piping is impractical, a mountable berm with 5:1 slopes will be permitted.

• Maintenance must be provided to assure continuous performance of the stabilized construction entrance, including wash rack when used.

• The need and potential locations for stabilized construction entrances should be discussed at the Field Inspection meeting or discussed with the appropriate District Hydraulic Engineer, District NPDES Coordinator, or Construction Engineer.

• Whether a construction entrance is used or not, all sediment tracked onto public or paved roads must be cleaned up by the Contractor immediately and at the end of each working day. Where Safety and the Maintenance of Traffic (MoT) requires, tracked sediment should also be removed.

10.1.15 Temporary Diversion Channel - R&B Standard EC-12 (Reference VESCH Standards 3.24 and 3.25)

• A Temporary Diversion Channel should be used where culvert installation is proposed in a live stream environment (perennial or intermittent) and where it will be necessary to divert the stream in order for the culvert to be installed in the dry.

• The designer, using USGS Topographical Maps and/or field observations, shall determine the need for a Temporary Diversion Channel and identify the most feasible location for the channel.

• A temporary diversion channel can also be used to divert offsite runoff around a regulated land-disturbing activity.

• When it is determined that a Temporary Diversion Channel is required, the designer shall determine the following:
  o The length of the Temporary Diversion Channel.
  o The bottom width of Temporary Diversion Channel necessary to essentially match that of the existing low water stream channel.
    ▪ The depth of the Temporary Diversion Channel (average ground surface elevation minus average natural streambed elevation).
    ▪ The class of lining required based on the following:
      • Specify Class A Lining where the Temporary Diversion Channel slope is less than 2 percent.
Chapter 10 – Erosion and Sediment Control

- Specify Class B Lining where the Temporary Diversion Channel slope is equal to or greater than 2 percent.

- The location of the Temporary Diversion Channel should be shown on the appropriate ESC plan sheet, when using the Multiple Phase ESC Plan concept, or the Construction plan sheet, when using the Single Phase ESC Plan concept.

- Temporary Silt Fence shall be provided along both sides of the Temporary Diversion Channel.


- When a culvert requires multiple lines or barrels to convey flow under a stream crossing, and countersinking of the culverts is required for aquatic organism passage, a Riprap Weir should be specified to divert low flow and base flow to one culvert barrel. Final design of the Riprap Weir should be coordinated with the Environmental staff to assure that water quality (401/404) permit conditions are satisfied.

- The hydraulic engineer shall design the Riprap Weir structure to promote aquatic organism passage at low flows, but ensure the hydraulic capacity of the multiple line culvert is maintained at design flows and check flows.

- As an alternative design, one of the barrels in a multiple line culvert could be countersunk and the other barrel inverts specified at a higher elevation than the countersunk barrel to divert low flow and base flow to the lower barrel.


- Minimum Standard 12 of the Virginia Erosion and Sediment Control Regulations (9VAC25-840-40 12.) requires when a vehicle or equipment cross a watercourse more than twice in a 6 month period, as stabilized vehicular crossing must be utilized to reduce erosion and sedimentation associated with the crossing.

- VDOT requires that vehicle or equipment crossing of the regulated waters of the United States or waters of the State be conducted using a Temporary Vehicular Watercourse Crossing.
• Standard EC-14 uses a temporary culvert crossing constructed of nonerodible fill consisting of Class I Dry Riprap underlain with geotextile materials and topped with #1 Coarse Aggregate.

• The culverts shall be designed to convey the peak flow from a 2-year storm event.

• The water surface elevation for the 2-year design storm event shall be at or below the lowest surface elevation of the crossing.

• The standard EC-14 design should only be used when the total drainage area at the crossing is less than 1 square mile.

• The depth of riprap and aggregate over the culverts shall be the minimum specified in R&B Standard PC-1.

• Alternative temporary stream crossing design may be used provided it is submitted to the Engineer for review and approval in accordance with the R&B Specifications 105 and 107.

• Note that temporary vehicular stream crossings must meet federal, state, and local requirements for work in or crossing a live watercourse (9VAC25-840-40 14.)

10.1.18 Slope Interrupter Devices – R&B Standard EC-15

• Temporary slope interrupter devices such as fiber rolls may be provided to trap sediment and moisture on slopes until vegetation can provide long-term stabilization.

• Provide slope interrupter devices for all slopes 3:1 and steeper with a minimum slope length of 25’. Maximum spacing of 25’ is required for installation of slope interrupter devices when their use is required.

• Note that slope interrupter devices are not meant to replace the use of temporary silt fence (EC-5) at the perimeter of a project where sediment laden sheet flow runoff is expected.

10.1.19 Temporary Check Dams – R&B Standard EC-16

• Temporary Check Dams may be specified on the ESC plan for ditches or swales where the total drainage area to the check dam is less than or equal to 1-acre and the peak flow to the check dam is 1 cfs or less in a 2-year, 24-hour storm event.
• Drainage areas greater than 1 acre or peak flows greater than 1 cfs require the use of EC-4 Rock Check Dams.

• Temporary Check Dams are not allowed as a substitute for EC-4 Rock Check Dams.

10.1.20 Turbidity Curtains – R&B Specification 303.03(i) (Reference VESCH Standard 3.27)

• A Turbidity Curtain is used to provide sedimentation protection for a watercourse from up-grade land disturbance or from dredging or filling operations within the watercourse.

• A Turbidity Curtain may be used in both non-tidal and tidal watercourses where intrusion into the watercourse by construction activities or sediment movement is unavoidable.

• Turbidity Curtains should not be placed across the main flow of a significant body of moving water but instead should be located parallel to the direction of flow.

• The Turbidity Curtain should extend for the entire depth of the water to the bed (bottom) of the channel except in locations subject to tidal action and/or significant wind or wave forces.

• At locations subject to tidal action and/or significant wind and wave forces, the bottom of the Turbidity Curtain should extend no closer than 1.0’ (0.3 m) above the bed (bottom) of the channel at mean low water.

• An impervious material should be used for the Turbidity Curtain for general applications.

• A pervious material should be used for the Turbidity Curtain for special applications in areas of tidal or moving water where there is a need to extend the curtain all the way to the bed (bottom) of the channel.

• The maximum depth (height) of the curtain shall be no greater than 10 feet (3.0 m) for all stages of water level anticipated during the duration of the curtain’s installation.

• The designer is referred to the Virginia Erosion and Sediment Control Handbook for further design parameters and construction details.
10.2 DESIGN CONSIDERATIONS

10.2.1 Right-of-way/Easement

- Prior to the Public Hearing Stage of the project, the need for fee right-of-way, permanent easement or temporary easement to accommodate the construction and maintenance of temporary diversion channels, sediment traps, sediment basins or other perimeter erosion and sediment control devices should be addressed.

- All right-of-way or easements needed to accommodate the construction and maintenance of temporary diversion channels and erosion and sediment control measures shall be shown on the plans prior to their submission for right-of-way acquisition.

10.2.2 Safety

- Guardrail or fencing around sediment traps or sediment basins should be specified where it is determined to be needed for the safety of pedestrians or vehicles.

- The need for guardrail or fencing should be determined by the District Construction Engineer or other person so designated.

10.2.3 Maintenance Access

- The need to maintain erosion and sediment control, control measures during construction shall be considered in the development of the ESC plan.

- The plan design shall incorporate a means of access (e.g., sufficient right-of-way, easements, flattened slopes, etc.) for the maintenance of sediment traps, sediment basins and other erosion and sediment control measures.

10.3 PLAN DETAILS

10.3.1 Symbols

Standard symbols are to be used to depict erosion and sediment control items on the plans in accordance with General Note E-3 shown in the latest Location and Design Instructional and Informational Memorandum 110 (IIM-LD-110) and in accordance with instructions in the VDOT CADD Manual.
10.3.2 Check Dams

Rock Check Dams that are to function as a part of the permanent SWM Plan for the project should be designated on the plans as follows:

“Rock Check Dam Type (specify I or II) - Permanent SWM Structure (to remain in place after project completion).”

10.3.3 Dewatering Basins

- Do not show specific locations on the plans.
- The description of the applicable drainage structure (or a separate description note when utilizing individual sheets to depict a phased ESC Plan) should note the need for a Dewatering Basin(s) and specify the number required.

10.3.4 Stabilized Construction Entrances

The specific location(s) of Stabilized Construction Entrances will not be shown on the construction plans prior to the construction phase. A note should be included on the appropriate plan sheet(s) specifying the general location (station, lane, roadway, etc.) where it is anticipated that Stabilized Construction Entrances will be required. During the construction phase, the contractor shall show the location of the stabilized construction entrance(s) in the SWPPP or associated documents.

10.3.5 Silt Fence Geotextile

Where existing fence is available for the attachment of the silt fence geotextile, the plans are to specify the following: “Silt Fence Geotextile Req’d. (Attach to Exist. Fence).”

10.3.6 Slope Drains

The specific locations of Slope Drains will not be shown on the plans. A note should be included on the appropriate plan sheet(s) specifying the general location (station to station, lane, roadway, etc.) and estimated quantity of Slope Drains and Culvert Outlet Protection Class 1, Standard EC-1 required. During the construction phase, the contractor shall show the location of slope drains in the SWPPP or associated documents.
10.3.7 Temporary Diversion Channel

- When the location is shown on an individual phased ESC Plan Sheet, the description for the Temporary Diversion Channel should specify the width of the channel required and the class of lining required (A or B). Note that Temporary Silt Fence is required along both sides of the Temporary Diversion Channel as specified in the standard.

- When the location is shown on the Construction plan sheet, the description for the Temporary Diversion Channel should be included in the description for the applicable drainage structure. The following information should be included in the drainage description:

  “Temporary Diversion Channel Req’d. Width = (specify) (specify) cu. yds. Temporary Diversion Channel Excavation (specify) sq. yds. Temporary Diversion Channel Lining, Class (specify) (specify) ft. (m) Temporary Silt Fence Req’d.”

- The plan description calls attention to the need for a Temporary Diversion Channel and defines the width of the channel and the class of lining required.

- The designer should be liberal when estimating the length of Temporary Diversion Channel required in order to avoid significant cost overruns during construction.

- The Contractor, with approval of the Project Engineer, District Hydraulics Engineer, and/or the District Environmental Staff, will have the latitude to field locate the Temporary Diversion Channel where needed to best fit his planned construction sequencing. The Contractor is paid for the actual quantity of excavation and quantity of lining installed.

- Sufficient right-of-way and/or temporary/permanent easement should be provided in order to allow the contractor the latitude to locate the Temporary Diversion Channel on either side of the proposed structure. Location of wingwalls or other appurtenances that protrude beyond the neat lines of the culvert’s barrel shall be considered when locating the Temporary Diversion Channel and establishing the required R/W or Easement.

10.3.8 General Notes

See the latest Location and Design Instructional and Informational Memorandum 110 (IIM-LD-110) for the applicable Erosion and Sediment Control Notes that are to be included on the General Notes Sheet of the plans.
10.4 MAINTENANCE

Accumulated sediment shall, at a minimum, be removed from erosion and sediment control facilities as follows:

- Sediment Traps & Basins - When the wet storage volume has been reduced by approximately 50%
- Temporary Silt Fence – When it retains sediment up to ½ of its height
- Rock Check Dams – When the storage capacity behind the dam has been reduced by approximately 50%
- Dewatering Basins – When the excavated volume has been reduced by approximately 33%
- Slope Interrupter – When it retains sediment up to ½ of its height
- All other erosion and sediment control facilities – When the capacity, height or depth has been reduced by approximately 50%.

10.5 BASIS OF PAYMENT

10.5.1 Siltation Control Excavation

All silt removal and sediment cleanout from erosion and sediment control items will be measured and paid for as “cubic yards of Siltation Control Excavation.”

10.5.2 Check Dams

To be measured and paid for per each for the type specified.

10.5.3 Temporary Silt Fence

To be measured and paid for in linear feet.

10.5.4 Temporary Sediment Basins and Sediment Traps

Excavation for Temporary Sediment Basins or Sediment Traps will be measured and paid for as “cubic yards Temporary Sediment Basin Excavation.” If additional fill material is needed for dams or berms, it will be measured and paid for as “cubic yards of Regular Excavation, Borrow Excavation or Embankment.”
10.5.5 Dewatering Basins

To be measured and paid for per each.

10.5.6 Inlet Protection

- Types A and B to be measured and paid for per each for the type specified.
- Type C will be measured and paid for in accordance with the individual pay items and pay units shown in the Standard Drawing for EC-6, Type C.

10.5.7 Temporary Diversion Dike

Temporary Diversion Dike will be measured in linear feet, complete-in-place, and will be paid for at the contract unit price per linear foot.

10.5.8 Stabilized Construction Entrance

- A minimum number of stabilized construction entrances should be considered for a project and a quantity provided on the ESC Summary Table in the plans.
- Will not be measured for payment but the cost shall be included in the price bid for other appropriate items.

10.5.9 Slope Drains

- To be measured and paid for per each regardless of size or length.
- EC-1 Class 1 at slope drain outlet to be measured and paid for per square yard or ton as specified in the plan.

10.5.10 Brush Silt Barriers

Will not be measured for payment but the cost shall be included in the price bid for other appropriate items.

10.5.11 Geotextile Fabric

When attached to brush barriers or an existing fence, payment will be made for square yards of Geotextile Fabric.
10.5.12 Culvert Outlet Protection

- Std. EC-1, Class A1 & Class I & Class II - to be measured and paid for in square yds or tons.
- Std. EC-3 RECP Permanent, soil stabilization mat – to be measured and paid for in square yds.

10.5.13 Turbidity Curtains

To be measured and paid for in linear feet of the type specified, measured from edge of curtain to edge of curtain along the support cable.

10.5.14 Temporary Diversion Channel

To be measured and paid for in cubic yards Temporary Diversion Channel Excavation and square yards Temporary Diversion Channel Lining for the Class specified.

10.5.15 Slope Interrupter

To be measured and paid for in linear feet.

10.6 QUANTITY ESTIMATES

10.6.1 Summary Sheet

- All estimated quantities for erosion and sediment control items are to be summarized on the Erosion Control Summary Sheet.
- Estimated quantities are to be shown for each phase of the ESC Plan.

10.6.2 Rock Check Dams

- Summarize a quantity of 4.74 cubic yards of Siltation Control Excavation for each Rock Check Dam Type I specified. This should allow for two cleanouts.
- Summarize a quantity for 0.32 cubic yards of Siltation Control Excavation for each Rock Check Dam Type II specified. This should allow for two cleanouts.

Due to the magnitude of the changes to this section, shading has been omitted.
10.6.3 Temporary Silt Fence

Summarize a quantity for cubic yards of Siltation Control Excavation as 0.17 Cubic yards of Siltation Control Excavation for each linear foot of Temporary Silt Fence specified.

10.6.4 Brush Silt Barrier

The estimated linear feet are to be shown on the Erosion Control Summary Sheet.

10.6.5 Temporary Sediment Basins and Traps

- Summarize the cubic yards of Temporary Sediment Basin Excavation on the Erosion Control Summary Sheet. If Borrow or Embankment is needed, it is to be included in roadway totals on the Grading Diagram and Summary Sheet.

- The Grading Diagram is to reflect how the cubic yards of Temporary Sediment Basin Excavation and cubic yards of Embankment are to be distributed.

- Temporary Sediment Basin control structure (riser pipe) – Summarize pay item as linear feet of Temporary Sediment Riser Pipe (size) on the Erosion Control Summary Sheet.

- Any culvert pipe necessary for a temporary sediment basin shall be included with other applicable pipe on the Drainage Summary Sheet.

- Summarize a quantity for cubic yards of Siltation Control Excavation that is equal to 50% of the total volume (wet storage volume plus dry storage volume) of the basin or trap. This will allow for two cleanouts.

10.6.6 Dewatering Basin

- The number of Dewatering Basins specified for each applicable site shall consider any potential phased construction of the proposed drainage structure. At a minimum, the following number of dewatering Basins shall be specified:
  - One Dewatering Basin for each pipe(s) or major structure that has a combined hydraulic opening of 12.6 square feet (48" diameter pipe or equivalent) or greater including bridges 20’ or less in length.
  - Two Dewatering Basins for each bridge over 20’ in length.
• Summarize a quantity of 3 cubic yards of Siltation Control Excavation for each Dewatering Basin specified, based on a minimum Dewatering Basin size of 6’ x 6’ x 3’. This will allow for two cleanouts.

10.6.7 Inlet Protection

• Type A
  o Summarize a quantity of 15 cubic yards of Siltation Control Excavation for each Drop Inlet Silt Trap Type A specified at St’d DI-5, DI-7A,7B and DI-12,12A,12B,12C Drop Inlet locations. This should allow for two cleanouts.
  o Summarize a quantity of 5 cubic yards of Siltation Control Excavation for each Drop Inlet Silt Trap Type A specified at Standard DI-1 and DI-7 Drop Inlet locations. This should allow for two cleanouts.

• Type B
  Summarize a quantity of 5 cubic yards of Siltation Control Excavation for each Drop Inlet Silt Trap Type B specified at curb drop inlet locations. This should allow for two cleanouts.

• Type C
  See the payment method for Rock Check Dams (EC-4) as specified on the plans.

10.6.8 Stabilized Construction Entrance

The estimated number of Stabilized Construction Entrances is to be shown on the Erosion Control Summary Sheet.

10.6.9 Slope Drains

• Summarize the estimated number of Slope Drains and the quantity of Culvert Outlet Protection Class 1, Standard EC-1 on the Erosion Control Summary Sheet.

• The number of Slope Drains required is to be estimated as follows:

  One Slope Drain for each 250 linear feet, or portion thereof, for fills 8 feet in height or greater, for each roadway baseline; e.g., 200’ of fill = 1 Slope Drain; 580’ of fill = 3 Slope Drains.
10.6.10 Erosion Control Mulch

- Summarize a quantity on the Erosion Control Summary Sheet when recommended by the Roadside Development Manager.

- This material is estimated at the rate of 50 square yards per 100 feet of roadway alignment.

10.6.11 Turbidity Curtain

Summarize as linear feet of Turbidity Curtain for the type specified (Pervious or Impervious) on the Erosion Control Summary Sheet.

10.6.12 Temporary Diversion Channel

- An estimated quantity of Temporary Diversion Channel Excavation and Temporary Diversion Channel Lining for the Class specified (A or B) is to be shown on the Erosion Control Summary Sheet.

- Silt fence along both sides of channel is to be measured and paid for separately and summarized on the Erosion Control Summary Sheet.

- The designer shall estimate the cubic yards of temporary Diversion Channel Excavation and the square yards of Temporary Diversion Channel Lining based on the estimated width and depth of the channel using Table 1.

10.1.13 Slope Interrupter

Summarize a quantity for cubic yards of Siltation Control Excavation as 0.17 Cubic yards of Siltation Control Excavation for each linear foot of Slope Interrupter specified.

10.1.14 Temporary Check Dams

Summarize a quantity for 0.17 cubic yards of Siltation Control Excavation for each Temporary Check Dam specified.
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<td>10.00</td>
<td>10.37</td>
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EROSION AND SEDIMENT CONTROL PLAN

EXAMPLE NO PLAN PROJECT
Chapter 10 – Erosion and Sediment Control

Sta. 16 + 00
Existing wood
Deck bridge
Structure #3062
To be removed
60'-single 8X8
Box culvert req’d.
(2'-5’ fill)
Std. BCS-05 w/
Std. BCW-21
Wing walls req’d.

Sta. 12 +00
Private Entrance
No existing or
Proposed pipe

Sta. 10 + 00 to
13 + 50 all widening
To be done to this side.

Final Version
Not to Scale
All Locations, Stations,
Dimensions, Lengths,
Cuts/Fills, etc. are
Approximate.

Proj: 0000-00-000_N000
Any County
AD Date:11/02

Wetland Area
This area cannot be affected

End fill Begin 2’ cut
16 + 00

End cut Begin 1’ fill
13 +00

Sta. 20+00
Private Entrance
Existing 20’-18” pipe
(min. cover)
Do not disturb
Std. EC-1, Class I req’d.

Pre & post pictures
Req’d.

Begin 1’ cut
Begin Project
Route 0000

Station 18+50
Existing 36” pipe
To be removed
42’-48” pipe req’d.
(3’ cover) (skew 30°)
2 std. EW-28’s req’d.
Chapter 10 – Erosion and Sediment Control

Sta. 28 +50
Field Entrance
No existing pipe
20'-15" pipe req’d.
(1’ cover)

Sta. 25 + 10
Private Entrance
Existing 25'-15" pipe
To be removed
30'-15" pipe req’d.
(4’ cover)

Sta. 24 + 80
Existing 36" cross pipe
To be removed
40'-72" pipe req’d.
(8’ cover)
2 Std. EW-2’s req’d.
Std. EC-1, Class II req’d.

Permanent 20’X50’
Drainage Easement
Req’d.

Station 21 + 00 To 24 + 00
Drain ditch req’d.
W=3’, D=2’, Side Slope=2:1
Class I dry riprap req’d.
D=18”

Sta. 20 + 80
Field Entrance
Existing 12'-15" pipe
To be removed
25'-15" pipe req’d.
(1’ cover)
Chapter 10 – Erosion and Sediment Control

End Project

Sta. 38 + 50
Field Entrance
Existing 20'-15" pipe
To be removed
30'-15" pipe req’d.
(2’ cover)

Sta. 35 + 10
Private Entrance
Existing 28'-15" pipe
To be removed
36'-18" pipe req’d.
(3’ cover)

Sta. 34 + 80
Existing 24" pipe
To be removed
40'-30" pipe req’d.
(4’ cover)
3.5 Sq. Yds. Std.
EC-I, Class II
Req’d, Type B
Installation

Station 33 + 00 to
Station 35+20 project
will be shifted to the left
approximately 50’
on to new alignment to
 lessen the encroachment
on the stream

Sta. 30 + 80
Field Entrance
Existing 25'-15" pipe
To be removed
32'-18" pipe req’d.
(8’ cover)

End fill

39 + 00
38 + 00
37 + 00
36 + 00
35 + 00
34 + 00
33 + 00
32 + 00
31 + 00
30 + 00

Begin 12’ fill

30’ Class I dry riprap
Req’d, D= 24”

Type I Rock
Check Dam

50’

Station 32+15
Std. EC-3 Sediment Trap
VOL=168 Wet 168 Dry
L= 18 ft. 
T=2 ft.
B= 25 ft. SS= 2:1
W= 15 ft. Depth= 4 ft.
Refer to R&B Standards,
Vol. I

Approx. 100’

Farm Pond
### KEY
#### 0000-00-000,N000

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Pipe Existing" /></td>
<td>Pipe Existing</td>
</tr>
<tr>
<td><img src="image2" alt="Pipe - 42” or smaller" /></td>
<td>Type II rock check dam</td>
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<td><img src="image3" alt="Pipe - 48” or larger" /></td>
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<td>Rolled Erosion Control Product (RECP) Temporary</td>
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<td>Rolled Erosion Control Product (RECP) Permanent</td>
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<td><img src="image7" alt="Guardrail" /></td>
<td>EC-1 Outlet protection</td>
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<td>Temporary slope drain</td>
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</table>

### NOTE:
- A temporary diversion channel St’d EC-12 may be required for work in live streams. Must be reviewed and approved by Environmental. (R&B Standard)
- All E&S controls need to be removed within 30 days after project is stabilized. (MS 18)
- All referenced standards and E&S controls should conform to the latest edition of the VDOT Road & Bridge Standards.
- Refer to contract documents for all quantities. (e.g.: minor structure excavation, bedding, backfill etc.)
- Dewatering devices may be required at live stream pipe installations.
- All disturbed areas will be stabilized with seed and mulch in accordance with the Roadside Development Sheet.
# Chapter 11 – Stormwater Management

## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.1</td>
<td>Introduction</td>
<td>11-1</td>
</tr>
<tr>
<td>11.2</td>
<td>Design Policy</td>
<td>11-2</td>
</tr>
<tr>
<td>11.3</td>
<td>General Design Criteria</td>
<td>11-3</td>
</tr>
<tr>
<td>11.3.1</td>
<td>Introduction</td>
<td>11-3</td>
</tr>
<tr>
<td>11.3.2</td>
<td>Pre-Development Conditions</td>
<td>11-3</td>
</tr>
<tr>
<td>11.3.3</td>
<td>Hydrology</td>
<td>11-3</td>
</tr>
<tr>
<td>11.3.4</td>
<td>Multi-Use Facilities</td>
<td>11-4</td>
</tr>
<tr>
<td>11.3.4.1</td>
<td>Quality versus Quantity</td>
<td>11-4</td>
</tr>
<tr>
<td>11.3.4.2</td>
<td>Temporary versus Permanent</td>
<td>11-4</td>
</tr>
<tr>
<td>11.3.5</td>
<td>Impounding Facilities</td>
<td>11-4</td>
</tr>
<tr>
<td>11.3.6</td>
<td>Regional Facilities</td>
<td>11-5</td>
</tr>
<tr>
<td>11.3.7</td>
<td>Right of Way/Permanent Easements</td>
<td>11-6</td>
</tr>
<tr>
<td>11.3.8</td>
<td>Fencing</td>
<td>11-6</td>
</tr>
<tr>
<td>11.3.9</td>
<td>Plan Details</td>
<td>11-7</td>
</tr>
<tr>
<td>11.3.9.1</td>
<td>Stormwater Management Profiles and Cross Sections</td>
<td>11-7</td>
</tr>
<tr>
<td>11.3.9.2</td>
<td>Stormwater Management Details – R&amp;B Standard SWM-DR</td>
<td>11-7</td>
</tr>
<tr>
<td>11.3.9.3</td>
<td>Stormwater Management Summary</td>
<td>11-7</td>
</tr>
<tr>
<td>11.3.9.4</td>
<td>Method of Measurement – Basis of Payment</td>
<td>11-8</td>
</tr>
<tr>
<td>11.3.10</td>
<td>Maintenance</td>
<td>11-8</td>
</tr>
<tr>
<td>11.4</td>
<td>Part II B Design Criteria</td>
<td>11-10</td>
</tr>
<tr>
<td>11.4.1</td>
<td>Water Quality</td>
<td>11-10</td>
</tr>
<tr>
<td>11.4.2</td>
<td>Water Quantity</td>
<td>11-12</td>
</tr>
<tr>
<td>11.4.2.1</td>
<td>Channel Protection</td>
<td>11-12</td>
</tr>
<tr>
<td>11.4.2.2</td>
<td>Flood Protection</td>
<td>11-14</td>
</tr>
<tr>
<td>11.4.2.3</td>
<td>Sheet Flow</td>
<td>11-15</td>
</tr>
<tr>
<td>11.5</td>
<td>Part II B Design Concepts</td>
<td>11-16</td>
</tr>
<tr>
<td>11.5.1</td>
<td>Water Quality</td>
<td>11-16</td>
</tr>
<tr>
<td>11.5.1.1</td>
<td>Offsite Water Quality Compliance</td>
<td>11-17</td>
</tr>
<tr>
<td>11.5.1.2</td>
<td>Compliance Spreadsheets</td>
<td>11-17</td>
</tr>
<tr>
<td>11.5.1.3</td>
<td>Land Cover and Soil Groups</td>
<td>11-18</td>
</tr>
<tr>
<td>11.5.2</td>
<td>Water Quantity</td>
<td>11-19</td>
</tr>
<tr>
<td>11.5.2.1</td>
<td>Channel Protection</td>
<td>11-19</td>
</tr>
<tr>
<td>11.5.2.1.1</td>
<td>Manmade Stormwater Conveyance System</td>
<td>11-20</td>
</tr>
<tr>
<td>11.5.2.1.2</td>
<td>Restored Conveyance System</td>
<td>11-20</td>
</tr>
<tr>
<td>11.5.2.1.3</td>
<td>Natural Conveyance System</td>
<td>11-21</td>
</tr>
<tr>
<td>11.5.2.1.4</td>
<td>Energy Balance</td>
<td>11-21</td>
</tr>
<tr>
<td>11.5.2.1.5</td>
<td>Improvement Factor (I.F.)</td>
<td>11-22</td>
</tr>
<tr>
<td>11.5.2.1.6</td>
<td>Forested Conditions</td>
<td>11-22</td>
</tr>
<tr>
<td>11.5.2.1.7</td>
<td>Limits of Analysis</td>
<td>11-23</td>
</tr>
<tr>
<td>11.5.2.1.8</td>
<td>Runoff Reduction</td>
<td>11-23</td>
</tr>
<tr>
<td>11.5.2.1.9</td>
<td>Increases in Peak Rate of Runoff</td>
<td>11-25</td>
</tr>
<tr>
<td>11.5.2.2</td>
<td>Flood Protection</td>
<td>11-25</td>
</tr>
<tr>
<td>11.5.2.2.1</td>
<td>Conveyance System Definition</td>
<td>11-26</td>
</tr>
<tr>
<td>11.5.2.2.2</td>
<td>Localized Flooding not Currently Experienced</td>
<td>11-26</td>
</tr>
<tr>
<td>11.5.2.2.3</td>
<td>Localized Flooding Currently Experienced</td>
<td>11-26</td>
</tr>
<tr>
<td>11.5.2.2.4</td>
<td>Compliance with the Flood Protection Criteria</td>
<td>11-27</td>
</tr>
<tr>
<td>11.5.2.2.5</td>
<td>Limits of Analysis</td>
<td>11-27</td>
</tr>
</tbody>
</table>
**11.5.2.3** Sheet Flow ......................................................................................................................... 11-27

**11.5.3** Part II B Design Procedures and Sample Problems .......................................................... 11-28

11.5.3.1 SWM Plan Requirements ........................................................................................................ 11-28

11.5.3.2 Water Quality – Runoff Reduction Method (RRM) Procedure ........................................ 11-29

11.5.3.2.1 BMPs In-Series/Treatment Trains .................................................................................. 11-33

11.5.3.2.2 Offsite Water Quality Compliance Options .................................................................. 11-34

11.5.3.2.3 Water Quality – Sample Problem – New Development .............................................. 11-34

11.5.3.2.4 Water Quality – Sample Problem – Redevelopment ................................................. 11-43

11.5.3.3 Water Quantity – Channel Protection Procedure .............................................................. 11-52

11.5.3.3.1 Channel Protection - Limits of Analysis Sample Problems ........................................... 11-55

11.5.3.3.2 Channel Protection - Adjusted CN Sample Problem ....................................................... 11-56

11.5.3.3.3 Channel Protection - Manmade System Sample Problem ....................................... 11-56

11.5.3.3.4 Channel Protection - Restored System Sample Problem .............................................. 11-57

11.5.3.3.5 Channel Protection - Natural System Sample Problem ............................................... 11-57

11.5.3.4 Water Quantity – Flood Protection Procedure ................................................................. 11-59

11.5.3.4.1 Flood Protection - Limits of Analysis ......................................................................... 11-62

11.5.3.4.2 Flood Protection - Limits of Analysis Sample Problems ........................................... 11-63

11.5.3.4.3 Flood Protection Sample Problem – Localized Flooding Not Currently Experienced ................................................................................................................. 11-64

11.5.3.4.4 Flood Protection Sample Problem – Localized Flooding Currently Experienced ................................................................................................................. 11-67

11.5.4 Pretreatment ............................................................................................................................. 11-68

11.5.5 Treatment Volume Computation ............................................................................................ 11-69

11.5.6 Detention Time Computation and Orifice Sizing .................................................................. 11-69

11.5.6.1.1 Average Hydraulic Head Method (Method #2 from VSWMH) - VDOT Preferred Method ......................................................................................................................... 11-70

11.5.6.1.2 Maximum Hydraulic Head Method (Method #1 from VSWMH) .................................. 11-71

11.5.6.1.3 Tv Hydrograph ............................................................................................................. 11-72

11.5.6.1.4 Alternative Method of Routing WQV to Find Drawdown Time .................................. 11-72

11.5.7 Preliminary Detention Volume Computation ........................................................................ 11-73

11.5.7.1.1 Modified Rational Method, Simplified Triangular Hydrograph .................................. 11-73

11.5.7.1.2 Critical Storm Duration Method .................................................................................. 11-74

11.5.7.1.3 Pagan Volume Estimation Method .............................................................................. 11-75

11.5.7.1.4 Sample Problems – Using 3 Methods to Estimate Volume of Storage for Quantity Control .......................................................................................................................... 11-76

11.5.8 Preliminary Basin Sizing ........................................................................................................ 11-78

11.5.9 Final Basin Sizing – Reservoir Routing ................................................................................ 11-80

11.5.9.1 Storage – Indication Method Routing Procedure ........................................................... 11-80

11.5.9.2 Storage – Indication Method Routing Sample Problem #1 ........................................... 11-83

**11.6** Part II C Design Criteria ........................................................................................................ 11-97

11.6.1 Water Quality ....................................................................................................................... 11-97

11.6.2 Water Quantity .................................................................................................................... 11-99

11.6.3 Compensatory Treatment ..................................................................................................... 11-100

11.6.4 Embankment (Dam) .............................................................................................................. 11-101

11.6.5 Basin Grading ....................................................................................................................... 11-103

11.6.6 Sediment Forebay .................................................................................................................. 11-104

**11.7** Part II C Design Concepts ...................................................................................................... 11-106

11.7.1 Water Quality ....................................................................................................................... 11-106

11.7.2 Water Quantity .................................................................................................................... 11-107

11.7.3 Extended Detention vs. Retention ......................................................................................... 11-107

11.7.4 Detention Time ...................................................................................................................... 11-108

11.7.5 Release Rates ........................................................................................................................ 11-109

11.7.5.1 Channel Erosion Control – Q; Control ............................................................................ 11-109

11.7.5.2 Flooding .......................................................................................................................... 11-111
11.7.5.3 Water Quality Control ................................................................. 11-112
11.7.6 Outlet Hydraulics ........................................................................ 11-117
  11.7.6.1 Orifice .................................................................................. 11-117
  11.7.6.2 Weirs .................................................................................. 11-117
  11.7.6.3 Types of Outlet Structures ................................................... 11-118
  11.7.6.3.1 General ........................................................................... 11-118
  11.7.6.3.2 SWM-1 (VDOT Standard) ................................................ 11-118
  11.7.6.3.3 Weir Wall (Stormwater Management Dam) ................... 11-119
  11.7.6.4 Routing .............................................................................. 11-119

11.8 Part II C Design Procedures and Sample Problems ....................... 11-120
  11.8.1 Documentation Requirements ................................................. 11-120
  11.8.2 Water Quality Volume Computation and BMP Selection Procedure 11-120
  11.8.3 Detention Time Computation and Orifice Sizing ....................... 11-122
    11.8.3.1 Average Hydraulic Head Method (Method #2) - VDOT Preferred Method ................................................................. 11-123
    11.8.3.2 Maximum Hydraulic Head Method (Method #1) ......... 11-123
    11.8.3.3 WQV Hydrograph (HYG) ...................................................... 11-124
    11.8.3.4 Alternative Method of Routing WQV to Find Drawdown Time ....... 11-124
  11.8.4 Channel Erosion Control Volume – Q1 Control ......................... 11-125
  11.8.5 Preliminary Detention Volume Computation ......................... 11-127
    11.8.5.1 Modified Rational Method, Simplified Triangular Hydrograph Routing ................................................................. 11-127
    11.8.5.2 Critical Storm Duration Method ......................................... 11-128
    11.8.5.3 Pagan Volume Estimation Method .................................... 11-129
    11.8.5.4 Sample Problems – Using 3 Methods to Estimate Volume of Storage for Quantity Control ................................................................. 11-130
  11.8.6 Determine Preliminary Basin Size ........................................... 11-132
  11.8.7 Final Basin Sizing-Reservoir Routing ....................................... 11-133
    11.8.7.1 Storage – Indication Method Routing Procedure ............ 11-133
    11.8.7.2 Storage – Indication Method Routing Sample Problem .......... 11-136
  11.8.8 Final Basin Sizing-Reservoir Routing ....................................... 11-142

11.9 References ..................................................................................... 11-148

List of Tables

Table 11-1. WQV Orifice Sizes ................................................................. 11-70
Table 11-2. Storage Characteristics ......................................................... 11-82
Table 11-3. Runoff Hydrographs ............................................................... 11-84
Table 11-4. Stage-Discharge-Storage Data .............................................. 11-85
Table 11-5. Storage Routing for the 2-yr Storm ...................................... 11-87
Table 11-6. Storage Routing for the 10-yr Storm .................................... 11-87
Table 11-7. BMP Selection Table ............................................................. 11-99
Table 11-8. Summary of Design Criteria for Dry and Wet Basins ........... 11-104
Table 11-9. Functional Goals of Stormwater BMPs ............................... 11-107
Table 11-10. WQV Orifice Sizes ............................................................... 11-122
Table 11-11. Storage Characteristics ......................................................... 11-134
Table 11-12. Runoff Hydrographs ............................................................... 11-137
Table 11-13. Stage-Discharge-Storage Data .............................................. 11-138
List of Figures

Figure 11-1. Simplified Triangular Hydrograph Method ......................................................... 11-74
Figure 11-2. Pagan Method Curve ........................................................................................ 11-76
Figure 11-3. Stage-Storage Curve ....................................................................................... 11-81
Figure 11-4. Stage-Discharge Curve .................................................................................... 11-81
Figure 11-5. Storage Characteristics Curve ........................................................................ 11-82
Figure 11-6. Runoff Hydrographs ....................................................................................... 11-88
Figure 11-7. Typical SWM Basin Dam ................................................................................ 11-102
Figure 11-8. Typical Sediment Forebay Plan and Section ................................................ 11-105
Figure 11-9. Removal Rates vs. Detention Time ................................................................. 11-108
Figure 11-10. Simplified Triangular Hydrograph Method .................................................... 11-128
Figure 11-11. Pagan Method Curve ................................................................................... 11-130
Figure 11-12. Stage-Storage Curve .................................................................................... 11-134
Figure 11-13. Stage-Discharge Curve ................................................................................. 11-134
Figure 11-14. Storage Characteristics Curve ..................................................................... 11-135
Figure 11-15. Runoff Hydrographs .................................................................................... 11-141

List of Appendices

Appendix 11B-1 SWM and TSB Summary Table
Appendix 11B-2 Redevelopment/Surplus Credit Tracking Form (LD-458)
Appendix 11C-1 SWM Facility Tabulation Sheet
Chapter 11 - Stormwater Management

11.1 Introduction

Development of watersheds has the potential for generally causing an increase in the peak flow rate of stormwater runoff. This increase is often associated with flood damage, erosion, and siltation control problems, and increased pollutant loads. Urban development has been identified as having a direct impact on the hydrologic cycle by reducing, or even eliminating, the natural storage capacity of the land. These natural storage areas are then replaced with impervious and managed pervious surfaces. Impervious cover prevents the infiltration of rainfall into the soil and increases the speed and quantity of rainfall runoff to the outfall. Increased stormwater runoff impacts water quality, stream channel erosion, and localized flooding. For a watershed with no defined, or inadequate, outfall, the total volume of runoff is critical and SWM facilities (SWMFs) can be used to store the increases in volume and control peak discharge rates.

The goal of stormwater management (SWM) is to inhibit the deterioration of the aquatic environment by instituting a program that maintains both water quantity and quality post-development runoff characteristics, as nearly as practicable, equal to or better than pre-development runoff characteristics, and to limit the post-development peak discharge flow rates to match the predevelopment peak discharge flow rates.

**Stormwater Quality Control**
Stormwater quality control pertains to reducing the amount of pollutants discharged by land development projects. This goal is achieved by reducing the volume of runoff to reduce pollutant loads, treating runoff to reduce pollutant concentration and loads, or a combination of both.

**Stormwater Quantity Control**
Stormwater quantity control (or flooding and erosion control) pertains to replicating the water quantity post-development runoff characteristics and site hydrology, as nearly as practicable, equal to or better than the existing pre-development runoff characteristics, including runoff volume and peak rate of runoff.
11.2 Design Policy


The general application to highway drainage design associated with these regulations is addressed here and also in the latest version of VDOT Location & Design Instructional & Informational Memorandum IIM-LD-195. Effective July 1, 2014, Water Quality and Water Quantity are now governed by two (2) distinct methods, depending on whether the project follows Part II B or Part II C technical criteria. Please refer to IIM-LD-195 which provides guidance in determining which technical criteria governs for a given project.
11.3 General Design Criteria

11.3.1 Introduction

Depending on which technical criteria (Parts II B & II C) governs a given project, the design elements for Stormwater Management Facilities (SWMFs) or Best Management Practices (BMPs) will be different. For example, Part II B includes 9VAC25-870-62 through 9VAC25-870-92, and Part II C includes 9VAC25-870-93 through 9VAC25-870-99.

For those projects following Part II C, BMPs will closely follow design criteria as still recognized by the Virginia Stormwater Management Handbook (Blue Book) at http://www.deq.virginia.gov/Programs/Water/StormwaterManagement/Publications.aspx and also pursuant to the information in this chapter. VDOT published a Part II C BMP Design Manual of Practice for designers and plan reviewers to use and reference at http://www.virgiadot.org/business/resources/LocDes/Part_II_C_BMP_Design_Manual.pdf

For those projects following Part II B, the design of BMPs will follow design criteria as identified in the VDOT Part II B BMP Design Manual of Practice located at http://www.virgiadot.org/business/resources/LocDes/Part_II_B_BMP_MOP_Combined_6-24-19.pdf; as well as DEQ-approved water quality BMP standards and special provisions. A designer can use the standards and specifications from the Virginia Stormwater BMP Clearinghouse; however, the designer is encouraged to follow the design practices in this Chapter, as these are specific to VDOT from a construction and maintenance standpoint.

11.3.2 Pre-Development Conditions

For purposes of computing pre-development runoff, all pervious lands on the site shall be assumed to be in “good” hydrologic condition in accordance with the U.S. Department of Agriculture’s Natural Resources Conservation Service (NRCS) standards, regardless of conditions existing at the time of computation. Pre-development runoff calculations utilizing other hydrologic conditions may be utilized provided that it is demonstrated to and approved by the DEQ that actual site conditions warrant such considerations.

11.3.3 Hydrology

Hydrology should be performed using the appropriate hydrograph procedures presented in Chapter 6, Hydrology.

Unless otherwise specified, the prescribed design storms are the 1-year (yr), 2-yr, and 10-yr 24-hour storms using the site-specific rainfall precipitation frequency data recommended by the U.S. National Oceanic and Atmospheric Administration (NOAA) Atlas 14. Partial duration time series shall be used for the precipitation data.
Pre-development and post-development runoff characteristics and site hydrology shall be verified by site inspections, topographic surveys, available soil mapping or studies, and calculations consistent with good engineering practices.

Unless otherwise specified, all hydrologic analyses shall be based on the existing watershed characteristics and how the ultimate development condition of the subject project will be addressed.

11.3.4 Multi-Use Facilities

11.3.4.1 Quality versus Quantity

SWMFs may function as both quantity control and quality control facilities, and are also known as a Best Management Practices or BMPs. Some facilities may only be needed for either quality or quantity control.

11.3.4.2 Temporary versus Permanent

Permanent SWMF may be utilized as temporary sediment basins during the construction phase of the project, and if so, the design of the SWMF will need to address this dual function. The design that is needed for a permanent SWMF may need to be altered to provide additional temporary sediment storage volume that is in excess of the applicable design volume. For design purposes, the two volumes (temporary sediment storage volume and post-construction volume) should not be added together, but rather the larger of the two should govern the facility’s design.

The additional volume needed for temporary sediment storage may be provided by excavating the bottom of the basin lower than that required for the WQV. The basin’s permanent outlet control structure can be temporarily altered to serve as the control structure for the temporary sediment basin (see Standard SWM-DR of VDOT’s R&B Standards and the Virginia ESC Handbook). When the project is nearing completion, and the basin is no longer needed for temporary sediment control, the basin can be converted to satisfy the permanent SWMF requirements by regrading (excavating and/or filling) and removing any temporary control structure appurtenances.

11.3.5 Impounding Facilities

SWM wet ponds and extended detention ponds that are not covered by the Impounding Structure Regulations (4VAC50-20 et. seq.) shall, at a minimum, be engineered for structural integrity for the 100-yr storm event.

Construction of SWM impoundment structures or facilities may occur in karst areas only after a study of the geology and hydrology of the area has been conducted to determine the presence or absence of karst features that may be impacted by stormwater runoff and facility placement.
Permanent SWM impoundment structures or facilities shall only be constructed in karst features after completion of a geotechnical investigation that identifies any necessary modifications to the facility to ensure its structural integrity and maintain its water quality and quantity efficiencies. Any Class V Underground Injection Control Well registration statements for stormwater discharges to improved sinkholes shall be included in the SWPPP.

11.3.6 Regional Facilities

There are many cases where it is more feasible to develop one major SWMF to control a large watershed area rather than a number of small individual facilities controlling small drainage areas within the large watershed. The concept of regional SWMFs is endorsed by VDOT provided that certain requirements are met.

When applicable, the regional facility shall comply with the impounding structure regulations (4VAC50-20 et. seq.).

The regional facility is allowed to address water quality requirements and where allowed, water quantity requirements, in accordance with sections 9VAC25-870-69 and 9VAC25-870-92 of the VSMP regulations.

Development and/or use of regional SWMFs must be a joint undertaking by VDOT and the local governing body. VDOT shall not be owner of any such facility. The site must be part of a Master SWM Plan developed and/or approved by the local VSMP Authority and/or DEQ and any agreements related to the VDOT use of these facilities must be consummated between VDOT and the local governing body. VDOT may enter into an agreement with a private individual or corporation provided the local governing body has a DEQ approved SWM program that complies with the VSMP Regulations and the proper agreements for maintenance and liability of the regional facility have been executed between the local governing body and the private individual or corporation and any such agreements are referenced in the agreement between VDOT and the private individual or corporation.

When VDOT agrees to the use of an existing or future VDOT roadway embankment as an impounding structure for a regional facility, the roadway embankment must be designed or retrofitted appropriately for such use. The VDOT R/W line will normally be set at the inlet face of the main drainage structure.

The design of regional SWMFs must address any mitigation needed to meet the water quality and quantity requirements of any known future VDOT projects within the contributing watershed. Regional SWMFs located upstream of a proposed VDOT roadway shall provide sufficient mitigation for any water quality and quantity impacts of runoff from the proposed roadway project which may not pass through the proposed facility.
Any questions or concerns related to the use of an offsite regional SWMF to satisfy the VDOT post-development SWM requirements should be discussed between the SWM Plan Designer and the appropriate DEQ regional office prior to entering into any agreements with either private or public entities.

### 11.3.7 Right of Way/Permanent Easements

Permanent SWMFs may be placed in fee R/W or in permanent easements.

It is recommended that all permanent SWM features (dams, risers, storage area etc.) be placed within fee R/W initially. Outfall ditches and similar features may initially be placed in permanent easements.

The final decision on R/W versus permanent easement should be made prior to the R/W (or similar) phase of the project development process based on information obtained at the Field Inspection, Design Public Hearing and/or other such plan review milestones.

VDOT will generally be amenable to the desires of the affected landowners regarding the fee R/W or permanent easement issue.

The multiple use of property for SWMFs and other features, such as utilities, is permissible. The decision on such use must be made on a case-by-case basis.

Permanent easements and/or other properties acquired through the R/W acquisition process, and which are considered a part of the “site” in determining the post-development SWM requirements for the project, are to remain under the ownership/control of VDOT for the life of the project and such property is to be identified/designated on the plans and legally encumbered for the purpose of SWM.

### 11.3.8 Fencing

Fencing of SWMFs is normally not required and should not be considered for most practices due to:

- **Insignificant Hazard** – For detention basins (no permanent water pool), significant ponding of water in the basin should only occur with very heavy rainfall events and the maximum ponded depth should typically be no more than about 3'. Ponds and lakes are almost never fenced, even though they may be located in subdivisions and have deep, permanent water pools.
- **Limits Maintenance Operations** – Fencing could hinder the performance of both routine and long term maintenance operations. Fencing could become damaged during major maintenance operations and have to be repaired or replaced.
Fencing of SWMFs may occasionally be needed and should be used when:

- The basin is deep with a ponded depth greater than about 3’ and/or has steep side slopes with two or more side slopes steeper than 3:1, or
- The basin is in close proximity to schools, playgrounds or similar areas where children may be expected to frequent, or
- It is recommended by the VDOT Field Inspection Review Team (or other such plan reviewing group), the VDOT Residency Administrator or the City/County (where City/County will take over maintenance responsibility), or
- A chain or gate will be needed on some basins to prohibit vehicular access for dumping or other undesirable access. The designer should seek input as appropriate from the District Hydraulic Engineer or the District Roadside Manager to determine any prohibition requirements.

Where fencing is proposed, access gate(s) of sufficient size to accommodate maintenance equipment are to be provided. Appropriate security mechanisms for the gates are to be provided to prevent/deter unauthorized entry.

For non-fenced basins, a chain barricade (see Standard CR-1 of VDOT’s R&B Standards) or gate may be needed across the vehicular entrance to prohibit non-authorized access if there is a concern with illegal dumping or other undesirable activities at the site.

“No Trespassing” signs shall be considered for use on all basins, whether fenced or unfenced, and should be recommended, as needed, by the VDOT Field Inspection Review Team or other such plan reviewing group.

11.3.9 Plan Details

11.3.9.1 Stormwater Management Profiles and Cross Sections
- To be provided for all SWMFs.

11.3.9.2 Stormwater Management Details – R&B Standard SWM-DR
- Includes details for debris rack, trash rack, concrete cradle, water quality orifice and modifications for use of SWMF as a temporary sediment basin.
- Specify at each SWMF location requiring any of the noted items.
- The location and the size of the water quality orifice or any other required openings in the control structure shall be specified in the description/details for the control structure for each SWMF.

11.3.9.3 Stormwater Management Summary
- All drainage items related to the construction of SWMFs shall be summarized, by location, in the Drainage Summary for the project.
- All water quality requirements related to redevelopment projects shall be summarized on the Water Quality Redevelopment Tabulation Sheet in Appendix 11B-2.
• All incidental items related to the construction of SWMFs shall be summarized, by location, in the Incidental Summary for the project.
• Stormwater Management Excavation and Borrow or Embankment fill, if needed, are to be included in the totals on the Grading Diagram and/or Summary.

11.3.9.4 Method of Measurement – Basis of Payment

• Stormwater Management Drainage Structure (SWM-1): Basis of payment to be linear feet (LF) measured from invert of structure to top of concrete. Price bid includes cost of trash rack, debris rack and holder, temporary dewatering device and temporary metal plates.
• Stormwater Management Dam (weir wall): Basis of payment to be cubic yards (CY) of Concrete Class A3 Miscellaneous and pounds (LBS) of Reinforcing Steel.
• Concrete Cradle: Basis of payment to be cubic yards (CY) of Concrete Class A3 Miscellaneous
• Grading:
  o Excavation for SWM basins will be measured and paid for as cubic yards (CY) of SWM Basin Excavation.
  o Fill material needed for dams or berms will be measured and paid for as cubic yards (CY) of Regular Excavation, Borrow Excavation or Embankment.
  o The Grading Diagram is to reflect how the cubic yards (CY) of SWM Management Basin Excavation and cubic yards (CY) of Embankment or Borrow is to be distributed.

11.3.10 Maintenance

An important step in the design process is identifying whether special provisions are warranted to properly construct or maintain proposed SWMFs. To assure acceptable performance and function, the designer should review the latest version of the VDOT Post-Construction BMP Inspection and Maintenance Manual as part of the design consideration.

The Manuals will identify the requirements for maintenance of SWMFs, the schedule for inspection and maintenance operations, and the identification of persons responsible for the maintenance. Proper design should focus on minimizing maintenance requirements by addressing the following potential problems:

• Both weed growth and grass maintenance should be addressed in the plan and design. When practical given R/W constraints, concerns may be addressed by constructing side slopes no steeper than 3:1 so that they can be maintained using available power-driven equipment, such as tractor mowers.
• Sedimentation may be controlled by constructing forebays to contain sediment for easy removal.
• Bank deterioration can be controlled with protective lining, vegetation, or by limiting bank slopes.
• Standing water or soggy surfaces may be eliminated by sloping basin bottoms toward the outlet, or by constructing underdrain facilities to lower water tables. These measures also assist in mosquito control.

• Outlet structures should be selected to minimize the possibility of blockage.

• Locate the facility for easy access so that maintenance associated with litter and damage to fences and perimeter plantings can be conducted on a regular basis.

• A 10’ wide access for inspection and maintenance personnel should be provided at each SWMF. The Standard PE-1 details shown in VDOT’s Road and Bridge Standards should be used for vehicular entrances. A turnaround should be provided on vehicular entrances when needed based upon accessibility. Appropriate surface material should be provided for each vehicular entrance. The designer should seek input as appropriate from the District Hydraulic Engineer or the District Roadside Manager to determine the vehicular access requirements.

• VDOT maintenance procedures include inspecting each SWMF on an annual basis, and inspecting each SWMF after any storm that causes the capacity of the principal spillway to be exceeded. Basins should also have accumulated sediment removed about every 5 to 10 years or as necessary based upon a SWMF inspection.
11.4 Part II B Design Criteria

11.4.1 Water Quality

The following minimum design criteria and statewide standards for SWM shall be applied to the site:

1) New development. The total phosphorus load of new development projects shall not exceed 0.41 lb/ac/yr, as calculated pursuant to 9VAC25-870-65.

2) For prior developed lands on linear projects, a designer can reduce 20% below the pre-development total phosphorus load. However, the total phosphorus load shall not be required to be reduced to below the applicable standard for new development unless a more stringent standard has been established by a locality. As such, the following approach is recommended for VDOT linear projects utilizing the VRRM spreadsheets:

a. The calculations shall be limited to the disturbed area of the project, plus any new right of way or new permanent easements secured specifically for the project. Areas that qualify as “routine maintenance” (see the Virginia Stormwater Management Act § 62.1-44.15:34), such as mill and pave areas, should not be included as they are not required to meet the water quality requirements of Part II B.

b. The designer can utilize the VRRM redevelopment spreadsheet provided by DEQ (ensuring to utilize the most current, corrected version and checking the “linear development” box) to calculate compliance. The DEQ VRRM redevelopment spreadsheet will also address new development by comparing changes in land cover between the existing and proposed conditions. It is no longer necessary to use two spreadsheets: one for new development and one for redevelopment (development on prior developed land).

3) Compliance with the water quality design criteria shall be determined utilizing the Virginia Runoff Reduction Method or another equivalent methodology that is approved by the State Water Control Board.
4) The following BMPs are accepted by VDOT to effectively reduce pollutant loads and/or runoff volume:
   - Soil Amendments
   - Permeable Pavement
   - Grass Channel
   - Bioretention (and Bioretention Filter)
   - Infiltration
   - Dry Swale
   - Wet Swale
   - Sheet Flow to Vegetated Filter/Open Space
   - Extended Detention Pond
   - Filtering Practice
   - Constructed Wetland
   - Wet Pond

5) Manufactured Treatment Devices (MTDs) or proprietary BMPs accepted by DEQ may be utilized, when accepted by VDOT, in accordance with the design guidance and efficiencies approved by DEQ. See the VDOT Approved Products Lists or Special Products Evaluation List for manufactured or proprietary BMPs acceptable to VDOT: [http://www.virginiadot.org/business/bu-materials-New-Products.asp](http://www.virginiadot.org/business/bu-materials-New-Products.asp).

6) Where a project drains to more than one 6th Order Hydrologic Unit Code (HUC), the pollutant load reduction requirements shall be applied independently within each HUC unless reductions are achieved in accordance with a comprehensive watershed SWM plan.

7) Offsite compliance options may be used to meet required pollutant reductions, including the following:
   a. Offsite controls utilized in accordance with a comprehensive SWM plan adopted pursuant to the VSMP Regulation,
   b. Pollutant loading pro rata share programs established pursuant to § 15.2-2243 of the Code of Virginia,
   c. The nonpoint nutrient offset program established pursuant to § 62.1-44.15:35 of the Code of Virginia,
   d. Other offsite options approved by VDOT or the DEQ, and
   e. When VDOT has additional properties available within the same HUC or upstream HUC that the project directly discharges to, or within the same watershed, offsite SWMFs on those properties may be utilized to meet the required pollutant reductions from the land-disturbing activity.
11.4.2 Water Quantity

Compliance with the minimum standards set out in this section is deemed to satisfy the requirements of subdivision 19 of 9VAC25-840-40 (Minimum Standard 19 or MS-19) for ESC Plans.

The U.S. Department of Agriculture's Natural Resources Conservation Service (NRCS) synthetic 24-hour rainfall distribution and models, including, but not limited to NRCS Methods (including TR-55, TR-20, or EFH-2); hydrologic and hydraulic methods developed by the U.S. Army Corps of Engineers; or other standard hydrologic and hydraulic methods, shall be used to conduct the analyses of SWM compliance.

11.4.2.1 Channel Protection

Concentrated stormwater flow shall be released into a stormwater conveyance system and meet the criteria in subdivision 1, 2, or 3 below, from the point of discharge to the limits of analysis as defined in subdivision 4 below.

1. Manmade stormwater conveyance systems. When stormwater from a development is discharged to a manmade stormwater conveyance system, following the land-disturbing activity, either:
   a. The manmade stormwater conveyance system shall convey the post-development peak flow rate from the 2-yr 24-hour storm event without causing erosion of the system. Detention of stormwater or downstream improvements may be incorporated into the approved land-disturbing activity to meet this criterion, at the discretion of VDOT; or
   b. The peak discharge requirements for concentrated stormwater flow to natural stormwater conveyance systems in subdivision 3 of this subsection shall be met.

2. Restored stormwater conveyance systems. When stormwater from a development is discharged to a restored stormwater conveyance system that has been restored using natural design concepts, following the land-disturbing activity, either:
   a. The development shall be consistent, in combination with other stormwater runoff, with the design parameters of the restored stormwater conveyance system that is functioning in accordance with the design objectives; or
   b. The peak discharge requirements for concentrated stormwater flow to natural stormwater conveyance systems in subdivision 3 of this subsection shall be met.

3. Natural stormwater conveyance systems. When stormwater from a development is discharged to a natural stormwater conveyance system, the maximum peak flow rate from the 1-yr 24-hour storm following the land-disturbing activity shall be calculated either:
a. In accordance with the following methodology (referred to as the Energy Balance):

\[
Q_{\text{Developed}} = \text{I.F.} \times \left( \frac{Q_{\text{Pre-Developed}} \times RV_{\text{Pre-Developed}}}{RV_{\text{Developed}}} \right)
\]

Under no condition shall \(Q_{\text{Developed}}\) be higher than \(Q_{\text{Pre-Developed}}\) nor shall \(Q_{\text{Developed}}\) be required to be less than that calculated in the following equation:

\[
Q_{\text{Developed}} = \left( \frac{Q_{\text{Forest}} \times RV_{\text{Forest}}}{RV_{\text{Developed}}} \right)
\]

Where:

I.F. (Improvement Factor) = 0.8 for sites > 1 acre LDA or 0.9 for sites ≤ 1 acre LDA

\(Q_{\text{Developed}}\) = the allowable peak flow rate of runoff from the developed site for the 1-yr 24-hour storm.

\(RV_{\text{Developed}}\) = the volume of runoff from the site in the developed condition for the 1-yr 24-hour storm.

\(Q_{\text{Pre-Developed}}\) = the peak flow rate of runoff from the site in the pre-developed condition for the 1-yr 24-hour storm.

\(RV_{\text{Pre-Developed}}\) = the volume of runoff from the site in pre-developed condition for the 1-yr 24-hour storm.

\(Q_{\text{Forest}}\) = the peak flow rate of runoff from the site in a forested condition for the 1-yr 24-hour storm.

\(RV_{\text{Forest}}\) = the volume of runoff from the site in a forested condition for the 1-yr 24-hour storm.

b. In accordance with another methodology that is demonstrated to achieve equivalent results and is approved by the Board.

4. Limits of analysis. Channel Protection criteria under subdivisions 1, 2, or 3 will apply when the regulated land-disturbing activity contributes more than 1% of the total watershed area or existing peak discharge at each outfall or point of discharge. If the energy balance under subdivision 3 of this section is applied at the outfall or point of discharge, no further analysis is required. If downstream analysis is required under subdivisions 1 or 2, the stormwater conveyance systems shall be analyzed for compliance with channel protection criteria up to a point where either:
a. Based on land area, the site's contributing drainage area is less than or equal to 1.0% of the total watershed area; or

b. Based on peak flow rate, the site's peak flow rate from the 1-yr 24-hour storm is less than or equal to 1.0% of the existing peak flow rate from the 1-yr 24-hour storm, prior to the implementation of any stormwater quantity control measures.

The point where the limit of analysis occurs does not need to be included in the evaluation of adequacy, but the system from the outfall or point of discharge up to (but not including) the limits of analysis must be analyzed. If there is a natural channel anywhere in the system from the outfall up to the limit of analysis (but not including the point defining the limit of analysis), then the energy balance will apply. Note that the limits of analysis are applied separately for each outfall or point of discharge from a regulated land-disturbing activity.

11.4.2.2 Flood Protection
For the purposes of this section, flooding and all flow rates are to be analyzed by the use of a 10-yr 24-hour storm event. Concentrated stormwater flow shall be released into a stormwater conveyance system and shall meet one of the following criteria as demonstrated by use of acceptable hydrologic and hydraulic methodologies:

1. If the stormwater conveyance system currently does not experience localized flooding: The point of discharge releases stormwater into a stormwater conveyance system that, following the land-disturbing activity, confines the post-development peak flow rate within the stormwater conveyance system, this provision is satisfied. Detention or downstream improvements may be incorporated into the land-disturbing activity to satisfy this criterion.

2. If the stormwater conveyance system currently experiences localized flooding the point of discharge either:
   a. Confine the post-development peak flow rate within the stormwater conveyance system to avoid the localized flooding (detention or downstream improvements may accomplish this), or;
   b. Release a post-development peak flow rate that is less than the pre-development peak flow rate (no downstream analysis is required if this option is employed).

3. Limits of analysis. Unless otherwise stated in 2.b above, stormwater conveyance systems shall be analyzed for compliance with flood protection criteria to a point where:
   a. The site's contributing drainage area is less than or equal to 1.0% of the total watershed area draining to a point of analysis in the downstream stormwater conveyance system;
b. Based on peak flow rate, the site’s peak flow rate is less than or equal to 1.0% of the existing peak flow rate prior to the implementation of any stormwater quantity control measures, or;

c. The stormwater conveyance system enters a mapped floodplain or other flood-prone area where development is prohibited. Flood-prone areas may include, but are not limited to, the floodplain, the floodway, the flood fringe, wetlands, riparian buffers, or other areas adjacent to the main channel.

The point where the limit of analysis occurs does not need to be included in the evaluation of adequacy, but the system from the outfall or point of discharge up to (but not including) the limits of analysis must be analyzed. If there is a natural channel anywhere in the system from the outfall up to the limit of analysis (but not including the point defining the limit of analysis), then the energy balance will apply. Note that the limits of analysis are applied separately for each outfall or point of discharge from a regulated land-disturbing activity.

11.4.2.3 Sheet Flow
Increased volumes of sheet flow resulting from pervious or disconnected impervious areas, or from physical spreading of concentrated flow through level spreaders, must be identified and evaluated for potential impacts on down-gradient properties or resources. Increased volumes of sheet flow that will cause or contribute to erosion, sedimentation, or flooding of down gradient properties or resources shall be diverted to a SWMF or a stormwater conveyance system that conveys the runoff without causing down-gradient erosion, sedimentation, or flooding. If all runoff from the site is at the point of analysis sheet flow and the conditions of this subsection are met, no further water quantity controls are required. The designer is required to document that increases in sheet flows meet these conditions.
11.5 Part II B Design Concepts

11.5.1 Water Quality

Stormwater runoff can have a significant impact on the environment. Various pollutants are found in stormwater runoff, and studies show that the common sources of these pollutants are atmospheric deposition, urban and agricultural land uses, and natural spaces. Impervious surfaces, such as parking lots, rooftops, and roads associated with land development serve to accumulate and transport these pollutants to receiving streams. Also, the conversion of pervious surfaces such as undisturbed forest, meadow, and other open spaces to managed turf can increase runoff and the amount of pollution in the runoff.

Control of stormwater quality offers the following potential benefits:

- Improved surface water quality through runoff reduction;
- Recharge of groundwater resources;
- Maintenance of historic base flow rates and stream hydrology; and
- Protection of surface water quality through treatment of runoff.

Under the Virginia Runoff Reduction Method (VRRM), water quality design relies on three mechanisms to control pollution on stormwater runoff: reduce pollutant sources by minimizing land disturbance through environmental site design; reducing runoff volume by retaining and infiltrating runoff; and treating the remaining runoff through the application of stormwater best management practices (BMPs). The selection and sizing of BMPs, which use a number of physical, chemical, and biological mechanisms to control pollutants, is generally based upon the contributing drainage area, storage and treatment volume required, and flow rate through the system.

One of the first considerations in selecting a stormwater BMP is the functional goal of the BMP. The main components of stormwater management (SWM) are: water quality, channel protection, and flood protection. Any one or a combination of these components will dictate the functional goal of the BMP design. In general, SWMFs can be categorized into water quality BMPs and water quantity (channel and flood protection) facilities.

The use of some BMPs is limited by site or watershed feasibility factors, such as environmental impacts, drainage area or watershed size, topographic constraints, underlying soils and geology, seasonal high groundwater table, and other constraints specific to a project.

The BMPs designed for water quality control provide varying levels of runoff reduction and pollutant removal. Phosphorus is the keystone pollutant targeted for removal in Virginia, although sediment and nitrogen remain pollutants of concern and are addressed in water quality BMP design.
11.5.1.1 Offsite Water Quality Compliance

Offsite alternatives as describe in 9VAC25-870-69 may be utilized to meet the design criteria for water quality under certain situations. Offsite options shall not be allowed:

- Unless the selected offsite option achieves the necessary pollutant load reductions prior to the commencement of the construction of the proposed project. Where the offsite option will be constructed as a part of the proposed VDOT project, the offsite option must be completed and functional prior to the completion of the VDOT project, or

- In violation of local water quality-based limitations at the point of discharge that are consistent with the determinations made pursuant to a TMDL Implementation Plan, contained in a MS4 Program Plan approved by DEQ, or as otherwise may be established or approved by DEQ.

A common offsite compliance option used for VDOT projects is the purchase of nutrient credits as discussed in IIM-LD-251.

11.5.1.2 Compliance Spreadsheets

Compliance with the water quality design criteria set out in subdivisions A 1 and A 2 of 9VAC25-870-63 shall typically be determined by utilizing the Virginia Runoff Reduction Method (Virginia Runoff Reduction Method: Instructions & Documentation, March 28, 2011) or another equivalent methodology that is approved by the State Water Control Board. VDOT may utilize the DEQ VRRM Compliance Spreadsheets or any proprietary or non-proprietary spreadsheet or software which properly incorporates the VRRM for assessing compliance with pollutant removal requirements. Designers are responsible for ensuring that their use of the software (inputs and outputs) is consistent with the VRRM and applicable technical criteria. Load reductions for new development and redevelopment will typically be calculated using these tools, unless separate calculations are required due to limitations in the tool (e.g. in some instances, the DEQ VRRM Redevelopment Spreadsheet overestimates required load reductions for prior developed lands with high existing turf loadings. In these instances, VDOT may elect to utilize an alternate spreadsheet which calculates required load reductions in accordance with the method). Additional calculations will be prepared in general accordance with the Virginia Runoff Reduction Method.

The BMPs addressed in the most recent VDOT BMP Design Manual of Practice and contained in the most recent versions of VDOT’s approved special provisions and standard insertable sheets for Runoff Reduction Practices are preferred to satisfy these criteria, and will be adapted by the Engineer as appropriate. Other approved BMPs found on the Virginia Stormwater BMP Clearinghouse Website may also be utilized, if deemed acceptable by VDOT, including manufactured BMPs approved by the DEQ.
Deviations in design from the VDOT BMP Design Manual of Practice, VDOT approved special provisions and standard insertable sheets, or the Virginia BMP Clearinghouse must be approved.

Compliance with the water quality criteria are evaluated generally using the entire site. However, where a site drains to more than one 6th Order HUC, the pollutant load reduction requirements shall be applied independently within each HUC unless reductions are achieved in accordance with the options described in Section 11.4.1 (7).

11.5.1.3 Land Cover and Soil Groups

There are three categories of land cover identified by the VRRM: Forest/Open Space, Managed Turf, and Impervious Cover. Definitions of the three categories of land cover are provided in the definitions chapter of the Drainage Manual and DEQ guidance on use of the VRRM.

There are limited circumstances in which forest/open space will be allowed within the development site, and subsequently allowed to be identified as such using the VRRM. These include the following:

1. Surface area of stormwater BMPs that are NOT wet ponds, have some type of vegetative cover, and that do not replace an otherwise impervious surface. (BMPs in this category include bioretention, dry swale, grass channel, ED pond that is not mowed routinely, stormwater wetland, soil amended areas that are vegetated, and infiltration practices that have a vegetated cover.)

2. Utility rights-of-way that will be left in a natural vegetated state (can include areas that will be bush-hogged no more than four times per year)

In general, areas to be disturbed during construction in the R/W that do not meet the criteria above will be considered managed turf as the area will be assumed to be compacted unless soil restoration practices are implemented following disturbance.

If a disturbed area is expected to remain forest/open space because compaction will not occur, this area must be identified on the plan including the rationale for it not being compacted. The plan must also include maintenance requirements (i.e. mowing and fertilizer requirements) for these areas to maintain forest/open space conditions after construction is complete. These areas must be identified early in the process to allow the District Hydraulics Engineer an opportunity to review and approve.

If an area is to remain undisturbed, the plan must include provisions to exclude equipment from entering the area during construction and must include maintenance requirements (i.e. mowing and fertilizer requirements) for these areas to remain forest/open space conditions after construction is complete. These areas must be identified early in the process to allow the District Hydraulics Engineer an opportunity to review and approve.

Hydrologic soil group determinations can be made using the National Resources Conservation Service (NRCS) web soil surveys.
11.5.2 Water Quantity

Controlling the quantity of stormwater runoff can provide the following potential benefits:

- Prevention or reduction of increases in peak runoff rate, volume, and other characteristics caused by land disturbance and urban development;
- Decrease downstream channel erosion;
- Mitigation of downstream drainage capacity problems (localized flooding);
- Recharge of groundwater resources;
- Reduction or elimination of the need for downstream outfall improvements; and,
- Maintenance of historic base flow rates and stream hydrology.

Note that the new Water Quantity criteria for channel and flood protection are applied at each outfall from a land-disturbing project. Compliance must be shown at each outfall, not as an aggregate for the entire land-disturbing activity. The application of Water Quantity criteria by outfall may result in the use of different criteria applied to demonstrate compliance at each outfall.

11.5.2.1 Channel Protection

Conventional channel protection in Virginia focused on the control of runoff peak rate of flow for specific design events (2-yr storm for natural channels, and the 2- and 10-yr storms for manmade channels), as defined under Minimum Standard 19 of the Virginia Erosion and Sediment Control Regulations (9VAC25-840-40).

The application of Minimum Standard 19 has not always resulted in protection of channels and properties downstream of land development projects from erosion and sedimentation. As a result, the amended Virginia Stormwater Act (§62.2-44.15:28) requires the maintenance of runoff peak rate of flow and other characteristics (such as runoff volume, frequency, and duration) that replicate the existing pre-development runoff and site hydrology, or improve upon the existing pre-development conditions if stream channel erosion already exists. In order to address this requirement, a Technical Advisory Panel (TAP) was formed by the DEQ to develop new channel protection criteria in 2011-2012. The results of the TAP work is the new channel protection criteria in the Virginia Stormwater Management Program Regulation.
The new channel protection criteria require that concentrated stormwater runoff is released to a manmade, restored, or natural stormwater conveyance system. For manmade and restored channels, the designer must demonstrate adequacy of the channel from the outfall up to the limits of analysis. If inadequate for the post-development conditions, the designer must provide for detention or channel improvements, or apply the new Energy Balance method. For discharges to natural channels, an adequacy analysis is not required, but the new Energy Balance criteria must be applied at each outfall or point of discharge.

Whenever the Energy Balance is used to achieve compliance for channel protection, a channel adequacy analysis is not necessarily required. However, a channel analysis is recommended for both manmade and restored channels to determine if a SWMF is required for compliance with the water quantity criteria, and to optimize the size of the necessary SWMFs, as the control requirements for the manmade and restored channels could be less than those required to satisfy the Energy Balance.

Defaulting to a design based upon the Energy Balance when the conveyance system is manmade or restored, without first conducting a channel adequacy analysis, could result in higher costs for R/W acquisition, design, construction, and maintenance. The designer should consider the additional cost for the SWMF versus the cost for the channel adequacy analysis. While the channel adequacy analysis may require additional field survey, hydrologic analysis, and hydraulic modeling, the cost for the analysis may be less than the cost of implementing a SWMF designed for the Energy Balance that is larger than needed to meet the criteria for a manmade or restored channel.

11.5.2.1.1 Manmade Stormwater Conveyance System
A manmade stormwater conveyance system, including ditches, swales, curb, gutter, storm sewer, culverts, or other components must convey the post-development peak flow rate from the 2-yr storm event without causing erosion of the system. The system must be shown adequate to the limits of analysis (see below). Where a manmade channel is inadequate for the post-development condition, site design, stormwater detention, or channel improvements can be used to obtain compliance in the 2-yr storm event. Alternatively, the Energy Balance criteria can be applied to achieve compliance, regardless of the adequacy of the manmade stormwater conveyance system. Engineering methods for evaluating the adequacy of manmade stormwater conveyance systems are discussed in more detail in Chapters 4, 6, 7, and 9 of the Drainage Manual.

11.5.2.1.2 Restored Conveyance System
A restored conveyance system, such as a restored or relocated stream based upon natural channel design concepts, must meet the hydrologic and hydraulic parameters used for the restoration design. Where a restored channel is inadequate for the post-development condition, site design, stormwater detention, or channel improvements can be used to obtain compliance. The Energy Balance criteria can also be applied to achieve compliance, regardless of the adequacy of the restored conveyance system. Design methods for evaluating and designing restored conveyance systems can be found in Chapters 4, 6, and 7 of the Drainage Manual.
11.5.2.1.3 Natural Conveyance System
For a natural conveyance system, the Energy Balance must be applied for compliance with the channel protection criteria. The methods for applying the Energy Balance are discussed below. A natural channel need not be analyzed for channel adequacy, as application of the Energy Balance is considered adequate for channel protection.

11.5.2.1.4 Energy Balance
In order to move beyond the traditional control of just the peak rate of runoff and address other runoff characteristics such as volume, frequency, and duration, a Technical Advisory Panel developed the Energy Balance method. While the term “energy” is used to describe the method, the computation is actually more loosely related to a “power” balance, but the difference is not important to the development and application of the method to protect channels.

The fundamental concept of the Energy Balance is to further reduce the post-developed peak rate of runoff to below the pre-developed peak rate of runoff in inverse proportion to the increase in runoff volume for the post-developed condition. While application of the Energy Balance may not directly reduce the runoff volume resulting from development, it allows more time for infiltration and reduces the frequency of channel-forming storm events, providing channel hydrology similar to existing low flow or base flow conditions.

The Energy Balance is based upon the 1-yr return period storm instead of the 2-yr storm previously required for natural channels in Minimum Standard 19 (and still required for manmade channels). The Energy Balance equation used to calculate the allowable peak rate of runoff was summarized in the Design Criteria section:

\[
Q_{\text{Developed}} = \text{I.F.} \times \left( \frac{Q_{\text{Pre-Developed}} \times RV_{\text{Pre-Developed}}}{RV_{\text{Developed}}} \right)
\]

Where:
I.F. (Improvement Factor) = 0.8 for sites > 1 acre LDA or 0.9 for sites ≤ 1 acre LDA
\(Q_{\text{Developed}}\) = the allowable peak flow rate of runoff from the developed site for the 1-yr 24-hour storm.
\(RV_{\text{Developed}}\) = the volume of runoff from the site in the developed condition for the 1-yr 24-hour storm.
\(Q_{\text{Pre-Developed}}\) = the peak flow rate of runoff from the site in the pre-developed condition for the 1-yr 24-hour storm.
\(RV_{\text{Pre-Developed}}\) = the volume of runoff from the site in pre-developed condition for the 1-yr 24-hour storm.
11.5.2.1.5  Improvement Factor (I.F.)

As noted previously, the amended Stormwater Management Act requires that a land-disturbing activity improve upon the existing runoff characteristics when the existing receiving channel is already eroding. To address this requirement, an Improvement Factor (I.F.) was added to the Energy Balance method to require a further reduction in the peak rate of runoff for the post-development condition. For small projects ≤ 1-acre in area, the I.F. is 0.9, resulting in a minimum 10% reduction in the peak rate of runoff. Projects > 1-acre must apply an I.F. of 0.8 or a minimum 20% reduction in the allowable peak runoff rate.

In some cases, where it can be demonstrated that the existing channel is not experiencing erosion in the pre-development condition, the I.F. may be waived for calculating the allowable peak rate of runoff for the post-development condition. However, use of the waiver will require a channel adequacy analysis to demonstrate that the existing channel is adequate for the existing conditions and that an improvement is not necessary. Waivers may be granted by DEQ provided that VDOT coordinates with DEQ and submits a written request to DEQ requesting the exception. Waiver/exception requests must be submitted in writing to the VDOT Central Office L&D MS4/SWM Program for review and coordination with the DEQ Central Office. The request shall include documentation of the need for the waiver/exception. The documentation shall describe all means and methods evaluated for meeting the water quality/quantity requirements and the reasons why specific means or methods were determined not feasible. The documentation shall also state that the exception being requested is the minimum necessary to afford relief.

11.5.2.1.6  Forested Conditions

When applying the Energy Balance method to a land-disturbing activity, the allowable peak rate of runoff could be less than the runoff from the site if it were undeveloped in a good forested condition. This can occur from the application of the I.F. on a site where the runoff peak rate or volume does not increase appreciably from the pre-developed condition. In this case, the allowable peak rate of runoff should not be less than the forested condition calculated as follows:

\[
Q_{\text{Developed}} = \left( \frac{Q_{\text{Forest}} \times RV_{\text{Forest}}}{RV_{\text{Developed}}} \right)
\]

Where:
- \(Q_{\text{Developed}}\) = the allowable peak flow rate of runoff from the developed site for the 1-yr 24-hour storm.
- \(RV_{\text{Developed}}\) = the volume of runoff from the site in the developed condition for the 1-yr 24-hour storm.
- \(Q_{\text{Forest}}\) = the peak flow rate of runoff from the site in a forested condition for the 1-yr 24-hour storm.
- \(RV_{\text{Forest}}\) = the volume of runoff from the site in a forested condition for the 1-yr 24-hour storm.
11.5.2.1.7 *Limits of Analysis*
Unlike Minimum Standard 19, where the receiving channel was analyzed for adequacy immediately below the land development site outfall (or at the outlet of a storm sewer when the project outfalls to a storm sewer), the new channel protection criterion requires that manmade and restored channels be analyzed and demonstrated to be adequate downstream to the point in the drainage system where the site contributing drainage area is ≤1% of the total watershed area, also known as the “limits of analysis”. Alternatively, the limits of analysis can be the point in the system where the peak rate of runoff from the 1-yr 24-hour storm for the post-developed condition prior to the implementation of any stormwater quantity controls is less than 1% of the total watershed peak rate of runoff.

The point where the limit of analysis occurs does not need to be included in the evaluation of adequacy, but the system from the outfall or point of discharge up to (but not including) the limits of analysis must be analyzed. If there is a natural channel anywhere in the system from the outfall up to the limit of analysis (but not including the point defining the limit of analysis), then the energy balance will apply. Note that the limits of analysis are applied separately for each outfall or point of discharge from a regulated land-disturbing activity.

11.5.2.1.8 *Runoff Reduction*
A consequence of the new Runoff Reduction Method (RRM) discussed previously under Water Quality is the reduction in runoff volume associated with BMPs that provide RRM credit. By reducing the runoff volume with RRM facilities, the allowable discharge under the Energy Balance method may be higher because the overall increase in runoff volume is lower for the post-development condition.

To reflect the reduced runoff volume when RRM facilities are included in the SWM design, an adjustment in the post-development Runoff Curve Number (CN) can be calculated using the VRRM spreadsheets or other engineering methods acceptable to VDOT and DEQ (see section on modeling).

An adjusted CN must be calculated individually for each storm event (1-, 2-, and 10-yr 24-hour), as the CN reduction provided will vary based upon the storm. In general terms, the CN reduction is calculated assuming that the RRM Treatment Volume provided is treated as additional initial abstraction (I_a) in the CN Method from the NRCS Technical Release 55 (TR-55) *Urban Hydrology for Small Watersheds*. As the RRM retention volume is constant and based upon the SWMF design, but the total runoff volume varies for each storm event, a reduced CN would be calculated and used in hydrologic calculations downstream of the RRM facilities.

Here is the method used to calculate adjusted CNs using modified equations 2-3 and 2-4 of TR-55:
\[ Q - R = \frac{(P - 0.2 \times S_{\text{adj}})^2}{(P + 0.8 \times S_{\text{adj}})} \]

Where:
- \( Q \) = runoff (watershed inches) for the 1-, 2-, or 10-yr 24-hour storm event before RRM practices are applied
- \( R \) = volume of retention storage provided by runoff reduction practices (watershed inches) = runoff reduction volume \((\text{ft}^3)/\text{watershed drainage area (acres}) \times 12 \) \((\text{inches/foot})/43,560 \) \((\text{ft}^3/\text{acre})\)
- \( P \) = rainfall (inches) for the 1-, 2-, or 10-yr 24-hour storm event
- \( S_{\text{adj}} \) = potential maximum retention after runoff begins (inches) adjusted for runoff reduction volume

The value of \( Q \) prior to runoff reduction is calculated for the watershed using standard TR-55 graphical peak discharge methodology; the retention storage volume \( R \) provided by runoff reduction practices is typically calculated in the VRRM spreadsheet in \( \text{ft}^3 \), converted to watershed inches, and subtracted from \( Q \); and the equation solved for an adjusted \( S \) value \((S_{\text{adj}})\). The \( S_{\text{adj}} \) value is then used to determine the adjusted \( CN \) using the following relationship from TR-55:

\[ CN_{\text{adj}} = \frac{1000}{S_{\text{adj}} + 10} \]

Where:
- \( CN_{\text{adj}} \) = adjusted \( CN \) calculated individually for the 1-, 2-, and 10-yr 24-hour storm events

The solution for \( S_{\text{adj}} \) involves a quadratic equation, and multiple techniques are available to solve. However, the Virginia RRM Spreadsheet solves the equation for \( S_{\text{adj}} \) and provides \( CN_{\text{adj}} \) for the 1-, 2-, and 10-yr storms to simplify the process for the designer.

Note that the RRM Spreadsheet adjusts the \( CN \) using units of watershed inches instead of acre-feet or cubic feet. As long as the pre- and post-development drainage areas remain the same at the outfall, the use of watershed inches is valid. However, when the drainage area at an outfall changes from the pre- to post-developed condition, the \( CN \) adjustments used for the Energy Balance should be made using units of volume, such as acre-feet or cubic feet.

While the RRM Spreadsheet is ideal for adjusting \( CN \)s for simple drainage areas, there are times when changes in drainage area due to development or more complex drainage networks exceed the capabilities of the RRM spreadsheet. In this case, the designer will have to use other means to apply the \( CN \) methodology of TR-55 and compute adjusted \( CN \) values for a project.

The adjusted \( CN \) for each storm event (1-, 2-, and 10-yr 24-hour) is then used in the revised hydrologic calculations to determine the adjusted peak rate of runoff and runoff volume after RRM practices in each storm. The adjusted peak rate of runoff and runoff
volume are used to determine compliance for water quantity as already discussed, including the Energy Balance method.

Note that the runoff retention volume provided via RRM practices should not be used to adjust CNs and also as storage for storm routing for water quantity control. The volume reduction provided should be used for one or the other, but not both. However, should a SWMF include non-retention storage (detention) for water quantity above or outside of the RRM practice retention volume, then the additional storage could be used in storm routing to demonstrate additional reductions in peak rate of runoff due to detention.

Typically, it is advantageous for storage-based practices (such as bioretention facilities) to disregard the potential CN adjustment value, and be routed including the pore storage (in the media, and the choker/reservoir stone), the surface storage, and the outlet characteristics. It is also commonplace to neglect underdrain flows (this assumption was used in the derivation of the Effective CN method) due to their relatively insignificant effect on the overall discharge, in effect modeling the practice as an empty plugged bathtub.

An example would be taking credit for the retention volume present in the engineered soil media, sump stone, and ponding in a bioretention facility. If the total retention volume is used to adjust the CN, then the same volume should not be used as storage to route a storm through the bioretention facility to take credit for peak runoff reduction. However, the retention volume in the engineered media and sump stone could be used to calculate an adjusted CN, and the ponding volume used for detention volume routing through the facility. In this case, the runoff reduction volume used to adjust the CN would need to be calculated separately from the RRM Spreadsheet, which assumes that all runoff retention storage is used for CN adjustment.

11.5.2.1.9 **Increases in Peak Rate of Runoff**
In no case shall the Energy Balance method be used to justify an increase in post-developed peak rate of runoff from a regulated land-disturbing project. This can occur when the site design results in a post-development runoff volume that is less than the pre-development volume, such as when runoff reduction controls are used. For all regulated land-disturbing activities, the post-developed peak rate of runoff shall not exceed the pre-developed peak rate of runoff.

11.5.2.2 **Flood Protection**
Local flood protection in Virginia for storm events smaller than the 100-yr event was addressed under Minimum Standard 19 of the Virginia Erosion and Sediment Control Regulations (9VAC25-840-40) and in the flooding criteria in Part II C of the Virginia Stormwater Management Program (VSMP) Regulation (9VAC-25-870-98).

In the case of linear transportation projects, the criteria in the VSMP Regulation did not require the control of the post-developed stormwater runoff for flooding, except in accordance with a watershed or regional SWM plan. With the amended VSMP Regulation, linear transportation projects are no longer exempt from the Flood
Protection criteria. The application of the new Flood Protection criteria to VDOT projects is discussed below.

11.5.2.2.1 Conveyance System Definition
The new VSMP Regulation defines a “stormwater conveyance system” as a combination of drainage components that are used to convey stormwater discharge, either within or downstream of the land-disturbing activity. This includes:

"Manmade stormwater conveyance system" means a pipe, ditch, vegetated swale, or other stormwater conveyance system constructed by man except for restored stormwater conveyance systems;

"Natural stormwater conveyance system" means the main channel of a natural stream and the flood-prone area adjacent to the main channel; or

"Restored stormwater conveyance system" means a stormwater conveyance system that has been designed and constructed using natural channel design concepts. Restored stormwater conveyance systems include the main channel and the flood-prone area adjacent to the main channel.

Note that both the natural and restored systems include the main channel and the flood-prone area adjacent to the main channel when considering localized flooding and Flood Protection.

An analysis of the system in the existing conditions is necessary to establish if localized flooding occurs in the 10-yr 24-hour storm event. Historic flood records and anecdotal evidence may also be useful in documenting existing flood conditions, although both should be supported by hydrologic and hydraulic modeling to support the existence of flooding.

11.5.2.2.2 Localized Flooding not Currently Experienced
When localized flooding does not occur under pre-developed conditions (i.e., the stormwater conveyance system contains the 10-yr 24-hour storm event), the post-development discharge from project must be confined within the stormwater conveyance system. If this does not occur, detention of stormwater, system improvements, or a combination of both may be used to make the system adequate.

11.5.2.2.3 Localized Flooding Currently Experienced
When localized flooding does occur under pre-developed conditions (i.e., the stormwater conveyance system does not contain the 10-yr 24-hour storm event), either:

a. The post-development peak discharge from the project must be confined within the stormwater conveyance system to avoid localized flooding, and detention of stormwater, system improvements, or a combination of both may be used to make the system adequate; or,

b. The post-development peak flow rate must be less than the pre-developed peak flow rate for the 10-yr 24-hour storm event.
11.5.2.2.4 Compliance with the Flood Protection Criteria

Compliance with the flood protection criteria can be achieved by detaining the post-development 10-yr 24-hour peak runoff at each project outfall to below the pre-development rate; however, there are cases where detention to the pre-developed peak runoff rate may not be necessary, and a system adequacy analysis is required to demonstrate the need.

If it is demonstrated via an adequacy analysis that the stormwater conveyance system does not contain the post-developed peak rate of runoff, then the designer must consider the options for detention and system improvements.

In general, detention may be preferred to system improvements due to the cost of acquiring additional R/W and long-term maintenance of the system improvements. However, the system improvements required to mitigate minor increases in peak runoff may be less intrusive and expensive than providing detention and meet other project goals for SWM and environmental protection. The designer must consider the total cost of options before making a final decision to provide detention, make system improvements, or a combination of both.

11.5.2.2.5 Limits of Analysis

Stormwater conveyance systems must be analyzed and demonstrated to be adequate downstream up to the point in the drainage system where the site contributing drainage area is ≤ 1% of the total watershed area.

The limits of analysis can also be the point in the downstream system where the peak rate of runoff from the 10-yr, 24-hour storm for the post-developed condition is less than 1% of the existing peak rate of runoff for the watershed.

The point where the limit of analysis occurs does not need to be included in the evaluation of adequacy, but the system from the outfall or point of discharge up to (but not including) the limits of analysis must be analyzed. If there is a natural channel anywhere in the system from the outfall up to the limit of analysis (but not including the point defining the limit of analysis), then the energy balance will apply. Note that the limits of analysis are applied separately for each outfall or point of discharge from a regulated land-disturbing activity.

Unlike the channel protection analysis, the flood protection analysis can also end when the stormwater conveyance system enters a mapped floodplain or other flood-prone area adopted by local ordinance.

11.5.2.3 Sheet Flow

When the post-developed condition results in increases in sheet flow runoff volume from a project, including the dispersal of concentrated flow using level spreaders and other energy dissipating techniques, the designer must identify the discharges and evaluate the runoff increase for impacts to waterways and properties. If the evaluation demonstrates the potential to cause or contribute to erosion, sedimentation, or flooding below the project, then the increased sheet flows must be directed to a SWMF or
stormwater conveyance system that is adequate for the increased runoff. When all discharges from a project are sheet flow and the criteria are met, no further controls are required for channel or flood protection.

It should be noted that the conversion of concentrated flow to sheet flow via level spreaders and energy dissipaters becomes more difficult as the volume and peak rate of runoff increases. This is especially true for concentrated discharges from a culvert or storm sewer system, or when the area below the outfall is sloped away from the discharge. If the designer chooses to convert concentrated flow to sheet flow, the evaluation and design of control structures must be carefully documented and supported via engineering computations. The area required to convert concentrated flow to sheet flow may exceed available R/W and easement for larger outfalls, so flow may need to remain concentrated to make the best use of existing land available for drainage and SWM.

The potential for erosion and sedimentation from increases in sheet flows from a project shall be evaluated. Flooding must be considered for increases in sheet flow volume as well, using good engineering practice and acceptable hydrologic and hydraulic evaluation. Designers shall describe how the potential for flooding, erosion and sedimentation from increased sheetflow was evaluated and how no adverse impact on downstream waterways and properties were determined.

11.5.3 Part II B Design Procedures and Sample Problems

11.5.3.1 SWM Plan Requirements
The following documentation will be required for SWMF/BMP design:

- Documentation requirements presented in Chapter 6, Hydrology.
- Computations for determination of the pre- and post-development peak runoff rates and runoff volumes for the design storms.
- Water Quantity (Channel and Flood Protection) computations for the relevant design storms, including determination of the limits of analysis.
- Water Quality computations based upon the Runoff Reduction Method (RRM).
- SWMF Tabulation Sheet when submitting final plans.
- Provide all documentation from storm routing. This would generally include inflow and outflow hydrographs, stage-storage curves, discharge rating curves for the spillway(s), and routing summaries. This information would be generated by various computer modeling software.
- Basin grading and spillway(s) details and specifications provided.
- Complete (C) and Minimum (M) plan projects shall show SWM measures in the plan assembly and may include insert sheets specific to each SWM measure for clarity.
- No-plan (N) and other types of projects (including maintenance activities) that have an abbreviated plan assembly must conform to the requirements of the VSMP Regulations and VPDES Construction General Permit where the land disturbance value exceeds the applicable land disturbance thresholds for such.
The plan design details for BMPs shall be appropriately sealed and signed by a qualified person licensed or registered in the Commonwealth of Virginia as an architect, professional engineer, land surveyor, landscape architect, or certified professional soil scientist, as regulated by the Virginia Department of Professional Regulation.

11.5.3.2 Water Quality – Runoff Reduction Method (RRM) Procedure

The Virginia Runoff Reduction Method is described and detailed in documents published by the Virginia DEQ, including Guidance Memorandums, supporting white papers from the method developer, training materials, and the VRRM Spreadsheets. Please see the DEQ documents for more in-depth procedures on implementation of the RRM.

For this process, it is assumed that the VRRM Spreadsheets are being used to demonstrate compliance with the water quality criteria. If another acceptable model or method is being used, follow the instruction and directions for that model or method. Some of the same steps may apply.

Step 1 - Select Project Type and Open VRRM Spreadsheet

Determine if the project qualifies as New Development or Redevelopment (see the definitions for “New Development” and “Redevelopment” in Chapter 1 Appendix A-1.)

Based on the type of development, start a new VRRM Spreadsheet for the project and enter information to identify the project.

Note that water quality compliance must be demonstrated by Hydrologic Unit Code (HUC). If a project drains to more than one HUC, a VRRM Spreadsheet may be required for each HUC to demonstrate compliance in each HUC individually. Overtreatment in one HUC is not allowed to compensate for compliance in another HUC using onsite BMPs, unless the overtreatment is in an adjacent upstream HUC.

Step 2 - Enter Project Information on Site Tab

Collect overall project site parameters and enter into the Site tab in the VRRM Spreadsheet, including:

- Land Cover type (Forest/Open Space, Managed Turf, or Impervious, not the land cover types from TR-55)
- Hydrologic Soil Group or HSG for each land cover type (A, B, C, or D as defined by the USDA for TR-55).
- Sub-area for each combination of Land Cover type and HSG.
Check that the total area for each Land Cover type and overall project area are correct in the summary table for the sub-areas entered.

Note that the New Development spreadsheet only needs the Post-Development project parameters.

The Redevelopment spreadsheet requires site information for the pre-development (or existing) conditions, as well as the post-development (or proposed) conditions.

The Redevelopment spreadsheet has the option to calculate alternate water quality reduction requirements for linear development projects, such as roadways. Make sure the cell is marked “Yes” for a linear redevelopment project.

Please note that the latest redevelopment spreadsheet from DEQ can compute the removal required for both new development and redevelopment on a project, so it is not necessary to break out the land cover by new development or prior development or use both the new development and redevelopment spreadsheets for a project.

Step 3 - Review Project Site Summary and Pollutant Removal Requirements

The spreadsheet should update the VRRM Site calculations and report the following information for the overall site on the tab:

- Runoff coefficients (Rv)
- Total Phosphorous (TP) Load Reduction Required (lb/yr)
- Land Cover Summary
- Treatment Volume (cubic feet or ft³)
- Nutrient Loads (lb/yr)

Step 4 - Identify Outfalls and Provide Drainage Area Information

Based upon the topography, proposed drainage network, and site design, identify the location(s) of project outfall(s).

Determine the contributing drainage area to each outfall, as well as the sub-areas for Land Cover type by HSG. The VRRM Spreadsheet can handle up to 5 outfalls, one each on the worksheet tabs labeled D.A. A to D.A. E.

For each outfall, enter the contributing sub-area to the outfall based upon the distribution of Land Cover type and HSG, as was done for the overall project on the Site tab. The D.A. tab will calculate the total drainage area and Runoff coefficient (Rv) for the outfall.
Step 5 - Selection and Application of Stormwater BMPs

Based upon the topography, drainage design, R/W available, site constraints, and other SWM requirements, select stormwater best management practices (BMPs) for use in the outfall.

Note that the entire contributing drainage area to an outfall may not be treated in one BMP. The contributing drainage area to the outfall can be broken into smaller areas for the selection and application of distributed BMPs. BMPs can also be placed in series to provide a treatment train with higher pollutant removal efficiencies (see Section 11.5.3.1.1).

Step 5a - Apply Runoff Reduction (RR) Practices

Begin with Runoff Reduction (RR) practices that are suitable for the contributing drainage area, as they generally provide a greater pollutant removal rate than conventional BMPs that only provide treatment. These include:

- Vegetated Roof (not likely to be used on transportation projects)
- Rooftop Disconnection (not likely to be used on transportation projects)
- Permeable Pavement
- Grass Channel
- Dry Swale
- Bioretention
- Infiltration
- Extended Detention
- Sheetflow to Filter/Open Space

See the VDOT BMP Design Manual of Practice or the Virginia BMP Clearinghouse for detailed information on the selection, application, and design of RR practices.

Once RR practices have been selected, enter the acreage of managed turf and impervious area draining to each practice in the spreadsheet. The spreadsheet will automatically calculate the runoff reduction, remaining runoff volume, total treatment volume, pollutant load, pollutant removed, and remaining pollutant load for each practice.

Note that RR practices can be placed downstream of other BMPs to create a treatment train of BMPs in series. See Section 11.5.3.1.1 for a discussion.

The purchase of nutrient credits to address post-construction water quality reduction requirements for construction activities shall be considered the preferred alternative when available and economically feasible.
If the project site area is less than 5 acres, up to 100% of the Phosphorous reduction can be achieved via offsite options; or, if the removal rate is less than 10 lb/yr, up to 100% of the Phosphorous reduction can be achieved via offsite options.

If the project site area is greater than 5 acres and the Phosphorous removal rate is greater than 10 lb/yr, up to 25% of the Phosphorous reduction can be achieved via offsite options. In some cases, more than 25% can be purchased if it can be shown that achieving 75% removal onsite is not practicable. This requires submitting a waiver to DEQ to secure approval for purchasing more than 25% of the credits.

See Section 11.5.3.2.2 below for a discussion of offsite water quality compliance options.

**Step 5b - Review Water Quality Compliance**

Once RR practices have been applied to each outfall, go to the Water Quality Compliance tab to determine if the selected BMPs meet the water quality requirements for the overall site.

If the TP load reduction target has been met for the project site, then proceed to detailed design for the BMPs selected, Step 7.

If the TP load reduction target has been exceeded for the project site, consider optimization of the RR practices selected to reduce the TP reduction to the target load. Return to Step 5a.

If the TP load reduction target has not been met, consider the application of additional RR practices, the use of RR practices in series, or move on to Step 5c to select conventional treatment options for compliance.

**Step 5c - Apply Conventional BMPs**

If the RR practices selected do not satisfy the Phosphorous Load reduction requirements for the project, then the use of conventional BMPs (no runoff reduction provided) should be considered. The conventional BMPs included in the VRRM Spreadsheet are:

- Wet Swales
- Filtering Practices
- Constructed Wetlands
- Wet Ponds
- Manufactured Treatment Devices (MTDs)

See the VDOT BMP Design Manual of Practice or the Virginia BMP Clearinghouse for detailed information on the selection, application, and design of conventional BMPs.
Note that conventional BMPs can be placed downstream of other BMPs to create a treatment train of BMPs in series. See Section 11.5.3.2.1 for a discussion.

Step 5d - Review Water Quality Compliance

Once conventional BMPs have been applied, go to the Water Quality Compliance tab to determine if the selected BMPs meet the water quality requirements for the overall site.

If the TP load reduction target has been met for the project, then proceed to detailed design for the BMPs selected, Step 7.

If the TP load reduction target has been exceeded for the project site, consider optimization of the BMPs selected to reduce the TP reduction to the target load. Return to Steps 5a and 5c.

If the TP load reduction target has not been met, apply additional BMPs, the use of BMPs in-series, or move on to Step 6 to investigate offsite compliance options.

Step 7 - BMP Design

Once compliance with the pollutant load reduction requirements is achieved by application of BMPs and offsite compliance options, the designer should proceed with detailed design of BMPs. The results of the VRRM Spreadsheet should be retained to verify the Treatment Volume required for each RR practice and conventional BMP.

For detailed information on the design of BMPs, see the VDOT BMP Design Manual of Practice or the Virginia BMP Clearinghouse.

11.5.3.2.1 BMPs In-Series/Treatment Trains

SWM BMPs (RR and conventional) can be placed downstream of other BMPs to create a “treatment train” of practices in-series. The use of BMPs in series provides greater pollutant removal rates and allows smaller controls to be distributed throughout a project, closer to the pollutant sources.

This is accomplished by going to the spreadsheet entries for each upstream BMP and selecting the Downstream Practice to be employed via a dropdown list. The spreadsheet will automatically enter the remaining runoff volume and pollutant load from the upstream BMP to the calculations for the downstream BMP.
Note that if other areas drain to the downstream BMP (but not the upstream BMP in series), then those areas must be entered separately into the spreadsheet in the section for the downstream BMP. Note that the water quality sizing for BMPs with a runoff reduction practice upstream is reduced by the volume of runoff reduction upstream. However, all downstream facilities should be evaluated for proper conveyance and freeboard, as appropriate, using the full contributing drainage area.

11.5.3.2.2 Offsite Water Quality Compliance Options
When a project meets the requirements that allow offsite water quality compliance, the designer should consider the option, especially when the R/W or easements available for SWM onsite are limited and site constraints make onsite BMPs difficult or costly to implement. A cost comparison of the onsite versus offsite options should include capital costs (R/W, easements, and construction) as well as the long-term maintenance costs. Often, the cost for purchasing offsite nutrient credits to meet water quality load reductions is less costly, especially when land costs and long term maintenance are included in the analysis.

Also, the designer must consider the need for water quantity control for the project. While conventional BMPs provide no Runoff Reduction, RR practices reduce the runoff volume and provide water quantity control. The Runoff Reduction provided by the RR practice and the water quantity control provided should be considered in a comparison of the onsite versus offsite design options, especially if a SWMF will be required for water quantity control.

The most common form of offsite compliance is the purchase of Nutrient Credits from a Nutrient Credit Bank trading under the nonpoint nutrient offset program established pursuant to § 62.1-44.15:35 of the Code of Virginia. VDOT maintains contracts with approved Nutrient Credit Banks across the Commonwealth serving most of the major tributaries in Virginia. The cost per credit (lb/yr) has been fixed under the competitive, negotiated contracts. Note that the contracts have a fixed term and are re-advertised and negotiated periodically. See IIM-LD-251 for details on the VDOT Nutrient Credit purchase program.

Note that there are other ways to achieve water quality compliance offsite for a project. These include the payment into a pro rata system used to construct nutrient reduction BMPs in the same watershed; development under a comprehensive SWM plan adopted pursuant to the VSMP Regulation; on other VDOT owned properties in the same HUC or upstream HUC as the land disturbance activity; and other options approved by VDOT and the DEQ. These options are all unique and will not be developed further in this document. Contact the District Hydraulics Engineer, State Hydraulics & Utilities Engineer, or State MS4 Program Manager to coordinate and develop offsite options other than Nutrient Credit purchases.

11.5.3.2.3 Water Quality – Sample Problem – New Development
Assume a VDOT project with a 1.7-acre site draining to an outfall. The soils are all classified as HSG C by the USDA Soil Survey. In the pre-development condition, the entire site is undisturbed forest. In the post-developed condition, the site will include 0.8
acres of impervious roadway and shoulder, with the balance managed turf. What are the Rv, total TP load, TP Load Reduction Required, and total Treatment Volume for the site?

Step 1 - Use the VRRM Spreadsheet for New Development for the project, as the pre-development condition is described as “undisturbed forest”.

Step 2 - On the Site tab, enter the project name and date of the calculations. Complete the Land Cover table for the post-development conditions:

**Post-Development Project (Treatment Volume and Loads)**

<table>
<thead>
<tr>
<th>Land Cover (acres)</th>
<th>A Soils</th>
<th>B Soils</th>
<th>C Soils</th>
<th>D Soils</th>
<th>Totals</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Forest/Open Space (acres)</strong> -- undisturbed, protected forest/open space or reforested land</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
</tr>
<tr>
<td>Managed Turf (acres) -- disturbed, graded for yards or other turf to be mowed/managed</td>
<td></td>
<td></td>
<td>0.90</td>
<td></td>
<td>0.90</td>
</tr>
<tr>
<td>Impervious Cover (acres)</td>
<td></td>
<td></td>
<td>0.80</td>
<td></td>
<td>0.80</td>
</tr>
<tr>
<td><strong>Totals</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.70</td>
</tr>
</tbody>
</table>

Note that the pre-development condition land cover is not required for the New Development tab, as the allowable TP load based upon the water quality criteria is 0.41 lb/acre/yr for New Development, regardless of the type of land cover in the pre-development condition.

Step 3 - The spreadsheet automatically calculates the post-development Requirements for the Site Area:

**Post-Development Requirement for Site Area**

| TP Load Reduction Required (lb/yr) | 1.49 |

This is the difference between the allowable TP load of 0.41 lb/acre/yr for new development and the total TP load in the post-development condition, which is provided in the Land Cover Summary.
A Land Cover Summary for the post-development condition is provided:

**LAND COVER SUMMARY -- POST DEVELOPMENT**

<table>
<thead>
<tr>
<th>Land Cover Summary</th>
<th>Treatment Volume and Nutrient Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forest/Open Space Cover (acres)</td>
<td>0.00</td>
</tr>
<tr>
<td>Weighted Rv (forest)</td>
<td>0.00</td>
</tr>
<tr>
<td>% Forest</td>
<td>0%</td>
</tr>
<tr>
<td>Managed Turf Cover (acres)</td>
<td>0.90</td>
</tr>
<tr>
<td>Weighted Rv (turf)</td>
<td>0.22</td>
</tr>
<tr>
<td>% Managed Turf</td>
<td>53%</td>
</tr>
<tr>
<td>Impervious Cover (acres)</td>
<td>0.80</td>
</tr>
<tr>
<td>Rv (impervious)</td>
<td>0.95</td>
</tr>
<tr>
<td>% Impervious</td>
<td>47%</td>
</tr>
<tr>
<td>Site Area (acres)</td>
<td>1.70</td>
</tr>
<tr>
<td>Site Rv</td>
<td>0.56</td>
</tr>
<tr>
<td><strong>Treatment Volume</strong></td>
<td><strong>Treatment Volume</strong></td>
</tr>
<tr>
<td>(acre-ft)</td>
<td>(cubic feet)</td>
</tr>
<tr>
<td><strong>0.0798</strong></td>
<td><strong>3,478</strong></td>
</tr>
<tr>
<td><strong>TP Load (lb/yr)</strong></td>
<td><strong>2.18</strong></td>
</tr>
<tr>
<td><strong>TN Load (lb/yr)</strong></td>
<td><strong>15.63</strong></td>
</tr>
</tbody>
</table>

Based upon the results reported by the VRRM Spreadsheet, Site Tab, the Site Rv is 0.56, the total TP Load for the site is 2.18 lb/yr, the TP Load reduction required is 1.49 lb/yr (68% reduction from the total TP Load), and the total Treatment Volume required to achieve complete runoff reduction (no increase in runoff volume) is 3,478 ft³.

**Step 4 -** Assume the entire site drains to one outfall, so move to the second tab labeled “D.A. A” and enter the Land Cover information for the project. For this example, the Land Cover information for Drainage Area A is the same as for the project site:
### Drainage Area A Land Cover (acres)

<table>
<thead>
<tr>
<th>Land Cover</th>
<th>A Soils</th>
<th>B Soils</th>
<th>C Soils</th>
<th>D Soils</th>
<th>Totals</th>
<th>Land Cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forest/Open Space (acres)</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Managed Turf (acres)</td>
<td>0.90</td>
<td>0.90</td>
<td></td>
<td>0.22</td>
<td>1.70</td>
<td></td>
</tr>
<tr>
<td>Impervious Cover (acres)</td>
<td>0.80</td>
<td></td>
<td>0.80</td>
<td>0.95</td>
<td>1.70</td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.70</td>
<td></td>
</tr>
</tbody>
</table>

**Step 5a -** The first step in providing onsite treatment is to consider the use of Runoff Reduction practices. For the example, assume the roadway is open section with roadside drainage via swales and ditches. Also, assume the soils have low permeability and are unsuitable for infiltration (based upon a site specific soil investigation), but the depth to seasonal high groundwater and bedrock is > 6’.

Based upon this information and the information provided in the VDOT BMP Design Manual of Practice and Virginia BMP Clearinghouse, select a Dry Swale #1 as our Runoff Reduction practice.

As the entire site drains to the outfall via proposed Dry Swales, enter the entire site area into the spreadsheet under the columns “Managed Turf Credit Area” and “Impervious Cover Credit Area”:

### Stormwater Best Management Practices (RR = Runoff Reduction)

<table>
<thead>
<tr>
<th>Practice</th>
<th>Runoff Reduction Credit (%)</th>
<th>Managed Turf Credit Area (acres)</th>
<th>Impervious Cover Credit Area (acres)</th>
<th>Volume From Upstream Practice (ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5. Dry Swale (RR)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.a. Dry Swale #1 (Spec #10)</td>
<td>40</td>
<td>0.90</td>
<td>0.80</td>
<td>0</td>
</tr>
</tbody>
</table>

Once the information for the contributing drainage area to the Dry Swales is entered, the spreadsheet will calculate the Runoff Reduction volume provided by the practice, the remaining volume of runoff not removed via the Runoff Reduction practice, the total Treatment Volume for the practice, the untreated TP load to the practice, the TP removed by the practice, and any remaining TP load after runoff reduction and treatment.
## Stormwater Best Management Practices (RR = Runoff Reduction)

<table>
<thead>
<tr>
<th>Practice</th>
<th>Runoff Reduction (ft³)</th>
<th>Remaining Runoff Volume (ft³)</th>
<th>Total BMP Treatment Volume (ft³)</th>
<th>Untreated Phosphorus Load to Practice (lb)</th>
<th>Phosphorus Removed By Practice (lb)</th>
<th>Remaining Phosphorus Load (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5. Dry Swale (RR)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.a. Dry Swale #1 (Spec #10)</td>
<td>1,391</td>
<td>2,087</td>
<td>3,478</td>
<td>2.18</td>
<td>1.13</td>
<td>1.05</td>
</tr>
</tbody>
</table>

Step 5b - Based upon the results above, has 100% Runoff Reduction been achieved by the application of Dry Swales to the project? Has the TP Load reduction required been satisfied to achieve compliance?

According to the Site tab, 3,478 ft³ of Runoff Reduction is required to reduce the runoff volume to pre-development condition, but only 1,391 ft³ is achieved in the Dry Swales, so 100% Runoff Reduction is not met. Also, 1.49 lb/yr of TP reduction is required for site compliance, but the Dry Swales only provide 1.13 lb/yr of reduction.

Compliance with the Water Quality requirements for the site can also be verified on the Water Quality Compliance tab:
Site Results (Water Quality Compliance)
Runoff Reduction Volume and TP By Drainage Area

<table>
<thead>
<tr>
<th>D.A. A</th>
</tr>
</thead>
<tbody>
<tr>
<td>RUNOFF REDUCTION VOLUME ACHIEVED (ft³)</td>
</tr>
<tr>
<td>TP LOAD AVAILABLE FOR REMOVAL (lb/yr)</td>
</tr>
<tr>
<td>TP LOAD REDUCTION ACHIEVED (lb/yr)</td>
</tr>
<tr>
<td>TP LOAD REMAINING (lb/yr)</td>
</tr>
</tbody>
</table>

NITROGEN LOAD REDUCTION ACHIEVED (lb/yr) | 8.59 |

Total Phosphorus

<p>| |</p>
<table>
<thead>
<tr>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>FINAL POST-DEVELOPMENT TP LOAD (lb/yr)</td>
</tr>
<tr>
<td>TP LOAD REDUCTION REQUIRED (lb/yr)</td>
</tr>
<tr>
<td>TP LOAD REDUCTION ACHIEVED (lb/yr)</td>
</tr>
<tr>
<td>TP LOAD REMAINING (lb/yr)</td>
</tr>
</tbody>
</table>

REMAINING TP LOAD REDUCTION REQUIRED (lb/yr): 0.35

Note that there is a remaining TP Load reduction required of 0.35 lb/yr, as shown in the tab in red font. Had the TP Load reduction been satisfied, the number would be green.

Step 5c - As the TP Load reduction was not satisfied with the first application of RR practices, further treatment is necessary. Assuming no additional RR practices can be used, look at adding a conventional BMP downstream of the Dry Swales for treatment in-series.

On Tab D.A. A, return to the Dry Swale #1 and go to the column labeled “Downstream Practice to be Employed”. Using the pull down menu, select a Filtering Practice #1 design for the conventional BMP.
Stormwater Best Management Practices (RR = Runoff Reduction) --Select from dropdown lists--

<table>
<thead>
<tr>
<th>Practice</th>
<th>Untreated Phosphorus Load to Practice (lb)</th>
<th>Phosphorus Removed By Practice (lb)</th>
<th>Remaining Phosphorus Load (lb)</th>
<th>...</th>
<th>Downstream Practice to be Employed</th>
</tr>
</thead>
<tbody>
<tr>
<td>5. Dry Swale (RR)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.a. Dry Swale #1 (Spec #10)</td>
<td>2.18</td>
<td>1.13</td>
<td>1.05</td>
<td>...</td>
<td>11.a. Filtering Practice #1</td>
</tr>
</tbody>
</table>

When this is done, note that the spreadsheet automatically populates information for the downstream BMP. Scroll down to the row labeled “11.a. Filtering Practice #1 (Spec #15)” and note that the remaining runoff from the upstream Dry Swales is now entering the proposed BMP:

Stormwater Best Management Practices (RR = Runoff Reduction)

<table>
<thead>
<tr>
<th>Practice</th>
<th>Volume from Upstream Practice (ft³)</th>
<th>Runoff Reduction (ft³)</th>
<th>Remaining Runoff Volume (ft³)</th>
<th>Total BMP Treatment Volume (ft³)</th>
<th>Phosphorus Load from Upstream Practices (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>11. Filtering Practices (no RR)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11.a. Filtering Practice #1 (Spec #12)</td>
<td>2,087</td>
<td>0</td>
<td>2,087</td>
<td>2,087</td>
<td>1.05</td>
</tr>
</tbody>
</table>

Note that the remaining volume of runoff and TP load from the Dry Swales is now in the calculation for the downstream BMP. Compare the value for “Volume from Upstream Practice” from the Filtering Practice #1 with the “Remaining Runoff Volume” for the Dry Swales. Also, compare the “Phosphorus Load from Upstream Practices” for the Filtering Practice #1 with the value reported as “Remaining Phosphorus Load” for the Dry Swales.

(Note that no Land Cover values were entered for the Filtering Practice #1, as it receives all of the remaining runoff from the upstream Dry Swales. If additional areas discharged to the Filtering Practice, but not through the Dry Swales, then the additional areas would be entered as Land Cover in acres for the Filtering Practice #1.)
Step 5d - Check water quality compliance for the overall project on the “Water Quality Compliance” tab:

**Site Results (Water Quality Compliance)**

Runoff Reduction Volume and TP By Drainage Area

<table>
<thead>
<tr>
<th>D.A. A</th>
<th>RUNOFF REDUCTION VOLUME ACHIEVED (ft³)</th>
<th>1,391</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TP LOAD AVAILABLE FOR REMOVAL (lb/yr)</td>
<td>2.18</td>
</tr>
<tr>
<td></td>
<td>TP LOAD REDUCTION ACHIEVED (lb/yr)</td>
<td>1.76</td>
</tr>
<tr>
<td></td>
<td>TP LOAD REMAINING (lb/yr)</td>
<td>0.42</td>
</tr>
</tbody>
</table>

Total Phosphorus

<table>
<thead>
<tr>
<th></th>
<th>FINAL POST-DEVELOPMENT TP LOAD (lb/yr)</th>
<th>2.18</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TP LOAD REDUCTION REQUIRED (lb/yr)</td>
<td>1.49</td>
</tr>
<tr>
<td></td>
<td>TP LOAD REDUCTION ACHIEVED (lb/yr)</td>
<td>1.76</td>
</tr>
<tr>
<td></td>
<td>TP LOAD REMAINING (lb/yr):</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>REMAINING TP LOAD REDUCTION REQUIRED (lb/yr):</td>
<td>0.00 **</td>
</tr>
</tbody>
</table>

**TARGET TP REDUCTION EXCEEDED BY 0.28 LB/YEAR **

Based upon the results above, has 100% Runoff Reduction been achieved for the project by the addition of a Filtering Device #1 in series with Dry Swales? Has the TP Load reduction required been satisfied to achieve compliance?

According to the Site tab, 3,478 ft³ of Runoff Reduction is required to reduce the runoff volume to pre-development condition, but only 1,391 ft³ is achieved in the Filtering Device and Dry Swales, so 100% Runoff Reduction is not met.

However, 1.76 lb/yr of TP reduction is provided for the proposed treatment train of Dry Swales in-series with a Filtering Practice #1. This exceeds the 1.49 lb/yr of TP reduction required for compliance by 0.28 lb/yr as shown with a green font for “TP Load Remaining” and a note in the tab.

As the TP load reduction exceeds the water quality criterion, the design should be optimized to reduce the excess, unless the additional 0.28 lb/yr of TP reduction can be used for compliance on another project in the same HUC, or for compliance with VDOT’s Chesapeake Bay TMDL Watershed Action Plan.
Step 6 - Instead of adding a Filtering Practice #1 in-series with the Dry Swales, consider the option to provide compliance offsite.

*Does the project qualify for the use of offsite options for compliance? If so, what % of the total TP Load Reduction Required can be met using offsite options?*

Recall that the sample project area is 1.7-acres (< 5-acres) and the TP Load Reduction Required is 1.49 lb/yr (< 10 lb/yr). For the sample project, 100% of the TP Load Reduction Required could be purchased according to the water quality offsite compliance criteria.

Another option would be to construct the Dry Swales as a RR practice and drainage system, but purchase the remaining credits, instead of putting a Filtering Practice #1 downstream of the Dry Swales. In this case, the Dry Swales alone provide a TP Load Reduction of 1.13 lb/yr, so the designer could choose to purchase the 0.35 lb/yr difference to bring the project into compliance with the water quality criterion.

Also, the designer must consider the need for water quantity control for the project. While the Filtering Device #1 provides no Runoff Reduction, the proposed Dry Swales reduce the runoff volume and provide water quantity control. The Runoff Reduction provided by the RR practice should be considered in a comparison of the design options, especially if a SWMF will be required for water quantity control.

In order to select the best option, the designer should consider the cost of the Dry Swales and Filtering Device #1 versus the cost of obtaining offsite compliance. The need for SWMFs for water quantity control should also be considered. The cost comparison should include both the capital costs (R/W, easements, and construction) as well as the long-term maintenance costs.

Step 7 - Using the information from the VRRM Spreadsheet and the selected water quality treatment design option (including the Treatment Volume required), design each BMP (RR practice or conventional) using the design standards in the VDOT BMP Design Manual of Practice, supplemented by the information provided in the Virginia BMP Clearinghouse.
11.5.3.2.4 **Water Quality – Sample Problem – Redevelopment**

Assume for the previous sample that the pre-development condition included 0.2 acres of impervious area and 0.3 acres of managed turf, with the balance undisturbed forest. The soils remain classified as HSG C by the USDA Soil Survey. In the post-developed condition, the site will include 0.8 acres of total impervious area (roadway and shoulder) and 0.9 acres of managed turf.

Step 1 - As the project site was previously developed with impervious area and managed turf, the project constitutes “development on prior developed lands”, also referred to as Redevelopment. Use the VRRM Spreadsheet for Redevelopment for this sample project.

Step 2 - On the Redevelopment Site tab, enter the project name and date of the calculations. As the project includes impervious roadway and shoulder, assume it is a linear development and mark the appropriate box “Yes”.

Complete the Land Cover table for both the pre- and post-development conditions:

<table>
<thead>
<tr>
<th>Pre-ReDevelopment Land Cover (acres)</th>
<th>A Soils</th>
<th>B Soils</th>
<th>C Soils</th>
<th>D Soils</th>
<th>Totals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forest/Open Space (acres) -- undisturbed, protected forest/open space or reforested land</td>
<td></td>
<td></td>
<td>1.20</td>
<td></td>
<td>1.20</td>
</tr>
<tr>
<td>Managed Turf (acres) -- disturbed, graded for yards or other turf to be mowed/managed</td>
<td></td>
<td></td>
<td>0.30</td>
<td></td>
<td>0.30</td>
</tr>
<tr>
<td>Impervious Cover (acres)</td>
<td></td>
<td></td>
<td>0.20</td>
<td></td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.70</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Post-Development Land Cover (acres)</th>
<th>A Soils</th>
<th>B Soils</th>
<th>C Soils</th>
<th>D Soils</th>
<th>Totals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forest/Open Space (acres) -- undisturbed, protected forest/open space or reforested land</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
</tr>
<tr>
<td>Managed Turf (acres) -- disturbed, graded for yards or other turf to be mowed/managed</td>
<td></td>
<td></td>
<td>0.90</td>
<td></td>
<td>0.90</td>
</tr>
<tr>
<td>Impervious Cover (acres)</td>
<td></td>
<td></td>
<td>0.80</td>
<td></td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.70</td>
</tr>
</tbody>
</table>

Area Check OK. OK. OK. OK. 1.70
Note that the pre-development condition land cover is required for the Redevelopment tab, as the TP load reduction requirement for development on prior developed lands where the net impervious cover area remains the same is a 20% reduction below existing conditions (for projects 1-acre or larger). Projects less than 1-acre with no net increase in impervious cover only require a 10% reduction below the existing pollutant load.

Also note that the spreadsheet compares the pre- and post-development impervious cover to determine what portion of a project is considered redevelopment (requiring a 20%/10% reduction from existing) versus additional net impervious area. When the post-development net impervious cover area increases above the pre-development area, the additional impervious area added must meet the new development criteria of 0.41 lb/acre/yr.

There is compliance criterion for linear development projects on prior developed lands (redevelopment). When the box in the Site tab is marked “Yes”, noting the project is a linear redevelopment, the spreadsheet automatically calculates the alternate criterion by applying a 20% reduction to the pre-development TP load. The spreadsheet applies the allowable load for linear development project as the compliance goal for the sample project, as the project is a linear development.

Step 3 - The spreadsheet automatically calculates the post-development pollutant reduction requirements for the Site:

<table>
<thead>
<tr>
<th>Post-Development Requirement for Site Area</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>TP Load Reduction Required (lb/yr)</strong></td>
</tr>
<tr>
<td>1.43</td>
</tr>
<tr>
<td><strong>Linear Project TP Load Reduction Required (lb/yr):</strong></td>
</tr>
<tr>
<td>1.49</td>
</tr>
</tbody>
</table>

A detailed look at the results on the Site tab includes a Land Cover Summary and pollutant loading computation results for the pre- and post-development conditions. Note that two columns are included in the Pre-Redevelopment Land Cover Summary: the first is for the overall pre-development site conditions (1.70 acres), while the second is for the pre-development conditions adjusted to remove the additional area converted to new impervious cover.
### LAND COVER SUMMARY -- PRE-REDEVELOPMENT

<table>
<thead>
<tr>
<th>Land Cover Summary-Pre</th>
<th>Listed</th>
<th>Adjusted¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forest/Open Space Cover (acres)</td>
<td>1.20</td>
<td>0.60</td>
</tr>
<tr>
<td>Weighted Rv(forest)</td>
<td>0.04</td>
<td>0.04</td>
</tr>
<tr>
<td>% Forest</td>
<td>71%</td>
<td>55%</td>
</tr>
<tr>
<td>Managed Turf Cover (acres)</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>Weighted Rv(turf)</td>
<td>0.22</td>
<td>0.22</td>
</tr>
<tr>
<td>% Managed Turf</td>
<td>18%</td>
<td>27%</td>
</tr>
<tr>
<td>Impervious Cover (acres)</td>
<td>0.20</td>
<td>0.20</td>
</tr>
<tr>
<td>Rv(impervious)</td>
<td>0.95</td>
<td>0.95</td>
</tr>
<tr>
<td>% Impervious</td>
<td>12%</td>
<td>18%</td>
</tr>
<tr>
<td>Total Site Area (acres)</td>
<td>1.70</td>
<td>1.10</td>
</tr>
<tr>
<td>Site Rv</td>
<td>0.18</td>
<td>0.25</td>
</tr>
</tbody>
</table>

¹Adjusted Land Cover Summary:
Pre-ReDevelopment land cover minus pervious land cover (forest/open space or managed turf) acreage proposed for new impervious cover.
Adjusted total acreage is consistent with Post-ReDevelopment acreage (minus acreage of new impervious cover).
Column I shows load reduction requirement for new impervious cover (based on new development load limit, 0.41 lbs/acre/year).

The redevelopment post-development (Post-ReDevelopment) summary is more complicated than the New Development spreadsheet results, because of the application of the redevelopment criteria to the pre-development load (including the pre-development impervious cover, but not the new impervious cover) and application of the new development criteria for the additional impervious cover. Also, the spreadsheet calculates the alternate criterion required for linear development.
<table>
<thead>
<tr>
<th>Land Cover Summary-Post (Final)</th>
<th>Land Cover Summary-Post</th>
<th>Land Cover Summary-Post</th>
</tr>
</thead>
<tbody>
<tr>
<td>Post ReDev. &amp; New Impervious</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Forest/Open Space Cover (acres)</td>
<td>0.00</td>
<td></td>
</tr>
<tr>
<td>Weighted Rv(forest)</td>
<td>0.00</td>
<td></td>
</tr>
<tr>
<td>% Forest</td>
<td>0%</td>
<td></td>
</tr>
<tr>
<td>Managed Turf Cover (acres)</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>Weighted Rv (turf)</td>
<td>0.22</td>
<td></td>
</tr>
<tr>
<td>% Managed Turf</td>
<td>53%</td>
<td></td>
</tr>
<tr>
<td>Impervious Cover (acres)</td>
<td>0.80</td>
<td></td>
</tr>
<tr>
<td>Rv(impervious)</td>
<td>0.95</td>
<td></td>
</tr>
<tr>
<td>% Impervious</td>
<td>47%</td>
<td></td>
</tr>
<tr>
<td>Final Site Area (acres)</td>
<td>1.70</td>
<td></td>
</tr>
<tr>
<td>Final Post Dev Site Rv</td>
<td>0.56</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Land Cover Summary-Post</th>
<th>Land Cover Summary-Post</th>
<th>Land Cover Summary-Post</th>
</tr>
</thead>
<tbody>
<tr>
<td>Post-Development</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Forest/Open Space Cover (acres)</td>
<td>0.00</td>
<td></td>
</tr>
<tr>
<td>Weighted Rv(forest)</td>
<td>0.00</td>
<td></td>
</tr>
<tr>
<td>% Forest</td>
<td>0%</td>
<td></td>
</tr>
<tr>
<td>Managed Turf Cover (acres)</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>Weighted Rv (turf)</td>
<td>0.22</td>
<td></td>
</tr>
<tr>
<td>% Managed Turf</td>
<td>82%</td>
<td></td>
</tr>
<tr>
<td>ReDev. Impervious Cover (acres)</td>
<td>0.20</td>
<td></td>
</tr>
<tr>
<td>Rv(impervious)</td>
<td>0.95</td>
<td></td>
</tr>
<tr>
<td>% Impervious</td>
<td>18%</td>
<td></td>
</tr>
<tr>
<td>Total ReDev. Site Area (acres)</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td>ReDev Site Rv</td>
<td>0.35</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Land Cover Summary-Post</th>
<th>Land Cover Summary-Post</th>
</tr>
</thead>
<tbody>
<tr>
<td>Post-Development New Impervious</td>
<td></td>
</tr>
<tr>
<td>New Impervious Cover (acres)</td>
<td>0.60</td>
</tr>
<tr>
<td>Rv(impervious)</td>
<td>0.95</td>
</tr>
</tbody>
</table>

Treatment Volume and Nutrient Load calculation results are reported for the pre- and post-redevelopment conditions as well:

<table>
<thead>
<tr>
<th>Treatment Volume and Nutrient Load</th>
<th>Pre-ReDevelopment Treatment Volume (acre-ft)</th>
<th>Pre-ReDevelopment Treatment Volume (cubic feet)</th>
<th>Pre-ReDevelopment TP Load (lb/yr)</th>
<th>Pre-ReDevelopment TP Load per acre (lb/acre/yr)</th>
<th>Baseline TP Load (lb/yr) (0.41 lbs/acre/yr applied to pre-redevelopment area excluding pervious land proposed for new impervious cover)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.0253</td>
<td>1,104</td>
<td>0.69</td>
<td>0.41</td>
<td>0.45</td>
</tr>
</tbody>
</table>

Chapter 11-46 of 148
## Treatment Volume and Nutrient Load

<table>
<thead>
<tr>
<th>Final Post-Development Treatment Volume (acre-ft)</th>
<th>0.0798</th>
<th>Post-ReDevelopment Treatment Volume (acre-ft)</th>
<th>0.0323</th>
<th>Post-Development Treatment Volume (acre-ft)</th>
<th>0.0475</th>
</tr>
</thead>
<tbody>
<tr>
<td>Final Post-Development Treatment Volume (cubic feet)</td>
<td>3,478</td>
<td>Post-ReDevelopment Treatment Volume (cubic feet)</td>
<td>1,408</td>
<td>Post-Development Treatment Volume (cubic feet)</td>
<td>2,069</td>
</tr>
<tr>
<td>Final Post-Development TP Load (lb/yr)</td>
<td>2.18</td>
<td>Post-ReDevelopment Load (TP) (lb/yr)*</td>
<td>0.88</td>
<td>Post-Development TP Load (lb/yr)</td>
<td>1.30</td>
</tr>
<tr>
<td>Final Post-Development TP Load per acre (lb/acre/yr)</td>
<td>1.29</td>
<td>Post-ReDevelopment TP Load per acre (lb/acre/yr)</td>
<td>0.80</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. Reduction Required (Below Pre-ReDevelopment Load)</td>
<td>20%</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The load reduction required for the standard redevelopment and the net increase in impervious area are calculated and reported separately:

| TP Load Reduction Required for Redeveloped Area (lb/yr) | 0.37 | TP Load Reduction Required for New Impervious Area (lb/yr) | 1.05 |

Note that the TP Load reduction required for the standard redevelopment criterion, including a redevelopment reduction of 20% from the pre-development load (minus the pervious cover area converted to impervious cover in post-development) and an allowable loading of 0.41 lb/acre/yr for the net increase in impervious area. Based upon the results of the spreadsheet, the standard redevelopment reduction required is (0.37 lb/yr + 1.05 lb/yr) =
1.42 lb/yr rounded to 1.43 lb/yr in the spreadsheet. Standard redevelopment on VDOT projects would not apply to linear roadways, but could apply to design for facilities and rest areas.

The alternate TP Load required for linear redevelopment projects is 20% less than the pre-development TP load. The pre-development TP load is reported as 0.69 lb/yr, so the allowable TP Load for linear development would be (1.0 – 0.2) x 0.69 lb/yr = 0.55 lb/yr. The TP Load required reduction from the post-development load of 2.18 lb/yr is (2.18 lb/yr – 0.55 lb/yr) = 1.63 lb/yr.

Lastly, the water quality criteria include a minimum allowable load based upon conditions of forest/open space, calculated using 0.41 lb/acre/yr as the allowable load. For the 1.70 acre sample project, the minimum allowable TP Load is (0.41 lb/acre/yr x 1.70 acres) = 0.70 lb/yr. The TP Load required reduction in this case is the post-development load minus the minimum allowable, or (2.18 lb/yr – 0.70 lb/yr) = 1.48 lb/yr rounded to 1.49 lb/yr in the spreadsheet.

A comparison of the TP Load reduction required by the three criteria shows that the least stringent reduction requirement would be the standard redevelopment reduction of 1.43 lb/yr, followed by the minimum allowable load reduction of 1.49 lb/yr, while the most stringent TP load reduction is based upon the linear development criterion at 1.63 lb/yr.

Based on the spreadsheet results reported above, which TP Load Reduction Requirement (lb/yr) must the sample project satisfy? What is the Treatment Volume required for 100% Runoff Reduction in ft³?

In the end, the spreadsheet requires the minimum allowable load criterion and reports a TP Load reduction requirement of 1.49 lb/yr as the “Linear Project TP Load Reduction Required” on the Site spreadsheet. The Water Quality Compliance tab confirms the value applied. Note that this is more stringent than the standard redevelopment load reduction of 1.43 lb/yr, but if the project is a linear development, then the standard redevelopment criterion does not apply.

The Treatment Volume required for the post-redevelopment condition, in order to provide for 100% Runoff Reduction to address the increase in runoff volume, is reported as 3,478 ft³.

Step 4 - Unlike the first example for New Development, assume that only the post-development impervious cover area of 0.8 acres is being treated before discharging to the outfall, with the balance of the project site (managed turf) not being treated.
Move to the second tab labeled “D.A. A” and enter the post-development Land Cover information for the project. For this example, the Land Cover information for Drainage Area A is the same as for the project site:

**Drainage Area A Land Cover (acres)**

<table>
<thead>
<tr>
<th></th>
<th>A Soils</th>
<th>B Soils</th>
<th>C Soils</th>
<th>D Soils</th>
<th>Totals</th>
<th>Land Cover Rv</th>
</tr>
</thead>
<tbody>
<tr>
<td>Forest/Open Space (acres)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Managed Turf (acres)</td>
<td></td>
<td></td>
<td>0.90</td>
<td></td>
<td>0.90</td>
<td>0.22</td>
</tr>
<tr>
<td>Impervious Cover (acres)</td>
<td></td>
<td>0.80</td>
<td></td>
<td></td>
<td>0.80</td>
<td>0.95</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>1.70</strong></td>
<td></td>
</tr>
</tbody>
</table>

Step 5a - The first step in providing onsite treatment is to consider the use of Runoff Reduction practices. For the redevelopment example, assume only the post-development impervious cover is being treated to achieve water quality compliance for the site. Also, assume the soils have low permeability and are unsuitable for infiltration (based upon a site specific soil investigation), but the depth to seasonal high groundwater and bedrock is > 6’.

Based upon this information and the information provided in the VDOT BMP Design Manual of Practice and Virginia BMP Clearinghouse, start by selecting a Bioretention #2 as our RR practice.

Unlike the New Development example where the entire site was treated by BMPs, only the post-development impervious cover is being treated in this example. Enter the post-development impervious cover area into the spreadsheet under the column “Impervious Cover Credit Area” for practice “6.b. Bioretention #2 or Micro-Bioretention #2 (Spec #9)”:  

**Stormwater Best Management Practices (RR = Runoff Reduction)**

<table>
<thead>
<tr>
<th>Practice</th>
<th>Runoff Reduction Credit (%)</th>
<th>Managed Turf Credit Area (acres)</th>
<th>Impervious Cover Credit Area (acres)</th>
<th>Volume From Upstream Practice (ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6. Bioretention (RR)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.b. Bioretention #2 or Micro-Bioretention #2 (Spec #9)</td>
<td>80</td>
<td>0.80</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>
Once the information for the drainage to the BMP is entered, the spreadsheet will calculate the Runoff Reduction volume provided by the practice, the remaining volume of runoff not removed via the Runoff Reduction practice, the total Treatment Volume for the practice, the untreated TP load to the practice, the TP removed by the practice, and any remaining TP load after runoff reduction and treatment:

**Stormwater Best Management Practices (RR = Runoff Reduction)**

<table>
<thead>
<tr>
<th>Practice</th>
<th>Runoff Reduction (ft³)</th>
<th>Remaining Runoff Volume (ft³)</th>
<th>Total BMP Treatment Volume(ft³)</th>
<th>Untreated Phosphorus Load to Practice (lb)</th>
<th>Phosphorus Removed By Practice (lb)</th>
<th>Remaining Phosphorus Load (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6. Bioretention (RR)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.b. Bioretention #2 or Micro-Bioretention #2 (Spec #9)</td>
<td>2,207</td>
<td>552</td>
<td>2,759</td>
<td>1.73</td>
<td>1.56</td>
<td>0.17</td>
</tr>
</tbody>
</table>

**Step 5b** - *Based upon the results above, has 100% Runoff Reduction been achieved by the application of RR practice to the project? Has the TP Load reduction required been satisfied to achieve compliance?*

According to the Site tab, 3,478 ft³ of Runoff Reduction is required to reduce the runoff volume to pre-development condition, but only 2,207 ft³ is achieved in the Bioretention #2, so 100% Runoff Reduction is not met.

Also, 1.49 lb/yr of TP reduction is required for site compliance, and the Bioretention #2 practice provides 1.56 lb/yr of reduction, so the water quality criteria should be satisfied.

Compliance with the water quality requirements for the site can also be verified on the Water Quality Compliance tab:
Site Results (Water Quality Compliance)

Runoff Reduction Volume and TP By Drainage Area

<table>
<thead>
<tr>
<th>D.A. A</th>
</tr>
</thead>
<tbody>
<tr>
<td>RUNOFF REDUCTION VOLUME ACHIEVED (ft³)</td>
</tr>
<tr>
<td>TP LOAD AVAILABLE FOR REMOVAL (lb/yr)</td>
</tr>
<tr>
<td>TP LOAD REDUCTION ACHIEVED (lb/yr)</td>
</tr>
<tr>
<td>TP LOAD REMAINING (lb/yr)</td>
</tr>
</tbody>
</table>

Total Phosphorus

<table>
<thead>
<tr>
<th>LINEAR PROJECT:</th>
</tr>
</thead>
<tbody>
<tr>
<td>FINAL POST-DEVELOPMENT TP LOAD (lb/yr)</td>
</tr>
<tr>
<td>TP LOAD REDUCTION REQUIRED (lb/yr)</td>
</tr>
<tr>
<td>TP LOAD REDUCTION ACHIEVED (lb/yr)</td>
</tr>
<tr>
<td>TP LOAD REMAINING (lb/yr):</td>
</tr>
<tr>
<td>REMAINING TP LOAD REDUCTION REQUIRED (lb/yr):</td>
</tr>
</tbody>
</table>

** TARGET TP REDUCTION EXCEEDED BY 0.07 LB/YEAR **

Note that there is excess TP Load reduction of 0.07 lb/yr, as shown in the tab in green font and noted by the text. Had the TP Load reduction not been satisfied, the number would be red.

Step 5c - The TP Load reduction required for the entire project was satisfied with the first application of RR practices to just the post-development impervious cover; therefore, further treatment is not necessary.

As the TP load reduction exceeds the water quality requirement, the design could be optimized to reduce the excess. The designer should check with the District Hydraulics Engineer to determine if the excess reduction should remain for use as offsite credit or for TMDL compliance.

According to the Site tab, 3,478 ft³ of RR retention storage is required to reduce the runoff volume to pre-development conditions. Only 2,207 ft³ is achieved in the RR practice, so 100% Runoff Reduction is not met, but the Bioretention #2 provides 63% of the RR retention storage, which will decrease the post-development runoff considerably and help achieve significant water quantity control.
Step 6 - Instead of using the RR practice to achieve water quality compliance onsite, consider the option to provide compliance offsite.

*Does the project qualify for the use of offsite nutrient credits for compliance? If so, what % of the total TP Load Reduction Required can be met using offsite options?*

Based upon the offsite criterion, a project with a site area less than 5-acres or a TP Load Reduction Requirement less than 10 lb/yr can use offsite options for water quality compliance, up to 100% of the TP Load Reduction Required. Recall that the sample project area is 1.7-acres (< 5-acres) and the TP Load Reduction Required is 1.49 lb/yr (< 10 lb/yr). Therefore, 100% of the TP Load Reduction Required could be purchased.

In order to select the best option (onsite versus offsite compliance), the designer should consider the cost of the Bioretention #2 versus the cost of obtaining offsite compliance through a value engineering analysis. The need for SWMFs for water quantity control should also be considered, since the Bioretention #2 provides a significant decrease in post-development runoff to meet water quantity goals for channel and flood protection. The value engineering cost comparison should include both the capital costs (R/W, easements, and construction) as well as the long-term maintenance costs over the operating life of the project and BMP.

The designer should consider the need for water quantity control for the project. The Runoff Reduction provided by the RR practice should be considered in a comparison of the design options, especially if a SWMF will be required for water quantity control.

Step 7 - Using the information from the VRRM Spreadsheet and the selected water quality treatment option, including the Treatment Volume required, design the BMP using the design standards in the VDOT BMP Design Manual of Practice, supplemented by the design standards in the Virginia BMP Clearinghouse.

11.5.3.3 Water Quantity – Channel Protection Procedure

*Step 1 - Identify project outfalls and receiving stormwater conveyance systems.*

*Determine project site (the project site, for the purposes of determining water quality and quantity compliance includes only the regulated land-disturbing activity) drainage area for each project outfall and the total drainage area to each stormwater conveyance system (system).*
Step 2 - Determine the limits of analysis in the system based upon the drainage area at each outfall and the total drainage area to the system:

\[ DA_{\text{outfall}} \leq \frac{DA_{\text{system}}}{100} \]

Where:
- \( DA_{\text{outfall}} \) = site drainage area at the outfall (acres or square miles)
- \( DA_{\text{system}} \) = total drainage area for the system at the limits of analysis (use units consistent with \( DA_{\text{outfall}} \))

Step 3 - Determine the type of system below each outfall to the limits of analysis in order to identify the applicable channel protection criteria.

Step 4 - Determine the pre- and post-development peak rate and volume of runoff for each outfall and the system (see Chapter 4 Hydrology)

Step 5 - Using runoff information developed in Step 5, determine the limits of analysis at the point in the system where the total pre-development peak rate of runoff is 100 times greater than the outfall peak rate of runoff for each outfall during the 1-yr 24-hour storm:

\[ Q_{\text{outfall}} \leq \frac{Q_{\text{system}}}{100} \]

Where:
- \( Q_{\text{outfall}} \) = post-development peak rate of runoff from the site at the outfall in the absence of SWM/BMP for the 1-yr 24-hour storm (cubic feet or second or cfs)
- \( Q_{\text{system}} \) = pre-development peak rate of runoff for the system at the limits of analysis (use units consistent with \( Q_{\text{outfall}} \))

Use the limits of analysis for the point closest to the outfall based upon the two methods in Steps 3 and 5.

Step 6 - Determine the adequacy of the stormwater conveyance system from the outfall or point of discharge up to (but not including) the limits of analysis using hydraulic calculation methods discussed in Chapters 6 to 9 of the Drainage Manual.

Include CN adjustment (see Section 11.5.2.1.8) where runoff reduction practices with retention storage are provided to adjust the allowable peak rate of runoff.

If the system is **manmade**, go to Step 6a.

If the system is **restored**, go to Step 6b.

If the system is **natural**, go to Step 6c.
Step 6a - For a **manmade** stormwater conveyance system (system), assess the system’s ability to resist erosion for the post-development 2-yr 24-hour storm.

*If the system is unable to resist erosion in the post-development condition, use the methods outlined in Chapter 11, Sections 11.5.6 to 11.5.9 to design and incorporate SWMFs/BMPs into the plan to meet the channel protection criterion.*

Alternatively, consider the option to make downstream system improvements to meet the criterion; a combination of SWM and system improvements; or apply the Energy Balance method at the outfall. (Go to Step 7c)

Step 6b - For **restored** systems designed using natural channel design, evaluate the hydrologic and hydraulic design parameters used to restore the system to determine if the restored channel will function as designed in the post-development runoff conditions.

*If the system is unable to function as originally designed in the post-development condition, use the methods outlined in Chapter 11, Sections 11.5.6 to 11.5.9 to design and incorporate SWMFs/BMPs into the plan to meet the criterion for the system.*

Alternatively, consider the option to make downstream system improvements to meet the criterion; a combination of SWM and system improvements; or apply the Energy Balance method at the outfall. (Go to Step 7c)

Step 6c - For **natural** systems (or when selected as the design option under Steps 7a or 7b), apply the Energy Balance (see Section 11.5.2.1.4) using the pre- and post-development peak rate and volume of runoff for the contributing site area to each outfall. Where the post-development runoff from the site increases or the improvement factor (I.F.) applies, provide SWMFs or BMPs to reduce runoff and detain to allowable peak rates.

Check to confirm that the allowable developed peak discharge resulting from the Energy Balance is not less than that required if the project site was forested. If it is less, than the allowable peak discharge should be the same as the project site in the forested condition.

Step 7 - Confirm that the **limits of analysis** identified and used for applying the channel protection criterion is the closest to the outfall based upon the two methods used in Steps 2 and 5.
11.5.3.1 **Channel Protection - Limits of Analysis Sample Problems**

Step 1 - Given an outfall with a drainage area of 0.5 acres, what would the minimum total drainage area in the downstream system be at the limit of analysis for the channel protection criterion?

Step 2 - Determine the limits of analysis in the system based upon the post-development drainage area at the outfall and the pre-development total drainage area in the system:

\[ DA_{\text{outfall}} \leq \frac{DA_{\text{system}}}{100} \]

\[ DA_{\text{system}} \geq DA_{\text{outfall}} \times 100 \]

\[ DA_{\text{system}} \geq 0.5 \text{ acres} \times 100 \geq 50 \text{ acres} \]

The adequacy analysis can end just before the point in the manmade system where the total pre-development system drainage area is 50 acres.

Step 3 - Assume the system below the outfall to the limits of analysis based upon drainage area comparison is a manmade system.

Step 4 - By applying standard hydrologic methods, it is determined that the post-development peak discharge from the site at the project outfall is 0.8 cfs, while the pre-development peak discharge in the system at the limits of analysis based upon the drainage areas is 105 cfs.

Step 5 - Could the limits of analysis change based upon the peak discharges?

Determine the total pre-development peak rate of runoff at the point in the system is 100 times the outfall post-development peak rate of runoff:

\[ Q_{\text{outfall}} \leq \frac{Q_{\text{system}}}{100} \]

\[ Q_{\text{system}} \geq Q_{\text{outfall}} \times 100 \]

\[ Q_{\text{system}} \geq 0.8 \text{ cfs} \times 100 = 80 \text{ cfs} \]

Compare the peak rate of runoff at the limits of analysis identified in Step 2 based upon drainage area comparison (105 cfs) against the calculated peak rate of runoff that is 100 times greater than the outfall peak rate (80 cfs)

\[ 105 \text{ cfs} \geq 80 \text{ cfs} \]

As 105 cfs is greater than 80 cfs, the limits of analysis could be moved upstream from the point where the system drainage area is 100 times the project outfall drainage area to the point where the peak rate of runoff in pre-development conditions is 80 cfs, which is 100 times the project outfall peak rate of runoff of 0.8 cfs.
11.5.3.3.2  Channel Protection - Adjusted CN Sample Problem

Step 6 - A 0.5 acre site drains to a project outfall both in the pre- and post-development conditions. The site area has a post-development CN of 88. In order to meet water quality criteria, the RRM was applied and a BMP selected that provides a runoff reduction volume of 478 cubic feet. The 1-yr 24-hour storm precipitation depth is 2.75 inches and the runoff for the drainage area before applying runoff reduction practices is 1.60 inches. What is the adjusted CN for the 1-yr 24-hour storm after application of the runoff reduction BMP?

\[
Q - R = \frac{(P - 0.2 \times S_{adj})^2}{(P + 0.8 \times S_{adj})}
\]

Where:

\[
Q = 1.60 \text{ inches}
\]

\[
R = \frac{478 \text{ cubic feet}}{(0.5 \text{ acre} \times 43560 \text{ square feet/acre}) \times 12 \text{ inches/foot}} = 0.26 \text{ inches}
\]

\[
Q - R = 1.60 - 0.26 = 1.34 \text{ inches}
\]

\[
P = 2.75 \text{ inches}
\]

\[
1.34 = \frac{(2.75 - 0.2 \times S_{adj})^2}{(2.75 + 0.8 \times S_{adj})}
\]

Compute \(S_{adj}\) using by solving the quadratic equation: \(S_{adj} = 1.85 \text{ inches}\)

Solve for \(CN_{adj}\):

\[
CN_{adj} = \frac{1000}{S_{adj} + 10} = \frac{1000}{1.85 + 10} = \frac{1000}{11.85} = 84
\]

Alternatively, the adjusted CN for the 1-yr 24-hour storm can be obtained from the VRRM Spreadsheet.

11.5.3.3.3  Channel Protection - Manmade System Sample Problem

Step 6a - The process for conducting an adequacy analysis of a manmade stormwater conveyance system is discussed in detail in Chapters 7 (Ditches and Channels), 8 (Culverts), and 9 (Storm Drains). The analysis must demonstrate that the manmade system will resist the erosive forces in a 2-yr 24-hour storm for the post-development condition, including the total runoff to the system from offsite.

If the system is inadequate at any location from the outfall down to the limits of analysis, use detention design methods to provide attenuation in a SWMF such that the system is adequate; design system improvements using the methods in Chapters 7, 8, and 9 to make the system adequate; or apply the Energy Balance method to comply with the Channel Protection criterion.
11.5.3.3.4 Channel Protection - Restored System Sample Problem

Step 6b - Assume the receiving system at the limits of analysis is a restored channel designed by the application of natural channel design techniques. The process for conducting an adequacy analysis of a restored stormwater conveyance system designed and constructed using natural channel design techniques is more complicated than for a manmade system. Some of the methods discussed in detail in Chapters 6 for open systems may apply, but more importantly the post-development condition must be reviewed to determine if the original restoration design is adequate for the post-development hydrologic and hydraulic conditions. The analysis must demonstrate that the restored system was designed and constructed to resist the flow conditions used for the natural channel design event(s) used in the system restoration, including the total runoff to the system.

If the restored system is inadequate at any location from the outfall down to the limits of analysis, use the detention design methods to provide attenuation in a SWMF such that the restored system is adequate for the post-developed condition; design system improvements using natural channel design methods to address the post-development runoff conditions; or apply the Energy Balance method to comply with the Channel Protection criterion.

11.5.3.3.5 Channel Protection - Natural System Sample Problem

Step 6c - Assume a VDOT project outfall with a site area of 1.7 acres (pre- and post-development conditions) discharging to a natural system. The USDA classifies the soils in the drainage area as HSG C, with pre-development land cover of 1.7 acres of open space in grass in good condition (CN = 74), and post-development land cover of 0.9 acres grass in good condition and 0.8 acres of impervious roadway (weighted CN = 85). The pre-development time of concentration is 20 minutes and the post-development time of concentration is 10 minutes.

Pre- and post-development hydrology has been developed using the TR-55 methodology presented in Chapter 4 and a VRRM Spreadsheet prepared for water quality computations. The 1-yr 24-hour precipitation for the project (obtained from Atlas 14) is 2.73 inches. The pre-development peak flow rate of runoff and runoff volume for the 1-yr 24-hour storm are 1.28 cfs and 0.603 in (0.0854 ac-ft), respectively.

For the post-development condition, the peak flow rate of runoff for the 1-year 24-hour storm is 3.22 cfs and the runoff volume is 1.233 in (0.1747 ac-ft), before the application of any runoff reduction practices.

What is the allowable discharge rate of runoff from the site at the outfall?
As the receiving system is natural below the outfall to the limits of analysis, the channel protection criteria require the application of the Energy Balance equation:

\[ Q_{\text{Allowable}} = Q_{\text{Developed}} = I.F. \times \left( \frac{Q_{\text{Pre-Developed}} \times RV_{\text{Pre-Developed}}}{RV_{\text{Developed}}} \right) \]

Where:
- I.F. = 0.80 (site > 1-acre)
- \( Q_{\text{Pre-Developed}} = 1.28 \text{ cfs} \)
- \( RV_{\text{Pre-Developed}} = 0.603 \text{ in (0.0854 ac-ft)} \)
- \( RV_{\text{Developed}} = 1.233 \text{ in (0.1747 ac-ft)} \)

\[ Q_{\text{Allowable}} = Q_{\text{Developed}} = 0.80 \times \left( \frac{1.28 \times 0.114}{1.233} \right) = 0.10 \text{ cfs} \]

To meet the Energy Balance, the allowable Q must be reduced from 3.22 cfs to 0.10 cfs, which is a 3.12 cfs or 97% reduction in post-development peak flow for the 1-yr 24-hour storm.

Check the peak discharge from the 1.7 acre site assuming the land cover is forest (woods in good condition), using TR-55 Methodology. For woods in good condition (CN = 70 for HSG C), the peak rate of runoff for the 1-yr 24-hour is computed to be 1.21 cfs, which is higher than the computed allowable peak rate using the Energy Balance (0.10 cfs). Therefore, the allowable peak rate of runoff for the sample problem is the value for woods in good condition or 1.21 cfs. This requires a reduction of 2.01 cfs or 62% from the post-development peak rate of runoff.

In situations where the point of discharge includes drainage from the site (regulated land-disturbing activity) and additional undisturbed area from outside the project site, the improvement factor applies only to the regulated land-disturbing activity. In these instances it is necessary to separate the total drainage to the outfall into that coming from the project site and that coming from undisturbed off-site areas. Designers should determine the total peak discharge at the point of discharge (pre and post-development) including the project site and any undisturbed areas that may drain to the point of discharge. BMPs or SWMFs should be applied such that the post-development peak flow rate at the point of discharge is equal to or less than the total pre-development discharge from undisturbed areas plus the allowable Q from the project site.

If RRM practices are provided for the 1.7 acres drainage area to address water quality in the form of a dry swale #2 design providing 2,087 ft³ of runoff reduction, what would the allowable peak discharge at the outfall be for the developed condition?
First, determine the adjusted CN for the 1-yr 24-hour storm at the project outfall:

\[ Q - R = \frac{(P - 0.2 \times S_{adj})^2}{(P + 0.8 \times S_{adj})} \]

Where:
\[ Q = 1.233 \text{ inches} \]
\[ R = 2,087 \text{ cubic feet/ (1.7 acres x 43560 square feet/acre) x 12 inches/foot} = 0.338 \text{ inches (from the RRM Spreadsheet)} \]
\[ Q - R = 1.233 - 0.338 = 0.895 \text{ inches} \]
\[ P = 2.73 \text{ inches} \]

\[ 0.895 = \frac{(2.73 - 0.2 \times S_{adj})^2}{(2.73 + 0.8 \times S_{adj})} \]

Solving the quadratic equation for \( S_{adj} = 2.97 \).

Solve for \( CN_{adj} \):

\[ CN_{adj} = \frac{1000}{S_{adj} + 10} = \frac{1000}{2.97 + 10} = \frac{1000}{12.97} = 77 \]

The RRM Spreadsheet “Runoff Volume and CN” tab for the project calculates the adjusted CN for the 1-yr 24-hour storm as 79, based upon the RV numbers computed in the spreadsheet, as opposed to the Q values (watershed inches) calculated via TR-55.

Using \( CN_{adj} = 77 \) in the TR-55 hydrologic calculations, determine the adjusted peak runoff (\( Q_{Developed} \)) and runoff volume (\( RV_{Developed} \)) for the post-development condition after the application of the RRM practices: 2.05 cfs and 0.756 inches, respectively.

With the CN adjusted post-development peak rate of runoff, the required reduction would be \( (2.05 \text{ cfs} - 1.21 \text{ cfs}) = 0.84 \text{ cfs} \) or \( (0.84 \text{ cfs}/3.22 \text{ cfs} \times 100\%) = 26\% \) of the unadjusted post-development peak rate of runoff.

By applying dry swales in the project drainage area to address water quality via the RRM, the designer also reduced the runoff volume through curve number (CN) adjustment. The CN adjustment decreased the reduction in peak rate of runoff from the developed site from 2.01 cfs (62% reduction) to 0.84 cfs (26%). This will reduce the storage volume required for detention to meet the natural channel protection criteria.

**11.5.3.4 Water Quantity – Flood Protection Procedure**

**Step 1** - Identify project outfalls, receiving stormwater conveyance systems, and mapped floodplain(s) or flood prone area(s) identified in a study (such as a
FEMA floodplain identified in a FIRM or a FIS, or a local floodplain map supported by a study

Step 2 - If the stormwater conveyance system (system) immediately below an outfall is mapped as a floodplain or flood prone area, then the flood protection criteria is satisfied and no further analysis is required for that outfall.

Otherwise, proceed to Step 3.

Step 3 - Determine project site drainage area for each project outfall.

Step 4 - Determine the limits of analysis in the system based upon the drainage area at each outfall and the total drainage area to the system:

\[ DA_{\text{outfall}} \leq \frac{DA_{\text{system}}}{100} \]

Where:

- \( DA_{\text{outfall}} \) = project drainage area at the outfall (acres or square miles)
- \( DA_{\text{system}} \) = total drainage area for the system at the limits of analysis (use units consistent with \( DA_{\text{outfall}} \))

If a floodplain or flood prone area is mapped in the system upstream of the limits of analysis determined in Step 4, then the flood protection analysis can stop at the mapped floodplain or flood prone area. Go to Step 8 to determine the adequacy of the system to the limits of analysis.

Otherwise, continue to Step 5.

Step 5 - Determine the pre- and post-development peak rate and volume of runoff for each outfall and the system for the 10-yr 24-hour storm (see Chapter 4 Hydrology)

Step 6 - Using runoff information developed in Step 5, determine the limits of analysis at the point in the system where the total pre-development peak rate of runoff is 100 times greater than the outfall peak rate of runoff for each outfall during the 10–yr 24-hour storm:

\[ Q_{\text{outfall}} \leq \frac{Q_{\text{system}}}{100} \]

Where:

- \( Q_{\text{outfall}} \) = post-development peak rate of runoff at the outfall in the absence of SWM/BMP for the 10-yr 24-hour storm (cubic feet or second or cfs)
- \( Q_{\text{system}} \) = pre-development peak rate of runoff for the system at the limits of analysis (use units consistent with \( Q_{\text{outfall}} \))

If a floodplain or flood prone area is mapped in the system upstream of the limits of analysis determined in Step 6, then the flood protection analysis can
stop at the mapped floodplain or flood prone area. Go to Step 8 to determine the adequacy of the system to the limits of analysis.

Otherwise, continue to Step 7.

Step 7 - If a floodplain or flood prone area is mapped in the system upstream of the limits of analysis determined in Steps 4 and 6, then the flood protection analysis can stop at the mapped floodplain or flood prone area. Go to Step 8 to determine the adequacy of the system to the limits of analysis.

Use the limits of analysis for the point closest to the outfall based upon the two methods in Steps 4 and 6.

Step 8 - Is there documentation available to demonstrate the performance of the system below the outfall in the pre-development conditions during the 10-yr 24-hour the system is currently flooding or not flooding?

If so, go to Step 8a.

If not, go to Step 8b.

Step 8a - If the available documentation demonstrates that the system below an outfall does not currently experience localized flooding during a 10-yr 24-hour storm event, then go to Step 9.

If there is documentation that the system below an outfall currently experiences localized flooding during the 10-yr 24-hour event, go to Step 10.

Step 8b - If there is no documentation that a system below an outfall does or does not currently experiencing flooding during the 10-yr 24-hour storm, hydraulic modeling of the system in the pre-development condition should be completed to determine if flooding is reasonably expected. See Chapters 6, 7, 8, and 9 for details on hydraulic modeling of systems.

If modeling of the pre-development condition demonstrates that no flooding is reasonably expected in the system down to the limits of analysis, proceed to Step 9. If the modeling documents potential flooding in the system to the limits of analysis below the outfall under the pre-development condition, then proceed to Step 10.

Note that this step may be necessary just to define the stormwater conveyance system for flood protection, which includes both the main channel and adjacent flood prone areas.

Step 9 - For a system that does not currently experiencing localized flooding:

Using the hydrologic modeling results from Step 5, prepare a hydraulic analysis for system using the post-development 10-yr 24-hour storm to
determine if the post-development peak flow rate is confined within the system to the limits of analysis.

If the post-development peak flow rate is contained within the system to the limits of analysis, then the system is adequate below the outfall and no further SWM is required for flood protection.

If the post-development peak flow rate is not contained within the system to the limits of analysis, provide stormwater detention or system improvements to make the system adequate. Go to Step 11.

Step 10 - For a system that currently experiences localized flooding:

Using the hydrologic modeling results from Step 5, prepare a hydraulic analysis of the system using the post-development 10-yr 24-hour storm to determine if the post-development peak flow rate is confined within the system to the limits of analysis.

If the post-development peak flow rate is contained within the system to the limits of analysis, then the system is adequate below the outfall and no further SWM is required for flood protection.

If the post-development peak flow rate is not contained within the system to the limits of analysis:

• Provide stormwater detention or system improvements to make the system adequate for the post-development condition, or

• Provide a design that releases the post-development peak flow rate to less than the pre-development peak flow rate for the 10-yr 24-hour storm.

Go to Step 11.

Step 11 - See the methods in Chapter 11, Sections 11.5.6 to 11.5.9 for design of detention facilities to make the system adequate for the post-development condition, or to provide a post-development peak runoff rate that is less than pre-development when the system below the outfall is currently experiencing flooding.

See the methods in Chapters 6, 7, 8, and 9 for making the system below the outfall adequate for the post-development peak rate of runoff.

11.5.3.4.1 Flood Protection - Limits of Analysis

Determining the limits of analysis is critical in demonstrating compliance for Flood Protection. The limits of analysis for flood protection must extend downstream to (but not including) the point in the system below the outfall where the total system drainage area is 100 times greater than the project outfall drainage area; to the point where the total system peak rate of runoff is 100 times greater than the project outfall peak rate of
runoff, both based upon the 10-yr 24-hour storm; or to the point in the system where a mapped floodplain exists.

Note that the system must be analyzed from the outfall to the limits of analysis, not just below the outfall or just at the limits of analysis. This may require extensive hydrologic and hydraulic modeling to demonstrate that the system is adequate.

Before just choosing to provide detention for a project outfall, the designer should assess the adequacy of the existing system to determine the extent of the system (main channel and adjacent flood prone area), whether flooding exists pre-development, and what peak flows the system can adequately convey without causing flooding of properties and waterways. The adequacy analysis could result in higher allowable peak runoff for the post-development condition, especially if there is existing flooding occurring.

Note that systems that are not currently experiencing flooding require detention to the peak rate that is adequate, not just detention to the pre-development peak rate of runoff from the project at that outfall. Depending upon the system, this could mean a peak runoff less than or greater than the pre-development condition. A system adequacy determination to the limits of analysis will allow the designer to optimize the SWM plan for flood protection.

11.5.3.4.2 Flood Protection - Limits of Analysis Sample Problems

Step 4 - Given an outfall from a project with a contributing site area of 10.3 acres, what would the minimum total drainage area in the downstream system be at the limit of analysis for the flood protection criterion?

\[
\text{DA}_{\text{outfall}} \leq \frac{\text{DA}_{\text{system}}}{100}
\]

\[
\text{DA}_{\text{system}} \geq \text{DA}_{\text{outfall}} \times 100
\]

\[
\text{DA}_{\text{system}} \geq 10.3 \text{ acres} \times 100 \geq 1,030 \text{ acres}
\]

The adequacy analysis can end at the point in the manmade system where the total system drainage area is 1,030 acres.

Step 6 - The post-development peak discharge for the 10-yr 24-hour storm from the site is computed to be 187 cfs, while the pre-development peak discharge in the system at the limits of analysis based upon the drainage area comparison above is 15,895 cfs. Could the limits of analysis change based upon the peak discharges?

\[
\text{Q}_{\text{outfall}} \leq \frac{\text{Q}_{\text{system}}}{100}
\]

\[
\text{Q}_{\text{system}} \geq \text{Q}_{\text{outfall}} \times 100
\]
As $Q_{\text{system}}$ (15,895 cfs) is less than $Q_{\text{outfall}} \times 100$ (18,700 cfs), the limits of analysis cannot be moved upstream based upon flow comparison.

Step 7 - If the system below the outfall is mapped in a Flood Insurance Rate Map (FIRM) adopted by FEMA as a Zone AE Special Flood Hazard Area (SFHA). The floodplain mapping begins at a point in the system below the outfall where the total drainage area is 1 square mile. *Can the limits of analysis for the flood protection criterion be moved?*

The system drainage area below the outfall at the limits of analysis based upon comparison of drainage areas is 1,030 acres. The floodplain mapping generally begins in the system at a point with a total drainage area of 1 square mile, or 640 acres. As the system drainage area where floodplain mapping begins (640 acres) is smaller than the system drainage area for $D_{\text{outfall}} \times 100$, the flood prone area is located upstream. Therefore, the limits of analysis for flood protection can be moved upstream to the point where flooding is mapped. This can be confirmed by using the FIRM to determine if the floodplain mapping begins before the point in the system where the drainage area is equal to $D_{\text{outfall}} \times 100$.

**11.5.3.4.3 Flood Protection Sample Problem – Localized Flooding Not Currently Experienced**

Step 1 - A project with a single outfall discharges to a natural stormwater conveyance system. No mapped floodplain or flood prone areas (as identified in studies) are found immediately below the outfall.

Step 2 - As the system at the outfall is not mapped as a floodplain or flood prone area, a system adequacy analysis is required to satisfy the flood protection criteria.

Step 3 - The post-development drainage area at the project outfall is 2.3 acres.

Step 4 - Based upon drainage area comparisons, the limits of analysis are located in the receiving system where the total drainage area is 230 acres.

Step 5 - The post-development 10-yr 24-hour peak runoff rate at the project outfall is 12.6 cfs. At the limits of analysis identified by drainage area comparison, the pre-development 10-yr 24-hour peak rate of runoff is 990 cfs.

Step 6 - The pre-development peak rate of runoff for the 10-yr 24-hour storm at the limits of analysis based upon drainage areas (990 cfs) is less than 100 times the post-development project outfall peak rate of runoff for the 10-yr 24-hour storm (100 x 12.6 cfs = 1,260 cfs). This means that the limits of analysis based upon a comparison of the peak rate of runoff is located downstream of the limits based upon the drainage area comparison.
Step 7 - As no floodplain or flood prone areas are mapped downstream of the project outfall, and the limits of analysis based upon the drainage area comparison is closer to the project outfall than the limits based upon comparison of the peak rates of runoff, the adequacy analysis can end at the limits of analysis based upon the drainage area comparison.

Step 8 - Historic flood data and anecdotal information for the area downstream of the project provide no evidence of flooding currently experienced in the 10-yr 24-hour storm.

Step 8a - As there exists documentation demonstrating that the system below an outfall does not currently experience localized flooding during a 10-yr 24-hour storm event, the flood adequacy analysis can move to Step 9.

Step 9 - Given the hydrologic information developed in Step 5 for the 10-yr 24-hour storm event, a hydraulic analysis of the receiving system capacity from the project outfall to the limits of analysis is conducted using good hydrologic engineering methods and practices presented in Chapters 6 and 12 of the Drainage Manual.

For the sample problem, the receiving system (main channel and adjacent flood prone areas) is well defined, so the pre-development peak rate of runoff does not need to be modeled hydraulically to determine the pre-development flood prone areas. However, the system is modeled to determine that the capacity at the limits of analysis is 1,050 cfs. Is the system adequate for the post-development flows in the 10-yr 24-hour storm?

In Step 6 it was determined that the post-development peak rate of runoff for the system at the limits of analysis is 1,260 cfs. As 1,260 cfs is greater than the system capacity of 1,050 cfs, the system does not confine the post-development peak rate of runoff and the flood protection criteria is not met. To confirm this conclusion, the system can be modeled hydraulically with the post-development peak rate of runoff to determine if the water surface elevations increase and flooding goes outside of the flood prone areas adjacent to the main channel. Move on to Step 11 to make changes to the land-disturbing activity or receiving system such that the criterion is met.

Note that the system must adequately convey the post-development peak rate of runoff from the outfall to the limits of analysis, not just at the limits of analysis. While a system may prove adequate at the limits of analysis, there may be segments between the outfall and limits of analysis that are not adequate to convey the post-development peak rate of runoff for the 10-yr, 24-hr storm. The best way to demonstrate the overall adequacy of the system for the post-developed conditions it to generate modeling for the system along its length downstream to the limits of analysis.
Step 11 - As the existing system below the outfall does not convey the post-development peak rate of runoff in the sample problem, either detention or stormwater system improvements may be incorporated into the land-disturbing activity to meet the flood protection capacity.

Note that as the pre-development system is deemed not to currently experience flooding based on historic record or anecdotal evidence, the design goal for detention is to meet the system capacity, not to discharge less than the pre-development peak rate of runoff. If system improvements are chosen to address flood protection, then the improvements should be designed for the post-development runoff conditions. Also, a combination of detention and system improvements is an alternative.

Because the system was deemed adequate for the pre-development conditions based upon existing evidence that flooding was not occurring, it is possible that the peak rate of runoff for the system capacity may actually be higher than the pre-development peak rate of runoff. When that is the case, detention should be designed to achieve the system capacity, when the existing system is not currently experiencing flooding.

Given that system improvements usually require work outside of the project area, necessitating additional R/W or easements, and requiring water quality permits for work in waters of the U.S., detention of runoff within the project limits is likely to be the most efficient and effective option. A cost comparison of detention onsite versus offsite system improvements should be made to determine the best option to address flood protection.

However, if system improvements are pursued due to cost effectiveness or other project benefits or site constraints, which require additional R/W or easements, work outside of the project area, and water quality permits for work in waters of the U.S., then it may make sense to use just system improvements.

What if the flood prone area adjacent to the main channel is modeled for the pre-development peak rate of runoff and it is determined that the 10-yr 24-hour storm event is not confined within the system?

If this occurs, then there is now information available that counters the historic record and anecdotal evidence that supported a conclusion of no localized flooding. In this case, it may be necessary to re-evaluate the flood protection criteria assuming that the system does currently experience localized flooding. An example of applying the flood protection criteria for systems that currently experience flooding is presented in the next section.
11.5.3.4.4  Flood Protection Sample Problem – Localized Flooding Currently Experienced

What if the system in the previous sample problem was currently experiencing flooding in pre-development flow conditions, based upon historic flood record, anecdotal evidence, a system specific floodplain study, or by the designer when routing the pre-development peak rate of runoff through the system in a hydraulic model?

Step 8a - As there exists documentation demonstrating that the system below the outfall does currently experience localized flooding during a 10-yr 24-hour storm event, the flood adequacy analysis can move to Step 10.

Step 10 - Given the hydrologic information developed in Step 5 for the 10-yr 24-hour storm event, a hydraulic analysis of the receiving system capacity from the project outfall to the limits of analysis is conducted using good hydrologic engineering methods and practices presented in Chapters 6 and 12 of the Drainage Manual.

For the sample problem, the receiving system (main channel and adjacent flood prone areas) is well defined, so the pre-development peak rate of runoff does not need to be modeled hydraulically to determine the pre-development flood prone areas. However, the system is modeled to determine that the capacity at the limits of analysis is 1,050 cfs. Is the system adequate for the post-development flows in the 10-yr 24-hour storm?

In Step 6 it was determined that the post-development peak rate of runoff for the system at the limits of analysis is 1,260 cfs. As 1,260 cfs is greater than the system capacity of 1,050 cfs, the system does not confine the post-development peak rate of runoff and the flood protection criteria is not met. Move on to Step 11 to make changes to the land-disturbing activity or receiving system such that the criterion is met.

Note that the system must adequately convey the post-development peak rate of runoff from the outfall to the limits of analysis, not just at the limits of analysis. While a system may prove adequate at the limits of analysis, there may be segments between the outfall and limits of analysis that are not adequate to convey the post-development peak rate of runoff for the 10-yr, 24-hr storm. The best way to demonstrate the overall adequacy of the system for the post-developed conditions is to generate modeling for the system along its length downstream to the limits of analysis.

Step 11 - As the existing system below the outfall does not convey the post-development peak rate of runoff in the sample problem, either detention or stormwater system improvements may be incorporated into the land-disturbing activity to meet the flood protection capacity.

Note that as the pre-development system is deemed to currently experience flooding based on historic record or anecdotal evidence, the design goal for
detention is to either meet the system capacity or to discharge less than the pre-development peak rate of runoff from the project at the outfall in the post-development conditions. This is different than the requirement for a system that is not currently experiencing flooding, as it allows the option to provide onsite detention to pre-development peak rates of runoff instead of providing detention to the system capacity. In this case, the designer should evaluate both detention criteria to determine which requires the least onsite detention and design for that criterion. 

For example, if the detention required to meet the system adequacy is a 1-acre basin area storing 4 acre-feet of runoff, and the detention required to detain to less than the pre-development peak rate of runoff only requires a 0.7-acre basin area storing 2.5 acre-feet, then the smaller detention basin is likely to be more cost effective to implement. The designer should do a cost comparison, including the cost for R/W and easements as well as construction and maintenance, to demonstrate which basin is most cost effective for the final design.

If system improvements are chosen to address flood protection, then the improvements should be designed for the post-development runoff conditions. Also, a combination of detention and system improvements is an alternative. 

As noted in the previous example, system improvements usually require work outside of the project area, necessitating additional R/W or easements, and requiring water quality permits for work in waters of the U.S. For this reason, detention of runoff is likely to be the most efficient and effective option. A cost comparison of detention versus offsite system improvements should be made to determine the best option to address flood protection.

If system improvements are pursued due to cost effectiveness or other project benefits or site constraints, which require additional R/W or easements, work outside of the project area, and water quality permits for work in waters of the U.S., then it may make sense to use just system improvements.

11.5.4 Pretreatment

Pretreatment is an important component of most water quality BMPs. Its purpose is to remove gross pollutants (sand, grit, gravel, trash, and debris) from stormwater runoff in an area that is easier to access and maintain, protecting the primary BMP downstream from contamination and extending the maintenance life of the overall BMP. Pretreatment features may require more frequent maintenance themselves, but their intent is to lengthen the time between maintenance activities in the primary BMP. Examples of pretreatment practices include vegetated filter strips, sediment forebays, energy dissipaters, and manufactured treatment devices.
Some of the new RR practices require more than one form of pretreatment in order to achieve runoff reduction and higher pollutant removal efficiencies. Details on pretreatment selection and design are found in the VDOT Part II B BMP Design Manual of Practice and the Virginia BMP Clearinghouse Standards and Specifications.

11.5.5 Treatment Volume Computation

Treatment Volume (Tv) for water quality replaces the old concept of “Water Quality Volume (WQV)” in the Virginia Runoff Reduction Method (VRRM). The value of Tv is the product of the “Target Rainfall Event” (1.00 inch), the site area (acres), and the site Rv from the VRRM, with a unit conversion:

\[
Tv = 1.00 \text{ inches} \times \text{Site Area acres} \times \text{Rv} \times \left(\frac{\text{1 foot}}{12 \text{ inches}}\right)
\]

What is the Tv for a 1.7 acre site with a computed post-development Rv of 0.56?

\[
Tv = 1.00 \text{ inches} \times 1.7 \text{ acres} \times 0.56 \times \left(\frac{\text{1 foot}}{12 \text{ inches}}\right) = 0.0793 \text{ acre - feet}
\]

The VRRM Spreadsheet automatically calculates the water quality Tv requirements for the entire site on the Site tab after the land cover information is entered.

Once BMPs are applied to a drainage area, the Tv for the contributing drainage area to each BMP must be calculated for proper sizing of the BMP. The VRRM Spreadsheet also calculates the Tv to each BMP in the drainage area tabs, using the contributing drainage area and the Rv calculated for the contributing drainage area to each BMP. For BMPs in-series, the Tv for a downstream BMP is based upon the Tv for the upstream BMP minus the RR retention storage volume provided by the upstream RR practice.

Some BMP designs require a storage or treatment volume that is more than one Tv to achieve the published pollutant reduction rates. This is presented in the VDOT BMP Design Manual of Practice and the Virginia BMP Clearinghouse Standards. For example, the Bioretention Level 2 and the Extended Detention Level 2 standards require a design using 1.25 times Tv to achieve higher removal rates. A Wet Pond #2 design is based upon 1.5 times the Tv, but has a higher pollutant removal rate than a Wet Pond #1.

11.5.6 Detention Time Computation and Orifice Sizing

A water quality extended-detention basin treats runoff by detaining it and releasing it over a specified amount of time. In theory, extended-detention of the required Treatment Volume will allow the particulate pollutants to settle out of runoff, functioning similarly to a permanent pool in a Wet Pond. The Virginia BMP Clearinghouse Standard for Extended Detention Level 1 specifies 24-hours or less draw down for the average Tv
time, while the Extended Detention Level 2 design requires an average $T_v$ draw down time of 36 hours.

This is a brim draw down time, beginning at the time of peak storage of the water quality volume. Brim drawdown time means the time required for the entire calculated volume to drain out of the basin.

This assumes that the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. The extended detention orifice can be sized using either of the following methods, taken from the Virginia Stormwater Management Handbook (VSWMH), 1999 edition:

- Using the **average hydraulic** head associated with the required $T_v$ and draw down time. This is the VDOT preferred option.

- Using the **maximum hydraulic** head associated with the $T_v$, calculate the orifice size needed to achieve the required draw down time and route the $T_v$ through the basin to verify the actual storage volume used and the drawdown time.

After calculating the needed orifice size the designer should select the nearest nominal size opening from Table 11-10.

<table>
<thead>
<tr>
<th>Diameter</th>
<th>Square Feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>½</td>
<td>0.001</td>
</tr>
<tr>
<td>¾</td>
<td>0.003</td>
</tr>
<tr>
<td>1</td>
<td>0.005</td>
</tr>
<tr>
<td>1 ½</td>
<td>0.012</td>
</tr>
<tr>
<td>2</td>
<td>0.022</td>
</tr>
<tr>
<td>2 ½</td>
<td>0.034</td>
</tr>
<tr>
<td>3</td>
<td>0.049</td>
</tr>
<tr>
<td>3½</td>
<td>0.067</td>
</tr>
<tr>
<td>4</td>
<td>0.087</td>
</tr>
<tr>
<td>4 ½</td>
<td>0.110</td>
</tr>
<tr>
<td>5</td>
<td>0.136</td>
</tr>
<tr>
<td>5 ½</td>
<td>0.165</td>
</tr>
<tr>
<td>6</td>
<td>0.196</td>
</tr>
</tbody>
</table>

### 11.5.6.1.1 Average Hydraulic Head Method (Method #2 from VSWMH) - VDOT Preferred Method

The average hydraulic head method is the preferred method for determining the required orifice size. It is quicker and easier than the maximum hydraulic head method, which requires a routing to verify the drawdown time. It is also noted that the difference in orifice size produced by the two different methods is insignificant, (i.e. 2 inches versus 2½ inches.)
Average Hydraulic Head Sample Problem:

Find the orifice size for the required treatment volume for an Extended Detention Level 1 design using the average hydraulic head method, where:

\[ h_{\text{max}} = 1.1 \text{ ft.} \]
\[ T_v = 8,720 \text{ ft}^3 \]

Step 1 - Calculate the average head:

\[ h_{\text{avg}} = \frac{1.1 \text{ ft}}{2} = 0.55 \text{ ft} \]

Note: Actual h for an orifice is measured from the center of the orifice. Since the size of this orifice is unknown and assumed small, use \( h_{\text{max}} = 1.1 \text{ ft.} \)

Step 2 - Determine the drawdown time for an Extended Detention Level 1 design from the VDOT BMP Manual of Practice: average 24-hr detention time.

Step 3 - Calculate the discharge through the orifice based on the required treatment volume based upon the required detention time:

\[ Q_{\text{avg}} = \frac{T_v}{\text{Extended Detention Time}} = \frac{8,720 \text{ ft}^3}{24 \text{ hr} \times 3,600 \text{ sec/hr}} = 0.101 \text{ cfs} \]

Step 4 - Calculate the orifice area by rearranging the orifice equation:

\[ A = \frac{Q_{\text{avg}}}{C \sqrt{2 \times g \times h_{\text{avg}}}} = \frac{0.101 \text{ cfs}}{0.6 \sqrt{2 \times 32.2 \times 0.55}} = 0.0283 \text{ ft}^2 \]

Step 5 - From Table 11-10, select a 2-inch orifice with \( A = 0.022 \text{ ft}^2 \).

Step 6 - The \( T_v \) hydrograph should then be routed through the basin to determine if the residence time is approximately 24 hours.

11.5.6.1.2 Maximum Hydraulic Head Method (Method #1 from VSWMH)

The maximum hydraulic head method uses the maximum discharge and results in a slightly larger orifice than the same procedure using the average hydraulic head method. The routing allows the designer to verify the performance of the calculated orifice size. However, as a result of the routing effect, the actual basin storage volume used to achieve the drawdown time will be less than the computed brim drawdown volume.

Maximum Hydraulic Head Sample Problem:

Find the orifice size for the required treatment volume for an Extended Detention Level 1 design using the maximum hydraulic head method, where:

\[ h_{\text{max}} = 1.1 \text{ ft.} \]
\[ Q_{\text{avg}} = 0.101 \text{ cfs} \]
Step 1 - Calculate the maximum $Q_{\text{max}}$:

$$Q_{\text{max}} = 2 \times Q_{\text{avg}} = 2 \times 0.101 \text{ cfs} = 0.202 \text{ cfs}$$

Step 2 - Using $h_{\text{max}}$ and $Q_{\text{max}}$, compute the orifice size for the required treatment volume using the maximum hydraulic head method.

Calculate the orifice area by rearranging the orifice equation:

$$A = \frac{Q_{\text{max}}}{C\sqrt{2 \times g \times h_{\text{max}}}} = \frac{0.202 \text{ cfs}}{0.6\sqrt{2 \times 32.2 \times 1.1}} = 0.040 \text{ ft}^2$$

Step 3 - From Table 11-10, select a 2½-inch orifice with $A = 0.034 \text{ ft}^2$

Step 4 - Route the Tv hydrograph through the basin using the 2½-inch orifice.

NOTE: The routing of the Tv hydrograph thru a basin may not be possible with some routing software where hydrographs lasting longer than 24 hours are not accommodated. The problem is due to detention times greater than 24-hours to achieve hydrograph draw down to 0 cfs, possibly greater than 30 or more hours at very low flows.

11.5.6.1.3 **Tv Hydrograph**

To develop a runoff hydrograph for the Tv, the designer should use the “Target Rainfall Event” of 1 inch, HSG and land cover for post-development conditions, and the hydrograph development techniques presented in Chapter 6 of the Drainage Manual.

The NRCS unit hydrographs will probably be the easiest hydrographs to provide the required treatment volume for an extended detention basin or other SWMF. The land cover conditions should be based upon the NRCS CN method. Based upon the VRRM, “Forest/Open Space” uses the CNs for “Woods, Good”; “Managed Turf” uses the CNs for “Open Space, Good”; and “Impervious Cover” has a CN = 98 for all HSG.

11.5.6.1.4 **Alternative Method of Routing WQV to Find Drawdown Time**

The Stormwater Management Handbook, Vol. II, 1999 edition, defines “brim drawdown time” as the time the treatment volume elevation is reached until the basin is emptied. This is based upon a storm producing only the amount of runoff required for the Tv, based upon a “Target Rainfall Event” of 1 inch.

The normally required routing of a storm larger than the “Target Rainfall Event” of 1 inch for quantity control can also be used for drawdown time with some adjustment providing that the routing software will accommodate a duration greater than 24-hours. The receding limb of the inflow hydrograph will need to be showing either 0.0 or 0.01 cfs inflow up to a time of 30 hours for an Extended Detention Level 1 design, and up to 48 hours for an Extended Detention Level 2 design.
By this method the drawdown time for $T_v$ is actually from the time that the ponded depth recedes to the treatment volume elevation with no more inflow (remember that this method is for storm events > 1-inch) until the basin is “empty” (receding limb of the inflow hydrograph will need to be showing either 0.0 or 0.01 cfs).

11.5.7 Preliminary Detention Volume Computation

Three methods are presented for estimating the volume of storage needed for peak flow attenuation (quantity control). The estimated storage volumes are approximate and the designer will need to select the most appropriate volume in order to determine the preliminary basin size.

11.5.7.1.1 Modified Rational Method, Simplified Triangular Hydrograph Routing

Information needed includes the hydrology and hydrographs for the watershed or drainage area to be controlled, calculated by using one of the methods as outlined in Chapter 6, and the allowable release rates for the facility, as established by the water quantity criteria for channel and flood protection.

**Step 1 - Determine BMP requirements**

Calculate the water quality Treatment Volume ($T_v$) using the steps presented previously.

**Step 2 - Compute allowable release rates**

Compute the pre- and post-developed hydrology for the site outfall using the methods presented previously. In either case, the post-developed hydrology will provide the peak discharge into the basin as a peak discharge (cfs), a runoff volume (watershed inches, acre-feet, or ft$^3$), or a runoff hydrograph (cfs over storm and runoff duration). Refer to Chapter 6, Hydrology, on developing peak discharge, runoff volume, and runoff hydrographs.

**Step 3 - Estimate the required storage volume**

The information required includes the developed condition peak rate of runoff, or runoff hydrograph, and the allowable release rates for the appropriate design storms. These methods provide a preliminary estimate of the storage volume required for peak flow attenuation.
Figure 11-1. Simplified Triangular Hydrograph Method

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

\[ V_s = \frac{1}{2} T_b (Q_i - Q_o) \]

Where:
- \( V_s \) = Storage volume estimate, ft\(^3\)
- \( Q_i \) = Peak inflow rate, cfs
- \( Q_o \) = Peak outflow rate, cfs
- \( T_b \) = Duration of basin inflow, sec.

11.5.7.1.2 Critical Storm Duration Method

The critical storm duration method is used to calculate the maximum storage volume for a detention facility. This critical storm duration is the storm duration that generates the greatest volume of runoff and, therefore, requires the most storage. The required storage volume is represented by the area between the inflow hydrograph and the outflow hydrograph. The area can be approximated using the following equation:

\[ V = \left[ Q_i T_d + \frac{Q_i t_c}{4} - \frac{q_o T_d}{2} - \frac{3q_o t_c}{4} \right] 60 \]
Where:
\( V \) = Required storage volume, ft\(^3\)
\( Q_i \) = Inflow peak discharge, cfs, for the critical storm duration, \( T_d \)
\( T_c \) = Time of concentration, min.
\( q_o \) = Allowable peak outflow, cfs
\( T_d \) = Critical storm duration, min.

The first derivative of the critical storage volume equation with respect to time is an equation that represents the slope of the storage volume curve plotted versus time. When the equation above is set to equal zero, and solved for \( T_d \), it represents the time at which the slope of the storage volume curve is zero, or at a maximum.

The equation for the critical storm duration is:

\[
T_d = \sqrt{\frac{2CAa(b - \frac{t_c}{4})}{q_o}} - b
\]

Where:
\( T_d \) = Critical storm duration, min.
\( C \) = Runoff coefficient
\( A \) = Drainage area, ac.
\( a \) & \( b \) = Rainfall constants developed for storms of various recurrence intervals and various geographic locations
\( t_c \) = Time of concentration, min.
\( q_o \) = Allowable peak outflow, cfs

* The \( a \) & \( b \) rainfall constants are not to be used for any other purpose.

The Department has developed a computer program entitled “CRITSTRM” for performing these computations. Access is available upon request at the following web address:

http://www.virginiadot.org/business/locdes/notification.asp

### 11.5.7.1.3 Pagan Volume Estimation Method

This method is appropriate for use with small basins serving watersheds of 200 acres or less. For this method, data from many small basins was compiled and the curve in Figure 11-11 was developed. This curve is used to determine the storage volume for a given drainage area by dividing the pre-development peak inflow by the post-development peak inflow.

Knowing the percentage of peak inflow, the storage parameter (peak storage in cubic feet over peak inflow in cubic feet per second) can be found by moving horizontally over the y-axis to the curve and down to the x-axis.
By multiplying the storage parameter by the peak inflow, the approximate peak storage can be found. This method should be used only as a first trial. Experience has shown that this method is conservative.

**Figure 11-2. Pagan Method Curve**

**Step 1** - Determine pre- and post-development peak discharges.

**Step 2** - Determine the Storage Parameter \( (SP) \).

\[ SP = \frac{\text{Peak storage in cubic feet}}{\text{Peak inflow in cfs}} = \frac{\text{STO}}{I} \]

**Step 3** - Compute the Maximum Storage Volume \( (STO) \):

\[ STO = SP(I) \]

### 11.5.7.1.4 Sample Problems – Using 3 Methods to Estimate Volume of Storage for Quantity Control

Given the following information, estimate the volume of storage required for water quantity control:

<table>
<thead>
<tr>
<th>Condition</th>
<th>Rational Method</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DA</td>
</tr>
<tr>
<td>Pre-developed</td>
<td>25 ac</td>
</tr>
<tr>
<td>Post-developed</td>
<td>25 ac</td>
</tr>
</tbody>
</table>
Method 1: Modified Triangular Hydrograph Method

Based on the triangular hydrograph method, solve for $V_s$ as follows:

$$V_s = \frac{1}{2} T_b (Q_i - Q_o)$$

Where:
- $V_s$ = Storage volume estimate, ft$^3$
- $Q_i$ = 65 cfs
- $Q_o$ = 24 cfs
- $T_b = 2 \times T_c$ (post-development) = 2 x 21 min = 42 min = 2,520 sec

$$V_s = \frac{1}{2} (2,520)(65 - 24) = 51,660 \text{ ft}^3$$

Method 2: Critical Storm Duration Method

Based on the critical storm duration method, determine the critical storm duration $T_d$ as follows:

- $a = 189.2$
- $b = 22.1$
- $C = 0.59$ (post-development)
- $A = 25$ acres
- $t_c = 21$ min (post-development)
- $q_o = 24$ cfs (Allowable outflow based on pre-development)

$$T_d = \sqrt{\frac{2CAa(b - \frac{t_c}{4})}{q_o}} - b$$

$$T_d = \sqrt{\frac{2(0.59)(25)(189.2)(22.1 - \frac{21}{4})}{24}} - 22.1$$

$$T_d = 40.5 \text{ min}$$

Solve for the critical storm duration intensity ($I$):

$$I = \frac{a}{b + 40.5} = \frac{189.2}{22.1 + 40.5} = 3.02 \text{ in/hr}$$

Determine the peak inflow ($Q$) using the Rational Method equation and the critical storm duration intensity ($I$):

$$Q = C_iC_iA = 1.0(0.59)(3.02)(25) = 44.5 \text{ cfs}$$
Determine the required storage volume \( V \) for the critical storm duration \( T_d \):

\[
V = \left[ Q_i T_d + \frac{Q_i t_c}{4} - \frac{q_o T_d}{2} - \frac{3q_o t_c}{4} \right] 60
\]

\[
V = \left[ 44.5(40.5) + \frac{44.5(21)}{4} - \frac{24(40.5)}{2} - \frac{3(24)(21)}{4} \right] 60
\]

\[
V = 70,313 \text{ ft}^3
\]

Method 3: Pagan Method

Based on the Pagan Method, solve for the estimated storage volume as follows:

\[
\frac{Q_o}{Q_i} = \frac{24}{65} = 0.37 \times 100\% = 37\%
\]

Determine the value for SP from Figure 11-5:

\[
SP = 3,100 \text{ sec}
\]

Use the relationship between STO, SP, and I to calculate STO:

\[
STO = SP(I) = 3,100 \times 65 = 201,500 \text{ ft}^3
\]

A comparison of the results for the 3 methods applied to the sample problem:

<table>
<thead>
<tr>
<th>Method</th>
<th>Estimated Storage Volume, V (ft$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triangular Hydrograph</td>
<td>51,660</td>
</tr>
<tr>
<td>Critical Storm Duration</td>
<td>70,313</td>
</tr>
<tr>
<td>Pagan Method</td>
<td>201,500</td>
</tr>
</tbody>
</table>

Note that the Pagan Method estimates a much higher detention volume than either the Triangular Hydrograph or Critical Storm Duration methods.

11.5.8 Preliminary Basin Sizing

Based upon the estimated storage volume requirements calculated by the three methods above, determine the preliminary size of the basin. Assume the basin will have a rectangular shaped base, 2:1 length to width ratio, and an optimum depth of 4 feet. The basin will have 3:1 side slopes, but for the first size estimate, size the basin assuming vertical sides for a first estimate.
Method 1: Simplified Triangular Hydrograph Method

Calculate the footprint assuming a 4-ft depth:
\[
\frac{51,660}{4} = 12,915 \text{ ft}^2
\]

Assuming a rectangular shape with 2:1 length to width ratio:
\[
(L) \times (W) = 12,915 \text{ ft}^2 \\
L = 2(W) \\
2(W) \times 1(W) = 2(W^2) = 12,915 \text{ ft}^2
\]
\[
W = \sqrt{\frac{12,915 \text{ ft}^2}{2}} = 80 \text{ ft}
\]
\[
L = 2(W) = 2(80) = 160 \text{ ft}
\]

Check the volume using the dimensions calculated:
\[
V = L \times W \times D = 160 \times 80 \times 4 = 51,200 \text{ ft}^3 > 51,660 \text{ ft}^3 \checkmark
\]

Method 2: Critical Storm Duration Method

Calculate the footprint assuming a 4-ft depth:
\[
\frac{70,313}{4} = 17,578 \text{ ft}^2
\]

Assuming a rectangular shape with 2:1 length to width ratio:
\[
(L) \times (W) = 17,578 \text{ ft}^2 \\
LL = 2(WW) \\
2(W) \times 1(W) = 2(W^2) = 17,578 \text{ ft}^2
\]
\[
W = \sqrt{\frac{17,578 \text{ ft}^2}{2}} = 94 \text{ ft}
\]
\[
L = 2(W) = 2(94) = 188 \text{ ft}
\]

Check the volume using the dimensions calculated:
\[
V = L \times W \times D = 188 \times 94 \times 4 = 70,688 \text{ ft}^3 > 70,313 \text{ ft}^3 \checkmark
\]

Method 3: Pagan Method

Calculate the footprint assuming a 4-ft depth:
\[
\frac{201,500}{4} = 50,375 \text{ ft}^2
\]
Assuming a rectangular shape with 2:1 length to width ratio:

\[(L) \times (W) = 50,375 \text{ ft}^2\]

\[L = 2(W)\]

\[2(W)(W) \times 1(W)(W) = 2(W^2W^2) = 50,375 \text{ ft}^2\]

\[W = \sqrt{\frac{50,375 \text{ ft}^2}{2}} = \sqrt{\frac{50,375 \text{ ft}^2}{2}} = 159 \text{ ft}\]

\[L = 2(W) = 2(159) = 318 \text{ ft}\]

Check the volume using the dimensions calculated:

\[V = L \times W \times D = 318 \times 159 \times 4 = 202,248 \text{ ft}^3 > 201,500 \text{ ft}^3\]

A comparison of the results for the 3 methods applied to the sample problem:

<table>
<thead>
<tr>
<th>Method</th>
<th>Estimated Dimensions (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Length</td>
</tr>
<tr>
<td>Triangular Hydrograph</td>
<td>160</td>
</tr>
<tr>
<td>Critical Storm Duration</td>
<td>188</td>
</tr>
<tr>
<td>Pagan Method</td>
<td>318</td>
</tr>
</tbody>
</table>

Note the differences in the results for the 3 methods. The only way to confirm the actual storage required to detain to allowable peak flow rates is to design the basin using the estimated storage and route the storms to confirm that detention is achieved. The design for the basin should be optimized to reflect the dimensions that provide sufficient storage without oversizing and driving up the cost to construct and maintain the facility.

11.5.9 Final Basin Sizing – Reservoir Routing

11.5.9.1 Storage – Indication Method Routing Procedure

The following procedure presents the basic principles of performing routing through a reservoir or storage facility (Puls Method of storage routing). Routing is most often completed with computer software, which develops the stage-discharge and stage-storage curves within the program.

Step 1: *Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility. Example stage-storage and stage-discharge curves are shown in Figure 11-12 and Figure 11-13 respectively.*
Step 2: Select a routing time period (\(\Delta t\)) to provide at least five points on the rising limb of the inflow hydrograph. Use \(t_p\) divided by 5 to 10 for \(\Delta t\).

Step 3: Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of \(s_{\Delta T}^O\) versus stage. An example tabulation of storage characteristics curve data is shown in Table 11-2.
### Table 11-2. Storage Characteristics

<table>
<thead>
<tr>
<th>Stage (H) (ft.)</th>
<th>Storage (^1) (S) (ac-ft)</th>
<th>Discharge (^2) (Q) (cfs)</th>
<th>Discharge (^2) (Q) (ac-ft/hr)</th>
<th>S (\Delta Q) (^2) (ac-ft)</th>
<th>S +(\Delta Q) (^2) (ac-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>0.05</td>
<td>0</td>
<td>0</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>101</td>
<td>0.05</td>
<td>15</td>
<td>1.24</td>
<td>0.20</td>
<td>0.40</td>
</tr>
<tr>
<td>102</td>
<td>0.05</td>
<td>35</td>
<td>2.89</td>
<td>0.56</td>
<td>1.04</td>
</tr>
<tr>
<td>103</td>
<td>1.6</td>
<td>63</td>
<td>5.21</td>
<td>1.17</td>
<td>2.03</td>
</tr>
<tr>
<td>104</td>
<td>2.8</td>
<td>95</td>
<td>7.85</td>
<td>2.15</td>
<td>3.45</td>
</tr>
<tr>
<td>105</td>
<td>4.4</td>
<td>143</td>
<td>11.82</td>
<td>3.41</td>
<td>5.39</td>
</tr>
<tr>
<td>106</td>
<td>6.6</td>
<td>200</td>
<td>16.53</td>
<td>5.22</td>
<td>7.98</td>
</tr>
</tbody>
</table>

\(^1\) Obtained from the Stage-Storage Curve.
\(^2\) Obtained from the Stage-Discharge Curve.

Note: \(t = 10\) minutes = 0.167 hours and 1 cfs = 0.0826 ac-ft/hr.

**Step 4:** For a given time interval, \(I_1\) and \(I_2\) are known. Given the depth of storage or stage \((H_i)\) at the beginning of that time interval, \(-S \Delta Q\) \(^2\) can be determined from the appropriate storage characteristics curve, Figure 11-14.

![Figure 11-5. Storage Characteristics Curve](image-url)
Step 5  **Determine the value of** \( s_2 \Delta T \) **from the following equation:**

\[
S_2 + \frac{O_2}{2} \Delta T = S_1 - \frac{O_1}{2} \Delta T + \frac{I_1 + I_2}{2} \Delta T
\]

(11.6)

Where:
- \( S_2 \) = Storage volume at time 2, ft\(^3\)
- \( O_2 \) = Outflow rate at time 2, cfs.
- \( \Delta T \) = Routing time period, sec
- \( S_1 \) = Storage volume at time 1, ft\(^3\)
- \( O_1 \) = Outflow rate at time 1, cfs
- \( I_1 \) = Inflow rate at time 1, cfs
- \( I_2 \) = Inflow rate at time 2, cfs

Other consistent units are equally appropriate.

Step 6:  **Enter the storage characteristics curve at the calculated value of** \( s_2 \Delta T \) **determined in Step 5 and read off a new depth of water \((H_2)\).**

Step 7:  **Determine the value of** \( O_2 \), **which corresponds to a stage of** \( H_2 \) **determined in Step 6, using the stage-discharge curve.**

Step 8:  **Repeat Steps 1 through 7 by setting new values of** \( I_1, O_1, S_1, \) **and** \( H_1 \) **equal to the previous** \( I_2, O_2, S_2, \) **and** \( H_2, \) **and using a new** \( I_2 \) **value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.**

**11.5.9.2 Storage – Indication Method Routing Sample Problem #1**

This example demonstrates the application of the methodology presented for the design of a typical detention storage facility used for water quantity control.

Storage facilities shall be designed for allowable runoff from the applicable design storms. The analysis should also include the 100-yr design storm runoff to ensure that the structure can accommodate runoff from this storm without damaging adjacent and downstream property and structures due to flooding or overtopping the dam and causing it to fail.

For this sample problem, assume the receiving system is manmade, the energy balance is not being used for compliance, and there is no mapped floodplain or flood prone area immediately downstream. The peak discharges from the 2- and 10-yr 24-hour design storms are as follows:

- Allowable 2-yr 24-hour peak discharge = 150 cfs
- Allowable 10-yr 24-hour peak discharge = 200 cfs
- Post-development 2-yr 24-hour peak discharge = 190 cfs
- Post-development 10-yr 24-hour peak discharge = 250 cfs
Since the post-development peak discharge must not exceed the allowable peak discharge for channel and flood protection, the allowable design discharges are 150 cfs and 200 cfs for the 2- and 10-yr 24-hour design storms, respectively.

**Step 1:** Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility.

Runoff hydrographs are shown in Table 11-12 below. Inflow durations from the post-development hydrographs are about 1.2 and 1.25 hours, respectively, for runoff from the 2- and 10-yr 24-hour storms.

<table>
<thead>
<tr>
<th>Table 11-3. Runoff Hydrographs</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pre-Development Runoff</strong></td>
</tr>
<tr>
<td>(1) Time (hrs)</td>
</tr>
<tr>
<td>0</td>
</tr>
<tr>
<td>0.1</td>
</tr>
<tr>
<td>0.2</td>
</tr>
<tr>
<td>0.3</td>
</tr>
<tr>
<td>0.4</td>
</tr>
<tr>
<td>0.5</td>
</tr>
<tr>
<td>0.6</td>
</tr>
<tr>
<td>0.7</td>
</tr>
<tr>
<td>0.8</td>
</tr>
<tr>
<td>0.9</td>
</tr>
<tr>
<td>1.0</td>
</tr>
<tr>
<td>1.1</td>
</tr>
<tr>
<td>1.2</td>
</tr>
</tbody>
</table>

Preliminary estimates of required storage volumes are obtained using the simplified triangular hydrograph method outlined in Section 11.5.4.1. For runoff from the 2- and 10-yr storms, the required storage volumes, $V_s$, are computed using Equation 11.3:

$$V_s = \frac{1}{2} T_b (Q_i - Q_o)$$

$$V_{s2} = \frac{1}{2} \left(1.2\right) (3600) (190-150) \quad 43,560 = 1.98 \text{ ac. ft.}$$

$$V_{s10} = \frac{1}{2} \left(1.25\right) (3600) (250-200) \quad 43,560 = 2.58 \text{ ac. ft.}$$

Stage-discharge and stage-storage characteristics of a storage facility that should provide adequate peak flow attenuation for runoff from both the 2- and 10-yr design storms are presented below in Table 11-13. The storage-discharge relationship was
developed and required that the preliminary storage volume estimates of runoff for both the 2- and 10-yr design storms to coincide with the occurrence of the corresponding allowable peak discharges.

Discharge values were computed by solving the broad-crested weir equation for head (H) assuming a constant discharge coefficient of 3.1, a weir length of 4’, and no tailwater submergence. The capacity of storage relief structures was assumed to be negligible.

**Step 2:** Select a routing time period (\(\Delta t\)) to provide at least five points on the rising limb of the inflow hydrograph. Use \(t_p\) divided by 5 to 10 for \(\Delta t\).

\[
\Delta T = \frac{t_p}{5} = \frac{0.5}{5} = 0.1 \text{ hr}
\]

**Step 3:** Use the storage-discharge data from Step 1 to develop storage characteristics curves (Stage-Discharge-Storage) that provide values of \(S + \frac{S}{2} \Delta T\) versus stage.

<table>
<thead>
<tr>
<th>(1) Stage (H) (ft)</th>
<th>(2) Discharge (Q) (cfs)</th>
<th>(3) Storage (S) (ac-ft)</th>
<th>(4) (S + \frac{S}{2} \Delta T) (ac-ft)</th>
<th>(5) (S + \frac{S}{2} \Delta T) (ac-ft)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
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<tr>
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<td>0.33</td>
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<td>1.8</td>
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<td>0.68</td>
<td>0.43</td>
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<tr>
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<td>0.60</td>
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<td>3.2</td>
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<td>0.76</td>
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<td>2.13</td>
<td>1.14</td>
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<td>2.76</td>
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<td>6.0</td>
<td>180</td>
<td>2.34</td>
<td>3.08</td>
<td>1.60</td>
</tr>
</tbody>
</table>
Storage routing was conducted for runoff from both the 2- and 10-yr design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Routing results are shown below for runoff from the 2- and 10-yr design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 2- and 10-yr design storms.

Step 4: For a given time interval, $I_1$ and $I_2$ are known. Given the depth of storage or stage ($H_1$) at the beginning of that time interval, $s_{1}^{O_1\frac{\Delta T}{2}}$ can be determined from the appropriate storage characteristics curve.

Step 5 Determine the value of $s_{2}\frac{O_2\Delta T}{2}$ from the following equation:

$$S_2 + \frac{O_2}{2} \Delta T = S_1 - \frac{O_1}{2} \Delta T + \frac{I_1 + I_2}{2} \Delta T$$

(11.7)

Summarized in Tables 11-5 and 11-6 for the 2-yr and 10-yr storms.

Step 6 Enter the storage characteristics curve at the calculated value of $s_{2}\frac{O_2\Delta T}{2}$ determined in Step 5 and read off a new depth of water ($H_2$).

Summarized in Tables 11-5 and 11-6 for the 2-yr and 10-yr storms.

Step 7 Determine the value of $O_2$, which corresponds to a stage of $H_2$ determined in Step 6, using the stage-discharge curve.

Summarized in Tables 11-5 and 11-6 for the 2-yr and 10-yr storms.

Step 8 Repeat Steps 1 through 7 by setting new values of $I_1$, $O_1$, $S_1$, and $H_1$ equal to the previous $I_2$, $O_2$, $S_2$, and $H_2$, and using a new $I_2$ value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

Summarized in Tables 11-5 and 11-6 for the 2-yr and 10-yr design storms.
### Table 11-5. Storage Routing for the 2-yr Storm

<table>
<thead>
<tr>
<th>Time (T) (hrs)</th>
<th>Inflow (I) (cfs)</th>
<th>( \frac{I_1 + I_2 \Delta T}{2} ) (ac-ft)</th>
<th>Stage (H1) (ft)</th>
<th>( S_1 \Delta Q_1 ) (ac-ft)</th>
<th>Stage (H) (ft)</th>
<th>Outflow (O) (cfs)</th>
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</thead>
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### Table 11-6. Storage Routing for the 10-yr Storm

<table>
<thead>
<tr>
<th>Time (T) (hrs)</th>
<th>Inflow (I) (cfs)</th>
<th>( \frac{I_1 + I_2 \Delta T}{2} ) (ac-ft)</th>
<th>Stage (H1) (ft)</th>
<th>( S_1 \Delta Q_1 ) (ac-ft)</th>
<th>Stage (H) (ft)</th>
<th>Outflow (O) (cfs)</th>
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</thead>
<tbody>
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<td>0.00</td>
<td>0.00</td>
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<td>0.36</td>
<td>1.20</td>
</tr>
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<td>1</td>
<td>0.02</td>
<td>1.20</td>
<td>0.28</td>
<td>0.30</td>
<td>0.90</td>
</tr>
<tr>
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<td>0.00</td>
<td>0.90</td>
<td>0.22</td>
<td>0.22</td>
<td>0.60</td>
</tr>
</tbody>
</table>

Chapter 11-87 of 148
Since the routed peak discharge is lower than the maximum allowable peak discharges for both design storms, the weir length could be increased or the storage decreased. If revisions are desired, routing calculations should be repeated.

Although not shown for this sample problem, runoff from the 100-yr frequency storm should be routed through the storage facility to establish freeboard requirements and to evaluate emergency overflow and stability requirements. In addition, the preliminary design provides hydraulic details only. Final design should consider site constraints such as depth to water, side slope stability, maintenance, grading to prevent standing water, and provisions for public safety.

An estimate of the potential downstream effects (i.e., increased peak flow rate and recession time) of detention storage facilities may be obtained by comparing hydrograph recession limbs from the pre-development and routed post-development runoff hydrographs. Example comparisons are shown below.

![Figure 11-6. Runoff Hydrographs](image)

**11.5.9.3 SWM Basin Design: Sample Problem**

**Step 1: Determine Stormwater Requirements:**

- The receiving system is a manmade stormwater conveyance system that was determined not to be adequate for the uncontrolled post-development peak runoff, as the velocity and shear will exceed the allowable values for the channel materials and lining.
• In accordance with the water quantity flood protection criteria, the SWMF will need to attenuate the post-development $Q_2$ such that the manmade system is not subject to erosion. The design of the dam and the emergency spillway will also need to provide protection of the dam for $Q_{100}$.

• The allowable peak discharge at which the channel is not expected to erode is $Q_{2all} = 20.5$ cfs and the post development $Q_{2post} = 29.6$ cfs (time of concentration, $t_c = 0.333$ hr).

• A FEMA Zone A floodplain is mapped for the receiving stream immediately below the outfall.

• A VRRM Spreadsheet was developed for water quality and an Extended Detention Level 2 basin is required.

**Step 2:**  *Determine the required Treatment Volume (Tv) and Design Treatment Volume:*

• An Extended Detention Level 2 design has been selected to meet the water quality criteria for the site.

• The Treatment Volume (Tv) for the drainage area contributing to the BMP was calculated in the VRRM Spreadsheet as 8,654 ft$^3$.

• According to the VDOT BMP Design Manual of Practice, the total volume for an Extended Detention Level 2 design is 1.25 x Tv. With a Tv of 8,654 ft$^3$, the total design volume is 1.25 x 8,654 ft$^3$ = 10,817 ft$^3$.

**Step 3:**  *Determine the size of the sediment forebay:*

• For an Extended Detention Level #2 design, a minimum of 40% of the Tv should be in a permanent pool, such as a forebay, micropool, deep pool, or wetlands. For the sample project, assume a forebay is selected for pretreatment and to meet the permanent pool requirements.

• Compute the sediment forebay volume and determine its dimensions:

$$V_{forebay} = \left(\frac{40\%}{100\%}\right) \times 10,817 \text{ ft}^3 = 4,327 \text{ ft}^3$$

If forebay is 4 ft deep, then the area of the forebay (assuming vertical walls) is calculated as:

$$\frac{4,327 \text{ ft}^3}{4 \text{ ft}} = 1,082 \text{ ft}^2$$

• The shape of the forebay does not need to be square and should be shaped to fit the site.
The established design parameters for the basin:

- An Extended Detention Level 2 with a 36-hour drawdown time is required for water quality.
- Quantity control for the Q2 is required for channel protection. The required volume will be estimated in the design process.
- Quantity control for the Q10 is not required for flood protection, as a floodplain is mapped immediately below the outfall.
- The required design treatment volume for water quality is 10,817 ft³.
- The estimated forebay volume is 4,327 ft³.

**Step 4: Determining the Water Quality Volume Elevation**

- Required treatment volume (for Extended Detention Level 2) = 10,817 ft³
- From the Stage-Discharge-Storage table:
  - The design Tv required satisfied @ Elev. 423.25
  - Water depth @ Elev. 423.25 = 1.95 ft
  - Actual Volume = 11,051 ft³ @ Elev. 423.25

**Step 5: Determining the Extended Detention Orifice Size Required for Water Quality Using Method #2 Average Hydraulic Head (VDOT Preferred)**

- As 40% of the total treatment volume is contained in a permanent pool in the forebay, the remaining 60% must be detained for 36-hours. Calculate the remaining 60% volume for extended detention, V_ED:
  \[
  V_{ED} = \left( \frac{60\%}{100\%} \right) \times 11,051 \text{ ft}^3 = 6,631 \text{ ft}^3
  \]
- Compute the Q_avg for the remaining volume (V_ED) using the required 36-hour drawdown time:
  \[
  Q_{avg} = \frac{\text{Tv}}{\text{Time}} = \frac{6,631 \text{ ft}^3}{36 \text{ hr} \times (3600 \text{ sec/hr})} = 0.051 \text{ cfs}
  \]
- While the storage depth for the total treatment volume is h = 1.95 ft, the storage depth for 40% of the total treatment volume is at Elev. 422.05. This will be the invert for the extended detention orifice.
- Calculate the head for detaining 60% of the total storage volume for 36-hours:
  \[
  h = 423.25 - 422.05 = 1.20 \text{ ft}
  \]
  \[
  h_{avg} = \frac{1.2 \text{ ft}}{2} = 0.6 \text{ ft}
  \]
• Orifice sizing computations:

\[ A = \frac{Q_{\text{avg}}}{C \sqrt{2gh_{\text{avg}}}} = \frac{0.051}{0.6 \sqrt{2(32.2)(0.6)}} = 0.014 \text{ ft}^2 \]

• The depth (h) used in the orifice equation would normally be measured from the center of the orifice. Due to the small size of the water quality orifice it is acceptable to consider h as the depth to the invert of the orifice.

• From Table 11-10, use a 1½-inch orifice with an area = 0.012 ft². This is slightly smaller than calculated, so reservoir routing should be conducted to confirm the allowable peak discharge and required extended detention time are achieved.

**Step 6: Determining the Storage Volume and Orifice Size Required for Channel Protection Using Method #2 Average Hydraulic Head (VDOT Preferred)**

The uncontrolled post-development peak discharge for the site in the 2-yr storm is 29.6 cfs, with a \( t_c \) of 0.333 hr.

The allowable 2-yr storm peak discharge for channel protection is 20.5 cfs.

**Step 6a: Sizing the Storage Volume for the Channel Control**

Use the Modified Triangular Hydrograph method to estimate the volume needed:

\[ V_s = \frac{1}{2} T_b (Q_i - Q_o) \]

Where:
- \( V_s \) = Detention storage volume estimate, \( \text{ft}^3 \)
- \( Q_i = Q_{2\text{post}} = 29.6 \text{ cfs} \)
- \( Q_o = Q_{2\text{all}} = 20.5 \text{ cfs} \)
- \( T_b = 2 \times t_c \text{ (post-development)} = 2 \times 0.333 \text{ hr} = 0.666 \text{ hr} = 2,398 \text{ sec} \)

\[ V_s = \frac{1}{2} (2,398)(29.6 - 20.5) = 10,911 \text{ ft}^3 \]

• From the Stage-Discharge-Storage table:
  - The volume to Elev. 423.25 is reserved for water quality.
  - Determine the total volume required for water quality and channel protection:

\[ V_{\text{total}} = 11,051 \text{ ft}^3 + 10,911 \text{ ft}^3 = 21,962 \text{ ft}^3 \]

  - Based on the Stage-Storage table, the volume provided at Elev. 425.50 = 22,783 ft³, which is > 21,962 ft³ required
Step 6b: Determining the Weir Size Required for Channel Protection Using Method #2 Average Hydraulic Head (VDOT Preferred)

- Assume depth, \( h = 2.25 \) ft.

\[
h_{avg} = \frac{2.25 \text{ ft}}{2} = 1.13 \text{ ft}
\]

- Select a weir as the hydraulic control structure for channel protection. Weir sizing computation

Weir equation:

\[
Q_{avg} = CL(h_{avg})^{1.5}
\]

Where:
- \( Q = Q_{2all} = \) weir discharge, cfs
- \( C = \) weir coefficient of discharge (use 3.0 for sharp-crested weir)
- \( L = \) weir length, ft
- \( h_{avg} = \) average head, ft

Rearranged weir equation to solve for weir length:

\[
L = \frac{Q_{2all}}{C(h_{avg})^{1.5}} = \frac{20.5}{3.0(1.13)^{1.5}} = 5.69 \text{ ft}
\]

- Use a 5.7 ft long sharp crested weir for channel protection. Reservoir routing should confirm that the allowable peak discharge is not exceeded for the 2-yr storm.

- The invert for the weir should be at the Elev. 423.25 where the water quality storage volume ends and the channel protection storage volume begins.

Summary of basin design for water quality and water quantity:

- Water Quality
  - Extended Detention Level 2 (1.25 x T_v)
  - Treatment volume provided at Elev. 423.25
  - 40% of total treatment volume in permanent pool in a forebay
  - 60% of total treatment volume in extended detention for a minimum of 36-hours
o Extended Detention orifice 1½-inches diameter with invert at Elev. 422.05.

- Water Quantity
  - Channel Protection
    - Allowable peak discharge for 2-yr storm to manmade system is 20.5 cfs.
    - Detention volume achieved at Elev. 425.50
    - Control weir 5.7 ft long at Elev. 423.25
  - Flood Protection is not required as the system immediately below the outfall is a mapped FEMA floodplain

Step 7: Determining the Elevation and Sizing of an Auxiliary Spillway to Convey the 100-year Storm Using Method #2 Average Hydraulic Head (VDOT Preferred)

- The post-development peak discharge for the 100-yr storm was calculated to be 237 cfs.

- The criterion for a facility with an auxiliary spillway is conveyance of the 100-year peak rate of runoff with a freeboard of 1 ft.

- To be conservative, the designer can assume that the principal spillway orifice (extended detention) and weir (channel protection) are blocked during the 100-yr event and size the auxiliary spillway to pass the full 100-yr storm. Routing the 100-yr event will help the designer optimize the auxiliary spillway design.

Step 7a: Determine the Invert Elevation for the Auxiliary Spillway

- The elevation for the water quality and water quantity storage volumes is estimated to be at Elev. 425.50

- The auxiliary spillway invert should be designed to have a minimum freeboard of 1.0 ft above the water quantity storage volume = Elev. 425.5 + 1.0 ft = Elev. 426.5

Step 7b: Determine the Sizing for the Auxiliary Spillway Using Method #2 Average Hydraulic Head (VDOT Preferred)

Auxiliary (sometimes called “emergency”) spillways are generally designed as weirs with a fume or channel lined with appropriate material to resist erosion. The weir section is the hydraulic control structure.

- The final embankment height (including freeboard) has not been computed, but there are often site constraints that drive it. For the preliminary design, assume the storage depth for conveying the 100-yr storm = 3.00 ft.

\[
\frac{h_{avg}}{2} = 1.50 \text{ ft}
\]
• Weir sizing computation

Weir equation:

\[ Q_{\text{avg}} = CL(h_{\text{avg}})^{1.5} \]

Where:
\( Q = Q_{100} = \) weir discharge, cfs
\( C = \) weir coefficient of discharge (use 2.6 for broad crested weir)
\( L = \) weir length, ft
\( h_{\text{avg}} = \) average head, ft

Rearranged weir equation to solve for weir length:

\[ L = \frac{Q_{100}}{C(h_{\text{avg}})^{1.5}} = \frac{237}{2.6(1.50)^{1.5}} = 49.6 \text{ ft} \]

• Use a 50 ft long broad crested weir to safely convey the 100-yr storm without overtopping the embankment.

• The final top elevation for the embankment should be set 1 ft above the peak water surface elevation for the routed 100-yr storm. Reservoir routing should be used to confirm that the 100-yr storm peak discharge is conveyed in the auxiliary spillway with a minimum of 1 ft of freeboard to the top of the embankment.

• Note that the flume or channel below the broad crested weir must be designed to adequately convey the 100-yr storm to the system without causing erosion of the embankment or flooding of property above the BMP. If the BMP is located within a mapped 100-yr floodplain, then the final design must not have an adverse effect on the mapped floodplain and base flood elevations (where present).

Summary of basin design for water quality and water quantity:

• Water Quality
  - Extended Detention Level 2 (1.25 x Tv)
  - Treatment volume provided at Elev. 423.25
  - 40% of the total treatment volume (1.25 c Tv) is in a permanent pool in a forebay
  - 60% of total treatment volume (1.25 c Tv) is in extended detention for a minimum of 36-hours
  - Extended Detention orifice is 1½-inches diameter with invert at Elev. 422.05
• Water Quantity
  o Channel Protection
    ▪ Allowable peak discharge for 2-yr storm to manmade system is 20.5 cfs
    ▪ Detention volume achieved at Elev. 425.50
    ▪ Control weir 5.7 ft long at Elev. 423.25
  o Flood Protection is not required as the system immediately below the outfall is a mapped FEMA floodplain

• 100-yr Storm Conveyance
  o An auxiliary spillway is proposed to convey the 100-yr storm event
  o The invert for the auxiliary spillway is set 1 ft above the SWM storage at Elev. 426.50
  o The auxiliary spillway design consists of a broad crested weir with a length of 50 ft
  o The peak water surface elevation for the 100-yr storm is Elev. 426.50 + 3.00 ft = Elev. 429.50
  o The low point in the embankment should be a minimum of 1 ft above the peak water surface elevation for the 100-yr storm to provide freeboard = Elev. 429.50 + 1.00 ft = Elev. 430.50

**Step 8:** Route the Water Quality and Quantity Design Storms through the Basin

• To confirm that the water quality and quantity control criteria are satisfied for the design, a reservoir routing should be conducted for the basin and spillways to confirm that maximum peak discharges, minimum detention times, and adequate freeboard is provided.

• Use the Puls Method or another acceptable level-pool routing method to route the water quality storm, the 2-yr storm, and the 100-yr storm through the basin. Note that the water volume for an Extended Detention Level 2 is 60% of (1.25 x Tv), as 40% of (1.25 x Tv) is included in a permanent pool in the forebay.

• Use the results to confirm that:
  o The water quality storm is detained for a minimum of 36-hours for brim draw down.
  o The 2-yr storm peak runoff is less than or equal to the allowable peak discharge.
  o The maximum water surface elevation in the 100-yr storm is at least 1 ft below the invert of the embankment crest.

• Note that the initial design for a BMP/SWMF may not be the optimal design, providing too little or too much control, especially for extended detention and detention. This may not be apparent until the reservoir routing is complete. The designer should adjust the design to provide the control required for compliance with
the Part II.B stormwater management criteria, making sure that the requirements are met. A designer may choose to apply some factor of safety based upon professional judgment and documented in the design, but an excessively large design will increase the cost to construct and maintain the BMP/SWMF and should be avoided.
11.6 Part II C Design Criteria

11.6.1 Water Quality

SWM design for water quality control is to be in accordance with the latest revisions to the Virginia Stormwater Management Regulations. The regulations state that the water quality volume (WQV) is equal to the first 0.5" of runoff multiplied by the area of impervious surface associated with the land development project.

The following comments represent the significant points of the current regulations (the page numbers referenced are those in the DEQ SWM Handbook):

- SWM requirements for water quality control are “Performance Based” (9VAC25-870-96). The type of BMP required is determined by the comparison of the pre-developed, post-developed, and average cover conditions (% impervious area) of the site or stormwater planning area to classify the project as Situation 1, 2, 3, or 4. Unless otherwise defined by a local Chesapeake Bay Preservation Act program, the average cover condition is assumed to be 16% impervious.
  - Situation 1 occurs when the site pre-developed and developed conditions both result in a % imperviousness area < the average cover condition. No additional water quality controls are required as the low density development is considered the best management practice.
  - Situation 2 occurs when the site pre-developed % impervious area ≤ average cover condition, but the developed condition % impervious area is > average cover condition. In this situation, water quantity controls are provided to reduce the developed pollutant load to the pre-developed condition.
  - Situation 3 occurs when both the pre-developed and developed & impervious areas are > the average cover condition. In this case, controls are provided to reduce the developed pollutant loading to 10% below the pre-developed pollutant loading or to the pollutant loading associated with the average cover condition, whichever requires less pollutant removal.
  - Situation 4 occurs when the project site discharges to an existing stormwater Best Management Practice (BMP) and the existing BMP was designed to treat the developed project site.

- A BMP is selected from Table 11-1 below that provides the necessary removal rate to satisfy the Performance Based calculations and Situation applicable to the project.

- BMP requirements for water quantity control are determined by the ESC Regulation MS-19 (9VAC25-840-40) for adequate receiving channels.
• Extended Detention Basins and Enhanced Extended Detention Basins require 2 times the Water Quality Volume (WQV), or a total of 1” of runoff from the developed projected site within VDOT R/W or easement draining to the BMP.

• Extended Detention Basins and Enhanced Extended Detention Basins require a 30 hour drawdown time for the required WQV. If the required orifice size is found to be significantly less than 3”, an alternative outlet design or alternative BMP should be investigated for use, such as a linear facility that treats the first flush and allows larger storms to bypass. The calculation procedure for drawdown time and orifice sizing is shown on in the Virginia SWM Handbook Volume II, Pages 5-33 through 5-38. Alternative outlet designs for Extended Detention and Enhanced Extended Detention are presented in the Virginia SWM Handbook Volume I, Figures 3.07-3a to 3.08-3c, Pages 3.07-8 to 3.07-10.

• Suggested details for the Extended Detention Basin are shown on Pages 3.07-4 and 5 (Virginia SWM Handbook). The riprap-lined low-flow channel through the basin is not recommended due to maintenance concerns.

• Suggested details for the Enhanced Extended Detention Basin are shown on Pages 3.07-6 and 7 (Virginia SWM Handbook). The geometric design may need to be more symmetrical than that shown in order to facilitate construction of the basin to the dimensions needed.
Table 11-7. BMP Selection Table

<table>
<thead>
<tr>
<th>Water Quality BMP</th>
<th>Treatment Volume</th>
<th>Target Phosphorus Rem. Eff.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vegetated filter strip</td>
<td></td>
<td>10%</td>
</tr>
<tr>
<td>Grassed swale</td>
<td></td>
<td>15%</td>
</tr>
<tr>
<td>Constructed wetlands</td>
<td>2xWQV</td>
<td>20%</td>
</tr>
<tr>
<td>Extended detention</td>
<td>2xWQV</td>
<td>35%</td>
</tr>
<tr>
<td>Retention basin I</td>
<td>3xWQV</td>
<td>40%</td>
</tr>
<tr>
<td>Bioretention basin</td>
<td>1xWQV</td>
<td>50%</td>
</tr>
<tr>
<td>Bioretention filter</td>
<td>1xWQV</td>
<td>50%</td>
</tr>
<tr>
<td>Extended detention enhanced</td>
<td>2xWQV</td>
<td>50%</td>
</tr>
<tr>
<td>Retention basin II</td>
<td>4xWQV</td>
<td>50%</td>
</tr>
<tr>
<td>Infiltration</td>
<td>1xWQV</td>
<td>50%</td>
</tr>
<tr>
<td>Bioretention basin</td>
<td>2xWQV</td>
<td>65%</td>
</tr>
<tr>
<td>Bioretention filter</td>
<td>2xWQV</td>
<td>65%</td>
</tr>
<tr>
<td>Sand filter</td>
<td>2xWQV</td>
<td>65%</td>
</tr>
<tr>
<td>Infiltration</td>
<td>2xWQV</td>
<td>65%</td>
</tr>
<tr>
<td>Retention basin III with aquatic bench</td>
<td>4xWQV</td>
<td>65%</td>
</tr>
<tr>
<td>Manufactured BMP Systems</td>
<td></td>
<td>20%</td>
</tr>
<tr>
<td>Hydrodynamic Structures *</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Manufactured BMP Systems</td>
<td></td>
<td>50%</td>
</tr>
<tr>
<td>Filtering Structures *</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Filterra™ Bioretention Filter System **</td>
<td></td>
<td>74%</td>
</tr>
</tbody>
</table>

*Innovative or alternate BMPs not included in this table may be allowed at the discretion of DEQ and VDOT.
**See Technical Bulletin No.6 in the Virginia SWM Handbook.

11.6.2 Water Quantity

The Virginia Erosion and Sediment Control Regulations Minimum Standard 19 (9VAC25-840-40 section 19) and Virginia Stormwater Management Program Regulations (9VAC25-870-97) shall govern water quantity control. Linear development projects shall not be required to control post-developed stormwater runoff for flooding, except in accordance with a watershed or regional SWM plan (9VAC25-870-98). The following general criteria apply:

- Determination of flooding and channel erosion impacts to receiving streams due to land-disturbing activities shall be measured at each point of discharge from the land disturbance and such determination shall include any runoff from the balance of the watershed that also contributes to that point of discharge.
- The specified design storms shall be defined as either a 24-hour storm using the rainfall distribution recommended by the U.S. Department of Agriculture’s Natural Resources Conservation Service (NRCS) when using NRCS methods or as the
storm of critical duration that produces the greatest required storage volume at the site when using a design method such as the Modified Rational Method.

- For purposes of computing runoff, all pervious lands in the site shall be assumed prior to development to be in good condition (if the lands are pastures, lawns, or parks), with good cover (if the lands are woods), or with conservation treatment (if the lands are cultivated); regardless of conditions existing at the time of computation.

- Construction of SWMFs or modifications to channels shall comply with all applicable laws, regulations, and ordinances. Evidence of approval of all necessary permits shall be presented.

- Pre-development and post-development runoff rates shall be verified by calculations that are consistent with good engineering practices.

- Outflows from a SWMF or stormwater conveyance system shall be discharged to an adequate channel.

- Hydrologic parameters shall reflect the ultimate land disturbance and shall be used in all engineering calculations.

- Natural channel characteristics shall be preserved to the maximum extent practicable.

- Pre-development conditions should be that which exist at the time the road plans are approved for R/W acquisition.

- An adequate receiving channel is required for stormwater outflows from all projects with more than 10,000 ft² of land disturbance.

- The receiving channel at a pipe or storm drain outlet should be analyzed by use of a 2-yr storm for natural channel capacity and erosion protection; while the 10-yr storm shall be used for man-made channel capacity, with the 2-yr storm for man-made channel erosion protection.

### 11.6.3 Compensatory Treatment

Compensatory treatment for water quality requirements (over treating at one outfall in a local watershed to compensate for not treating at an adjacent outfall in the same watershed) can be considered for meeting the requirements provided:

- The SWMFs at the treated outfall are designed to account for the water quality volumes for those areas where SWMFs are determined to be impractical or unacceptable.

- The downstream impacts, if any, which would occur as a result of discharging untreated runoff at the untreated outfall, must be documented. The documentation should note that compensating treatment of SWMFs has been incorporated.
• The channel at the untreated outfall must be analyzed to determine its adequacy to convey the additional runoff in accordance with the requirements of MS-19 of the VESCR and any necessary channel protection or improvements must be provided.

• The project is to be reviewed either by the State Hydraulics Engineer or his assistant when the project reaches the Field Inspection stage.

11.6.4 Embankment (Dam)

The following details are to be incorporated into the design of dams for VDOT SWM basins.

• The design of the dam and the basin should provide only a relatively shallow depth of ponded water in order to prevent the basin from being a hazard. It is desirable to have the ponded depth no more than about 2’ for water quality and about 4’ for the 10-yr storm (Q_{10}) quantity control.

• Foundation data for the base of the dam should be secured from the Materials Division for all SWM basins in order to determine if the native material will support the dam and not allow ponded water to seep under the dam. An additional boring near the center of the basin should also be requested if:
  o Excavation from the basin may, potentially, be used to construct the dam, or
  o There is potential for rock to be encountered in the area of excavation, or
  o A high water table is suspected that may alter the performance of the SWM basin.

• For large basins, more than one boring for the dam and one boring for the area of the basin shall be needed. The number and locations of the borings are to be determined by the VDOT SWM Plan Designer/Hydraulics Engineer and/or the VDOT District Materials Engineer.

• The foundation material under the dam and the material used for the embankment of the dam should be an ML or CL Type in accordance with the Unified Soil Classification System (ASTM D2487) (type A-4 or finer in accordance with the AASHTO Classification System M145) and/or meet the approval of the Materials Division. If the native material is not adequate, the foundation of the dam is to be undercut a minimum of 4’ or to a depth recommended by the Materials Division. The backfill and embankment material must meet the above soil classification, or the design of the dam may incorporate a trench lined with a membrane such as bentonite penetrated fabric, HDPE, or LDPE liner, to be approved by the Materials Division.

• The pipe culvert under or through the dam is to be concrete pipe with rubber gaskets, and the joint must be leak-resistant as per AASHTO PP-63, and shall be included in the Department’s Approved List No. 14.
• The foundation data for the SWM basin should be requested by the VDOT SWM Plan Designer/Hydraulics Engineer at the same time that the request for culvert foundation data is initiated.

• A concrete cradle is to be used under the pipe to prevent seepage through the dam barrel. The concrete cradle extends from the riser or inlet end of the pipe to the outlet of the pipe, and extend the full length of the pipe. For details of the concrete cradle, see Std. SWM-DR of the 2016 VDOT Road & Bridge Standards.

• If the height of the dam is greater than 15', or if the basin includes a permanent water pool, the design of the dam is to include a homogenous embankment with seepage controls or zoned embankment or similar design conforming to DEQ design standards for earth dams and is to be approved by the Materials Division.

• The minimum top width should be 10’. This helps facilitate both construction and maintenance and allows the embankment to be used for access. The side slopes should also be a minimum of 3:1, to permit mowing and maintenance access. The design of the design should include a seepage analysis as well as a slope stability analysis (minimum Factor of Safety of 1.5 (checked with normal pool level on storage side and rapid drawdown conditions). A typical cross-section of a SWM basin dam is shown in Figure 11-7.

 Figure 11-7. Typical SWM Basin Dam

• Construction of SWM impoundment structures within a FEMA designated 100-yr flood plain shall be avoided whenever possible. When this is unavoidable, a thorough review shall be made to ensure that the SWMF will operate effectively for its intended purpose during the passage of the 10-yr flood event on the flood plain. All SWMF construction within a designated 100-yr flood plain shall be in compliance with all applicable regulations under the FEMA’s National Flood Insurance Program. The SWMF shall be reviewed for any potential impacts to the 100-yr flood event characteristics of the floodplain and designed for structural stability during the passage of the 100-yr flood event on the flood plain.
• Impounding structures (dams) that are not covered by the Virginia Dam Safety Regulations shall be designed in accordance with this manual and reviewed for floodplain impacts during the passage of the 100-yr storm event.

### 11.6.5 Basin Grading

The layout and grading of a basin has a major influence on how effective the basin will be in removing pollutants. The designer should try to blend the basin into the surrounding topography while keeping several criteria in mind. First, the basin should be designed and graded so that the desirable length-to-width ratio is about 3:1 with a minimum ratio of 2:1. This helps prevent short-circuiting of the basin’s storage areas. The basin’s longest dimension should run parallel to the contours, which helps minimize cut and fill. The wider dimension should also be located at the outlet end. If the length to width ratio is less than about 2:1, and there is concern that the velocity of flow through the basin is high, the designer should consider using baffles within the basin to reduce velocity and prevent short-circuiting by increasing travel length. Baffles should be constructed of a pervious type material such as snow fence, rather than earth berms, which do not reduce the velocity.

• Basin side slopes should be no steeper than 3:1 to permit mowing and maintenance access
• The bottom slope of dry detention basins should be no more than 2% and no less than 0.5%
• Where safety is a concern, and fencing is not practical, use 4:1 side slope
• The depth of basin from the bottom to the primary outflow point (crest of riser, or invert of weir) should be no more than 3’ if possible, in order to reduce the hazard potential. If the depth needs to be more than 3’, fencing should be considered and a safety ledge considered around the perimeter to prevent people from falling in, and to facilitate their escape from the basin.
• Construction of SWMFs within a sinkhole is prohibited. If SWMFs are required along the periphery of a sinkhole, the design of such facilities shall comply with the guidelines in Chapter 5 of this manual and the DEQ’s Technical Bulletin No. 2 (Hydrologic Modeling and Design in Karst) and applicable sections of the Virginia SWM Handbook.
• Design of any SWMFs with permanent water features (proposed or potential) located within five (5) miles of a public use or military airport is to be reviewed and coordinated in accordance with Section A-6 of the VDOT Road Design Manual.
Table 11-8 summarizes the design criteria for dry and wet basin designs:

<table>
<thead>
<tr>
<th>Design Requirement</th>
<th>Dry Basin Design</th>
<th>Wet Basin Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quality control</td>
<td>Detain WQV for 30-hour minimum</td>
<td>Permanent pool volume is a function of the BMP selected (see Table 11-7)</td>
</tr>
<tr>
<td>Quantity control</td>
<td>Control 2- and 10-yr (when applicable) peak flows and maintain a non-erosive outfall velocity</td>
<td>Control 2- and 10-yr (when applicable) peak flows and maintain a non-erosive outfall velocity</td>
</tr>
<tr>
<td>Shape</td>
<td>3:1 length-to-width ratio; wedge shaped (wider at the outlet)</td>
<td>3:1 length-to-width ratio*; wedge shaped (wider at the outlet); permanent pool depth to 3’ max, if possible</td>
</tr>
<tr>
<td>Safety</td>
<td>Fence around basin if depth is greater than 3’; shallow safety ledge around basin. See following notes on fencing. (Section 11.3.8)</td>
<td>Fence around basin if depth is greater than 3’; shallow safety ledge around basin. See following notes on fencing. (Section 11.3.8)</td>
</tr>
<tr>
<td>Other Considerations</td>
<td>3:1 side slopes for easy maintenance access; 0.5-2% bottom slope to prevent ponding; sediment forebay to reduce maintenance requirements</td>
<td>3:1 side slopes for easy maintenance access; sediment forebay to reduce maintenance requirements; provide valve to drain pond for maintenance</td>
</tr>
</tbody>
</table>

* If this is not possible, every effort should be made to design the basin with no less than a 2:1 length to width ratio.

### 11.6.6 Sediment Forebay

A sediment forebay is a settling basin or plunge pool constructed at the incoming discharge points of a stormwater BMP. The purpose of a sediment forebay is to allow sediment to settle from the incoming stormwater runoff before it is delivered to the balance of the BMP. It is an essential component of most impoundment and infiltration BMPs including retention, detention, extended-detention, constructed wetlands, and infiltration basins. A sediment forebay also helps to isolate the sediment deposition in an accessible area, which facilitates BMP maintenance efforts.

A sediment forebay shall be located at each inflow point in the stormwater BMP. Storm drain piping or other conveyances may be aligned to discharge into one forebay or several, as appropriate for the particular site. Sediment forebays should always be installed in a location that is accessible by maintenance equipment. Figure 11-8 shows a typical sediment forebay.
In order to facilitate maintenance activities, sediment forebays are to be incorporated into the design of Extended Detention Basins and Extended Detention Basins Enhanced. The volume of the forebay should be $0.1'' - 0.25'' \times$ the impervious area treated by the facility or 10% of the required detention volume. See Pages 3.04-1 through 5 (SWM Handbook) for details. Where the overflow (emergency) spillway is incorporated as part of the dam/embankment, it shall be stabilized utilizing rip rap, concrete, or other non-erodible material (such as EC-3).

Figure 11-8. Typical Sediment Forebay Plan and Section
11.7  Part II C Design Concepts

11.7.1  Water Quality

Control of stormwater quality offers the following potential benefits:

- Control of sediment deposition
- Improved water quality through stormwater filtration
- Settling out of roadway runoff pollutants

Ideally, the pollutant removal mechanism should dictate the treatment volume or storm frequency for water quality BMPs. The sizing of BMPs, which uses gravitational settling of pollutants as the removal mechanism, can be based on a volume of runoff. The Virginia Stormwater Management Regulations require that the first flush of runoff be captured and treated to remove pollutants. The first flush, or water quality volume (WQV) is generally defined as the first one-half inch of runoff from impervious surfaces. Table 11-1 specifies the required treatment volume for each type of BMP based upon the WQV.

One of the first considerations in selecting a stormwater BMP is the functional goal of the BMP. The main components of SWM are: quality, stream channel erosion, and stormwater quantity or flooding. Any one or a combination of these components will dictate the functional goal of the BMP. In general, stormwater BMPs can be categorized into water quality BMPs and water quantity (stream channel erosion and flooding) BMPs.

Table 11-9 provides a general categorization of BMPs by functional goal. Note that some BMPs can be designed to satisfy both quality and quantity goals while others are specifically suited for only one.

The use of some BMPs is limited by site or watershed feasibility factors such as environmental impacts, drainage area or watershed size, and topographic constraints.

The BMPs designed for water quality control provide varying levels of pollutant removal and are suitable for specific development densities. Table 11-1 also provides a generic list of water quality BMPs and their target phosphorus removal efficiency. Phosphorus is the keystone pollutant targeted for removal in Virginia.
Table 11-9. Functional Goals of Stormwater BMPs

<table>
<thead>
<tr>
<th>Stormwater BMP</th>
<th>Quality</th>
<th>Stream Channel Erosion</th>
<th>Quantity/Flooding</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vegetated filter strip</td>
<td>+++</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grasses Swale (w/check dams)</td>
<td>+++</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Constructed wetlands</td>
<td>+++</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Extended detention</td>
<td>++</td>
<td>+++</td>
<td>+</td>
</tr>
<tr>
<td>Extended detention enhanced</td>
<td>+++</td>
<td>++</td>
<td>+</td>
</tr>
<tr>
<td>Bioretention</td>
<td>+++</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Retention basin</td>
<td>+++</td>
<td>++</td>
<td>+++</td>
</tr>
<tr>
<td>Sand filter</td>
<td>+++</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Infiltration</td>
<td>+++</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Infiltration Basin</td>
<td>++</td>
<td></td>
<td>+</td>
</tr>
<tr>
<td>Detention</td>
<td>++</td>
<td></td>
<td>+</td>
</tr>
<tr>
<td>Manufactured BMPs (Water Quality Structures)</td>
<td>+++</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Legend:  
+++ Primary functional goal  
++ Potential secondary functional goal  
+ Potential secondary functional goal with design modifications or additional storage


11.7.2 Water Quantity

Controlling the quantity of stormwater can provide the following potential benefits:

- Prevention or reduction of peak runoff rate increases caused by urban development
- Decrease downstream channel erosion
- Mitigation of downstream drainage capacity problems
- Recharge of groundwater resources
- Reduction or elimination of the need for downstream outfall improvements
- Maintenance of historic low flow rates by controlled discharge from storage

One concept that can be used to control the quantity of stormwater is to consider the use of offsite improvements or regional SWMFs.

11.7.3 Extended Detention vs. Retention

When evaluating the relative merits of extended dry detention versus wet retention basins, there are several factors to consider. Extended detention basins generally require much less storage volume than retention basins. However, wet basins generally provide more pollutant removal and are usually considered an amenity if designed properly. Wet basins require a reliable water/groundwater source and sometimes a significant size drainage area in order to maintain the desired permanent pool level and to prevent the basin from being objectionable. A typical extended detention basin plan
11.7.4 Detention Time

Settling or sedimentation is limited to particulate pollutants that drop out of the water column by means of gravitational settling. Pollutants attach themselves to heavier sediment particles or suspended solids and settle out of the water. Laboratory and field studies indicate that significant settling of urban pollutants occurs in the first 6 to 12 hours of detention. Figure 11-9 shows removal rate versus detention time for selected pollutants.

![Removal Rates vs. Detention Time](image)

Figure 11-9. Removal Rates vs. Detention Time

The brim drawdown requirement for water quality for extended detention design is 30 hours. The additional time is required to allow for ideal settling conditions to develop within the SWMF. In addition, the added time will allow for settling of smaller particle sizes and nutrients, as well as increasing the opportunity for biological processes to take place. Stormwater BMPs that utilize settling are usually suited for dual purposes that include providing storage volume for peak rate control, channel erosion, and flood control.
11.7.5 Release Rates

Control structure release rates are usually designed to approximate pre-developed peak runoff rates for the 2- and 10-yr design storms with an emergency spillway capable of handling the 100-yr peak discharge. Design calculations are required to demonstrate that the post-development release rates for the 2- and 10-yr design storms are equal to or less than the pre-development release rates. If it can be shown that the 2- and 10-yr design storms are controlled, then runoff from intermediate storm frequencies are assumed to be adequately controlled as well.

Multi-stage control structures may be required to control runoff from both the 2- and 10-yr storms. This can be accomplished through the use of orifices and weirs and is discussed in Section 11.4.7.

11.7.5.1 Channel Erosion Control – Q₁ Control

Water quantity control for the 1-year design storm (in lieu of the 2-yr design storm required by MS-19) may be needed if there is existing or anticipated erosion downstream. Control of the 1-year design storm requires detaining the volume of runoff from the entire drainage area and releasing that volume over a 24-hour period.

When the 1-year design storm is detained for 24 hours there will be no need to provide additional or separate storage for the WQV if it can be demonstrated that the WQV will be detained for approximately 24 hours. The control of the 1-year design storm may require a basin size that is 1.5 to 2 times larger than a basin used to control the increase in runoff from a 2- or 10-yr design storm.

Properties and receiving waterways downstream of any land-disturbing activity shall be protected from erosion and damage due to changes in stormwater flows and hydrologic characteristics, including but not limited to, changes in runoff volume, velocity, frequency, duration, and peak flow rate.

Requirements for stream channel erosion control shall be governed by the Virginia ESC Regulation MS19 for an adequate receiving channel for stormwater discharges.

Receiving channels shall be reviewed for adequacy based upon the following criteria:

- Natural channels shall be analyzed by the use of a post-development peak discharge from a 2-yr storm to verify that stormwater will not cause erosion of the channel bed and banks, and
- All previously constructed man-made channels shall be analyzed by the use of a post-development peak discharge from a 2-yr storm to verify that the stormwater will not cause erosion of the channel bed or banks.

When utilizing an existing culvert or storm sewer pipe as the outfall for stormwater runoff from the project site, the receiving channel at the outlet end of the existing culvert
or storm sewer pipe shall be analyzed for adequacy based on the type of receiving channel (natural or man-made).

If existing natural or previously constructed man-made receiving channels are not adequate, then one of the following measures must be implemented:

- Improve the receiving channel to a condition where the post-development peak runoff rate from a 2-yr storm will not cause erosion to the channel bed or banks or to the point where the drainage area within the channel complies with the requirements, or
- Develop a site design that will not cause the pre-development peak runoff rate from a 2-yr storm to increase (i.e., post development 2 year peak discharge is equal to or less than the pre-development 2 year peak discharge) when runoff discharges into a natural channel or will not cause the post-development peak runoff rate from a 10-year storm to increase (i.e., post development 10-yr peak discharge is equal to or less than pre-development 10-yr peak discharge) when runoff discharges into a man-made channel, or
- Provide a combination of channel improvements, stormwater detention or other measures to prevent downstream erosion.

Where determined necessary by the SWM Plan Designer or requested by DEQ, water quantity control for the 1-year storm may be required if there is existing or anticipated erosion concerns downstream of the project site. Such determination or request shall be made prior to the public participation phase of the project (or other such phase when no public participation process is required). Control of the 1-year storm requires detaining the volume of runoff from the entire drainage area and releasing that volume over a 24-hour period. See the Virginia SWM Handbook, Volume I, Page 1-23 and Volume II, Pages 5-38 thru 5-41 for additional information.

Post-development conditions for both offsite and onsite areas shall be those that exist at the time when the final receiving channel analysis is performed. All land cover shall be assumed to be in “good” condition regardless of actual conditions existing at the time the analysis is performed.

Post-development conditions for offsite areas shall be determined the same as for Pre-development conditions. Post-development conditions for the on-site areas shall be determined based on the proposed project plans and any known future plans of development within the project site.

One Percent (1%) Rule - If it can be demonstrated that the total drainage area to the point of analysis within the receiving channel is 100 times greater than the contributing drainage area from within the project site, the receiving channel may be considered adequate, with respect to the stability (erosion) requirements, without further analysis.
11.7.5.2 Flooding
Properties and receiving waterways downstream of any land-disturbing activity shall be protected from localized flooding due to changes in stormwater flows and hydrologic characteristics including, but not limited to, changes in runoff volume, velocity, frequency, duration, and peak flow rate.

For non-linear projects, the 10-yr post-development peak rate of runoff from the site shall not exceed the 10-yr pre-development peak rate of runoff. For linear projects, requirements for downstream flooding control shall be governed by the Virginia ESC Regulation MS19 for adequate receiving channel for stormwater discharges.

Receiving channels shall be reviewed for adequacy based upon the following criteria:

- Natural channels shall be analyzed by the use of a post-development peak discharge rate from 2-yr storm to verify that stormwater will not overtop the channel banks, and

- All previously constructed man-made channels shall be analyzed by the use of a post-development peak discharge rate from a 10-yr storm to verify that the stormwater will not overtop the channel banks, and

- Existing culvert and storm sewer systems, utilized as stormwater outfalls for the development site, shall be analyzed by the use of a post-development peak discharge rate from a 10-yr frequency storm to verify that the stormwater will be contained within the pipe or storm sewer system.

When utilizing an existing culvert or storm sewer pipe as the outfall for stormwater runoff from the project site, the receiving channel at the outlet end of the existing culvert or storm sewer pipe shall be analyzed for adequacy based on the type of receiving channel (natural or man-made).

If existing natural or previously constructed man-made receiving channels or existing culvert or storm sewer pipe systems are not adequate, then one of the following measures must be implemented:

- Improve the channel to a condition where the post-development peak runoff rate from a 10-yr storm will not overtop the channel banks or to the point where the drainage area within the channel complies with the requirements, or

- Improve the culvert or storm sewer system to a condition where the post-development peak runoff rate from a 10-yr storm is contained within the appurtenances, or
• Develop a site design that will not cause the pre-development peak run-off rate from a 2-yr storm to increase (i.e., post development 2-yr peak discharge is equal to or less than pre-development 2-yr peak discharge) when runoff from the site discharges into a natural channel or will not cause the pre-development peak runoff rate from a 10-yr storm to increase (i.e., post development 10-yr peak discharge is equal to or less than pre-development 10-yr peak discharge) when runoff from the site discharges into a man-made channel or a culvert/storm sewer system, or

• Provide a combination of channel/culvert/storm sewer system improvements, stormwater detention or other measures in order to prevent downstream flooding.

One Percent (1%) Rule - If it can be demonstrated that the total drainage area to the point of analysis within the receiving channel is 100 times greater than the contributing drainage area from within the project site, the receiving channel may be considered adequate, with respect to the flooding requirements, without further analysis.

Pre-development conditions for both the offsite and onsite areas shall be those that exist at the time when the final receiving channel analysis is performed. All land cover shall be assumed to be in good condition regardless of actual conditions existing at the time the analysis is performed.

Post-development conditions for offsite areas shall be determined the same as for Pre-development conditions. Post-development conditions for the on-site areas shall be determined based on the proposed project plans and any known future plans of development within the project site.

11.7.5.3 Water Quality Control

Unless otherwise exempt, a water quality control plan that provides compliance with the VSMP Regulations Part II C technical criteria shall be developed for each grandfathered VDOT land-disturbing activity exceeding the land disturbance thresholds noted in IIM-LD-195 (see Section 11.5.9 for additional information on grandfathered projects).

Compliance with the water quality criteria may be achieved by applying the performance-based criteria (see below for discussion and application of this methodology). Additional discussion and application of this methodology can also be found in Volumes I and II of the Virginia SWM Handbook.

Evaluation of water quality requirements may be performed considering the site area at each individual stormwater discharge (outfall) point from the proposed land-disturbing-activity/project or may be performed considering the site area for the entire limits of the proposed land-disturbing activity/project.

Where the proposed land-disturbing activity/project drains to more than one 6th Order HUC, the required pollutant load reductions shall be applied independently within each HUC unless reductions are proposed to be achieved under a project specific or a comprehensive SWM plan developed in accordance with Section 9VAC25-870-92 of the VSMP Regulations.
Performance-Based Criteria

- The calculated post-development pollutant load from the site shall be compared to the calculated pre-development pollutant load from the site based upon the average land cover condition or the existing site condition as related to the site’s percent impervious.

- The site’s percent impervious shall be determined as follows:
  - For pre-development conditions - The amount of pre-development impervious area within the site divided by the total area of the site times 100.
  - For post-development conditions - The amount of post-development impervious area within the site divided by the total area of the site times 100.

- A BMP shall be located, designed, and maintained to achieve the target pollutant removal efficiencies specified in Table 11-1 for the purposes of reducing the post-development pollutant load from the site to the required level based upon the following four applicable land development situations for which the performance-based criteria apply:
  - Situation 1 consists of land-disturbing activities where the pre-development percent impervious cover of the site is less than or equal to the average land cover condition (16%) and the proposed improvements will create a total post-development percent impervious cover of the site which is less than the average land cover condition (16%).
    - Water Quality Requirement: No reduction in the post-development pollutant discharge from the site is required.
  - Situation 2 consists of land-disturbing activities where the pre-development percent impervious cover of the site is less than or equal to the average land cover condition (16%) and the proposed improvements will create a total post-development percent impervious cover of the site which is greater than the average land cover condition (16%).
    - Water Quality Requirement: The post-development pollutant discharge from the site shall not exceed the pre-development pollutant discharge from the site based on the average land cover condition (16%).
  - Situation 3 consists of land-disturbing activities where the pre-development percent impervious cover of the site is greater than the average land cover condition (16%).
    - Water Quality Requirement: The post-development pollutant discharge from the site shall not exceed (a) the pre-development pollutant discharge from the site less 10% or (b) the pollutant discharge based on the average land cover condition (16%), whichever is greater.
  - Situation 4 consists of land-disturbing activities where the pre-development impervious cover of the site is served by an existing BMP that addresses water quality.
Water Quality Requirement: The post-development pollutant discharge from the site shall not exceed the pre-development pollutant discharge from the site based on the existing percent impervious cover of the area being served by the existing BMP. The existing BMP shall be shown to have been designed and constructed in accordance with proper design standards and specifications, and to be in proper functioning condition.

When the applicable percent impervious cover of the site is less than the statewide “average land cover condition” of 16%, no water quality BMPs are required. (Exception - Where a locality has established a lower “average land cover condition” than the statewide average, the provisions of IIM-LD-195 shall govern.)

The applicable post-development percent impervious cover of the site shall be as follows:

- For linear development projects:
  - “Old” criteria - The net increase in impervious area of the site (total post-development impervious area of the site minus the total pre-development impervious area of the site) divided by the total post-development area of the site times 100.
  - “New” criteria – See Performance-Based Criteria
- For Non-Linear Projects – See Performance-Based Criteria

The water quality volume for any required BMP shall be based on the total post-development impervious area draining to the BMP from within the R/W of the proposed project/activity and from within any VDOT R/W adjacent to the proposed project/activity.

Alternative BMPs
BMPs included on the Virginia SWM BMP Clearing House website [https://www.swbmp.vwwrc.vt.edu/](https://www.swbmp.vwwrc.vt.edu/) may be used with the Performance-Based water quality criteria. Unless otherwise approved by DEQ, the maximum removal efficiency allowed for the BMP will be that shown for phosphorus removal by treatment and any removal efficiency associated with phosphorus removal by runoff reduction will not be allowed.

Other alternative BMPs not included in Table 11-1 or the Virginia SWM BMP Clearing House website may be allowed at the discretion and approval of DEQ.

Approval to use alternative BMPs is to be coordinated between the VDOT District or Central Office SWM Plan Designer and the DEQ Regional Stormwater Program Manager. The VDOT State Stormwater Management Program Administrator and the DEQ Central Office Director of the Office of Water Permits shall be copied on any correspondence related to a request for approval of the use of any alternative BMPs.
Use of Low Impact Development (LID) and Better Site Design (BSD) practices are encouraged to the maximum extent practicable in order to reduce the stormwater runoff impacts of the proposed development. LID practices include, but are not limited to, the preservation/protection of riparian buffers, wetlands, steep slopes, mature trees, flood plains, woodlands and highly permeable soils. BSD practices include, but are not limited to, reduction of impervious cover, conservation of natural areas and the more effective use of pervious areas to treat stormwater runoff.

When the 1-year storm is detained for 24 hours, there will be no need to provide additional or separate storage for the WQV if it can be demonstrated that the WQV will be detained for approximately 24 hours.

Offsite Water Quality Compliance Options
Where the water quality requirements for the land development activity cannot be satisfied onsite, offsite options may be used to achieve compliance with the requirements of the VSMP Regulations.

Offsite compliance options allowed for use in meeting required phosphorus load reductions include one or more of the following:

- Offsite controls utilized in accordance with a comprehensive SWM plan adopted pursuant to Section 9VAC25-870-95 of the VSMP regulations for the local watershed within which a project is located (e.g., a regional SWMF).
- A locality pollutant loading pro rata share program established pursuant to § 15.2-2243 of the Code of Virginia or similar local funding mechanism (e.g., a stream restoration fund).
- The Nonpoint Nutrient Offset Program established pursuant to § 62.1-44.15:35 of the Code of Virginia (i.e., the purchase of phosphorus credits from a Nutrient Credit Bank).
- Any other offsite option approved by DEQ.
- When VDOT has additional properties located within the same 6th Order HUC or upstream HUC of the land-disturbing activity or within the same watershed as determined by DEQ, SWMFs located on those properties may be utilized to meet the required phosphorus load reductions from the land-disturbing activity.

VDOT may utilize offsite options if the project meets any one of the following conditions:

- The activity will disturb less than five acres of land (100% offsite compliance allowed).
- The activity’s post-developed phosphorus load reduction requirement is less than 10 pounds per year (100% offsite compliance allowed).
- At least 75% of the required phosphorus load reductions can be achieved onsite (up to 25% offsite compliance allowed).
If at least 75% of the activity’s required phosphorus load reductions cannot be achieved onsite, then the required phosphorus load reductions may be achieved, in whole or in part, through the use of offsite compliance options (up to 100% offsite compliance may be allowed) provided VDOT can demonstrate to the satisfaction of the DEQ that:

- Alternative site designs have been considered that may accommodate onsite BMPs, and
- Onsite BMPs have been considered in alternative site designs to the maximum extent practicable, and
- Appropriate onsite BMPs will be implemented, and
- Full compliance with post-development nonpoint nutrient runoff compliance requirements cannot practicably be met onsite,

Offsite options shall not be allowed:

- Unless the selected offsite option achieves the necessary phosphorus load reductions prior to the commencement of the construction of the proposed project. Where the offsite option will be constructed as a part of the proposed VDOT project, the offsite option must be completed and functional prior to the completion of the VDOT project, or
- In violation of local water quality-based limitations at the point of discharge that are consistent with the determinations made pursuant to a TMDL Implementation Plan, contained in a MS4 Program Plan approved by DEQ or as otherwise may be established or approved by DEQ.

Non-structural practices including, but not limited to, minimization of impervious areas and curbing requirements, open space acquisition, floodplain management, and protection of wetlands may be utilized as appropriate in order to at least partially satisfy water quality requirements. Approval to use such non-structural measures is to be secured in advance from DEQ and is to be coordinated between the VDOT State Stormwater Management Program Administrator and the DEQ Central Office Director of the Office of Water Permits.
11.7.6 Outlet Hydraulics

11.7.6.1 Orifice
An orifice is an opening into a standpipe, riser, weir, or concrete structure. Openings smaller than 12 inches may be analyzed as a submerged orifice if the headwater to depth ratio (HW/D) is greater than 1.5. An orifice for water quality is usually small (less than 6 inches) and round. VDOT has determined that the orifice is less prone to clogging when located in a steel plate rather than a 6- or 8-inch hole in a concrete wall. Details are shown in the latest version of VDOT Location & Design Instructional & Informational Memorandum IIM-LD-195. For square-edged entrance conditions, the orifice equation is expressed as:

\[ Q = CA\sqrt{2gh} \]  \hspace{1cm} (11.1)

Where:
Q = Discharge, cfs
C = Orifice entrance coefficient (generally 0.6)
A = Cross-sectional area of orifice, ft²
g = Acceleration due to gravity, 32.2 ft/s²
h = Head on orifice, ft.

11.7.6.2 Weirs
The most common type of weir associated with SWM is the broad-crested weir as is defined by Equation 11.2:

\[ Q = CLH^{3/2} \]  \hspace{1cm} (11.2)

Where:
Q = Discharge, cfs
C = Broad-crested weir coefficient (Range from 2.67 to 3.33 and is generally assumed to be 3.0.) For additional information, refer to King and Brater, Handbook of Hydraulics, 1976, which lists coefficients and instructions on determining an appropriate coefficient.
L = Broad-crested weir length, ft.
H = Head above weir crest, ft.

If the upstream edge of a broad-crested weir is rounded so as to prevent contraction and if the slope of the crest is as great as the headless due to friction, flow will pass through critical depth at the weir crest; this gives the maximum entrance coefficient (C) of 3.00. For sharp corners on the broad-crested weir; however, a minimum (C) of 2.67 should be used. The designer should also check to make certain the weir or orifice is not submerged by the downstream tailwater.
11.7.6.3 Types of Outlet Structures

11.7.6.3.1 General
Outlet structures typically include a principal spillway and an emergency overflow, and must accomplish the design functions of the facility. Outlet structures can take the form of combinations of drop inlets, pipes, weirs, and orifices. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency outlet. If site restrictions prevent the use of an emergency spillway, then the principal spillway should be sized to safely pass the 100-yr design storm without overtopping the facility. The designer should consider partial clogging (50%) of the principal spillway during the 100-yr design storm to ensure the facility would not be overtopped. For large SWMFs, selecting a flood magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The minimum flood to be used to size the emergency spillway is the 100-yr design storm flood. The sizing of a particular outlet structure should be based on results of hydrologic routing calculations.

A principal spillway system that controls the rate of discharge from a stormwater facility will often use a multi-stage riser for the drop inlet structure, such as the VDOT standard SWM-1. A multi-stage riser is a structure that incorporates separate openings or devices at different elevations to control the rate of discharge from a stormwater basin during multiple design storms. Permanent multi-stage risers are typically constructed of concrete to help increase their life expectancy. The geometry of risers will vary from basin to basin. The designer can be creative to provide the most economical and hydraulically efficient riser design possible.

The primary control structure (riser or weir) should be designed to operate in weir flow conditions for the full range of design flows. Where this is not possible or feasible and the control structure will operate in orifice flow conditions at some point within the design flow range, an anti-vortex device, consistent with the design recommendations in the SWM Handbook, shall be utilized.

In a SWM basin design, the multi-stage riser is of utmost importance because it controls the design water surface elevations. In designing the multi-stage riser, many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. Each iterative routing requires that the facility’s size and outlet shape be designed and tested for performance.

Two types of outlet structures are discussed below.

11.7.6.3.2 SWM-1 (VDOT Standard)
The VDOT standard riser outlet structure is identified as a SWM drainage structure (VDOT Standard SWM-1). This structure should be used at all applicable locations where a drop inlet type control structure is desired. Water quality orifices and additional orifices and weirs can be designed for use with the SWM-1. In addition, the SWM-1 can be modified during construction to serve as the outlet for a temporary sediment basin. The subsurface base of a SWM-1 is typically loaded with Class I stone to counter buoyancy forces.
Anti-vortex vanes are usually not needed on risers for SWM basins due to the VDOT practice of designing relatively shallow basins with emergency spillways. A small trash rack in front of the water quality orifice is included in the SWM-1 details. SWM-1 details can be found in VDOT Road and Bridge Standards Volume I.

Culverts under or through the dam of a SWM Basin are to be reinforced concrete pipe with rubber gaskets, and the joint must be leak-resistant as per AASHTO PP-63, and shall be included in the Department’s Approved List No. 14. A concrete cradle is to be used under the pipe to prevent seepage through the dam. The concrete cradle is to extend the full length of the pipe. (See Road and Bridge Standard SWM-DR)

11.7.6.3.3 Weir Wall (Stormwater Management Dam)
Another type of outlet structure that can be used is a weir wall. The weir wall may be constructed either in place of a riser or as part of a pipe culvert’s wingwalls.

A weir wall in lieu of a riser may be used in areas of shallow basins where the weir wall is no higher than about 5’. The weir wall will have an outlet channel instead of a pipe and will operate efficiently with fewer maintenance concerns than a riser and pipe configuration.

In conjunction with a culvert, the weir is created by building a wall between the culvert’s wingwalls. A concrete apron extends from the pipe to the weir wall at a distance of approximately 1.5 times the culvert diameter. The top of the wall is used to provide the required storage volume and flow attenuation. Notches can also be used in the weir wall to attenuate various storms, and a water quality orifice can be installed at the base in order to drain the basin and provide quality treatment. In addition, the weir wall can be modified during construction to serve as the outlet for a sediment basin. Weir wall outfall structures have proven useful in providing online SWMFs at culvert crossings with dry, intermittent drainage swales by providing the required storage on the upstream side of the crossing. Online facilities should not be used in live streams.

11.7.6.4 Routing
The following data is needed to complete storage design and routing calculations using the appropriate computer program:

- Inflow hydrographs for all selected design storms
- Allowable release rates
- Stage-storage curve or data for proposed SWMF
- Stage-discharge curve or data for the outlet control structures based upon the preliminary design of the outlet control structure and emergency spillway
- Receiving channel performance curve or data
11.8 Part II C Design Procedures and Sample Problems

11.8.1 Documentation Requirements

The following documentation will be required for SWMF design:

- Documentation requirements presented in Chapter 6, Hydrology
- Computations for determination of the pre- and post-development peak runoff rates for the design storms
- Receiving channel adequacy to include Q₂ velocity and Q₁₀ capacity
- Water quality volume based on new impervious area calculation and BMP selection
- WQV orifice size
- Drawdown time for WQV
- Compensatory treatment for uncontrolled new impervious areas
  The designer will complete the SWM and TSB Summary Sheet as provided in Appendix 11B-1
- SWMF Tabulation Sheet when submitting final plans
- Provide all documentation from routing. This would generally include inflow and outflow hydrographs and storage computations for sizing the primary spillway. This information would be generated by various computer modeling software
- Basin grading and primary spillway details and specifications
- Complete (C) and Minimum (M) plan projects shall show SWM measures in the plan assembly as directed in the VDOT Drainage Manual and the VDOT Road Design Manual.
- No-plan (N) and other types of projects (including maintenance activities) that have an abbreviated plan assembly must conform to the requirements of the VSMP Regulations and VPDES General Construction Permit where the land disturbance value exceeds the applicable land disturbance thresholds for such. For the definition of these types of projects, and the procedures for addressing the SWM plan details for such projects, see the VDOT Drainage Manual and the VDOT Road Design Manual.

The plan design details for BMPs shall be appropriately sealed and signed by a person registered in the Commonwealth of Virginia as a professional architect, engineer, land surveyor or landscape architect.

11.8.2 Water Quality Volume Computation and BMP Selection Procedure

Step 1: Determine the new impervious area within that area at the outfall being evaluated.
Step 2: Determine the area within the R/W and easement(s) at the outfall being evaluated.

Step 3: Compute the percentage new impervious (Step 1/Step 2)

Step 4: Compute the WQV by multiplying ½ inch by the new impervious area and convert the units to cubic feet.

Step 5: Refer to Table 11-7 to determine which type of BMP is best suited for the percentage of impervious area

Step 6: Multiply the WQV by the basin treatment factor based (Table 11-1) on the BMP determined from Step 5. This provides the required treatment volume.

Water Quality Volume Computation and BMP Selection Sample Problem:

Assume the basin is to be an extended detention basin based upon 35% new impervious area within the R/W.

Step 1: Determine the new impervious area within that area at the outfall being evaluated.

   New Impervious Area = 2.4 acres

Step 2: Determine the area within the R/W and easement(s) at the outfall being evaluated.

Step 3: Compute the percentage new impervious (Step 1/Step 2).

   Given in the problem statement as 35%.

Step 4: Compute the WQV by multiplying ½ inch by the impervious area and convert the units to cubic feet.

   \[ WQV = \frac{1}{2} \text{ inch} \times \text{Impervious Area} \]

   \[ \frac{1}{2} \text{ inch} \times (1 \text{ ft/12 inches}) = 0.04126 \text{ ft} \]

   \[ 1 \text{ acre} = 43,560 \text{ ft}^2 \]

   \[ WQV = 0.04167 \times 43,560 \times 2.4 \text{ ac.} = 4,356 \text{ ft}^3 \text{ (say 4,360 ft}^3) \]

Step 5: Refer to Table 11-7 to determine which type of BMP is best suited for the percentage of impervious area

   For 35% impervious cover, an extended detention basin will be used.
Step 6: Multiply the WQV by the basin treatment factor based on the BMP determined from Step 4. This provides the treatment volume.

Required Treatment Volume = 2 x WQV = 2(4360) = 8720 cu. ft.

11.8.3 Detention Time Computation and Orifice Sizing

A water quality extended-detention basin treats the water quality volume by detaining it and releasing it over a specified amount of time. In theory, extended-detention of the water quality volume will allow the particulate pollutants to settle out of the first flush of runoff, functioning similarly to a permanent pool. Virginia’s Stormwater Management Regulations pertaining to water quality specify a 30-hour draw down time for the water quality volume. This is a brim draw down time, beginning at the time of peak storage of the water quality volume. Brim drawdown time means the time required for the entire calculated volume to drain out of the basin. This assumes that the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. The extended detention orifice can be sized using either of the following methods:

- Using the average hydraulic head associated with the water quality volume (WQV) and the required drawdown time. This is the VDOT preferred option.
- Using the maximum hydraulic head associated with the water quality volume (WQV, calculate the orifice size needed to achieve the required draw down time and route the water quality volume through the basin to verify the actual storage volume used and the drawdown time.

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<tr>
<td>6</td>
<td>0.196</td>
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</table>
After calculating the needed orifice size the designer should select the nearest nominal size opening from Table 11-10.

11.8.3.1 Average Hydraulic Head Method (Method #2) - VDOT Preferred Method
The average hydraulic head method is the preferred method for determining the required orifice size. It is quicker and easier than the maximum hydraulic head method, which requires a routing to verify the drawdown time. It is also noted that the difference in orifice size produced by the two different methods is insignificant, (i.e. 2 inches versus 2½ inches.

Average Hydraulic Head Sample Problem:
Find the orifice size for the required treatment volume using the average hydraulic head method.

\[ h_{\text{max}} = 1.1 \text{ ft.} \]
Volume = 8,720 ft\(^3\) (from Sample Problem 11.5.2.1)
\[ h_{\text{avg}} = \frac{1.1}{2} = 0.55 \text{ ft.} \]
Note: Actual \( h \) on orifice is to the center of the orifice. Since the size of this orifice is unknown and assumed small, use \( h_{\text{max}} = 1.1 \text{ ft.} \)

Calculate the discharge through the orifice based on the required treatment volume.

\[ Q_{\text{avg}} = \frac{\text{Treatment Volume}}{\text{Time}} = \frac{8720 \text{ cu. ft.}}{30 \text{ hr} \left(\frac{3600 \text{ sec}}{\text{hr}}\right)} = 0.081 \text{ cfs} \]

Calculate the orifice area by rearranging Equation 11.1.

\[ A = \frac{Q}{C \sqrt{2gh_{\text{avg}}}} = \frac{0.081}{0.6 \sqrt{2(32.2)(0.55)}} = 0.0223 \text{ sq. ft.} \]
From Table 11-10, select a 2-inch orifice with \( A = 0.022 \text{ ft}^2 \).

11.8.3.2 Maximum Hydraulic Head Method (Method #1)
The maximum hydraulic head method uses the maximum discharge and results in a slightly larger orifice than the same procedure using the average hydraulic head method. The routing allows the designer to verify the performance of the calculated orifice size. As a result of the routing effect however, the actual basin storage volume used to achieve the drawdown time will be less than the computed brim drawdown volume.

Maximum Hydraulic Head Sample Problem:
Using the data provided in sample problem 11.5.3.1.1, determine the orifice size using the maximum hydraulic head method: Use the maximum hydraulic head (not the
average) and the maximum Q \((Q_{avg} \times 2)\). The WQV hydrograph (HYG) should then be routed through the basin to determine if the residence time is approximately 30 hours. Find the orifice size for the required treatment volume using the maximum hydraulic head method.

\[ h_{max} = 1.1 \text{ ft.} \]
\[ Q_{max} = 2Q_{avg} = 2(0.081) = 0.16 \text{ cfs} \]

Calculate the orifice area by rearranging Equation 11.1.

\[ A = \frac{Q}{C \sqrt{2gh_{avg}}} = \frac{0.16}{0.6 \sqrt{2(32.2)(1.1)}} = 0.0320 \text{ sq. ft.} \]

From Table 11-10, select a 2½-inch orifice with \(A = 0.034 \text{ ft}^2\)

Next step: Route the WQV hydrograph thru the basin using the 2½-inch orifice.

**COMMENTS:** The routing of the WQV hydrograph thru a basin may not be possible with some routing software. The problem can be due to the need for using a hydrograph for a minimum of about 30 hours and with possibly the last 29-hours inflow of 0.0 or 0.01 cfs. The problem could also be due to the need for small orifice sizes < 3-inches in diameter.

**11.8.3.3 WQV Hydrograph (HYG)**

To develop a hydrograph for the WQV following the sample problem in Section 11.5.3.2.1, you need only to calculate the hydrograph for the new impervious area and use the time of concentration that applies to the new impervious area and its proximity to the basin. The NRCS Method hydrograph will probably be the easiest hydrograph to provide the required treatment volume of 1 inch of runoff for an extended detention basin. The time of concentration \((t_c)\) may be found by methods discussed in Chapter 6, Hydrology, since the \(t_c\) has the same definition in the Rational Method as in TR-55. The process will involve using a \(CN=98\) for the impervious area, Rainfall \((RF) = 1.2\) inches to produce RUNOFF \((RO) = 1\) inch and the NRCS 24-hour Type II storm distribution. All VDOT designers should have the TR-55 software and the above values can be used to produce the hydrograph.

**11.8.3.4 Alternative Method of Routing WQV to Find Drawdown Time**

The Stormwater Management Handbook, Vol. II, defines brim drawdown time as from the time the WQV elevation is reached until the basin is emptied. This is based upon a treatment volume storm producing only the amount of runoff required for the WQV.

The normally required routing of the 2-yr storm for quantity control can also be used for drawdown time with some slight adjustment providing that the routing software will accommodate a 30-hour duration and a small size orifice. The receding limb of the inflow hydrograph will need to be showing either 0.0 or 0.01 cfs inflow up to a time of about 30 hours. By this method the drawdown time for WQV is actually from the time that the ponded depth recedes to the treatment volume elevation (with no more inflow)
until the basin is empty. For practical purposes, if the routing shows that the basin is empty at about 30 hours, the design is adequate.

### 11.8.4 Channel Erosion Control Volume – Q₁ Control

Extended detention of a specified volume of stormwater runoff can also be incorporated into a basin design to protect downstream channels from erosion. Virginia’s Stormwater Management Regulations recommend 24-hour extended detention of the runoff from the 1-year frequency storm as an alternative to the 2-yr peak discharge reduction required by MS-19 of the VESCR.

The design of a channel erosion control extended-detention orifice is similar to the design of the water quality orifice in that previous orifice sizing methods can be used:

1. Using the average hydraulic head method (VDOT Preferred Method), approximate the orifice size associated with the channel erosion control volume ($V_{ce}$) and the drawdown time.

2. Using the maximum hydraulic head method, approximate the orifice size associated with the channel erosion control volume ($V_{ce}$) and the required drawdown time and route the 1-year frequency storm through the basin to verify the storage volume and drawdown time.

The routing procedure takes into account the discharge that occurs before maximum or brim storage of the channel erosion control volume ($V_{ce}$). The routing procedure provides a more accurate accounting of the storage volume used while water is flowing into and out of the basin, and may result in less storage volume being used than the calculated brim storage volume associated with the maximum hydraulic head. The actual storage volume needed for extended detention of the runoff generated by the 1-year frequency storm will be approximately 60 percent of the calculated volume ($V_{ce}$) of runoff for curve numbers between 75 and 95 and with times of concentration between 0.1 and 1 hour.

**Channel Erosion Control Volume, (Q₁ Control) Sample Problem:**

The following sample problem illustrates the design of the extended-detention orifice for channel erosion control volume using the average hydraulic head method.

**Drainage Area = 25 ac.**

1-year rainfall = 2.7"

CN = 75

1-year rainfall depth of runoff = 0.8"

**Step 1** Determine the rainfall amount (inches) of the 1-year frequency storm for the local area where the project is located.
Step 2: With the rainfall amount and the runoff curve number (CN), determine the corresponding runoff depth using the runoff equation.

Step 3: Calculate the channel erosion control volume ($V_{ce}$)

$$V_{ce} = 25ac.\, (0.8\, \text{in.}) \left( \frac{1\, \text{ft.}}{12\, \text{in.}} \right) = 1.67\, \text{ft.}$$

To account for the routing effect, reduce the channel erosion control volume by 60%:

$$V_{ce} = 0.60(1.67) = 1.0\, \text{ac. ft. or 43,560 cu. ft.}$$

Step 4: Determine the average hydraulic head ($h_{avg}$) corresponding to the required channel erosion control volume.

$$h_{avg} = \frac{2 - 0}{2} = 1.0\, \text{ft.}$$

Note: When considering the maximum depth of ponding, the WQV is generally limited to 2'.

Step 5: Determine the average discharge ($Q_{avg}$) resulting from the 24-hour drawdown requirement.

$$Q_{avg} = \frac{43,560\, \text{cu ft}}{(24\, \text{hr})(3,600\, \text{sec/hr})} = 0.50\, \text{cfs}$$

Step 6: Determine the required orifice diameter by rearranging the Equation 11.1.

$$A = \frac{Q}{C\sqrt{2gh_{avg}}} = \frac{0.50}{0.6\sqrt{2(32.2)(1.0)}} = 0.104\, \text{sq. ft.}$$

Calculate the orifice diameter:

$$d = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4(0.104)}{\pi}} = 0.364\, \text{sw ft}$$

$$d = 4.4\, \text{in. (Say 4.5 in)}$$

The designer can also use Table 11-10 to determine a 4½-inch diameter extended detention orifice for channel erosion control.
11.8.5 Preliminary Detention Volume Computation

Three methods are presented for estimating the volume of storage needed for peak flow attenuation (quantity control). The estimated storage volumes are approximate and the designer will need to select the most appropriate volume in order to determine the preliminary basin size.

11.8.5.1 Modified Rational Method, Simplified Triangular Hydrograph Routing

Information needed includes the hydrology and hydrographs for the watershed or drainage area to be controlled, calculated by using one of the methods as outlined in Chapter 6, and the allowable release rates for the facility, as established by ordinance or downstream conditions.

Step 1: Determine BMP requirements

Determine the percent of new impervious area within the R/W. Select the type of BMP needed from Table 11-7. Calculate the water quality volume.

Some considerations for BMP selection include:

- Water Quality Extended-Detention Basin: The water quality volume must be detained and released over 30 hours. The established pollutant removal efficiency is based on a 30-hour drawdown.

- Water Quality Retention Basin: The volume of the permanent pool is established by the site impervious cover or the desired pollutant removal efficiency.

- Channel Erosion Control Extended-Detention Basin: The channel erosion control volume based upon $Q_1$, for the entire drainage area, must be detained and released over 24 hours.

Step 2: Compute allowable release rates

Compute the pre- and post-developed hydrology for the watershed. Sometimes, the pre-developed hydrology will establish the allowable release rate from the basin. Other times, the release rate will be established by downstream conditions. In either case, the post-developed hydrology will provide the peak discharge into the basin, as a peak discharge (cfs) or a runoff hydrograph. Refer to Chapter 6, Hydrology, on developing runoff hydrographs and peak discharge.

Step 3: Estimate the required storage volume

The information required includes the developed condition peak rate of runoff, or runoff hydrograph, and the allowable release rates for each of the
appropriate design storms. These methods provide a preliminary estimate of the storage volume required for peak flow attenuation.

![Simplified Triangular Hydrograph Method](image)

**Figure 11-10. Simplified Triangular Hydrograph Method**

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_s = \frac{1}{2} T_b (Q_i - Q_o)$$

Where:
- $V_s$ = Storage volume estimate, ft$^3$
- $Q_i$ = Peak inflow rate, cfs
- $Q_o$ = Peak outflow rate, cfs
- $T_b$ = Duration of basin inflow, sec.

**11.8.5.2 Critical Storm Duration Method**

The critical storm duration method is used to calculate the maximum storage volume for a detention facility. This critical storm duration is the storm duration that generates the greatest volume of runoff and, therefore, requires the most storage. The required storage volume is represented by the area between the inflow hydrograph and the outflow hydrograph. The area can be approximated using the following equation:
\[ V = \left[ Q_i T_d + \frac{Q_i t_c}{4} - \frac{q_o T_d}{2} - \frac{3q_o t_c}{4} \right] 60 \]

Where:
- \( V \) = Required storage volume, ft\(^3\)
- \( Q_i \) = Inflow peak discharge, cfs, for the critical storm duration, \( T_d \)
- \( T_c \) = Time of concentration, min.
- \( q_o \) = Allowable peak outflow, cfs
- \( T_d \) = Critical storm duration, min.

The first derivative of the critical storage volume equation with respect to time is an equation that represents the slope of the storage volume curve plotted versus time. When Equation 11.4 is set to equal zero, and solved for \( T_d \), it represents the time at which the slope of the storage volume curve is zero, or at a maximum. Equation 11.5 for the critical storm duration is:

\[ T_d = \sqrt{\frac{2CAa(b - \frac{t_c}{4})}{q_o}} - b \]

Where:
- \( T_d \) = Critical storm duration, min.
- \( C \) = Runoff coefficient
- \( A \) = Drainage area, ac.
- \( a \) & \( b^* \) = Rainfall constants developed for storms of various recurrence intervals and various geographic locations‡
- \( t_c \) = Time of concentration, min.
- \( q_o \) = Allowable peak outflow, cfs

*The \( a \) & \( b \) rainfall constants are not to be used for any other purpose.

The Department has developed a computer program entitled “CRITSTRM” for performing these computations. Access is available upon request at the following web address: [http://www.virginiadot.org/business/locdes/notification.asp](http://www.virginiadot.org/business/locdes/notification.asp).

### 11.8.5.3 Pagan Volume Estimation Method

This method is appropriate for use with small basins serving watersheds of 200 acres or less. For this method, data from many small basins was compiled and the curve in Figure 11-11 was developed. This curve is used to determine the storage volume for a given drainage area by dividing the pre-development peak inflow by the post-development peak inflow.

Knowing the percentage of peak inflow, the storage parameter (peak storage in cubic feet over peak inflow in cubic feet per second) can be found by moving horizontally over the y-axis to the curve and down to the x-axis.
By multiplying the storage parameter by the peak inflow, the approximate peak storage can be found. This method should be used only as a first trial. Experience has shown that this method is conservative.

![Figure 11-11. Pagan Method Curve](image)

**Limit to Watershed Areas of 1 sq. mi. or less**

\[
SP = \frac{\text{Peak storage in cubic feet}}{\text{Peak inflow in cfs}} = \frac{STO}{I}
\]

**Figure 11-11. Pagan Method Curve**

**Step 1:** Determine pre- and post-development peak discharges.

**Step 2:** Determine the Storage Parameter (SP).

SP is determined from Figure 11-11 drawing a line from the percentage of peak inflow \((Q_o/Q_i)\) to the line and reading the factor along the base of the figure.

**Step 3:** Compute the Maximum Storage Volume \((STO)\):

\[
STO = SP(I)
\]

### 11.8.5.4 Sample Problems – Using 3 Methods to Estimate Volume of Storage for Quantity Control

<table>
<thead>
<tr>
<th>Condition</th>
<th>Rational Method</th>
<th>D.A</th>
<th>C</th>
<th>T_c</th>
<th>Q_{10}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-developed</td>
<td>25ac.</td>
<td>0.38</td>
<td>52 min.</td>
<td>24 cfs</td>
<td></td>
</tr>
<tr>
<td>Post-developed</td>
<td>25 ac</td>
<td>0.59</td>
<td>21 min.</td>
<td>65 cfs</td>
<td></td>
</tr>
</tbody>
</table>
Method 1: Modified Triangular Hydrograph Method

Based on the methodology from 11.5.4.1, solve for $V_{s10}$ as follows:

$$V_s = \frac{1}{2} T_b (Q_i - Q_o)$$

Where:

- $V_{s10} = $ Storage volume estimate, ft$^3$
- $Q_i = 65$ cfs
- $Q_o = 24$ cfs
- $T_b = 2520$ sec. = 42 min.

$$V_s = \frac{1}{2} (2520)(65 - 24) = 51,660 \text{ cu. ft.}$$

Method 2: Critical Storm Duration Method

Based on the methodology in 11.5.4.2, determine the 10-yr critical storm duration $T_{d10}$ as follows:

$$a = 189.2$$

$$b = 22.1$$

$$C = 0.59 \text{ (Post-development)}$$

$$A = 25 \text{ acres}$$

$$t_c = 21 \text{ min (Post-development)}$$

$$q_{o10} = 24 \text{ cfs (Allowable outflow based on pre-development)}$$

$$T_d = \sqrt{\frac{2CAa(b - t_c)}{q_o} - b}$$

$$T_{d10} = \sqrt{\frac{2(0.59)(25.0)(189.2)(22.1 - \frac{21}{4})}{24} - 22.1}$$

$$T_{d10} = 40.5 \text{ min}$$

**Solve for the 10-yr critical storm duration intensity ($I_{10}$)**

$$I_{10} = \frac{189.2}{22.1 + 40.5} = 3.02 \text{ in/hr}$$
Determine the 10-yr peak inflow \( (Q_{10}) \) using the Rational Equation and the critical storm duration intensity \( (I_{10}) \)

\[
Q = C_f C_i A
\]

\[
Q_{10} = 1.0(0.59)(3.02)(25) = 44.5 \text{ cfs}
\]

Determine the required 10-yr storage volume \( (V_{10}) \) for the 10-yr critical storm duration \( (T_{d10}) \)

\[
V = \left[ Q_i T_d + \frac{Q_i t_c}{4} - \frac{q_o T_d}{2} - \frac{3q_o t_c}{4} \right] 60
\]

\[
V = \left[ (44.5)(40.5) + \frac{(44.5)(21)}{4} - \frac{(24)(40.5)}{2} - \frac{3(24)(21)}{4} \right] 60
\]

\[
V = 70,313 \text{ cu. ft. (Say 70,300 cu. ft.)}
\]

**Method 3: Pagan Method**

Based on the methodology in 11.5.4.3, solve for the storage volume as follows:

\[
\frac{Q_o}{Q_i} = \frac{24}{65} = 0.37 \ (37\%)
\]

\[
\text{SP} = 3100 \text{ seconds}
\]

\[
\text{STO} = \text{SP}(I) = 3100(65) = 201,500 \text{ cu. ft.}
\]

**11.8.6 Determine Preliminary Basin Size**

Based upon the estimated storage volume requirements calculated by the three methods in Section 11.5.4.4, determine the preliminary size of the basin. Assume the basin will have a rectangular shaped base, about 2:1 length to width ratio and optimum depth for \( Q_{10} \) about 4’. The basin will have 3:1 side slopes, but for the first size estimate, the size of the base using vertical sides will provide an adequate first estimate.

**From Method 1: Simplified Triangular Hydrograph Method**

\[
V_{10} = 51,660 \text{ ft}^3
\]

For a 4-ft depth, \( \frac{51,660}{4} = 12,915 \text{ sq. ft.} \)

About 80’ x 160’
From Method 2: Critical Storm Duration Method

\[ V_{10} = 70,300 \text{ ft}^3 \]

For 4' depth, \[ \frac{70,300}{4} = 12,915 \text{ sq. ft.} \]

About 90' x 195'

From Method 3: Pagan Method

\[ V_{10} = 201,500 \text{ ft}^3 \]

For a 4' deep, \[ \frac{201,500}{4} = 50,375 \text{ sq. ft.} \]

About 150' x 335'

Summary: Preliminary trial size basin would be recommended about 100' x 200'

11.8.7 Final Basin Sizing-Reservoir Routing

11.8.7.1 Storage – Indication Method Routing Procedure

The following procedure presents the basic principles of performing routing through SWMF (Puls Method of storage routing). Routing is most often completed with computer software, which develops the stage-discharge and stage-storage curves within the program.

Step 1: Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed SWMF. Example stage-storage and stage-discharge curves are shown in Figure 11-12 and Figure 11-13 respectively.
Step 2: Select a routing time period ($\Delta t$) to provide at least five points on the rising limb of the inflow hydrograph. Use $t_p$ divided by 5 to 10 for $\Delta t$.

Step 3: Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of $s \pm \frac{O}{2}\Delta T$ versus stage. An example tabulation of storage characteristics curve data is shown in Table 11-11.

Table 11-11. Storage Characteristics

<table>
<thead>
<tr>
<th>(1) Stage (H) (ft.)</th>
<th>(2) Storage $^1$ (S) (ac-ft)</th>
<th>(3) Discharge $^2$ (Q) (cfs)</th>
<th>(4) Discharge $^2$ (Q) (ac-ft/hr)</th>
<th>(5) $S\frac{Q}{2}$ (ac-ft)</th>
<th>(6) $S+\frac{Q}{2}$ (ac-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>0.05</td>
<td>0</td>
<td>0</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>101</td>
<td>0.05</td>
<td>15</td>
<td>1.24</td>
<td>0.20</td>
<td>0.40</td>
</tr>
<tr>
<td>102</td>
<td>0.05</td>
<td>35</td>
<td>2.89</td>
<td>0.56</td>
<td>1.04</td>
</tr>
<tr>
<td>103</td>
<td>1.6</td>
<td>63</td>
<td>5.21</td>
<td>1.17</td>
<td>2.03</td>
</tr>
<tr>
<td>104</td>
<td>2.8</td>
<td>95</td>
<td>7.85</td>
<td>2.15</td>
<td>3.45</td>
</tr>
<tr>
<td>105</td>
<td>4.4</td>
<td>143</td>
<td>11.82</td>
<td>3.41</td>
<td>5.39</td>
</tr>
<tr>
<td>106</td>
<td>6.6</td>
<td>200</td>
<td>16.53</td>
<td>5.22</td>
<td>7.98</td>
</tr>
</tbody>
</table>

$^1$ Obtained from the Stage-Storage Curve.

$^2$ Obtained from the Stage-Discharge Curve.
Step 4: For a given time interval, $I_1$ and $I_2$ are known. Given the depth of storage or stage ($H_i$) at the beginning of that time interval, $s - \frac{O_1}{2} \Delta T$ can be determined from the appropriate storage characteristics curve, Figure 11-14.

![Figure 11-14. Storage Characteristics Curve](image)

Step 5 Determine the value of $s_2 + \frac{O_2}{2} \Delta T$ from the following equation:

$$s_2 + \frac{O_2}{2} \Delta T = s_1 - \frac{O_1}{2} \Delta T + \frac{I_1 + I_2}{2} \Delta T$$

Where:
- $s_2$ = Storage volume at time 2, ft³
- $O_2$ = Outflow rate at time 2, cfs.
- $\Delta T$ = Routing time period, sec
- $s_1$ = Storage volume at time 1, ft³
- $O_1$ = Outflow rate at time 1, cfs
- $I_1$ = Inflow rate at time 1, cfs
- $I_2$ = Inflow rate at time 2, cfs

Other consistent units are equally appropriate.
Step 6: Enter the storage characteristics curve at the calculated value of $s_2 + \frac{O_2}{2} \Delta T$ determined in Step 5 and read off a new depth of water ($H_2$).

Step 7: Determine the value of $O_2$, which corresponds to a stage of $H_2$ determined in Step 6, using the stage-discharge curve.

Step 8: Repeat Steps 1 through 7 by setting new values of $I_1$, $O_1$, $S_1$, and $H_1$ equal to the previous $I_2$, $O_2$, $S_2$, and $H_2$, and using a new $I_2$ value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

11.8.7.2 Storage – Indication Method Routing Sample Problem

This example demonstrates the application of the methodology presented for the design of a typical detention SWMF used for water quantity control.

SWMFs shall be designed for runoff from both the 2- and 10-yr design storms and an analysis done using the 100-yr design storm runoff to ensure that the structure can accommodate runoff from this storm without damaging adjacent and downstream property and structures.

The peak discharges from the 2- and 10-yr design storms are as follows:

- Pre-developed 2-yr peak discharge = 150 cfs
- Pre-developed 10-yr peak discharge = 200 cfs
- Post-development 2-yr peak discharge = 190 cfs
- Post-development 10-yr peak discharge = 250 cfs

Since the post-development peak discharge must not exceed the pre-development peak discharge, the allowable design discharges are 150 and 200 cfs for the 2- and 10-yr design storms, respectively.

Step 1: Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed SWMF.

Runoff hydrographs are shown in Table 11-12 below. Inflow durations from the post-development hydrographs are about 1.2 and 1.25 hours, respectively, for runoff from the 2- and 10-yr storms.
Chapter 11-137 of 148

Table 11-12. Runoff Hydrographs

<table>
<thead>
<tr>
<th>Time (hrs)</th>
<th>Pre-Development Runoff</th>
<th>Post-Development Runoff</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(1) 2-yr (cfs)</td>
<td>(2) 10-yr (cfs)</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.1</td>
<td>18</td>
<td>24</td>
</tr>
<tr>
<td>0.2</td>
<td>61</td>
<td>81</td>
</tr>
<tr>
<td>0.3</td>
<td>127</td>
<td>170</td>
</tr>
<tr>
<td>0.4</td>
<td>150</td>
<td>200</td>
</tr>
<tr>
<td>0.5</td>
<td>112</td>
<td>150</td>
</tr>
<tr>
<td>0.6</td>
<td>71</td>
<td>95</td>
</tr>
<tr>
<td>0.7</td>
<td>45</td>
<td>61</td>
</tr>
<tr>
<td>0.8</td>
<td>30</td>
<td>40</td>
</tr>
<tr>
<td>0.9</td>
<td>21</td>
<td>28</td>
</tr>
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<td>15</td>
</tr>
<tr>
<td>1.2</td>
<td>8</td>
<td>13</td>
</tr>
</tbody>
</table>

Preliminary estimates of required storage volumes are obtained using the simplified triangular hydrograph method outlined in Section 11.5.4.1. For runoff from the 2- and 10-yr storms, the required storage volumes, $V_s$, are computed using Equation 11.3:

$$V_s = \frac{1}{2} T_b (Q_i - Q_o)$$

$$V_{s2} = \frac{1}{2} \frac{(1.2)(3600)(190 - 150)}{43,560} = 1.98 \text{ ac. ft.}$$

$$V_{s10} = \frac{1}{2} \frac{(1.2)(3600)(250 - 200)}{43,560} = 2.58 \text{ ac. ft.}$$

Stage-discharge and stage-storage characteristics of a SWMF that should provide adequate peak flow attenuation for runoff from both the 2- and 10-yr design storms are presented below in Table 11-13. The storage-discharge relationship was developed and required that the preliminary storage volume estimates of runoff for both the 2-and 10-yr design storms to coincide with the occurrence of the corresponding allowable peak discharges.

Discharge values were computed by solving the broad-crested weir equation for head (H) assuming a constant discharge coefficient of 3.1, a weir length of 4', and no tailwater submergence. The capacity of storage relief structures was assumed to be negligible.
Step 2: Select a routing time period ($\Delta t$) to provide at least five points on the rising limb of the inflow hydrograph. Use $t_p$ divided by 5 to 10 for $\Delta t$.

$$\Delta T = \frac{t_p}{5} = \frac{0.5}{5} = 0.10 \text{ hr}$$

Step 3: Use the storage-discharge data from Step 1 to develop storage characteristics curves (Stage-Discharge-Storage) that provide values of $s \pm \frac{0}{2} \Delta T$ versus stage.

<table>
<thead>
<tr>
<th>(1) Stage (H) (ft)</th>
<th>(2) Discharge (Q) (cfs)</th>
<th>(3) Storage (S) (ac-ft)</th>
<th>(4) $S + \frac{Q^2}{2}$ (ac-ft)</th>
<th>(5) $S - \frac{Q^2}{2}$ (ac-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>0.9</td>
<td>10</td>
<td>0.26</td>
<td>0.30</td>
<td>0.22</td>
</tr>
<tr>
<td>1.4</td>
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<td>0.50</td>
<td>0.33</td>
</tr>
<tr>
<td>1.8</td>
<td>30</td>
<td>0.56</td>
<td>0.68</td>
<td>0.43</td>
</tr>
<tr>
<td>2.2</td>
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<td>0.69</td>
<td>0.85</td>
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<tr>
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<td>50</td>
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<td>1.02</td>
<td>0.60</td>
</tr>
<tr>
<td>2.9</td>
<td>60</td>
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<td>1.18</td>
<td>0.68</td>
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<tr>
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<td>1.50</td>
<td>0.84</td>
</tr>
<tr>
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<td>90</td>
<td>1.28</td>
<td>1.66</td>
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</table>

Storage routing was conducted for runoff from both the 2- and 10-yr design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Routing results are shown below for runoff from the 2- and 10- year design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 2- and 10-yr design storms.
Step 4: For a given time interval, $I_1$ and $I_2$ are known. Given the depth of storage or stage ($H_1$) at the beginning of that time interval, $S_1 + \frac{O_1}{2} \Delta T$ can be determined from the appropriate storage characteristics curve.

Step 5 Determine the value of $s_2 + \frac{O_2}{2} \Delta T$ from the following equation:

$$S_2 + \frac{O_2}{2} \Delta T = S_1 - \frac{O_1}{2} \Delta T + \frac{I_1 + I_2}{2} \Delta T$$

Summarized in Tables 11-14 and 11-15 for the 2-yr and 10-yr storms.

Step 6 Enter the storage characteristics curve at the calculated value of $s_2 + \frac{O_2}{2} \Delta T$ determined in Step 5 and read off a new depth of water ($H_2$).

Summarized in Tables 11-14 and 11-15 for the 2-yr and 10-yr storms.

Step 7 Determine the value of $O_2$, which corresponds to a stage of $H_2$ determined in Step 6, using the stage-discharge curve.

Summarized in and Table 11-14 and 11-15 for the 2-yr and 10-yr storms.

Step 8 Repeat Steps 1 through 7 by setting new values of $I_1$, $O_1$, $S_1$, and $H_1$ equal to the previous $I_2$, $O_2$, $S_2$, and $H_2$, and using a new $I_2$ value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

Summarized in Tables 11-14 and 11-15 for the 2-yr and 10-yr design storms.
### Table 11-14. Storage Routing for the 2-yr Storm

<table>
<thead>
<tr>
<th>Time (T) (hrs)</th>
<th>Inflow (I) (cfs)</th>
<th>( \frac{I_1 + I_2 \Delta T}{2} ) (ac-ft)</th>
<th>Stage (H₁) (ft)</th>
<th>( \frac{S_1 \Delta T^1}{2} ) (6)-(8) (ac-ft)</th>
<th>( \frac{S_2 \Delta T^2}{2} ) (3)+(5) (ac-ft)</th>
<th>Stage (H) (ft)</th>
<th>Outflow (O) (cfs)</th>
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### Table 11-15. Storage Routing for the 10-yr Storm

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<tr>
<th>Time (T) (hrs)</th>
<th>Inflow (I) (cfs)</th>
<th>( \frac{I_1 + I_2 \Delta T}{2} ) (ac-ft)</th>
<th>Stage (H₁) (ft)</th>
<th>( \frac{S_1 \Delta T^1}{2} ) (6)-(8) (ac-ft)</th>
<th>( \frac{S_2 \Delta T^2}{2} ) (3)+(5) (ac-ft)</th>
<th>Stage (H) (ft)</th>
<th>Outflow (O) (cfs)</th>
</tr>
</thead>
<tbody>
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<td>0.00</td>
<td>0.00</td>
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<td>0.00</td>
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<td>0</td>
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</table>
Since the routed peak discharge is lower than the maximum allowable peak discharges for both design storms, the weir length could be increased or the storage decreased. If revisions are desired, routing calculations should be repeated.

Although not shown for this sample problem, runoff from the 100-yr frequency storm should be routed through the SWMF to establish freeboard requirements and to evaluate emergency overflow and stability requirements. In addition, the preliminary design provides hydraulic details only. Final design should consider site constraints such as depth to water, side slope stability, maintenance, grading to prevent standing water, and provisions for public safety.

An estimate of the potential downstream effects (i.e., increased peak flow rate and recession time) of detention SWMFs may be obtained by comparing hydrograph recession limbs from the pre-development and routed post-development runoff hydrographs. Example comparisons are shown below for the 10-yr design storms.

![Figure 11-15. Runoff Hydrographs](image)

Potential effects on downstream facilities should be minor when the maximum difference between the recession limbs of the pre-developed and routed outflow hydrographs is less than about 20%. As shown in Figure 11-15, the sample problem results are well below 20%; downstream effects can thus be considered negligible and downstream flood routing or $Q_1$ control omitted.
11.8.7.3 SWM Basin Design: Sample Problem

Step 1: Determine the type of BMP required:

- New impervious area draining to this outfall = 2.98 ac.
- Total drainage area at the outfall within the R/W and easements = 9.03 ac.
- Percentage Impervious Cover = \( \frac{2.98}{9.03} = 0.33 \) (33%)

From Table 11-7 select an extended detention basin

Step 2: Determine Quantity Control Requirements:

- The receiving channel is a natural channel that was determined not to be adequate. The post construction \( Q_2 \) will overtop the banks of the channel. The pre-construction \( Q_2 \) is also above the banks of the channel, but that is not a factor.
- In accordance with MS-19 of the VESCR, the BMP will need to attenuate the post-development \( Q_2 \) to not be greater than pre-development \( Q_2 \). The design of the dam and the emergency spillway will need to provide protection of the dam for \( Q_{100} \).
- The \( Q_{2pre} = 20.5 \) cfs and the \( Q_{2post} = 29.6 \) cfs. The usual design process would be to now estimate the quantity control volume needed for the basin.

Step 3. Determine if quantity control for \( Q_1 \) is required:

- Flood control for the 1-year frequency storm in lieu of the 2-yr frequency storm may be needed if there is existing or anticipated erosion downstream.
- A field review of the receiving channel has shown no significant erosion and none is anticipated.

Therefore, the alternative \( Q_1 \) control is not needed.

Step 4. Determine the required water quality volume and treatment volume:

- From Table 11-7 the required treatment volume for an extended detention basin is \( 2 \times WQV \). The WQV being equal to \( \frac{1}{2} \) inches x New Impervious area. \( 2 \times \frac{1}{2} \) inch = 1 in or 0.083 ft
- New pavement within the drainage area for this outfall = 2.98 ac or 129,809 ft\(^2\)

\[
\text{Treatment Volume} = 2 \times WQV
\]
\[
= 2 \left[ \frac{0.5(2.98)(43560)}{12} \right] = 10,817 \text{ cu.ft.}
\]
Step 5. Determine the temporary sediment storage requirements:

- The total drainage area to this outfall from a storm drain system is 12.98 ac.
- All of the drop inlets in the storm drain will have erosion control measures.
- Temporary sediment storage is not required because all of the inlets can be protected from sediment. However, temporary sediment storage will be provided with the volume equal to the treatment value due to the convenience of the basin and as a supplement to the erosion and sediment controls.
- If a temporary sediment basin were needed, the quantities would be:
  67 cu. yd. x 13 ac = 23,517 ft³ for wet storage
  67 cu. yd. x 13 ac = 23,517 ft³ for dry storage
  The total volume required for temporary sediment storage, wet plus dry = 47,034 ft³. This is much larger than the 10,817 ft³ required for the WQV.

Step 6. Determine the size of the sediment forebay:

- A sediment/debris forebay is recommended for extended detention basins and the volume should be between 0.1 to 0.25 in/ac of new impervious area or 10% of the required detention volume. This range establishes the minimum to maximum desirable sediment storage volumes needed. The actual size of the forebay is dependent upon the site conditions. It is desirable to size the forebay as near to the maximum sediment storage volume as possible.
- Compute the sediment forebay volume and determine its dimensions:
  \[ \text{Vol.} = 0.1 \text{ in.} \left(\frac{1 \text{ ft}}{12 \text{ in.}}\right) 2.98 \text{ ac.} \left(\frac{43560 \text{ sq. ft.}}{1 \text{ ac.}}\right) = 1082 \text{ cu. ft.} \]
  If forebay is 1 ft. deep: Size = 33 ft. x 33 ft.
  For 0.25 inch, volume = 2,704 ft³
  If basin is 1 ft. deep: Size = 50 ft. x 50 ft.

The shape of the forebay does not need to be square and should be shaped to fit the site. The volume of the forebay that cannot be drained should not be considered as part of the required storage volume for the basin.
The established design parameters for the basin:

- An extended detention basin is required for this site.
- QUANTITY CONTROL FOR Q_2 PEAK IS REQUIRED. The required volume will be estimated in the design process.
- Alternative Q_1 control is not needed.
- The required WQV is 10,817 ft³.
- The temporary sediment volume (if needed) is 47,034.
- The estimated forebay volume is 1,082 to 2,704 ft³.

**Determining the Water Quality Volume**

Calculate required WQV (for extended detention) = 10,817 ft³.

From Preliminary Elevation/Storage Table:

- The WQV required is met @ Elev. 423.25
- Depth = 1.95 ft.
- Actual Volume = 11,051 ft³ @ Elev. 423.25

**WQV Computations – Determining the Orifice Size Required**

Using Method #2 Average Hydraulic Head (VDOT Preferred)

- Assume depth, h = 1.95 ft. (Say 2.0 ft)

\[
\frac{h_{avg}}{2} = \frac{2.0}{2} = 1.0 \text{ ft.}
\]

- Compute the \(Q_{avg}\) for the WQV using the required 30-hour drawdown time:

\[
Q_{avg} = \frac{Treatment\ Volume}{Time} = \frac{11,051 \text{ cu. ft.}}{30 \text{ hr} \left( \frac{3600 \text{ sec}}{hr} \right)} = 0.102 \text{ cf/s}
\]

- Orifice sizing computations:

\[
A = \frac{Q_{avg}}{C\sqrt{2gh_{avg}}} = \frac{0.102}{0.6\sqrt{2(32.2)(1.0)}} = 0.021 \text{ sq. ft.}
\]

- The depth (h) used in the orifice equation would normally be measured from the center of the orifice. Due to the small size of the water quality orifice it is acceptable to consider the h as the depth to the invert of the orifice.

From Table 11-10, use a 2-inch orifice with an area = 0.022 ft².
Q₁ Control – Alternative Quantity Control

Assume that a field review of the receiving channel shows that there is significant erosion and it has been decided that the channel should be protected from the Q₁ instead of the Q₂ as required by MS-19. Control of the Q₁ requires containing the entire volume of the Q₁ from the total drainage area and releasing that volume over a 24-hour period. The computations are similar to those used for WQV storage and released over a 30-hour period. When Q₁ is detained and released over the 24-hour period, there will be no need to provide additional or separate storage for the WQV if it can be demonstrated that the treatment volume will be detained for approximately 24 hours.

Determine the Q₁ Control Volume:
Use Method #2 – Average Hydraulic Head (Recommended Method)

Find the Q₁ Control volume.

Given from design computations:

\[ DA = 12.98 \text{ ac} \]
\[ C = 0.67 \]
\[ T_c = 16 \text{ min} \]
\[ Q_2 = 29.6 \text{ cfs} \]

- **Use TR-55 to find the volume for Q₁:**
- **Convert the runoff coefficient, C = 0.67 from the Rational Method to CN = 80.**
- **Find the 1-year frequency 24-hour rainfall (RF) using the site-specific rainfall precipitation frequency data recommended by the U.S. National Oceanic and Atmospheric Administration (NOAA) Atlas 14. Partial duration time series shall be used for the precipitation data.**

RF = 2.8 inches.

- **Find the runoff depth for CN = 80 and RF = 2.8 inches using TR-55.**

Runoff (RO) = 1.1 inches

- **Compute the Q₁ Control volume:**

\[
V_{ce} = 12.98 \text{ ac. (1.1 in.)} \left(\frac{1 \text{ ft.}}{12 \text{ in.}}\right) \left(\frac{43,560 \text{ sq. ft.}}{1 \text{ ac.}}\right) = 51,829 \text{ cu. ft.}
\]

- **To account for the routing effect, reduce the channel erosion control volume by 60%:**

\[
V_{ce} = 0.60(51,829) = 31,097 \text{ cu. ft.}
\]
Sizing the Basin for the Q₁ Volume

1. Use the Rational Method triangular hydrograph (HYG) to estimate the volume needed:
   - From 24 hour rainfall (RF) table
     \[\text{RF}_1 = 2.8 \text{ inches} \quad \text{RF}_2 = 3.5 \text{ inches}\]
     \[\frac{\text{RF}_1}{\text{RF}_2} = \frac{2.8}{3.5} = 0.80 \ (80\%)\]
     Thus \(Q_1 = 80\% \text{ of } Q_2\)
     \[Q_2 = 29.6 \text{ cfs}\]
     \[Q_1 = 0.80 \times Q_2 = 0.80(29.6) = 23.7 \text{ cfs}\]
   - Compute the volume from a triangular HYG:
     Using \(t_c = 16 \text{ min.}, \ T_b = 2t_c = 32 \text{ min.}\)
     \[V_1 = 0.5(Q_1)(T_b) \left(\frac{60 \text{ sec}}{\text{min.}}\right)\]
     \[V_1 = 0.5(23.7 \text{ cfs})(32 \text{ min.}) \left(\frac{60 \text{ sec}}{\text{min.}}\right) = 22,752 \text{ cu. ft.}\]
   - Compute the volume from a trapezoidal HYG:
     Using \(t_c = 16 \text{ min.}\) and determining the critical storm duration, \(T_d = 22 \text{ min.}\)
     \[T_b = t_c + T_d = 38 \text{ min.}\]
     \[V_1 = 0.5(Q_1)[(T_d - t_c) + T_b] \left(\frac{60 \text{ sec}}{\text{min.}}\right)\]
     \[V_1 = 0.5(23.7 \text{ cfs})[(22 \text{ min} - 16 \text{ min}) + 38 \text{ min}] \left(\frac{60 \text{ sec}}{\text{min.}}\right) = 31,284 \text{ cu. ft.}\]

NOTE: Calculation is for entire volume of hydrograph

It is noted that this drainage area is sensitive to the critical storm duration of 22 minutes. For the \(Q_1 = 23.7 \text{ cfs}\) with \(t_c = 16 \text{ minutes}\) and the duration = 22 minutes, the volume of the HYG = 31,284 ft³ which is very close to the volume of 31,097 ft³ as calculated using the average hydraulic head method.
2. Determine the required orifice size:
   - To achieve the $Q_1$ volume at a safe ponded depth, assume a depth, $h = 3.0$ ft.
   - Find $Q_{avg}$ for the required 24-hour drawdown for $Q_1$ Control:
     \[
     Q_{avg} = \frac{V_{ce}}{\text{Time}} = \frac{31,097 \text{ cu. ft.}}{24 \text{ hr (3600 sec/hr)}} = 0.360 \text{ cfs}
     \]

3. Determine the orifice size:
   - Determine $h_{avg}$
     \[
     h_{avg} = \frac{3.0}{2} = 1.5 \text{ ft.}
     \]
   - Using the rearranged orifice equation:
     \[
     A = \frac{Q_{avg}}{C \sqrt{2gh_{avg}}} = \frac{0.360}{0.6 \sqrt{2(32.2)(1.5)}} = 0.061 \text{ sq. ft.}
     \]
   - From Table 11-10, use a 3 ½-inch orifice with an area = 0.067 ft$^2$. 
11.9 References


Virginia Department of Environmental Quality Technical Bulletin #6: Minimum Standard 3.11C Filterra Bioretention Filter System (Revised 11/01/02)

Virginia Department of Environmental Quality, Guidance Memo No. 16-2001 - Updated Virginia Runoff Reduction Method Compliance Spreadsheets - Version 3.0, May 2016


Sandvik, A. March, Proportional Weirs for Stormwater Pond Outlets, Civil Engineering, ASCE, pp. 54-56, March 1985


### Appendix 11B-1

**Stormwater Management and Temporary Sediment Basin Summary**

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<thead>
<tr>
<th>OUTLET LOCATION STA</th>
<th>Total Drainage Area (AC)</th>
<th>Road Area (AC)</th>
<th>Area of Impervious Area (AC)</th>
<th>Peak Discharge (CFS)</th>
<th>Peak Discharge (10-year)</th>
<th>Peak Discharge (2-year)</th>
<th>% Diff.</th>
<th>% Diff.</th>
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# Appendix 11B-2 Redevelopment/Surplus Credit Tracking Form

Sample sheet:

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<td>State</td>
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<td>River Basin</td>
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<td>Was the redevelopment project implemented after 7/1/09?</td>
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<tr>
<td>Did the project result in net reductions in pollutants?</td>
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<tr>
<td>Is the credit for treatment of VDOT ROW exclusively?</td>
<td>Yes ☐ NO ☐</td>
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<td>Stormwater Quality Criteria Utilized:</td>
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<tr>
<td>Part 11B (new)</td>
<td>☐</td>
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<tr>
<td>Part 3C (old)</td>
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<tr>
<td>Is the project located in an urbanized area?</td>
<td>Yes ☐ NO ☐</td>
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<td><strong>Notes</strong></td>
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<td><strong>Crediting Information</strong></td>
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<td>(*Include only the creditable portion associated with redevelopment or surplus that was not offsetting new development)</td>
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<td>Summary of Nutrient Crediting</td>
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<td>TP Credit (lb/yr)*:</td>
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<tr>
<td>TN Credit (lb/yr)*:</td>
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<tr>
<td>TSS Credit (lb/yr)*:</td>
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<tr>
<td>DA Entirely in MS4 UA?</td>
<td>Yes ☐ NO ☐</td>
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| *These values represent load reductions creditable for C8 TN05 purposes.
## Chapter 11 – Stormwater Management

### BMP Information

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<th>Type of Permanent BMP</th>
<th>Gross Swale Level 1</th>
<th>TP (CBP60)</th>
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<td>Total Imp to BMP (ac)</td>
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<td>Total Pervious to BMP (ac)</td>
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</tr>
<tr>
<td>% Vol Red</td>
<td></td>
<td></td>
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<td>% Storm Eff</td>
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<tr>
<td>% Total Mass Load</td>
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<td>Load Removed (CBP)</td>
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<td>Load Removed (VIRM)</td>
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Regional BMP (T/H): CBP60 BMP Classification

*Replicate above information for each BMP as needed.

### Site Summary

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<td>Total Existing Impervious (ac)</td>
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<td>Total Existing Pervious (ac)</td>
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<td>New Impervious (ac)</td>
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<td>Required TP Reduction, New IC (lbs/yr)</td>
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<td>Allowable TP Load, New IC (lbs/yr)</td>
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<td>Required TP Total Reductions, (lbs/yr)</td>
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<td>TP Load Removed by BMPs (lbs/yr)</td>
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<td>TP Creditable Reduction, Adjusting for New IC (lbs/yr)</td>
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<td>TN Creditable Reduction (lbs/yr)</td>
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<tr>
<td>TSS Creditable Reduction (lbs/yr)</td>
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Location of Crediting Forms: [ ]

Link to Crediting Forms: [ ]

Notes: [ ]

Preparer Name: [ ]

Title: [ ]

Date: [ ]

Address: [ ]

Email: [ ]

Phone: [ ]
### Implementation Information

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<th>Est. Time to Construct (mo):</th>
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<td>Project Accepted and Completed*</td>
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<tr>
<td>*Includes planning and initial maintenance</td>
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*Inspection/Acceptance Form Location:*

**Notes:**

**Certified by:**

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<th>Date:</th>
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</table>

**Address:**

<table>
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### Maintenance/Inspection Information*

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* All ongoing maintenance and verification is assumed to be associated with the VDOT Maintenance database and Inspection/O&M program. Note the data referred.

### Documents/Tracking Information

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<th>Location of Plans on File:</th>
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<tr>
<td>Location of Photos on File:</td>
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<tr>
<td>Location of Permits on File:</td>
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## Appendix 11C-1 Equivalent Runoff Curve Number (RCN) for Rational ‘C’

### LAND COVER & HYDROLOGIC COVER TYPE & HYDROLOGIC CONDITIONS

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<th>LAND COVER</th>
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<th>0.15 to 0.20</th>
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<td>Commercial and business</td>
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<td>58</td>
<td>57</td>
<td>56</td>
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### EQUIVALENT RUNOFF CURVE NUMBERS

| LAND COVER | Curve Numbers for Hydrologic Soil Group* | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 |
| Residential | 95 | 94 | 93 | 92 | 91 | 90 | 89 | 88 | 87 | 86 | 85 | 84 | 83 | 82 | 81 | 80 | 79 | 78 | 77 | 76 | 75 |
| Commercial and business | 72 | 71 | 70 | 69 | 68 | 67 | 66 | 65 | 64 | 63 | 62 | 61 | 60 | 59 | 58 | 57 | 56 | 55 | 54 | 53 | 52 | 51 |

### FOOTNOTES

- If the accurate soil information is not available, use Soil Group B.
- **The Rational method, including drop losses.
- ****The S.I.P.O.M. method.
# Chapter 12 – Riverine Analysis

## Table of Contents

**CHAPTER 12 - RIVERINE ANALYSIS** ................................................................................................... 12-1

12.1 Introduction ................................................................................................................................. 12-1  
12.1.1 Purpose 12-1  
12.1.2 Definitions .......................................................................................................................... 12-1  
12.1.3 Analysis/Design ..................................................................................................................... 12-2  

12.2 Design Policy/Criteria .................................................................................................................. 12-2  
12.2.1 Federal Floodplain Compliance .............................................................................................. 12-2  
12.2.2 AASHTO General Criteria ..................................................................................................... 12-2  
12.2.3 Department Criteria .............................................................................................................. 12-3  
12.2.3.1 Design Storm .................................................................................................................... 12-3  
12.2.3.2 Bridges ............................................................................................................................ 12-3  
12.2.3.2.1 Freeboard ..................................................................................................................... 12-3  
12.2.3.2.2 Scour ............................................................................................................................ 12-3  
12.2.3.2.3 Riprap Abutment Slope Protection ................................................................................ 12-3  
12.2.3.3 Culverts ............................................................................................................................ 12-3  
12.2.3.3.1 Multibarrel Culverts ..................................................................................................... 12-3  
12.2.3.3.2 Culvert Countersinking ................................................................................................. 12-4  
12.2.3.3.3 Culvert Headwalls ......................................................................................................... 12-4  
12.2.3.3.4 Inlet and Outlet Protection ........................................................................................... 12-4  
12.2.3.4 Stream Reach Impacts ....................................................................................................... 12-4  
12.2.3.4.1 Longitudinal Encroachments ....................................................................................... 12-4  
12.2.3.4.2 Stream Relocation ........................................................................................................ 12-4  
12.2.3.4.3 Stream Restoration ....................................................................................................... 12-4  
12.2.3.5 Flood control Structure .................................................................................................... 12-4  

12.3 Design Concepts ........................................................................................................................ 12-5  
12.3.1 Hydraulic Computation Methodologies ................................................................................ 12-5  
12.3.2 Bridge Scour or Aggradation ................................................................................................. 12-5  
12.3.3 Riprap 12-5  

12.4 Riverine H&HA .......................................................................................................................... 12-6  
12.4.1 Background .......................................................................................................................... 12-6  
12.4.2 Necessary Resources ........................................................................................................... 12-6  
12.4.3 Hydrologic Analysis ............................................................................................................ 12-6  
12.4.4 Hydraulic Analysis ............................................................................................................... 12-7  
12.4.4.1 Existing Effective Model ................................................................................................. 12-7  
12.4.4.2 Existing Conditions Model ............................................................................................... 12-7  
12.4.4.3 Calibration ....................................................................................................................... 12-7  
12.4.4.4 Proposed Conditions Model ............................................................................................ 12-7
12.5 Tidal H&HA ................................................................................................................... 12-8
12.5.1 Background ....................................................................................................... 12-8
12.5.2 Necessary Resources ....................................................................................... 12-8
12.5.3 Coastal Bridge and Culvert Design Techniques ............................................... 12-9
12.5.4 Computer Modeling ......................................................................................... 12-10
12.5.5 Hydrologic Analysis ........................................................................................... 12-11
  12.5.5.1 Bridge Location ................................................................................ 12-11
  12.5.5.2 Channel Cross Section .................................................................... 12-11
  12.5.5.3 Drainage Area Characteristics ......................................................... 12-12
  12.5.5.4 Storm Tides ..................................................................................... 12-12
  12.5.5.5 Flow Velocity .................................................................................... 12-13
12.5.6 Hydraulic Analysis ........................................................................................... 12-13

12.6 Riprap for Protection of Bridge Abutments and Piers ............................................. 12-15

12.7 Removal of Existing Bridge and Approach Embankments ................................... 12-16

12.8 Temporary Construction Causeway Design ........................................................... 12-18
12.8.1 Background ..................................................................................................... 12-18
12.8.2 Causeway Design ........................................................................................... 12-18
  12.8.2.1 Design Objectives ............................................................................ 12-18
  12.8.2.2 Plans ................................................................................................ 12-18
  12.8.2.3 General Notes .................................................................................. 12-19
12.8.3 Design Procedure ........................................................................................... 12-19

12.9 Documentation ........................................................................................................... 12-20

12.10 References .................................................................................................................. 12-21

List of Figures

  Figure 12-1. Removal of Approach Embankment.......................................................... 12-17
  Figure 12-2. Quantifying Removal of Approach Embankment ...................................... 12-17
  Figure 12-3. Temporary Construction Causeway Design.............................................. 12-19

List of Appendices

  Appendix 12B-1 LD-23 Structure and Bridge Data Sheet
  Appendix 12C-1 Tidal Bridge Scour Data and Worksheet
  Appendix 12C-2 Table of Storm Tide Description of Virginia Coast
  Appendix 12C-3 Virginia Coastal Maps Showing Predicted Water Surface Elevations
Chapter 12 - Riverine Analysis

12.1 Introduction

12.1.1 Purpose

The purpose of this chapter is to describe the means and methods for performing riverine analysis associated with stream crossings, longitudinal flood plain encroachments, stream restorations and other projects as necessary to document impacts to the base flood elevations and provide relevant design parameters to other divisions.

This chapter will apply when Bridge Scour Estimates are required, or when a Design Hydraulic Study is required AND:

- The base flood exceeds 500 cfs, (unless otherwise exempted by VDOT Hydraulics Staff).
- The combined stream crossing span exceeds 20 feet.
- An open bottom stream crossing is used (Bridge, arch culverts, 3-side structure, etc.)
- Detailed hydraulic results for natural channels or large man-made channels are required

12.1.2 Definitions

Freeboard – Distance from the stream crossing overtopping elevation to the lowest low cord of a bridge.

Longitudinal Encroachment – consists of fill placed within the base flood plain that impacts a longer reach of stream than is typically associated with a stream crossing and is typically for the purpose of constructing new roadway or widening an existing roadway.

Stream Crossing – consists of one or more openings in a roadway embankment to allow water to pass from one side of the roadway to the other.

Stream Realignment – The relocation of a stream into a channel that is not constructed using natural channel design principals.

Stream Restoration – The relocation or reconstruction of a stream using natural channel design principals. The analysis performed in this chapter will be before assessing higher flow and base flood impacts. The natural channel design will be performed and assessed by others.
12.1.3 Analysis/Design
Proper hydraulic analysis and design is as vital as the structural design. Transportation elements within the base flood plains should be designed for:

- Minimum cost subject to criteria
- Desired level of hydraulic performance up to an acceptable risk level
- Mitigation of impacts on stream environment
- Accomplishment of social, economic, and environmental goals

12.2 Design Policy/Criteria

12.2.1 Federal Floodplain Compliance
VDOT is subject to the federal guidance provided in 23 CFR 650 with respect to proposed actions in the base flood plain. VDOT is to apply the guidance to all base flood plains mapped or otherwise. This guidance is in addition to that provided in Chapters 4 and 17.

- Freeboard shall be provided where practical to protect the bridge structured from debris and scour related failure.
- For stream crossings and longitudinal encroachment that are subject to high flows degradation or aggradation of the river should be estimated and contraction and local scour determined. The appropriate positioning of the foundation, below the total scour depth if practicable, should be included as part of the final design.

12.2.2 AASHTO General Criteria
Design criteria are the tangible means for placing accepted policies into action and become the basis for the selection of the final design configuration of the stream-crossing system. Criteria are subject to change when conditions so dictate as approved by the Department.

Following are certain American Association of State Highway Transportation Officials (AASHTO) general criteria adopted by the Department related to the hydraulic analyses for bridges as stated in their highway drainage guidelines and Drainage Manual:

- Velocities through the structure(s) will not damage either the highway facility or increase damages to adjacent property.
- Bridge Pier spacing and orientation and abutment designed to minimize flow disruption and potential scour.
- Foundation design and/or scour countermeasures to avoid failure by scour.
- Minimal disruption of ecosystems and values unique to the floodplain and stream.
12.2.3 Department Criteria

These criteria augment the general criteria. They provide specific, quantifiable values that relate to local site conditions. Evaluation of various alternatives according to these criteria can be accomplished by using a water surface profile program such as those identified in Chapter 16.

12.2.3.1 Design Storm

For stream crossing and longitudinal encroachments the inundation of the travelway and clearance below the low shoulder dictates the level of traffic services provided by the facility. New construction and projects that increase the level of service of the roadway shall have a minimum 18" clearance from the low shoulder of the crossing to the design storm as determined by the functional classification of roadways presented in Chapter 6, Hydrology. The analysis will document the flood elevations for the base flow, 2, 5, 10, 25, 50, 100, and 500 year events.

12.2.3.2 Bridges

12.2.3.2.1 Freeboard

Where possible and practical for bridges on new or upgraded stream crossings the feasibility of providing freeboard will be assessed and documented in the Design Hydraulic Report. For replacement and rehab projects the presence of freeboard will be documented.

12.2.3.2.2 Scour

Design and Check scoured bed elevations are reported to Structure and Bridge for their use to aid in the design of the bridge foundations. These should consider the magnitude of the flood that generates the maximum scour depth up to the 100-year and 500-year events respectively. However the Structure and Bridge Division may request that a lower event be considered the design scour event for certain rural low volume roadways.

12.2.3.2.3 Riprap Abutment Slope Protection

For Spill through bridges riprap slope protection shall be sized using the procedures in HEC-23 and reported to Structure and Bridge. The thickness and bedding requirements shall conform to those reported in section 12.6.

12.2.3.3 Culverts

12.2.3.3.1 Multibarrel Culverts

For Culvert Crossings with multiple barrels the hydraulic analysis shall consider any culvert opening area that is below the natural flood plain elevation up or downstream of the crossing to be obstructed.
12.2.3.3.2 **Culvert Countersinking**
All Culvert crossings should comply with the USACOE countersinking requirements as outlined in Chapter 8 of this manual.

12.2.3.3.3 **Culvert Headwalls**
All new and replacement culverts shall include headwalls as outlined in Chapter 8 of this manual.

12.2.3.3.4 **Inlet and Outlet Protection**
Culverts shall be assessed using the procedures in Chapter 8 to determine if outlet protection is necessary and if so the extent.

12.2.3.4 **Stream Reach Impacts**

12.2.3.4.1 **Longitudinal Encroachments**
When a project places fill that is parallel to a stream within a floodplain this is considered a longitudinal encroachment. This reduces the effective flow area of the floodplain and has the potential to increase the flood elevations. The analytical methods described in this chapter shall be applied to projects of this type.

12.2.3.4.2 **Stream Relocation**
In many cases it is necessary to shift a stream from its original alignment to facilitate roadway construction and it is not feasible to apply natural channel design methodology. The analytical methods described in this chapter shall be applied to projects of this nature.

12.2.3.4.3 **Stream Restoration**
To meet the environmental regulations VDOT is engaged in a number of projects to improve impaired streams through restoring sediment transport and creating wildlife habitats. The filling of oversized channels and the planting of riparian vegetation has the potential to adversely impact flood elevations. The analytical methods described in this chapter shall be applied to projects of this nature.

12.2.3.5 **Flood control Structure**
A structure that is not designed as a flood control structure, an impounding structure or a dam shall not be evaluated as such to reduce the hydrology or ultimate headwater elevation. VDOT does not encourage the use of roadways as dams for impounding water. It is not VDOT practice not accept responsibility for portions of new roadways that have been designed as dams. Where there are existing roadways that are located on dams, it may be necessary to address current DCR Dam Safety Regulations with the dam owner prior to making any improvements to the roadway or outfall structure.
12.3 Design Concepts

12.3.1 Hydraulic Computation Methodologies

At a minimum, a one dimensional step-backwater computer model will be employed to perform the hydraulic analysis in these situations due to the complexity of the hydraulic conditions and the risk involved. No single method is ideally suited for all situations. If a satisfactory computation cannot be achieved with a given method, an alternate method should be attempted. Where the use of a one-dimensional step backwater computer model is indicated, the Department accepts any computer model currently accepted by FEMA but prefers HEC-RAS. A two dimensional flow model may be necessary to adequately represent the hydraulic conditions found at the stream and road crossings. The Department accepts any current two dimensional flow model accepted by FEMA but prefers SRS2HD. See Chapter 16 for additional discussion regarding software.

12.3.2 Bridge Scour or Aggradation

The Department employs the procedures and criteria presented in the FHWA’s “Evaluating Scour at Bridges” (HEC-18) and “Stream Stability at Highway Structures” (HEC-20) to determine and counteract the impact of scour and long term aggradation/degradation on bridges.

12.3.3 Riprap

Riprap is not to be used for scour protection at piers for new bridges. Riprap may be used to protect exposed abutment slopes. It may be used as a scour countermeasure at existing bridge piers as well as new or existing abutments or lateral encroachments. Design guidelines for placement and sizing of riprap are presented in the FHWA’s “Bridge Scour and Stream Instability Countermeasures” (HEC-23) publication. To qualify as a scour countermeasure the riprap must be placed as prescribed in HEC-23, otherwise it is only considered as being slope protection.
12.4 Riverine H&HA

12.4.1 Background

A Detailed Hydraulic Study as called for by the Location Hydraulic Study should typically be performed for the Department's new or substantially modified replacement major drainage structures and bridged waterways, significant longitudinal encroachments and stream restoration projects. Detailed analysis, as used here, means that the hydraulic analysis shall be performed using an appropriate model. In the case of Department construction in or in proximity to a FEMA detailed study floodplain, the existing effective analysis should be obtained as the potential basis for a revised model.

12.4.2 Necessary Resources

The resources necessary to perform an H&HA usually include, but would not be limited to: topographic maps, aerial photographs, maintenance records for the existing bridge, bridge data sheet, bridge situation survey, proposed bridge plans, sufficient roadway plans, profiles, and typical sections to cover the width of the floodplain in the vicinity of the crossing.

In the event a FEMA floodplain (or other officially delineated floodplain) is involved, it will be necessary to have any available flood profiles, maps, and hydraulic model data. The Department will secure and provide any necessary hydraulic model data. In the event a bridged waterway is involved, it will be necessary to have a schematic bridge layout or proposed bridge plan, bridge situation survey, and bridge data sheet.

12.4.3 Hydrologic Analysis

It will be necessary to determine a range of design peak discharges to use in the subsequent hydraulic analysis. See Chapter 6, Hydrology, for detailed information on application and procedures of hydrologic methods.

Methods employing total storm runoff (i.e. a hydrograph) consideration, such as the USACE HEC-1 or HEC-HMS or the NRCS' TR-20 and TR-55 models can be employed but aren't normally be necessary unless a hydrograph (as opposed to an instantaneous peak) is otherwise needed as in the case of an impoundment structure.

If the site is covered by a FEMA (or other officially delineated) floodplain, the published discharges shall be compared to other established methods and assess if they are within acceptable limits. If so, the published peak discharges employed in making the official floodplain delineation shall be employed. If it is determined that the published discharges are not valid this should be documented and submitted in the Design Hydraulic Study.

In all cases the base flow, 2, 5, 10, 25, 50, 100, and 500-year flood magnitudes will either be determined or obtained (from appropriate sources) and employed in the subsequent hydraulic analysis.
12.4.4 Hydraulic Analysis

When an existing study is available the Department prefers a three-step procedure for performing the hydraulic analysis using an approved model. When there is no Existing Effective Model then proceed directly to the Existing Conditions Model. When the existing effective model includes a floodway this shall be included in the analysis procedure.

12.4.4.1 Existing Effective Model

If a FEMA (or other officially delineated) floodplain is involved, the first step will be to mathematically reproduce the hydraulic model if practicable using the same computer model on which the original floodplain was based. If the computer model used to perform the original hydraulic analysis is no longer available or is not readily available, one of the approved computer models may be employed as long as it is adjusted to match the official model as closely as practicable. In accordance with FEMA directives HEC-RAS may be used in lieu of the originally employed computer model provided: (1) the program version employed must be at least version 3.1.1 or higher and (2) it ties back in to the original study model within 0.5’ at the upstream and downstream ends for the reach being modeled. Any exception to this criterion must be approved by the VDOT Hydraulics Section. The first hydraulic model will be referred to as the "EXISTING EFFECTIVE" model.

12.4.4.2 Existing Conditions Model

The next step would be to use the most recent survey and detailed bridge data to create or update any natural ground cross sections to the locations necessary to subsequently model any proposed construction. Any new model should extend sufficiently downstream and upstream of the area of construction to adequately evaluate the conditions. This is typically at 1000’ in both directions. It should be emphasized that any changes made in the "EXISTING EFFECTIVE" model should be for the purpose of facilitating the modeling of proposed conditions, updating the survey and correcting any observed errors in modeling methodology within the area of the project. This model then becomes the basis for measurement of any changes that would take place as a result of the proposed construction. This second hydraulic model will be referred to as the "EXISTING CONDITIONS" model.

12.4.4.3 Calibration

Reasonable efforts should be made to assess any existing studies or historical data (if available) at the crossing and if the model can be calibrated using this information. Calibration efforts should be discussed in the submitted documentation.

12.4.4.4 Proposed Conditions Model

Once the Existing Condition Model has been calibrated the hydraulic model will be modified to include any and all proposed construction and will be referred to as the "PROPOSED CONDITIONS" model. The allowable impacts to the 100-year flood elevation are subject to the limitations discussed previously in Chapter 17.
12.5  **Tidal H&HA**

12.5.1  **Background**

A detailed hydrologic and hydraulic analysis (H&HA) should be performed for all of the Department's new or replacement major tidal drainage structures and/or bridged tidal waterways. It is necessary to do this in order that VDOT construction be in compliance with national (i.e., FHWA, FEMA, etc.), state (DCR, etc.), and municipal (locally delineated floodplains) rules and regulations. The recommended procedures are described in the FHWA publication “Tidal Hydrology, Hydraulics and Scour at Bridges” (HEC-25).

Detailed analysis, as used here, also means that a three-level analysis such as outlined in HEC-20 and HEC-18 will be employed to evaluate the potential for scour around bridge foundations in order to design new and replacement bridges to resist scour. The complexity of the hydraulic analysis increases if the tidal structure or bridge constrict the flow and affect the amplitude of the storm surge (storm tide) so that there is a large change in elevation between the ocean and the estuary or bay, thereby increasing the velocities in the constricted waterway opening.

12.5.2  **Necessary Resources**

The resources necessary to perform an H&HA of tidal crossings, as for riverine crossings, usually include, but would not be limited to: topographic maps, aerial photographs, maintenance records for the existing bridge, bridge data sheet, bridge situation survey, proposed bridge plans, sufficient roadway plans, profiles, and typical sections to cover the width of the floodplain in the vicinity of the crossing.

Other resources that may be necessary for tidal analysis are: velocity meter readings, cross section soundings, location of bars and shoals, magnitude and direction of littoral drift, presence of jetties, breakwater, or dredging of navigation channels, and historical tide records. Sources of data include NOAA National Ocean Service, USACE, FEMA, USGS, U.S. Coast Guard, local universities, oceanographic institutions and publications in local libraries. NOAA maintains tidal gage records, bathymetric charts, and other data on line at www.nos.noaa.gov. Also refer to Chapter 13, Shore Protection, for details on working with tidal datum.
12.5.3 Coastal Bridge and Culvert Design Techniques

The hydraulic design guidelines for coastal or tidally influenced waterway bridge openings are complicated phenomenon is difficult to simulate. Coastal waterways are subject to storm surges and astronomical tides which play an important role in hydraulic behavior. The collection of adequate data to represent the actual condition also adds to the complexity of the problem. Data such as flows and storm surge description may be difficult to estimate. For small bridges, complex modeling may not be cost effective since the cost of the study may exceed the cost of the bridge.

Presently there is no standard procedure for the design of tidally influenced waterways. In many cases, the bridge hydraulic opening is designed to extend across the normal open water section. This may be an appropriate design from an economic standpoint; since the total cost of a larger bridge may approximates the cost of a smaller bridge considering approach embankments and abutment protection measures. This design is also desirable from an environmental perspective since it results in minimal environmental impacts. In most designs, the extent of detail in the analysis must be commensurate with the project size or potential environmental impacts. However, analytical evaluation of the opening is often required and is necessary when a full crossing cannot be considered or when the existing exhibits hydraulic problems. The complexity of these analyses lends themselves to computer modeling.

Because of the lack of standard procedures for the design of coastal waterways, research is being conducted on this matter. A FHWA pooled fund study coordinated by the South Carolina Department of Transportation has developed recommendations for modeling of tidally influenced bridges. In addition to design guidelines, technical research needs to be conducted to better understand the hydraulics in tidally influenced waterways.

Research is needed in the following areas: sediment transport and scour processes, coastal and tidal marsh ecosystems, environmental impacts and the development of comprehensive coastal hydraulics models.
12.5.4 Computer Modeling

Existing models cover a wide range from simple analytical solutions to heavy computer intensive numerical models. Some models deal only with flows through inlets, while others describe general one-dimensional or two-dimensional flow in coastal areas. A higher level includes hurricane or other storm behavior and predicts the resulting storm surges.

One-dimensional steady state models are the most commonly used models because they demand less data and computer time than the more comprehensive models. Most analyses for tidal streams are conducted with steady state models where the tidal effects are not simulated. This may be an adequate approach if the crossing is located inland from the mouth where the tidal effects are insignificant. Computer modeling for steady state hydraulics is generally preformed with the Corps of Engineers HEC-RAS (or HEC-2).

In the event that either tidal fluctuations or tidal storage are significant, simulation of the unsteady hydraulics is more appropriate. Unsteady flow computer models were evaluated under a FHWA pooled fund research project administered by the South Carolina Department of Transportation (SCDOT). The purpose of this study was to identify the most promising unsteady tidal hydraulic models for use in scour analyses. The study identified UNET, FESWMS-2D, and RMA-2V as being the most applicable for scour analysis. The research funded by the FHWA pooled fund project is being continued to enhance and adapt the selected models so that they are better suited to the assessment of scour at tidal bridges.

The pooled fund research project also resulted in guidance on the appropriate methodology to use based on the geomorphic characteristics of the tidal waterway. Where complicated hydraulics exists, for instance as in wide floodplains with interlaced channels or where flow is not generally in one direction, a one-dimensional model may not represent adequately the flow phenomena and a two-dimensional model is more appropriate. Two-dimensional models in common use to model tidal flow hydraulics are FESWMS-2DH and RMA-2V. FESWMS-2DH, a finite element model was prepared for the FHWA by David C. Froehlich and includes highway specific design functions such as pier scour, weirs, and culverts. RMA-2V, also a finite element model, was developed by the US Army Corps of Engineers. FESWMS-2DH and RMA-2V can also incorporate surface stress due to wind. These models require considerable time for model calibration. Thus, they do not lend themselves for analysis of smaller structure sites.

The US Army Corps of Engineers’ UNET model is widely accepted in situations where the more complicated two-dimensional models are not warranted or for use in making preliminary evaluations. UNET is a one-dimensional, unsteady flow model. The Corps of Engineers has now modified HEC-RAS to incorporate dynamic routing features similar to UNET.
Alternatively, either a procedure by Neill for unconstricted waterways, or an orifice equation for constricted tidal inlets can be used to evaluate the hydraulic conditions at bridges influenced by tidal flows. The procedure developed by Neill can be used for tidal inlets that are unconstricted. This method, which assumes that the water surface in the tidal prism is level, and the basin has vertical sides, can be used for locations where the boundaries of the tidal prism can be well defined and where only small portions of the inundated overbank areas are heavily vegetated or consists of mud flats. The friction loss resulting from thick vegetation tends to attenuate tide levels thereby violating the assumption of a level tidal prism. The discharges and velocities may be overestimated using this procedure. In some more complex cases a simple tidal routing technique (TIDEROUT) or a simple UNET or other 1-dimensional model (HEC-RAS) can be substituted with a similar level of effort. UNET includes storage areas that are assumed to fill as level pools.

12.5.5 Hydrologic Analysis

The flow associated with a tidal bridge generally consists of a combination of riverine and tidal flows. VDOT’s Tidal Bridge Scour Data & Worksheet (Appendix 12C-2) will be used to calculate both the tidal and riverine flow components for tidal crossings. This worksheet utilizes a “VDOT only” modification of Neill’s method for calculating tidal flow and USGS Regression equations for riverine flow. The data required to complete this worksheet is generally available from field data and limited research. A discussion which addresses the information needed to complete the Tidal Bridge Scour Data & Worksheet follows.

12.5.5.1 Bridge Location

- Bridge Number, Route, County, Length and River Crossing can be obtained from bridge plans and inspection reports.

- Tidal Bridge Category:
  - **Islands**: Passages between islands or between an island and the mainland where a route to the open sea exists in both directions.
  - **Semi-Enclosed Bays & Inlets**: Inlets between the open sea and an enclosed lagoon or bay where most of the discharge results from tidal flows.
  - **Estuaries**: River estuaries where the discharge consists of river flow as well as tidal flow.

12.5.5.2 Channel Cross Section

Channel cross section data may be obtained from several sources such as VDOT Central or District offices, bridge plans and/or bridge inspection reports.
12.5.5.3 Drainage Area Characteristics

Drainage area characteristics are required for estimating peak flood discharges using the USGS regression equations for Virginia. (See FHWA Tidal Pooled Fund Study “Tidal Hydraulic Modeling for Bridges” Section 3.4 for guidance in combining storm surge and upland runoff.)

- Drainage area estimated from USGS topographic maps (1:24000), NOAA Navigation maps or similar topographic maps from other sources such as county topographic maps.
- Percentage of forested area, main channel slope, average basin elevation and main channel length can be estimated from USGS topographic maps, street maps or other types of topographic maps.

12.5.5.4 Storm Tides

- The surface area of the tidal basin is required for estimating tidal flows. From USGS topographic maps or NOAA navigation maps, the surface area of the tidal basin can be obtained by planimetering several different contour line levels, and then developing a graph of the surface area versus elevation. Since the maximum tidal flow normally occurs at mid-tide, the preferred method of analysis is to determine the surface area of the tidal basin at this elevation. The surface area of the tidal basin at the mid-tide elevation can be determined from the graph by interpolation.
- The 10, 50, 100 and 500-year storm tides can be obtained from the maps and figures of the coastal regions of Virginia located in the appendix. The maps and table of storm tide description have been compiled and developed from existing FEMA Flood Insurance Study reports, NOAA tidal records, US Army Corps’ tidal analysis and Ho’s Hurricane Tide Frequencies Along The Atlantic Coast.
- Tidal flow is the product of the surface area and the rate change of the tidal height and may be expressed by the following equation:

\[ Q = 24,312 \frac{A_s}{T} H \]

Where:
- \( Q \) = Tidal flow, in cfs
- \( A_s \) = Surface area of the tidal area upstream from the bridge at the mid-tide elevation, in sq. mi.
- \( H \) = Tidal height, between low tide and high tide, in feet.
- \( T \) = Period of the storm tide, in hours. (See Note 1)

Note 1: Obtain both \( H \) and \( T \) from the maps and table in Appendix 12C-3 and 12C-4.
12.5.5.5 Flow Velocity

The flow velocities should be calculated for the flow conditions that may result in higher velocities. These conditions include: (a) the peak riverine flow with a low downstream water level and (b) the combined tidal flow and the flood peak flow, with the water level at the mid-tide elevation.

There is an additional condition, (c), that needs to be investigated for tidal bridges located on estuaries some distance upstream from a bay or ocean. The flow depth at bridges in such cases is less likely to be controlled by the tidal elevation in the bay and more likely to be controlled by the channel slope, boundary roughness and channel geometry. Using the low sea level to calculate the flow velocity for such bridges may result in an unreasonably high velocity due to underestimation of the flow depth and cross-sectional area. Manning’s equation should be used to estimate the flow velocity in such cases. Engineering judgment should be applied when estimating the flow conditions and appropriate flow depth to be used in calculating the velocity of flow. Particular attention needs to be directed at determining the appropriate combination of riverine and tidal flows for use in estimating worst case scour conditions.

The flow velocities estimated by the above methods represent an approximate value for use in the screening process. A detailed H&H Study is required if a more accurate estimation of velocities is desired.

The analysis of the flow velocity in this Worksheet assumes steady flow even though tidal flow is an unsteady flow phenomenon. The resulting velocity will generally be slightly different from a velocity calculated on the basis of unsteady flow. Since the rate of the vertical motion of storm tides is on the order of only three to eight thousandths of a foot per second, the velocity estimates obtained from the method discussed above should be reasonable for locations in unconstricted bays and estuaries where velocities are on the order of 3 fps or less.

Maps and figures of the coastal regions of Virginia that describe the tidal storm surge periods and predicted water surface elevations for the 10, 50, 100 and 500-year storms are shown in Appendix 12C-3 and 12C-4

12.5.6 Hydraulic Analysis

VDOT’s Tidal Bridge Scour Data and Worksheet (Appendix 12C-2) will be used during the Level 1 Analysis (see HEC-18) in order to estimate the maximum flow velocities through the tidal bridge during the passage of a storm tide. This estimate should be considered as a first approximation for use in judging whether the proposed tidal bridge requires a more detailed H&HA.

Normally, Neill’s method of analysis should provide an acceptable degree of accuracy for tidal inlets and estuaries that are not significantly constricted and where flow velocities are 3 fps or less.

Due to the magnitude of the changes to this section, shading has been omitted.
Where the waterway is constricted and estimated flow velocities exceed 3 fps, it may be appropriate to route the storm tide through the structure for purposes of obtaining a more accurate estimate of storm tide velocities. The TIDEROUT computer program is recommended for use when making calculations involving tide routing through a structure. TIDEROUT is a BASIC computer program developed by Mr. Raja Veeranachaneni, MD SHA. A copy of the TIDEROUT program is available on request from the department’s Hydraulics Section. If the estimated flow velocity from the Tidal Worksheet is 7 fps second or greater, routing of the storm tide through the structure should be considered.

Where the simplified methods yield overly conservative results, the use of routing techniques or unsteady flow computer models (Level 2) will provide more realistic predictions of hydraulic properties and scour.

For certain types of open tidal waterway crossings, worst-case scour conditions may be caused by the action of the wind. In other cases, such as passages between islands or an island and the mainland, the worst-case condition may represent a combination of tidal flow and wind forces. These specialized cases require careful analysis and should be studied by engineers with a background in tidal hydraulics.

Electronic spreadsheets are available which assist in the generation of storm surge hydrographs for use in defining downstream boundary conditions during hydrodynamic modeling. These spreadsheets are available on request from the Department’s Hydraulics Section. Maps showing the locations of ADCIRC stations along the Virginia coast where storm surge hydrographs are available are included in Appendix C of the “Tidal Hydraulic Modeling for Bridges” publication. Also available are spreadsheets that assist in the computation of time dependent scour and wave heights for tidal sites.

The FHWA Tidal Pooled Fund Study’s “Tidal Hydraulic Modeling for Bridges” publication presents guidance on the appropriate methodology to use based on the geomorphic characteristics of the tidal waterway.
12.6 Riprap for Protection of Bridge Abutments and Piers

Riprap is frequently used for protection of the earthen fill slopes employed in spill-through abutments. In such situations, it can serve the two-fold purpose of protecting the underlying shelf abutment and piers against runoff coming from the approach roadway and bridge superstructure as well as from scouring due to impinging flow resulting from floodwaters. However, in order to qualify as a scour countermeasure, the riprap thickness, placement, and coverage must be in accordance with the procedures described in HEC-23. Riprap can also be used around solid, gravity abutments to protect against scour. Riprap is considered an acceptable scour countermeasure for protection of bridge abutments. The use of riprap at bridge piers, on the other hand, is not acceptable for use in new construction and is considered only as a temporary countermeasure in the case of rehabilitation. The Department employs the riprap design procedures presented in the FHWA publication “Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance” (HEC-23).

This riprap shall be placed over an appropriate bedding material. For geotextile bedding under the riprap being used for bridge spill slopes, a stone cushion layer consisting of VDOT no. 25 or 26 aggregate should be placed between the riprap and geotextile bedding in accordance with the following:

- In the case of Class AI and I riprap, the aggregate cushion layer should be 4 inches thick
- In the case of Class II, Class III, Type I, and Type II riprap, the aggregate cushion layer should be 6” thick

When it is found necessary to employ riprap as a scour countermeasure around existing bridge piers for rehabilitation purposes, the design shall be performed in accordance with HEC-23 and/or must be reviewed and approved by an appropriate Department River Mechanics Engineer. Bedding requirements in such situations shall be in accordance with the above or as directed by the District Materials Engineer.
12.7 Removal of Existing Bridge and Approach Embankments

In the process of building a new bridged waterway and approaches, it often becomes necessary to remove all or a portion of the existing bridge approach roadway fill throughout the floodplain area. This is necessary for two reasons. First, leaving portions of the old bridge approaches in place may hinder the hydraulic capacity and efficiency of the new facility. In most instances the hydraulic performance of the new facility is predicated on the complete removal of the old one. Second, many State and/or Federal Environmental review agencies require that the old bridge and roadway approaches be removed in their entirety and the land graded back to its natural contour as a contingency for the issuance of certain environmental permits. The hydraulic engineer responsible for the performance of the hydrologic and hydraulic analysis for the proposed bridged waterway will notify the road designer as to whether or not it will be necessary to remove all or portions of the existing bridge and approaches.

When an existing bridge is to be removed, the bid item for removal of the existing bridge will include the entire superstructure and all portions of the substructure, such as abutments, wing walls and piers, pilings and riprap or slope protection. No portion of the approach roadway embankment is to be included in this bid item.

The limits of the approach roadway embankment to be removed will be furnished to the road designer by the Hydraulics Section and shown included in the Design Hydraulics Study. These limits are to be shown on the road grading plans along with the following note:

“The existing approach roadway embankments will be removed between Station ____________ and Station ________________ and will be included in the quantity for regular excavation.”

When a portion of existing approach embankments are removed for flood control, the remaining approach embankment surface should be graded on an approximate 0.5% slope toward the waterway in such a manner as not to impound any water on the surface after the flood waters have receded or after normal rainfall as shown in Figure 12-1.
The determination of quantities for the removal of approach embankment should be set up on a cubic yard basis and included in the plan quantity for regular excavation. The limits for computing the quantity is a vertical plane through the joint between the approach pavement and the end of the bridge as shown in Figure 12-2.

The road designer will request such additional survey information as is necessary to delineate and estimate the quantities of the embankment to be removed.

The District Engineer must be afforded an opportunity to review and comment on the embankment removal proposal prior to completing the plans.
12.8 Temporary Construction Causeway Design

12.8.1 Background

The need to provide a construction access facility that will not have a significant impact on normal flow conditions is mandated by the state and federal agencies that issue and/or approve the issuance of appropriate environmental permits.

12.8.2 Causeway Design

12.8.2.1 Design Objectives

- Provide a design that is reasonably convenient, economical, and logistically feasible for the contractor to build and remove.

- Provide a design that will not be subject to failure due to normal stream flow conditions. This should consider in-stream obstructions such as piers or islands that could direct high velocity jets at points along the causeway.

- Provide a design that will not cause a significant increase in the base flow (Q=1.1 DA mi^2) stage, will not significantly increase the velocity of flow through the causeway opening(s) for that flow rate, will not significantly alter flow distribution, and will not concentrate flow on the piers and foundations that would subject them to forces for which they were not designed. The causeway’s influence on flood flow elevations should be checked in the event that it does not wash out during a significant flood.

- Provide a design that will not obstruct over 50% of the width of the normal stream unless sufficient temporary drainage structures are placed under the causeway to offset greater reduction.

12.8.2.2 Plans

The temporary construction causeway should be designed as a rock prism. The design details and required notes should be shown on the typical section sheets (series 2 plan sheets) in the project plans or on a separate detail sheet for "Bridge Only" projects. A note, "Temporary Construction Causeway Required, See Sheet ______ of _____ for details" should be shown on the road plan sheet where the causeway appears. The design details and required notes for the "Temporary Construction Causeway" will be shown on the front sheet of Bridge plans for "Bridge Only" projects. A typical causeway design detail is shown in Figure 12-3.

The pay item(s) for causeways will be included with the road plans. For "Bridge Only" projects, the causeway pay item(s) will be included in the bridge plans.
The contractor should bid the rock causeway as shown on the plans. The contractor may elect to revise the design or substitute another design after being awarded the contract. If so, he should submit a revised design including necessary sketches and notes for review by the district construction, hydraulic and environmental personnel. The Department should obtain a revised environmental permit if necessary, for the contractor's revised design.

The material used in construction of the causeway should normally be Standard Class I Dry Riprap. The Hydraulic Engineer performing the hydraulic design for the causeway may, at his discretion, specify or authorize larger stone for the causeway's slope faces but must have sufficient justification for doing so. Such justification should be fully addressed in the hydraulic design documentation.

![Figure 12-3. Temporary Construction Causeway Design](image)

Show the top of the causeway as being 3’ over “base flow”.

### 12.8.2.3 General Notes

1. The basis of payment for the temporary causeway will be lump sum, which price should include all labor, equipment, materials and incidentals needed for construction, maintenance, removal and disposal of the causeway.

2. The Project Engineer may make minor adjustment in the location of the causeway provided that the adjustment does not change the design of the causeway.

### 12.8.3 Design Procedure

Step 1 Set the alignment of the causeway to facilitate construction activity. Set the finished grade 3’± above the base flow elevation. Set the side slope angle at approximately 1.5:1.

Step 2: Determine the required waterway opening(s) and the resulting hydraulic performance using appropriate hydraulic design techniques. It is recommended that pipes be used whose diameter (or rise as appropriate) is 2’ less than the causeway is high. In other words, if the causeway is 6’ high, then use 48” pipe(s).
12.9 Documentation

The results of the detailed analysis shall be incorporated into the Design Hydraulic Report as documented in Chapter 17. This shall include:

- Survey Drawings (by reference)
- Structure Inspection report data as applicable

For all analyses:

- Table comparing the base flood elevations for the existing and proposed conditions throughout the project reach, note if any increases are contained within the VDOT ROW.
- Historical Flood magnitude (may be estimated based on nearest standard event)
- Causeway evaluation results if performed

For stream crossings:

- Table comparing all storms evaluated upstream of a proposed crossing (preferably in the vicinity of the ROW)
- Freeboard provided, if <=0.0’ or for a culvert omit reference or note ‘None’
- Design Storm results and clearance to low shoulder
- Riprap proposed for bridge fill slopes or as needed for culvert outlet protection
- Embankment removal notation if required by Section 12.7
- Countersinking for culverts

For lateral encroachments:

- Table comparing the design storm elevation and low shoulder through the project reach and clearance to low shoulder
12.10 References


U. S. Army Corps of Engineers - HEC-2 Water Surface Profiles

U. S. Army Corps of Engineers - HEC-RAS River Analysis System

Chapter 12 – Bridge & Structure Hydraulics

Appendix 12A-1  LD-293 Form

The documentation to include a completed electronic copy of the LD293 form which is available at the VDOT Website. Sections that are not applicable should be deleted.
Chapter 12 – Bridge & Structure Hydraulics
Appendix 12A-1  LD-293 Form

LD293 (Rev 8/15)  Page 2 of 6

VDOT MAJOR STREAM CROSSING REPORT
UPG: <xxxxx>  Route: <xx> over <xx> in <xx City/County>  Project Number: <xxxxx-xxxx-xxxx>
Prepared By: <xx>  Organization: <xx>  Date: <xx/xx/xxxx>

Allowable Headwater: <xx> ft.  HW/D: <xx>
Skew <xx> degrees to Flood Flow

3. HYDROLOGY
The hydrology used in the hydraulic analysis was based upon the Flood Insurance Data / method using data obtained by VDOT / [other].

Drainage Area at this location is <xx> square miles

4. HYDRAULIC PERFORMANCE
The hydraulic analysis was performed using the FHWA water surface profile computer model WSPRO / USACE water surface profile computer model HEC-RAS (or HEC-2) / [describe the approach, method or model].

<table>
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<th>Discharge (cfs)</th>
<th>Exceedance Probability %</th>
<th>Change in existing flood levels (ft.)</th>
<th>Flood Elev. at common upstream sect &lt;xx&gt; (ft.)</th>
<th>Velocity in the vicinity of the bridge (fps)</th>
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</thead>
<tbody>
<tr>
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<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>1% (natural)</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>1% (floodway)</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>0.2%</td>
<td></td>
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<table>
<thead>
<tr>
<th>Design Summary</th>
<th>Exceedance Probability %</th>
<th>Elevation (ft.)</th>
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<tbody>
<tr>
<td>100-Year Base Flood</td>
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<tr>
<td>Roadway Design Flood</td>
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</tr>
<tr>
<td>Bridge 1 ft. Freeboard Flood*</td>
<td></td>
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<tr>
<td>Overtopping Flood</td>
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<td></td>
</tr>
<tr>
<td>Historical Flood &lt;date&gt;</td>
<td></td>
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</tr>
</tbody>
</table>

* Report the nearest standard storm event analyzed that would provide at least 1 ft. minimum clearance to the lowest rail chord elevation on the bridge deck.

5. APPLICABLE FLOODPLAIN MANAGEMENT CRITERIA
Select applicable statement:
For a project within a FEMA delineated floodplain:
FEMA establishes flood level, flood velocity, and flow distribution and this project is within FEMA community panel number: ☑ and Zone ☑. This project complies with FEMA requirements because there will be no increase in flood levels, velocities or flow distribution. This project complies with DDM-7 of the VDOT Drainage Manual, and the general understanding of floodplain development process coordinated between VDOT, DCR and FEMA; or

For a project in a FEMA floodplain with a Zone A designation that does not have base flood elevations, an increase in the cumulative 100-year flood elevation not exceeding one foot (1.0') is acceptable:

FEMA establishes flood level, flood velocity, and flow distribution and this project is within FEMA community panel number: ☑ and Zone ☑. This project complies with FEMA requirements because there will be no more than a cumulative one foot (1.0') increase in the 100-year flood elevation and velocities and flow distribution will not be significantly altered. This project complies with DDM-7 of the VDOT Drainage Manual, and the general understanding of floodplain development process coordinated between VDOT, DCR and FEMA; or

For a project in a FEMA floodplain with a Zone A or AE designation that does has an increase greater than the allowable limits:

FEMA establishes flood level, flood velocity, and flow distribution and this project is within FEMA community panel number: ☑ and Zone ☑. This project complies with FEMA requirements because a CLOMR has been submitted to the county. This project complies with DDM-7 of the VDOT Drainage Manual, and the general understanding of floodplain development process coordinated between VDOT, DCR and FEMA; or

For projects not within a FEMA delineated floodplain, include the following statement:

This project is not within a delineated FEMA floodplain. There are no FEMA requirements applicable within the project area.

6. FEMA DATA COMPARISON

In order to provide additional data to FEMA to help identify potential candidate areas for restudy, a comparison of the VDOT analyses and the FEMA published study is provided below. Note that these comparisons show the differences in the two analyses and do not represent the actual project impact.

FEMA Datum: <NGVD29 / NAVD88> Adjustment needed

The VDOT data reported in the following tables are based upon the proposed conditions analyses:
Chapter 12 – Bridge & Structure Hydraulics

Appendix 12A-1  LD-293 Form

VDOT MAJOR STREAM CROSSING REPORT
UPC: <XXXXXXX> Route <X> over <X> in <X City/County> Project Number: <XXXX-XXXX-XXXX>
Prepared By: <X> Organization: <X> Date: <XX/XX/YYYY>

HYDROLOGY

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<th>Event</th>
<th>FEMA Discharge (cfs)</th>
<th>VDOT Discharge (cfs)</th>
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<td>100-year</td>
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<td></td>
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<tr>
<td>500-year</td>
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</table>

(if it can be determined, provide a brief discussion of why there are differences between the two analyses)

HYDRAULICS

<table>
<thead>
<tr>
<th>Event</th>
<th>Elevation Upstream of Structure (ft)</th>
<th>Elevation at Upstream Limit of VDOT Study (ft)</th>
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<tr>
<td></td>
<td>FEMA (adj)</td>
<td>VDOT</td>
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<tr>
<td>100-year</td>
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<tr>
<td>500-year</td>
<td></td>
<td></td>
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</tbody>
</table>

(if it can be determined, provide a brief discussion of why there are differences between the two analyses)

7. RIPRAP RECOMMENDATIONS
Riprap for abutment slope protection: <Bridge only - select applicable option or amend as necessary>
- Riprap protection is not being employed; or
- The following riprap protection is being employed:
  - 26" Class I Dry Riprap over 4" no. 25 or 26 aggregate over filter cloth
  - 38" Class II Dry Riprap over 6" no. 25 or 26 aggregate over filter; or
- The indicated riprap protection was sized in accordance with the FHWA's BRIDGE SCOUR AND STREAM INSTABILITY COUNTERMEASURES (HEC-23) publication. Slope protection alone does not qualify as a bridge scour countermeasure

Riprap for outlet protection: <Culvert only - select applicable option or amend as necessary>
- Riprap outlet protection is not required at this structure; or
- The following riprap protection is being employed:
  - 26" Class I Dry Riprap over 4" no. 25 or 26 aggregate over filter cloth
Page 5 of 6

Chapter 12 – Bridge & Structure Hydraulics

Appendix 12A-1  LD-293 Form

LD293 (Rev 8/15)  VDOT MAJOR STREAM CROSSING REPORT

5 of 6 VDOT Drainage Manual

UPC: <XXXXXXX>  Route <X> over <X> in <X> City/County>  Project Number: <XXXXX-XXXXX>
Prepared By: <X>  Organization: <X>  Date: <XX/XX/XXXX>

o  38" Class II Dry Riprap over 6" no. 25 or 26 aggregate over filter
   o The riprap apron should be <X> ft. wide and <X> ft. long; or
   o The design is based upon DEQ approved methodologies

8. STANDARD STATEMENTS <for both bridges and culverts>
CAUSEWAYS: The use of causeways for temporary construction access was not considered in this analysis. If it is subsequently found necessary to use causeways, they must be submitted to the Hydraulics Staff for analysis and documentation.

Select the statement(s) below, as applicable to the project:
Temporary construction access causeways for this project should be composed of <specify>.
The high flow profiles will not be affected.
The causeway will not affect the water surface profile of base flow.
The maximum causeway elevation is <X> ft.
From Abutment A to Station <X>
From Station <X> to Abutment B
Only one will be in place at a time

EROSION AND SEDIMENT CONTROL: An Erosion and Sediment Control Plan will be prepared and implemented in compliance with Virginia Erosion and Sediment Control Law and Regulations, and VDOT's Annual Erosion and Sediment Control Standards and Specifications approved by the Department of Environmental Quality.

STORMWATER MANAGEMENT: Design of this project will be in compliance with the Virginia Stormwater Management Act and Regulations, and VDOT's Annual Stormwater Management Standards and Specifications, as approved by the Department of Environmental Quality.

STREAM BANK STABILIZATION: <The banks should reestablish themselves to the natural conditions. / The Riprap should be placed on all areas that will not support vegetation. / Disturbed areas outside the bridge should be seeded.>

9. BRIDGE SCOUR <Bridge only>
Select the appropriate statement:

- A scour analysis has not been performed at this time. Please provide the Hydraulics Staff with the geotechnical report once it is available; or
- A review of the Geotechnical Report and coordination with the S&B geotechnical staff have determined that the Bridge Foundations will be constructed on scour resistant material. No scour computations have been performed; or
LD293 (Rev 8/15) Page 6 of 6

VDOT MAJOR STREAM CROSSING REPORT
UPC: <XXXXXX> Route <X> over <X> in <X> City/County Project Number: <XXXXXX-
Prepared By: <X> Organization: <X> Date: <XX/XX/XXXX>

- Scour computations and a sketch of the design and check scoured bed profiles are attached. If scour countermeasures are required, a request must be submitted to the Hydraulics Staff for their design and documentation.

10. COUNTERSINKING AND MULTIPLE BARRELL CULVERTS (Culvert only)
(Select ONLY the statements that are applicable)
- The upstream and downstream invert of culverts with diameters greater than 24" (or equivalent) will be countersunk a minimum of 6" below the stream bed; and/or
- The upstream and downstream invert of culverts with diameters equal to or less than 24" (or equivalent) will be countersunk a minimum of 3" below the stream bed; and/or
- At least one barrel of the multiple barrel culvert structure will be countersunk a minimum of 6" for a diameter greater than 24" (or equivalent) or a minimum of 3" for a diameter equal to or less than 24" (or equivalent); and/or
- The width of the countersunk culvert barrel(s) receiving the low flow is approximately the width of the normal stream bed; and/or
- Low flow design measures have been implemented for multiple barrel culverts in which all barrels will be countersunk; and/or
- Culverts on bedrock will be countersunk a minimum of 3" below the stream bed; and/or
- Culverts on bedrock will be countersunk a minimum of 3" at the upstream end and stone step pools, low rock weirs or other measures will be constructed at the downstream end; and/or
- Countersinking of the culverts is not practicable due to <X> (See Drainage Manual Section 8.3.7). See attached supporting documentation.

11. COMMENTS
(Note any channel modifications, floodplain impacts and impact mitigation measures, as well as other data pertinent to the design. Also comment on the feasibility of using a smaller structure.)

This analysis is only applicable to the structure(s) and approaches described. Any changes in these conditions may invalidate this analysis and should be reviewed by Hydraulics Staff.

This design represents the smallest structure practicable for use at this site.

If this project is an interstate or other NHS project and is expected to be in excess of $1,000,000.00, please notify the FHWA that (1) no hydraulic impacts are anticipated or (2) the following hydraulic impacts are anticipated: <list>

If you have any questions or need additional information, please contact <X> at <X> or via electronic mail at <X>.
VIRGINIA DEPARTMENT OF TRANSPORTATION
STRUCTURE AND BRIDGE DATA SHEET

Project_________________________________County_____________________________________
Federal Route Base No.__________________Situation data for design of bridge on Route________
Over____________________________________________________________________________
Plane Coordinates or Latitude and Longitude___________________________________________
Date of Survey:________________________Location (Nearest Town, etc.)____________________

GENERAL INSTRUCTION

Fill out all blanks carefully, giving information on all points. High water data is especially important and
should be thoroughly investigated. Comments on any item covered in Survey Instruction Manual which are
not covered below should be noted on an attached sheet.

HYDRAULIC SURVEY

Existing structure is any structure at, or in the vicinity of the proposed site have comparable drainage area.

Date of original construction: __________________________________________________________

Was present bridge in place at time of extreme high water? ________________________________

Elevation of maximum high water and location:

Upstream side of existing structure Elevation/Location ____________________________________

Downstream side of existing structure Elevation/Location ________________________________

At other locations on the flood plain (describe) _________________________________________

Date of maximum high water: ______Mo.______Yr._______Source of information _____________

Elev. of normal water: ______ Date:__________Mo.__________Yr._____________ 

Source of information ______________________________________________________________________

Amount and character of drift present: _____________________________________________________

Location, description and ID of any water-gaging stations in the immediate vicinity: _____________

Elevation_______________ on gage corresponds to elev.__________________________on survey datum.

REMARKS

_____________________________________________________________________________________

_____________________________________________________________________________________

_____________________________________________________________________________________

_____________________________________________________________________________________

_____________________________________________________________________________________
# Appendix 12C-1  Tidal Bridge Scour Data and Worksheet

## VIRGINIA DEPARTMENT OF TRANSPORTATION

### TIDAL BRIDGE SCOUR DATA & WORKSHEET

Hydraulic Engineer:_________
Date:_________

### I. BRIDGE LOCATION

<table>
<thead>
<tr>
<th>BRIDGE No.</th>
<th>Route:</th>
<th>County No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>_________</td>
<td>_______</td>
<td>_________</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TIDAL BRIDGE CATEGORY:</th>
<th>Islands</th>
<th>Semi-Enclosed</th>
<th>Estuary</th>
<th>Bays &amp; Inlets</th>
</tr>
</thead>
</table>

### II. CHANNEL CROSS SECTION

- Channel Width (U/S 100 ft) \( W_u \) = ______Ft.
- Channel Width (at Bridge) \( W_o \) = ______Ft.
- Width (between abutment) \( W_d \) = ______Ft.
- Average Water Depth (below MSL/MLW/MTL) \( D \) = ______Ft.
- Clearance (from MSL/MLW/MTL to Lower Chord) \( C \) = ______Ft.
- Skew Angle (Centerline of Bridge with Channel) \( \phi \) = ______\(^{\circ}\) (Degrees)

### II. DRAINAGE AREA CHARACTERISTICS

- Drainage Area: ___Sq. Mi.; Forest: \( F \) = ____%;
- Average basin elevation: \( EL \) = _____Ft.
- Main Channel Slope: \( SI \) = _____Ft/Mi;
- Main Channel length: \( L \) = _____Mi.

#### (Information per USGS Report 94-4148 for Virginia Department of Transportation dated 1995)

- Main Channel Slope: \( SI \) = _____Ft/Mi;
- Main Channel length: \( L \) = _____Mi.

#### Peak Discharge Region Used:

- Compute from USGS Regression Equation:
  \[ Q_{100} = \text{___________} \text{CFS}; \quad Q_{500} = 1.7 (Q_{100}) = \text{___________} \text{CFS} \]

### III. STORM TIDES

- 100-year High Tide: \( H_{100} \) = ______Ft. Period: \( T_{100} \) = ______Hrs.
- 500-year High Tide: \( H_{500} \) = ______Ft. Period: \( T_{500} \) = ______Hrs.

#### Surface Area of Tidal basin at MSL:
- at _____Ft.: \( As \) = ______Sq. Mi.
- at _____Ft.: \( As \) = ______Sq. Mi.
- at _____Ft.: \( As \) = ______Sq. Mi.

#### Compute Tidal Flows:
- \( Q_{100} = \text{___________} \text{CFS}; \quad Q_{500} = 1.7 (Q_{100}) = \text{___________} \text{CFS} \]

### IV. FLOW VELOCITY

#### a) Based on Cross Sectional Area at MSL/MLW

\[ V_{100} = Q_{100}/A_1 = \text{___________} \text{Ft}/\text{S}; \quad V_{500} = Q_{500}/A_1 = \text{___________} \text{Ft}/\text{S} \]

#### b) Based on Cross Sectional Area at Midtide Elevation

\[ V_{100} = (Q_{100} + Q_{1100})/(A_1 + W_uH_{100}/2) = \text{___________} \text{Ft}/\text{S}; \quad V_{500} = (Q_{500} + Q_{500})/(A_1 + W_uH_{500}/2) = \text{___________} \text{Ft}/\text{S} \]

#### c) Based on Manning Equation (n = 0.025; s = 0.0005)

\[ V_{100} = 1.2 ((Q_{100} = Q_{100})/W_u)^{0.4} = \text{___________} \text{Ft}/\text{S}; \quad V_{500} = 1.2 ((Q_{500} = Q_{500})/W_u)^{0.4} = \text{___________} \text{Ft}/\text{S} \]

### Attach a Sketch of Cross-Section at Upstream (U/S) Side of Bridge
## Table of Storm Tide Description of Virginia Coast

### DERIVED CHARACTERISTICS

<table>
<thead>
<tr>
<th>STORM TIDE DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>500-Year (P-0.002)</td>
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<tr>
<td>Suitable R/V Probability</td>
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### Part A – General Values for Virginia Tidal Waters

<table>
<thead>
<tr>
<th>Correspondence R/V Value</th>
<th>2 hr</th>
<th>1 hr</th>
<th>0.8 hr</th>
<th>0.7 hr</th>
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</thead>
<tbody>
<tr>
<td>Tide Duration, D – 10 R/V</td>
<td>20 hr</td>
<td>10 hr</td>
<td>8 hr</td>
<td>7 hr</td>
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</table>

### Part B – Specific Example for Hampton Roads

<table>
<thead>
<tr>
<th>Storm Tide Elevation, E</th>
<th>11.2 ft</th>
<th>8.8 ft</th>
<th>7.8 ft</th>
<th>5.8 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>E/D, ft/hr</td>
<td>0.56</td>
<td>0.88</td>
<td>0.97</td>
<td>0.83</td>
</tr>
</tbody>
</table>
Appendix 12C-3   Virginia Coastal Maps Showing Predicted Water Surface Elevations

VIRGINIA DEPARTMENT OF TRANSPORTATION
Maps of the Coastal Regions of Virginia

Figure 24A. Estimated tidal flood elevations for 50-year event.
Appendix 12C-3      Virginia Coastal Maps Showing Predicted Water Surface Elevations

VIRGINIA DEPARTMENT OF TRANSPORTATION
Maps of the Coastal Regions of Virginia

Figure 0. Estimated tidal flood elevations for 100-year event, in feet above NGVD. 10 hours for flood rise and fall is appropriate to hurricane passages causing this event.
Appendix 12C-3  Virginia Coastal Maps Showing Predicted Water Surface Elevations

VIRGINIA DEPARTMENT OF TRANSPORTATION
Maps of the Coastal Regions of Virginia

Figure 2AC. Estimated tidal flood elevations for 500-year event.
January 8, 2012

MEMORANDUM

TO: Bruce Shepard, PE, Central Office Structure and Bridge

FROM: John Matthews, Assistant State Hydraulic Engineer

CC: File

SUBJECT: Hydraulic Commentary for the Route 205 bridge replacement over Tide Mill Stream in Westmoreland County
0205-096-101 UPC 61028

The revised road profile and bridge plans for this project for the design dated October 2011 have been reviewed to assess the potential impact to flooding. In these plans the road profile at the bridge has been raise by approximately 1.5 feet above the existing bridge elevation. This increase in profile elevation tapers to 0 at 180 feet east of the bridge and 150 west of the bridge. The proposed bridge opening is slightly narrower than the existing bridge opening but the low chord has been raised to allow for easier access during inspections. This results in an overall increase in flow area under the bridge but an overall reduction of the area available for overtopping.

The flooding at this location is controlled by storm surge from the Chesapeake Bay. Due to this flood control situation the changes to the structure by this project will have no impact on the 100-year flood elevation at this location.

Coordination with the Structure and Bridge Division indicated that they did not need scour computations for this crossing.

Based on both these factors it was determined that no detailed hydraulic analysis of the crossing was necessary.

Standard Hydraulic Non-Study Permit Statement:
This project should not cause more than minimal changes to peak flow characteristics, should not increase the flooding potential, nor cause more than minimal degradation of the water quality of the stream. This project should pose no restriction to the normally expected range of flows, should withstand expected normal high flows and will not restrict low flows. This project complies with applicable FEMA-approved state floodplain management requirements and the Department of Conservation and Recreation’s annual approval of VDOT’s Erosion and Sediment Control and Stormwater Management Program.
MEMORANDUM

TO: Wendy McAbee, Suffolk District Structure and Bridge
FROM: John Matthews, Central Office L&D Hydraulics
CC: John Dewell, Assistant State Hydraulic Engineer
     David LeGrande, Assistant State Hydraulic Engineer
     Keith Rider, Central Office L&D Hydraulics
     Jack Harrell, Suffolk District L&D Hydraulics
     Jennifer Salyers, Suffolk District Environmental

SUBJECT: Hydraulic Impacts of Virginia Capital Trail over Shellbank Creek

I have been asked by John Dewell, Assistant State Hydraulic Engineer to evaluate the hydraulic impacts of the proposed Virginia Capital Trail crossing of Shellbank Creek in James City County. This crossing is located immediately south of Route 5 approximately 3000 feet east of the intersection with Monticello Avenue.

The FEMA Flood Insurance Maps for James City County reports that this area is subject to flooding from the anticipated 100-year storm surge to elevation 8.5. A detailed riverine analysis of Shellbank Creek was not performed. A detailed analysis does not seem to have been warranted based on the watershed area, which is less than 1 square mile.

Shellbank Creek flows under Route 5 through an existing 6’ x 6’ concrete box culvert approximately 1 mile from the mouth of the stream at the James River. No information concerning any existing problems has been provided. It does not appear that at this location on
Shellbank creek is subject to daily tide cycles. At present the detailed hydraulic computations for this crossing are not available. The existing roadway elevation is approximately elevation 15, which is 12 feet above the invert of the stream at elevation 3.2. The existing roadway is not overtopped by the 100 year surge elevation nor should it be overtopped by flooding from within the watershed. Because of the limited storage area upstream of the roadway excessive velocities are not anticipated during the ebb flows after a storm surge event.

The proposed bridge on the Capital Trail will span the entire flood plain on a timber and steel structure that is 260 feet long. The low chord of the proposed bridge is above elevation 16, which is well above the anticipated 100 year event. The bridge is to be founded on pile bents. Because the outfall of the Route 5 crossing is located only 30 feet upstream, a large portion of the flood plain is not effective for flood flow. Only Bents 1 and 2 will be subject to any potentially high velocities as they are in the vicinity of the flows exiting the Route 5 culvert.

Discussions with the District Structure and Bridge Division report that the bridge is to be founded on piles that may be over 70 feet long will extend well below the streambed. The anticipated scour at this location will be limited because of the upstream constraints imposed by Route 5 and the fact that the watershed is small.

Conclusion:
A detailed hydrologic study of this site to assess the hydraulic impacts of the proposed structure is not warranted. The proposed structure will have no impact to the 100 year flooding and will have very limited impact on the existing flood flows.

If an alternate foundation is considered in lieu of the very long piles currently planned a hydraulic engineer should evaluate the revised design to assess if there is a potential for scour to be a concern.
August 30, 2004

MEMORANDUM

TO: Jason L. Henry, EIT

FROM: John H. Matthews, PE

CC: David M. LeGrande, Assistant State Hydraulic Engineer
    Hurley F. Minish

SUBJECT: Fentress Airfield Road Improvements
0165-131-102, PE101, R201, C501

We recently discussed the potential flooding impacts of the Fentress Airfield Road improvements in the City of Chesapeake. You have asked that the Hydraulics Section to assess the project for any potential impact to a FEMA Designated Floodplain.

The proposed project is to widen and existing roadway from 2 to 4 lanes and improve an existing intersection with the addition of turning lanes. The roadway will not be raised in this process and the proposed lanes will be largely below the existing roadway elevation on the interior of a super elevated turn. Any existing crossing will be extended with the same size pipes that currently exist. There are no defined streams shown crossing the roadway on the USGS quad map. The area is shown to be a swamp. There are existing pipes through the roadway embankment to allow for water movement north towards the river. Route 165 crosses the North Landing River and Intercoastal Waterway just north and east of the project area. There are no proposed improvements at these crossings.
The proposed project is located in a FEME Zone AE. The 100 Year flooding for the North Landing River is noted as at elevation 5.0 for the entire reach evaluated in the FIS. There is no floodway designation on the North Landing River. The FIS reports states that the flooding elevations were established using storm surge conditions and not riverine flooding.

Base on the limited scope of the project and the nature of the flooding, this project should have no impact on the 100-year flooding of the North Landing River. Thus, it is not necessary to perform a detailed H&HA.
# Chapter 13 - Shore Protection

## TABLE OF CONTENTS

**CHAPTER 13 - SHORE PROTECTION**

<table>
<thead>
<tr>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.1 Introduction ........................................................................................................ 13-1</td>
</tr>
<tr>
<td>13.1.1 General ............................................................................................................. 13-1</td>
</tr>
<tr>
<td>13.1.2 Assessing Highway Protection Needs in the Coastal Zone ................................ 13-1</td>
</tr>
<tr>
<td>13.1.2.1 Wave Attack ................................................................................................. 13-1</td>
</tr>
<tr>
<td>13.1.2.2 Littoral Drift ............................................................................................ 13-2</td>
</tr>
<tr>
<td>13.1.2.3 Seasonal Changes in Beach Morphology .................................................... 13-2</td>
</tr>
<tr>
<td>13.1.2.4 Foundation Conditions ................................................................................. 13-2</td>
</tr>
<tr>
<td>13.1.2.5 Corrosion ...................................................................................................... 13-2</td>
</tr>
<tr>
<td>13.1.2.6 Highway Protection Measures ..................................................................... 13-3</td>
</tr>
<tr>
<td>13.1.3 Lakes .............................................................................................................. 13-3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.2 The Coastal and Shoreline Area ........................................................................ 13-4</td>
</tr>
<tr>
<td>13.2.1 Introduction ..................................................................................................... 13-4</td>
</tr>
<tr>
<td>13.2.2 Tidal Elevation Nomenclature ......................................................................... 13-5</td>
</tr>
<tr>
<td>13.2.3 Tidal Elevation Conversion ............................................................................. 13-7</td>
</tr>
<tr>
<td>13.2.4 Design High Water .......................................................................................... 13-8</td>
</tr>
<tr>
<td>13.2.5 Determination of Mean High/Low Tide Levels ................................................ 13-8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.3 Dynamic Beach Processes .................................................................................. 13-11</td>
</tr>
<tr>
<td>13.3.1 Introduction .................................................................................................... 13-11</td>
</tr>
<tr>
<td>13.3.2 Normal Conditions ........................................................................................ 13-11</td>
</tr>
<tr>
<td>13.3.3 Storm Conditions ........................................................................................... 13-12</td>
</tr>
<tr>
<td>13.3.4 Beach and Dune Recovery .............................................................................. 13-14</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.4 Design Waves .................................................................................................... 13-15</td>
</tr>
<tr>
<td>13.4.1 Introduction .................................................................................................... 13-15</td>
</tr>
<tr>
<td>13.4.2 Significant Height and Period .......................................................................... 13-15</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.5 Simplified Methods for Estimating Wave Conditions ..................................... 13-16</td>
</tr>
<tr>
<td>13.5.1 Introduction .................................................................................................... 13-16</td>
</tr>
<tr>
<td>13.5.2 Predicting Wind Generated Waves ................................................................ 13-16</td>
</tr>
<tr>
<td>13.5.2.1 Wave Height ............................................................................................... 13-16</td>
</tr>
<tr>
<td>13.5.2.2 Hindcasting ................................................................................................ 13-16</td>
</tr>
<tr>
<td>13.5.2.3 Forecasting .................................................................................................. 13-16</td>
</tr>
<tr>
<td>13.5.2.4 Breaking Waves ......................................................................................... 13-17</td>
</tr>
<tr>
<td>13.5.2.5 Prediction Procedure .................................................................................. 13-18</td>
</tr>
<tr>
<td>13.5.2.5.1 Wind Speed Estimation ......................................................................... 13-18</td>
</tr>
<tr>
<td>13.5.2.5.2 Site Maximization Procedure ................................................................ 13-18</td>
</tr>
<tr>
<td>13.5.2.5.3 Design Breaker Wave ............................................................................ 13-18</td>
</tr>
<tr>
<td>13.5.2.5.4 Wave Run-up .......................................................................................... 13-19</td>
</tr>
<tr>
<td>13.5.3 Adjustments for Flooded Vegetated Land ...................................................... 13-20</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.6 Flood Prediction Methods .................................................................................. 13-21</td>
</tr>
<tr>
<td>13.6.1 Introduction .................................................................................................... 13-21</td>
</tr>
<tr>
<td>13.6.2 Coastal Flooding ............................................................................................. 13-21</td>
</tr>
<tr>
<td>13.6.3 Tidal Flow Restrictions ................................................................................... 13-22</td>
</tr>
<tr>
<td>13.6.4 Lake Shore Flooding ........................................................................................ 13-22</td>
</tr>
</tbody>
</table>
13.7 Riprap Shore Protection ................................................................. 13-23
  13.7.1 Introduction ............................................................................. 13-23
  13.7.2 General Features ................................................................. 13-23
    13.7.2.1 Shore Protection Design .................................................. 13-23

13.8 References .................................................................................. 13-27

List of Tables

Table 13-1. Correction Factors For Wave Run-up ........................................ 13-19

List of Figures

Figure 13-1. Visual Definition of Terms Describing a Typical Beach Profile .......... 13-4
Figure 13-2. Nomenclature of Tidal Ranges ................................................ 13-6
Figure 13-3. Normal Wave Action on Beach and Dune .................................. 13-11
Figure 13-4. Initial Attack of Storm Waves on Beach and Dune ..................... 13-12
Figure 13-5. Storm Wave Attack of Foredune ............................................. 13-13
Figure 13-6. After Storm Wave Attack of Beach and Dune ........................... 13-13
Figure 13-7. Riprap Rock Shore Protection Typical Design Configuration ....... 13-25

List of Appendices

Appendix 13B-1. Nomographs of Significant Wave Height Prediction Curves as Functions of Windspeed, Fetch Length, and Wind Duration

Appendix 13B-2. Design Breaker Wave

Appendix 13B-3. Wave Run-up on Smooth Impermeable Slope

Appendix 13B-4. Nomographs for Design of Rock Slope Shore Protection (For Shoal Water)

Appendix 13B-5. Nomographs for Design of Rock Slope Shore Protection (For Deep Water)

Appendix 13B-6. Nomograph for Riprap Size to Resist Wave Action
Chapter 13 - Shore Protection

13.1 Introduction

General Highways that encroach upon coastal zones, including bays, estuaries and tidal basins, and the shore of lakes and reservoirs, present unique circumstances which require additional measures to protect the roadway from erosion. Much of the discussion in the other chapters of this manual applies to these unique areas but does not address in detail the special aspects of seasonal variation and extremes of wind, wave, current, and tide upon banks and shores covered in this chapter. The information contained within this chapter may be used for preliminary design, minor design, and general information. Major designs should involve, in addition to the hydraulics engineer, coastal engineers, coastal geologists, oceanographers or other specialists in the interaction of wind, tides and waves. The procedures used should also be from the source documents/manuals, so that the most current methodologies are used.

13.1.1 Assessing Highway Protection Needs in the Coastal Zone

Highways in the coastal environment experience a wide array of threats to long-term stability that are unique to the coastal zone.

13.1.1.1 Wave Attack

The primary threat is from wave attack. The susceptibility of a highway to wave attack is dependent on location relative to tidal elevations, the underlying geology, and the exposure of the coastline.

Headlands and rocks that have historically withstood the relentless pounding of tide and waves can usually be relied on to protect adjacent highway locations. However, because headlands project out from the coast, wave action is usually concentrated at these locations. The need for shore protection structures on headlands are generally limited to highway locations at the top or bottom of bluffs having a history of sloughing and along beach fronts.

Wave attack on a sloping beach is less severe than on a headland, due to the gradual shoaling of the bed that causes incoming waves to break before they reach the shoreline. However, on long shallow sloping beaches, waves can reform after passing over bars.

The relative degree of protection or exposure of the shoreline affects the strength of wave attack. Coastlines exposed to long fetch lengths can be subject to high wave attack from wind-generated waves.

* Rev 9/09

Chapter 13-1 of 27
13.1.1.2 Littoral Drift

Littoral drift of beach sands may either be an asset or a liability. Littoral drift is a normal beach process. If a beach is stable then the net littoral drift is zero. If the amount of sediment brought into a section of beach by littoral drift exceeds outgoing sediment, then a new beach could be built in front of the embankment, reducing the depth of water at its toe and the corresponding height of the waves attacking it. If the net littoral drift is negative (degrading) or subject to seasonal variations, then shore protection measures may be necessary to retain the beach and protect the roadway.

On the other hand, if sand is in scant supply, backwash from revetment tends to degrade the beach or bed, and an allowance should be made for this scour when designing the revetment, both as to weight of stones and depth of foundation. Groins would be ineffective for such locations; if they succeeded in trapping some littoral drift, beaches located down-drift may retreat due to undernourishment.

13.1.1.3 Seasonal Changes in Beach Morphology

Changes in the beach profile occur on a seasonal basis due to changes in the earth’s tilt and seasonal weather variation. Changes in the axial tilt can reverse littoral currents. They also change the heights and ranges of tidal elevations.

Oceans are generally warmer than land during the winter resulting in low pressures over the water surface. Lower pressure results in higher tide ranges than occurs during summer. Thus, beach erosion is increased and “winter beaches” possess a steeper beach profile and more predominant offshore bars. Generally the shift is a recession, increasing the exposure of beach locations to the hazard of damage by wave action. On strands or along extensive embayments, recession at one end may develop accession at the other. Observations made during location should include investigation of this phenomenon. For strands, the hazard may be avoided by locating the highway on the backshore facing the lagoon.

13.1.1.4 Foundation Conditions

Foundation conditions vary widely for beach locations. On a receding shore, good bearing may be found on soft but substantial rock underlying a thin mantel of sand. Bed stones and even gravity walls have been founded successfully on such foundations.

Long straight beaches, spits and strands, are radically different, often with softer clays or organic materials underlying the sand. Sand usually being plentiful at such locations, subsidence is greater hazard than scour and location should anticipate a "floating" foundation for flexible, self-adjusting types of protection.

13.1.1.5 Corrosion

The corrosive effect of salt water is a major concern for hydraulic structures located along the coastline. The long-term effect on special coatings should be monitored.
13.1.1.6 Highway Protection Measures

Highways located in the coastal zone may be protected through two alternative measures – location planning and armoring.

In planning oceanfront locations, alignments should be selected that minimize the threat of wave attack and are located on stable land surfaces.

Often existing roadways cannot be relocated and new roadways must be exposed to wave attack. Structural measures may be used to armor the embankment face, or offshore devices like groins may be used to aggrade the beach at embankment toe.

13.1.2 Lakes

Under the right set of conditions, wind can create large waves on lakes. Height of waves is a function of fetch, so that the larger (or longer) the lake, the higher waves break upon reaching shoals, reducing the effects of erosion along embankments behind shallow coves and increasing the threat at headlands or along causeways in deep water. Constant rippling of tiny waves may cause severe erosion of certain soils.

The erosive force of wave action is a function of the fetch and in most inland waters is not very serious. In fresh waters the establishment of vegetal cover can often provide effective protection, but planners should not overlook the possibility of moderate erosion before the cover becomes established. Any light armor treatment should be adequate for this transitional period.

Older lakes have built thick beds of precipitated silt and organic matter. Bank protection along or across such lakes must be designed to suit foundations available, it usually being more practical to use lightweight or self-adjusting types supported by soft bed materials than to excavate mud to stiffer underlying soils. The warning is especially applicable to protection of causeway embankments.
13.2 The Coastal and Shoreline Area

13.2.1 Introduction

The beach and near-shore zone of a coast is the region where the forces of the sea react against the land. The physical system within this region is composed primarily of the motion of the sea, which supplies energy to the system, and the shore, which absorbs this energy. Because the shoreline is the intersection of the air, land and water, the physical interactions which occur in this region are unique, very complex, and difficult to fully understand. As a consequence, a large part of the understanding of the beach and near-shore physical system is simply descriptive in nature.

Where the land meets the ocean at a sandy beach, the shore has natural defenses against attack by waves, currents, and storms. The first of these defenses is the sloping near-shore bottom that causes waves to break offshore, dissipating their energy over the surf zone. The process of breaking often creates an offshore bar in front of the beach that helps to trip following waves. The broken waves re-form to break again, and may do this several times before finally rushes up the beach foreshore. At the top of wave up-rush a ridge of sand is formed. Beyond this ridge, or crest of the berm, lies the flat beach berm that is reached only by higher storm waves. Figure 13-1 shows a visual definition of the terms used to describe a typical beach profile.

![Figure 13-1. Visual Definition of Terms Describing a Typical Beach Profile](image-url)
The motions of the sea that contribute to the beach and near-shore physical system include waves, tides, currents, storm surges, and tsunamis. Wind waves are by far the largest contribution of energy from the sea to the beach and near-shore physical system. As winds blow over the water, waves are generated in a variety of sizes from ripples to large ocean waves. Following is a discussion of the coastal forces of motion. For more information on the mechanics of waves and shore phenomena, refer to publications developed by the US Army Corps of Engineers, Coastal and Hydraulic Engineering Laboratory. Procedures use in evaluating shore protection were developed by the Corps of Engineers and published as the Shore Protection Manual (1984), referred to as SPM. The Corps of Engineers has updated the SPM. The updated manual is called the Coastal Engineering Manual and was published in 2002.

13.2.2 Tidal Elevation Nomenclature

A depiction of tidal elevations and nomenclature is presented in Figure 13-2. Note that a typical tidal cycle in the Chesapeake Bay and Virginia Atlantic Coast has two highs and two lows. The average of all the higher highs for a long period (preferably in multiples of the 19-year metonic cycle) is mean higher high water (MHHW), and the average of all the lower lows is Mean Lower Low Water (MLLW). The vertical difference MHHW and MLLW is the Diurnal Range (R).
The average of all highs (indicated graphically as the mean of higher high and lower high) is the mean high water (MHW). The average of all lows (indicated graphically as the mean of higher low and lower low) is mean low water (MLW). The vertical difference between MHW and MLW is the mean range.

The maximum tide range \( R_m \) is defined as the vertical difference between the Highest High Tide and the Lowest Low Tide.

Mean Sea Level (MSL) is defined as the mean height of the surface of the sea for all stages of the tide over a Tidal Epoch (19 year Metonic Cycle\(^*\)). Confusion often arises when MSL is referenced as a vertical datum. Mean Tide Level (MTL) is the midway point between MLW and MHW. MSL is not necessarily synonymous with MTL. Often, when a bridge drawing or other document refers to MSL as its vertical datum, it is actually referring to a fixed datum. When the datum is named as MSL, the user of the data must clarify whether the reference is equivalent to MTL for the current tidal epoch. If it is found that MSL actually refers to a fixed datum, the user must determine the relationship between that fixed datum and the datum used in the study at hand.

The fixed datums used most commonly in the United States are the National Geodetic Vertical Datum of 1929 (NGVD29) and the North American Vertical Datum of 1988.

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\* Rev 9/09
(NAVD88). These reference surfaces were established by NGS. High-accuracy benchmarks referenced to these datums have been set by NGS throughout the nation.

NGVD29 was called the Sea Level Datum of 1929 until the name was changed in 1973. Many older plan sets, maps and documents refer to a datum of Sea Level or Mean Sea Level that is actually equivalent to NGVD29. One must not assume this equivalency, however, without verification.

The National Oceanic and Atmospheric Association (NOAA) publishes information on tidal elevations for the east and west coasts of North America and the Gulf of Mexico. Typically, elevations are referenced to MLW, MLLW or a local gage datum instead of National Geodetic Vertical Datum (NGVD 1929 or NAVD 1988). Bathymetric maps are typically referenced to MLW so that mariners may know the depth of water at low tides.

Daily tide predictions for points in the Chesapeake Bay and the Virginia shoreline are based on daily predictions for Washington, D.C., and Hampton Roads, VA. Tidal differences and other constants for locations along the Chesapeake Bay and Virginia shoreline are available from at the following NOAA Internet web site: www.nos.noaa.gov.

### 13.2.3 Tidal Elevation Conversion

A conversion must be made to relate the elevation of Design High Water to the appropriate vertical datum (NGVD 1929 or NAVD 1988, whichever is being used on the project in question) used in Highway Design. A series of geodetic bench marks have been established which permit conversion of local MLLW datum to the more accepted NGVD-29 or NAVD-88. The relationships between MLW and NGVD-29 or NAVD-88 for gage locations in Virginia are available from the following NOAA Internet web site: 

http://tidesandcurrents.noaa.gov/map/index.shtml?type=BenchMarkSheets&region=Virginia

Conversions for intermediate points may be made by interpolating between stations. Additional information on MLLW and NGVD-29 and NAVD-88 may be found by consulting the NOAA publication *Tide Tables High and Low Water Predictions (for the East Coast of North and South America)*.

Conversion of tide elevations should be undertaken with care and independently checked. Common errors are:

- Forgetting to convert from water levels to a vertical datum
- Adding the factor instead of subtracting it
- Using half the diurnal range as the stage of high water

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* Rev 7/14
13.2.4 Design High Water

Small waves and conditions indicate that when protection is necessary, one of the key elements in a successful design is establishing the elevation appropriate for the protection features. The most frequently used term is Design High Water. Design High Water for shore protection is a high stage of the static or still-water level of the sea. The height of Design High Water is referenced to tidal elevations such as Mean Low Water or Mean Sea Level. Except for inland waters affected by wind tides, floods and seiches, the level usually used for design is the highest tide.

13.2.5 Determination of Mean High/Low Tide Levels

It is frequently necessary to determine Mean High Water (MHW) and Mean Low Water (MLW) levels as a requisite for obtaining various federal, state, and/or local environmental permits. The National Oceanic and Atmospheric Administration (NOAA) publishes such information for the east and west coasts of North America and the Gulf of Mexico. Unfortunately, the elevations shown in these publications are predicated on local, mean lower low water (MLLW) datum instead of National Geodetic Vertical Datum of 1929 (NGVD 29) or the newer North American Vertical Datum of 1988 (NAVD 88). A series of geodetic bench marks have been established which permit conversion of mean low water datum to the more meaningful NGVD 29 and/or NAVD 88 Datum. A table of these tidal bench marks for Virginia, including the Chesapeake Bay and its tributaries, is located on the following NOAA's internet web site:

http://tidesandcurrents.noaa.gov/map/index.shtml?type=BenchMarkSheets&region=Virginia

To determine the MHW and MLW elevations, proceed as follows:

(1) Go to the web site shown above and scroll up or down the table of the Tidal Bench Marks until you find the one closest to the location in which you are interested. Click on the link.

(2) The screen will change to show information for the site selected. Scroll down until you see a link designated as “National Geodetic Vertical Datum (NGVD 29)” and click on it. You may also click on “Datums Page” in the top left hand corner and it will take you to the same spot on the page.

(3) The screen will change again, displaying a bar graph showing pertinent tide levels including MHW (Mean High Water), MLW (Mean Low Water) and usually a correction factor for both NGVD-29 and NAVD-88 datums. The MHW and MLW values shown on the graph are in terms of local MLLW (Mean Lower Low Water) datum and must be converted to either NGVD-29 or NAVD-88 datums. An example graphic display is shown below.

* Rev 7/14
(4) If you want MHW and MLW elevations in terms of NAVD-88 datum, subtract the correction factor shown for NAVD-88 datum from the values shown for MHW and MLW.

The NAVD 88 and the NGVD 29 elevations related to MLLW were computed from Bench Mark, TIDAL 7 STA 81, at the station.

Displayed tidal datums are Mean Higher High Water(MHHW), Mean High Water (MHW), Mean Tide Level(MTL), Mean Sea Level (MSL), Mean Low Water(MLW), and Mean Lower Low Water(MLLW) referenced on 1983-2001 Epoch.

**Elevation Information**

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<thead>
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</tr>
</thead>
<tbody>
<tr>
<td>VM:</td>
<td>3457</td>
</tr>
<tr>
<td>Station ID:</td>
<td>3603449</td>
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<td>1903-2001</td>
</tr>
<tr>
<td>Date:</td>
<td>Wed Mar 15 14:54:40 EDT 2000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Datum</th>
<th>Height</th>
</tr>
</thead>
<tbody>
<tr>
<td>MHHW</td>
<td>2.12 feet (0.647 meters)</td>
</tr>
<tr>
<td>MHW</td>
<td>1.93 feet (0.589 meters)</td>
</tr>
<tr>
<td>MSL</td>
<td>1.16 feet (0.354 meters)</td>
</tr>
<tr>
<td>NAVD88</td>
<td>1.06 feet (0.323 meters)</td>
</tr>
<tr>
<td>MTL</td>
<td>1.04 feet (0.317 meters)</td>
</tr>
<tr>
<td>NGVD29</td>
<td>0.31 feet (0.095 meters)</td>
</tr>
<tr>
<td>MLW</td>
<td>0.15 feet (0.045 meters)</td>
</tr>
<tr>
<td>MLLW</td>
<td>0.00 feet (0.000 meters)</td>
</tr>
</tbody>
</table>

Chapter 13-9 of 27
Example: Find the mean low and high water elevations on the James River near Claremont, Virginia in terms of NAVD-88 datum.

Step 1 Consult NOAA’s internet web site, locate the link for the tidal benchmark for the James River near Claremont and click on it.

Step 2 When the screen changes, scroll down the page until you get to the link “National Geodetic Vertical Datum (NGVD 29)” and click on it.

Step 3 When the graphic is displayed note that MHW = 1.93’ and MLW = 0.15’ in terms of local, MLLW datum and that the NAVD-88 correction factor is 1.08’

Step 4 Subtract the NAVD-88 correction factor of 1.08’ from both values to get MHW and MLW in terms of NAVD-88 datum.

\[
\text{MHW} = 1.93 - 1.08 = 0.85' \quad \text{Use 0.9’ for Mean High Water (MHW)}
\]

\[
\text{MLW} = 0.15 - 1.08 = -0.93' \quad \text{Use } -0.9' \text{ for Mean Low Water (MLW)}
\]

Answer: Mean Low Water (MLW) elevation = -0.9’

Mean High Water (MHW) elevation = 0.9’
13.3 Dynamic Beach Processes

13.3.1 Introduction

The beach constantly adjusts its profile to provide the most efficient means of dissipating incoming wave energy. This adjustment is the beach's natural dynamic response to the sea.

There are two general types of dynamic beach response to wave motion: response to normal conditions and response to storm conditions. Under normal conditions, the wave energy is easily dissipated by the beach's natural defense mechanisms. However, when storm conditions generate waves containing increased amounts of energy, the coast must respond with extraordinary measures, such as sacrificing large sections of beach and dune. In time the beach may recover, but often not without a permanent alteration.

13.3.2 Normal Conditions

As a wave moves toward shore, it encounters the first beach defense in the form of the sloping near-shore bottom. When the wave reaches a water depth equal to about 1.3 times the wave height, the wave collapses or breaks. Thus a wave 1’ high will break in a depth of about 1.3’. Breakers are classified as four types—plunging, spilling, surging, or collapsing. The form of breakers is controlled by wave steepness and near-shore bottom slope. Breaking results in a dissipation of wave energy by the generation of turbulence in the water and by the transport of sediment lifted off the bottom and tossed around by the turbulent water. Broken waves often re-form to break again, losing additional energy. Finally, the water travels forward as a foaming, turbulent mass and expends most of its remaining energy in a rush up the beach slope (Figure 13-3).

![Figure 13-3. Normal Wave Action on Beach and Dune](image-url)
If there is an increase in the incoming wave energy, the beach adjusts its profile to facilitate the dissipation of the additional energy. This is most frequently done by the seaward transport of beach material to an area where the bottom water velocities are sufficiently reduced to cause sediment deposition. Eventually enough material is deposited to form an offshore bar that causes the waves to break farther seaward, widening the surf zone over which the remaining energy must be dissipated. Tides compound the dynamic beach response by constantly changing the elevation at which the water intersects the shore and by providing tidal currents.

13.3.3 Storm Conditions

Strong winds generate high, steep waves. In addition, these winds often create a storm surge that raises the water level and allows waves to attack higher parts of the beach not ordinarily subjected to waves. The storm surge allows the large waves to pass over the offshore bar formation without breaking. When the waves finally break, the remaining width of the surf zone is not sufficient to dissipate the increased energy contained in the storm waves. The remaining energy is spent in erosion of the beach, berm, and sometimes dunes that are now exposed to wave attack by virtue of the storm surge.

Eroded material is carried offshore in large quantities where it is deposited on the near-shore bottom to form an offshore bar. This bar eventually grows large enough to break the incoming waves farther offshore, forcing the waves to spend their energy in the surf zone. This process is illustrated in Figure 13-4.

![Figure 13-4. Initial Attack of Storm Waves on Beach and Dune](image)

Beach berms are built naturally by waves to about the highest elevation reached by normal storm waves. When storm waves erode the berm and carry the sand offshore, the protective value of the berm is reduced and large waves can overtop the beach. The width of the berm at the time of a storm is thus an important factor in the amount of upland damage a storm can inflict.
In severe storms, such as hurricanes, the higher water levels resulting from storm surges allow waves to erode parts of a dune. It is not unusual for 50' to 100' wide dunes to disappear in a few hours. Storm surges are especially damaging if they occur concurrently with high astronomical tides (Figure 13-5).

**Figure 13-5. Storm Wave Attack of Foredune**

In essence, the dynamic response of a beach under storm attack is a sacrifice of some beach, and often dune, to provide material for an offshore bar. This bar protects the shoreline from further erosion. After a storm or storm season, natural defenses may again be reformed by wave and wind action (Figure 13-6).

**Figure 13-6. After Storm Wave Attack of Beach and Dune**

The storm surge and wave action may succeed in completely overtopping the dunes causing extensive coastal flooding. When this occurs, beach and dune sediments are swept landward by the water, and in the case of barrier islands, are deposited as overwash fans on the backshore or in the lagoon. This process results in a loss of sand from the dynamic beach system. Often, storm overwash and storm flooding return flow will erode enough sand to cut a new tidal inlet through the barrier island.
13.3.4 Beach and Dune Recovery

Some beach systems may be in quasi-equilibrium. Sediment supplied to the beach from littoral transport replaces sediment lost to overwash. Sediment deposited into offshore bars is redeposited on the beach. Following a storm there is a return to more normal conditions that are dominated by low, long swells. These waves transport sand from the offshore bar, built during the storm, and place the material on the beach. The rebuilding process takes much longer than the short span of erosion which took place.

Alternate erosion and accretion may be seasonal on some beaches; the winter storm waves erode the beach, and the summer swell (waves) rebuilds it. Beaches also appear to follow long-term cyclic patterns, where they may erode for several years and then accrete for several years.

A series of violent local storms over a short period of time can disturb a system in equilibrium and result in severe erosion of the shore because the natural protection does not have time to rebuild between storms. Sometimes full recovery of the beach never occurs because sand is deposited too far offshore during the storm to be returned to the beach by the less steep, normal waves that move material shoreward.

Other beach systems may be in disequilibrium. Erosion of the beach and dune system results in retreat of the beach. Roadways and structures placed near the beach may become threatened over time.
13.4 Design Waves

13.4.1 Introduction

The pattern of waves on any body of water exposed to winds generally contains waves of many periods. Typical records from a recording gage during periods of steep waves indicate that heights and periods of real waves are not constant as is assumed in theory. Wave-lengths and directions of propagation are also variable. Further, the surface profile for waves near breaking in shallow water or for very steep waves in any water depth is distorted, with high narrow crests and broad flat troughs. Real ocean waves are so complex that some idealization is required.

Even for the simplest of cases, the estimation of water levels caused by meteorological conditions is complex. Elaborate numerical models requiring the use of a computer are available, but simplified techniques may be used to predict acceptable wind wave heights for the design of highway protection facilities along the shores of the ocean, embayments, and inland lakes and reservoirs.

Wave prediction is called hindcasting when based on past meteorological conditions and forecasting when based on predicted conditions. The same conditions are used for hindcasting and forecasting. The only difference is the source of meteorological data. Reference is made to the Army Corps of Engineers, Coastal Engineering Manual, and FHWA’s HEC-25, Highways in the Coastal Environment for more complete information on the theory of wave generation and predicting techniques.

The prediction of wave heights from vessel-generated waves may be estimated from observations. Research is underway to provide more information on vessel wakes.

13.4.2 Significant Height and Period

A given wave train contains individual waves of varying heights and period. The significant wave height $H_s$, is defined as the average height of the highest one-third of all the waves in a wave train. $H_s$ is the design wave height normally used for flexible revetments.

Other design wave heights can also be designated, such as $H_{10}$ and $H_1$. The $H_{10}$ design wave is the average of the highest 10% of all waves, and the $H_1$ design wave is the average of the highest 1% of all waves. The relationship of $H_{10}$ and $H_1$ to $H_s$ can be approximated as shown in Equations 13.1 and 13.2.

\[
H_{10} = 1.27H_s \\
H_1 = 1.67H_s
\]  

Economics and risk of catastrophic failure are the primary considerations in designating the design wave average height.

* Rev. 9/09
13.5  **Simplified Methods for Estimating Wave Conditions**

13.5.1  **Introduction**

Wave height estimates are based on wave characteristics that may be derived from an analysis of the following data:

- Wave gauge records
- Visual observations
- Published wave hindcasts
- Wave forecasts
- Maximum breaking wave at the site

It should be noted that deepwater ocean wave characteristics derived from offshore data analysis may also need to be transformed to the project site using refraction and diffraction techniques described in the Army Corps of Engineer's *Coastal Engineering Manual*.

13.5.2  **Predicting Wind Generated Waves**

13.5.2.1  **Wave Height**

The height of wind-generated waves is a function of:

- Fetch length
- Wind speed
- Wind duration
- Depth of water

13.5.2.2  **Hindcasting**

Wave hindcast information, based on historical weather records and observations, is available from the Army Corps of Engineer's Waterway Experiment Station (WES) in Vicksburg, Mississippi. Hindcasting methods should be used to determine the design wave height for coastal revetments.

13.5.2.3  **Forecasting**

Simplified wind wave prediction techniques may be used to establish probable wave conditions for the design of highway protection on bays, lakes, and other inland bodies of water. Wind data for use in determining design wind velocities and durations is usually available from weather stations, airports, and major dams and reservoirs.

* Rev. 9/09
The following assumptions pertain to these simplified methods.

- The fetch is short, 75 mi. or less
- The wind is uniform and constant over the fetch

It should be recognized that these conditions are rarely met and wind fields are not usually estimated accurately. The designer should therefore not assume that the results are more accurate than warranted by the accuracy of the input and the simplicity of the method. Good, unbiased estimates of all wind generated wave parameters should be sought and the cumulative results conservatively interpreted. The individual input parameter should not each be estimated conservatively, since this may bias the result.

The applicability of a wave forecasting method depends on the available wind data, water depth and overland topography. Water depth affects wave generation and for a given set of wind and fetch conditions, wave heights will be smaller and wave periods shorter if the wave generation takes place in a transitional or shallow water rather than in deep water. The height of wind-generated waves may also be fetch-limited or duration-limited. Selection of an appropriate design wave may require a maximization procedure considering depth of water, wind direction, wind duration, wind speed, and fetch length.

There is no single theory for the forecasting of wind-generated waves for relatively shallow water. Until further research results are available the interim method for predicting shallow-water waves presented in the Corp's "Coastal Engineering Manual" are to be used. It uses deepwater forecasting relationships and is based on successive approximations in which wave energy is added due to wind stress and subtracted due to bottom friction and percolation. An initial estimate of wind generated significant wave heights can be made by using Appendix 13B-1. If the estimated wave height from the nomograph is greater than 2.0' it is recommended that the Army Corps of Engineers procedures be used to refine the input parameters.

13.5.2.4 Breaking Waves

Waves generated in deeper water and shoaling as they approach the embankment have a maximum size wave that will reach the shore still in possession of most of its deep-water energy. Wave heights derived from hindcasts or any forecasting method should be checked against the maximum breaking wave that the design still-water level depth and near-shore bottom slope can support. The design height will be the smaller of either the maximum breaker height or the forecasted or hindcasted wave height. The relationship of the maximum height of breaker that will expend its energy upon the protection (H_b) and depth of water at the slope protection (d_s) which the wave must pass over are illustrated in Appendix 13B-2.

_____________________

* Rev. 9/09
13.5.2.5 Prediction Procedure

The following sections provide an outline of a wave prediction procedure.

13.5.2.5.1 Wind Speed Estimation

To estimate wind speed the following information is needed:

- Actual wind records from the site
- General wind statistics
- Best alternative source of wind information

13.5.2.5.2 Site Maximization Procedure

Using the method presented in the Army Corps of Engineer's "Coastal Engineering Manual" (CEM) the site maximization procedure consists of the following steps.

- Adjust wind information to 33’ above water surface
- Determine fetch limitations
- Adjust wind information for over water conditions
- Develop and plot a wind speed-duration curve
- When applicable, develop and plot a wind speed-duration curve for limited fetch
- Select design wind
- Forecast deepwater wave characteristics from deepwater significant wave prediction curves
- Determine if deepwater or shallow-water conditions are present
- For shallow-water conditions, forecast shallow-water significant wave height and period
- For deepwater conditions, refract and shoal the deepwater wave to the project site, if needed
- Compute wave run-up and wind set-up

13.5.2.5.3 Design Breaker Wave

The following example illustrates how to use Appendix 13B-2 to estimate the maximum breaker wave height.

Example

By using hindcast methods, the significant wave height ($H_s$) has been estimated at 3.9’ with a 3-second period. Find the design wave height ($H_b$) for the slope protection if the depth of water ($d_s$) is only 2.0’ and the near-shore slope is 1V:10H.

* Rev. 9/09
Solution

\[
\frac{d_s}{gT^2} = \frac{2.0}{32.2(3^2)} = 0.007
\]

From Appendix 13B-2, \( \frac{H_b}{d_s} = 1.4 \)

\( H_b = 2.8 \) ft

Since the maximum breaker wave height \( (H_b) \) is smaller than the significant deepwater wave height \( (H_s) \), the design wave height is 2.8’.

**13.5.2.3.4 Wave Run-up**

An estimate of wave run-up, in addition to design wave height, may also be necessary to establish the top elevation of highway slope protection. Wave run-up is a function of the design wave height, the wave period, bank angle, and the roughness of the embankment protection material. For wave heights of 2.0’ or less wave run-up can be estimated by using Appendix 13B-3. The wave run-up height given on the chart is for smooth concrete pavement. Correction factors for reducing the height of run-up are adequate for most highway projects. The application of more detailed procedures is rarely justified, but if needed they are provided in the U.S. Army Corps of Engineers Manual, "Design of Coastal Revetments, Seawalls, and Bulkheads."

If in doubt whether waves generated by fetch and wind velocity will be of sufficient size to be affected by shoaling, use both charts and adopt the smaller value.

**Table 13-1. Correction Factors For Wave Run-up**

<table>
<thead>
<tr>
<th>Slope Surface - Material Type</th>
<th>Correction Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete pavement</td>
<td>1.00</td>
</tr>
<tr>
<td>Concrete blocks (voids &lt; 20%)</td>
<td>0.90</td>
</tr>
<tr>
<td>Concrete blocks (20% &lt; voids &gt; 40%)</td>
<td>0.70</td>
</tr>
<tr>
<td>Concrete blocks (40% &lt; voids &gt; 60%)</td>
<td>0.50</td>
</tr>
<tr>
<td>Grass</td>
<td>0.35 - 0.90</td>
</tr>
<tr>
<td>Rock riprap (angular)</td>
<td>0.60</td>
</tr>
<tr>
<td>Rock riprap (round)</td>
<td>0.70</td>
</tr>
<tr>
<td>Rock riprap (hand placed or keyed)</td>
<td>0.80</td>
</tr>
<tr>
<td>Grouted rock</td>
<td>0.90</td>
</tr>
<tr>
<td>Wire enclosed rocks/gabions</td>
<td>0.80</td>
</tr>
</tbody>
</table>

* Rev. 9/09
13.5.3 Adjustments for Flooded Vegetated Land

When waves travel across a shallow flooded area, the initial heights and periods of the waves may increase; i.e., when the wind stress exceeds the frictional stress of the ground and vegetation underlying the shallow water. The initial wave heights may decay at other times when the frictional stress exceeds the wind stress.

For further discussions and example problems of estimating the growth and decay of wind waves over flooded, vegetated land, refer to the Corps of Engineers publication, *Coastal Engineering Manual.*
13.6 Flood Prediction Methods

13.6.1 Introduction

The prediction of the flood stage elevation for a specific exceedence probability event is of considerable importance to the designer. The methods of prediction that are applied to coastal and lake shorelines are quite different from those used on upland rivers and streams.

13.6.2 Coastal Flooding

The depth of coastal flooding for a specified event depends upon the velocity, direction and duration of the wind, the astronomical tide, and the size and depth of the body of water over which the storm acts. Such floods are considered to be comprised of two parts — still-water depth and wave height. The duration of flooding depends on the duration of the generating forces.

The still-water elevations are taken from sources such as National Oceanic and Atmospheric Administration (NOAA) technical memoranda or Corps of Engineers (COE) study reports. In these analyses, storm tides are computed from a full set of climatologically representative events using a numerical-dynamic storm surge model. Tidal flood records covering a significant period of time are used to determine the exceedence probability of selected flood magnitudes.

The methodology for analyzing the effects of wave heights associated with coastal storm surge flooding is described in the National Academy of Sciences (NAS) report "Methodology for Calculating Wave Action Effects Associated with Storm Surges," 1977. This method is based on three major concepts. First, depth-limited waves in shallow water reach a maximum breaking height that is equal to 0.78 times the still-water depth. The wave crest is 70% of the total wave height above the still-water level.

The second major concept is that wave height may be diminished by dissipation of energy due to the presence of obstructions such as sand dunes, dikes and seawalls, buildings and vegetation. The amount of energy dissipation is a function of the physical characteristics of the obstruction. The third major concept is that wave height can be regenerated in open fetch areas due to the transfer of wind energy to the water.
13.6.3 Tidal Flow Restrictions

Tidal flow, both at flood stage and under normal conditions, may be restricted in its entrance into lagoons and estuaries. Natural narrow and/or shallow passageways as well as man-made restrictions may be present. These restrictions will affect the timing cycle of high and low water, which, in turn, may affect the environmental quality of the lagoon or estuary and its adjacent wetlands.

The highway designer should be aware of these potential impacts, particularly when planning a new facility. The dynamic flow conditions caused by this type of restriction are difficult to analyze and this often leads to the use of generous waterway openings.

13.6.4 Lake Shore Flooding

The flood stage elevation on reservoirs and sometimes on natural lakes is usually the result of inflow from upland runoff. If water stored in the reservoir is used for power generation, irrigation, or low water augmentation, or if the reservoir is used for flood control, the level of the water at the time of a flood must be anticipated from a review of operating schedules. In the absence of such data, the designer should assume a conservative approach and use a high starting lake level. Wind generated waves will also be present in many flood instances.

A highway design should reflect consideration of flood levels, wave action, and reservoir operational characteristics. However, attempts to provide the highway facility with protection from the rare flood events normally used in the design of a reservoir rarely provide cost-effective designs.

Reservoir routing techniques are used to predict the still-water flood levels for most lakes and reservoirs. These levels should be increased appropriately to reflect the superimposition of waves.

Lakes have insignificant tidal variations, but are subject to seasonal and annual hydrologic changes in water level and to water level changes caused by wind setup, and barometric pressure variations. Additionally, some lakes are subject to occasional water level changes by regulatory control works.
13.7 Riprap Shore Protection

13.7.1 Introduction

Where wave action is dominant, design of rock slope protection should proceed as described below for shore protection. Where current velocity governs, rock size may be estimated by using the procedures in Chapter 7, Ditches and Channels.

Most of the protection measures provided in Chapter 7 can be considered when design waves are less than 2.0’. For design waves greater than 2.0’, rock riprap usually provides the most economical and effective protection. Design procedures suitable for waves between 2.0’ and 5.0’ are provided below. Alternate design procedures are contained in Corps of Engineers Coastal Engineering Manual that should be used for design waves greater than 5.0’. The following is a discussion of riprap shore protection measures.

13.7.2 General Features

Riprap protection when used for shore protection, in addition to general advantages listed in Chapter 7, reduces wave run-up as compared to smooth types of protection.

- Placement – Figure 13-7 illustrates typical placement of riprap for shore protection.

- Foundation treatment in shore protection - The foundation work may be controlled by tidal action as well as excavation quantities, and production may be limited to only two or three toe or foundation rocks per tide cycle. If these toe rocks are not properly bedded, the subsequent vertical adjustment may be detrimental to the protection above. Even though rock is self-adjusting, the bearing of one rock to another may be lost. It is often necessary to construct the toe or foundation in a triangular or trapezoidal shape to an elevation approximating high tide in advance of embankment construction to prevent erosion of the latter.

13.7.2.1 Shore Protection Design

Stone Size — For deep-water waves that are shoaling as they approach the protection the required stone size may be determined by using Appendix 13B-4. The nomograph is derived from Equation 13.3.

\[
W = \frac{0.003d^3 \rho - \alpha}{s_{gr} csc (\rho - \alpha)}
\]

(13.3)

* Rev. 9/09
Where:

\[
\begin{align*}
    d_s & = \text{Maximum depth of water at toe of the rock slope protection or bar, ft} \\
    sg_r & = \text{Specific gravity of stones} \\
    sg_w & = \text{Specific gravity of water (sea water = 1.0265)} \\
    \alpha & = \text{Angle of face slope from the horizontal, deg} \\
    \rho & = \text{Constant -- 70° for randomly placed rubble} \\
    W & = \text{Minimum weight of outside stones for no damage, tons}
\end{align*}
\]

In general, \(d_s\) will be the difference between the elevation at the scour line at the toe and the maximum still-water level. For ocean shore, \(d_s\) may be taken as the distance from the scour line to the mean sea level plus one-half the tidal range.

If the deep water waves reach the protection, the stone size may be determined by using Appendix 13B-5. The nomograph is derived from Equation 13.4.

\[
W = 0.00231 H_s^3 s g_r\csc^3(\rho - \alpha) \left(\frac{sg_r}{sg_w} - 1\right)^3
\]  

(13.4)

Where:

\[
H_s = \text{Significant wave height (average of the highest } \frac{1}{3} \text{), ft}
\]

Typical placement of shore protection riprap is illustrated in Figure 13-7. Rock should be founded in a toe trench dug to hard rock or keyed into soft rock. If bedrock is not within reach, the toe should be carried below the depth of the scour. If the scour depth is questionable, extra thickness of rock may be placed at the toe that will autonomously adjust and provide deeper support. In determining the elevation of the scoured beach line, the designer should observe conditions during the winter season, consult records, or ask persons who have knowledge of past conditions.
Wave run-up is reduced by the rough surface of rock slope protection. In order that the wash will not top the rock, it should be carried up to an elevation of twice the maximum depth of water plus the deep-wave height \((d_s + H_b)\), whichever is lower. Consideration should also be given to protecting the bank above the rock slope protection from splash and spray.

Thickness of the protection must be sufficient to accommodate the largest stones. Except for toes on questionable foundation, as explained above, additional thickness will not compensate for undersized stones. When properly constructed, the largest stones will be on the outside, and if the wave forces displace these, additional thickness will only add slightly to the time of complete failure. As the lower portion of the slope protection is subjected to the greater forces, it will usually be economical to specify larger stones in this portion and smaller stones in the upper portion. The important factor in this economy is that a thinner section may be used for the smaller stones. If the section is tapered from bottom to top, the larger stones can be selected from a single graded supply.

An alternate procedure for designing riprap protection from wave action due to wind or boat traffic is presented in the FHWA’s publication Design of Riprap Revetment (HEC-11). It is applicable in situations where wave heights are less than 5.0’ and there is no major overtopping of the embankment and is defined by Equation 13.5.

\[
W_{50} = \frac{1.67H^3}{\cot \theta}
\]  

(13.5)
Where:

\[ W_{50} = \text{Weight of the 50\% size stone, lbs} \]
\[ H = \text{Wave height, ft.} \]
\[ \theta = \text{Angle of the embankment with respect to horizontal, deg.} \]

Expressing the equation in terms of median grain diameter produces

\[ D_{50} = 0.57 \frac{H}{\cot \frac{1}{3} \theta} \]  \hspace{1cm} (13.6)

Where:

\[ D_{50} = \text{Mean spherical diameter of the 50\% size stone, ft.} \]

Equation 13.6 can be solved with the Hudson relationship nomograph in Appendix 13B-6.

* Rev. 9/09
13.8 References


State of California, Department of Transportation (Caltrans) Bank and Shore Protection in California Highway Practice. 1970.


State of California, Department of Transportation (Caltrans) Highway Design Manual, Chapter 870, 9/1/06

Federal Highway Administration, HEC-25 – Highways in the Coastal Environment, June 2008


* Rev. 1/17
Appendix 13B-1  Nomographs of Significant Wave Height Prediction Curves as Functions of Windspeed, Fetch Length, and Wind Duration

Source: HEC-11
Chapter 13 – Shore Protection

Appendix 13B-2  Design Breaker Wave

Source: Model Drainage Manual AASHTO 2005
Appendix 13B-3 Wave Run-up on Smooth Impermeable Slope

\[ R = \text{WAVE RUN UP HEIGHT (ft)} \]
\[ H'_O = \text{WAVE HEIGHT (ft)} \]
\[ \theta = \text{BANK ANGLE WITH THE HORIZONTAL} \]

Source: HEC-11
Chapter 13 – Shore Protection

Appendix 13B-4 Nomographs for Design of Rock Slope Shore Protection (For Shoal Water)

Source: California Highway Design Manual, Chapter 870
Appendix 13B-5
Nomographs for Design of Rock Slope Shore Protection
(For Deep Water)

Example:
\[ \text{sgr} = 2.75 \]
\[ \text{Find } W = 18 \text{T} \]

Source: California Highway Design Manual, chapter 870
Appendix 13B-6 Nomograph for Riprap Size to Resist Wave Action

\[ D_{50} = 0.57 \frac{H}{\cot^{33} \theta} \]

- \( D_{50} \) = Median Riprap Size
- \( H \) = Wave height
- \( \theta \) = Bank Angle with Horizontal

Example

Given: \( \cot \theta = 2:1 \) \( H = 3 \text{ ft.} \)

Find: \( D_{50} \)

Solution: \( D_{50} = 1.33 \text{ ft.} \)

Hudson relationship for riprap size required to resist wave erosion

Source: HEC-11
Chapter 14 - Subdivisions

TABLE OF CONTENTS

CHAPTER 14 - SUBDIVISIONS ........................................................................................................... 14-1

14.1 Introduction ........................................................................................................................... 14-1
  14.1.1 Objective ....................................................................................................................... 14-1

14.2 Policy ................................................................................................................................... 14-2
  14.2.1 Applicability .................................................................................................................. 14-2
  14.2.2 Agency Permits and Coordination .............................................................................. 14-2

14.3 Design Criteria ...................................................................................................................... 14-4
  14.3.1 Hydrology ..................................................................................................................... 14-4
  14.3.2 Hydraulic Design .......................................................................................................... 14-4
    14.3.2.1 Culvert Hydraulics ............................................................................................... 14-5
    14.3.2.2 Storm Drain Hydraulics ....................................................................................... 14-5
  14.3.3 Channels ....................................................................................................................... 14-5
  14.3.4 Structural Design of Culverts, Storm Drains, and Bridges ....................................... 14-6
  14.3.5 Dams ............................................................................................................................ 14-6
  14.3.6 Drainage Easements ...................................................................................................... 14-7

14.4 Design Procedures ................................................................................................................. 14-8
  14.4.1 Design Documentation .................................................................................................. 14-8

14.5 References ............................................................................................................................ 14-10

List of Appendices

Appendix 14B-1 Checklist
Appendix 14C-1 General Instructions and Criteria Pertaining to Use of Highway Embankment as Dams
Appendix 14D-1 Guidelines for the Design and Acceptance of Roadway Causeways
Chapter 14 - Subdivisions

14.1 Introduction

14.1.1 Objective

This chapter is devoted primarily to the design criteria and technical aspects of the design of drainage facilities for subdivision streets and roads that are designated to become a part of the State Secondary System of Highways.

It should be recognized that subdivision land drainage is the responsibility of the local government in whose jurisdiction the land lies. The policies, criteria, and design recommendations contained herein apply only to the streets and roads that are or will be maintained by VDOT. Once the streets and roads have been accepted into the System for maintenance they should be considered as another property within the watershed and the Department should be considered another property owner when assigning responsibility for drainage or drainage improvements within a watershed.

For more comprehensive information concerning administrative requirements for subdivisions, refer to the current editions of VDOT Subdivision Street Requirements and the Guide for Additions, Abandonments, and Discontinuance – Secondary System of State Highways. Both publications are produced by the VDOT Maintenance Division in Richmond and can be obtained on VDOT’s web site http://www.virginiadot.org.

For the purpose of administering the State Transportation’s Board’s policy concerning subdivisions, a subdivision is defined as “the division of lot, tract, or parcel into two or more lots, plats, sites, or other division of land for the purpose, whether immediate or future, of sale or of building development.”

Any re-subdivision of a tract or parcel of land is interpreted as a new subdivision under this definition and must satisfy all VDOT requirements for street additions to the Secondary System irrespective of the date of the original subdivision.

*Rev. 9/09

Chapter 14-1 of 10
14.2 Policy

14.2.1 Applicability

These requirements are applicable to all subdivision streets which are designated to become a part of the State Secondary System of highways. Department engineers are allowed to exercise discretionary judgment for the practical application, in peculiar individual situations, that will allow the optimum development of land without sacrificing the integrity of the policy.

The Department’s review and approval is applicable only to streets that are proposed to ultimately be added to the State Secondary System.

14.2.2 Agency Permits and Coordination

Plats and/or plans of all proposed subdivisions within a Residency’s geographical boundary, whose streets are intended to be added to the Secondary System, should be submitted to the appropriate Resident Engineer for his review. In counties which have administrative staffs who administer the county ordinance, these submissions should be made through the county staff instead of directly to the Department’s Resident Engineer. The plats and/or plans should include:

The complete drainage layout including all pipe sizes, types, drainage easements, and means of transporting the drainage to a natural watercourse (For a definition, in a legal sense, of a natural watercourse, see Chapter 4). Not only should we consider the present drainage of the immediate development, but the evaluations relative to future expansion or new adjacent development should be made as to their effect on the facilities proposed for the immediate development. Care must be taken to ensure that sufficient easements are provided to a natural watercourse or to furnish an acceptable agreement from county authorities to save the Department harmless from future claims.

• A typical cross section showing the proposed street construction, width, depth, type of base, type of surface, etc.

• A profile or contour map showing the proposed grades for the streets and drainage facilities

• A location map indicating the tie-in with the existing VDOT road system

• CBR tests for the Department’s review of pavement design

It is not intended that VDOT do the design work for the developer. Therefore, all computations utilized in determining the drainage facilities (including design calculations along with bridge plans that may be part of the subdivision) should be submitted for review. The Department’s engineers will check computations that are pertinent, but the original design work should be done by the developer’s representatives who are licensed by law to do such work.
Upon receipt of the plats and/or plans, the Resident Engineer is to study the layout thoroughly and determine if it is in compliance with all requirements of the Department, noting thereon any changes he feels should be made and:

- The drainage features may be referred to the district drainage engineer for review. Should there be a subdivision on which the district feels it should obtain further advice, the matter should be referred to the Hydraulics Section of the Location and Design Division

- Where a situation other than drainage appears to be complicated, and if the Resident Engineer has any doubt regarding it, he is to forward the prints and all data to the District Engineer for advice. Likewise, the District Engineer should consult further with the Maintenance Division and the Location and Design Division on any matter which he feels is necessary. After appropriate corrections or changes have been noted on the plats and/or plans by those making the review, they should be returned to the Resident Engineer for his further processing

- The Resident Engineer will return to the developer, or where applicable to the county official, the plats and/or plans approved subject to notations thereon, keeping one copy for his files. He should list the required changes in his letter of transmittal. In counties where the plats and/or plan are not signed by the Resident Engineer, the board of supervisors of the county should be notified that the subdivision prints have been reviewed, certain recommendations made, and, if the subdivision is developed according to plans, that the streets will be eligible for State maintenance funding

- Plan approval by the Resident Engineer signifies his recommendation for VDOT approval of that which was shown on the plats and/or plans at the time of submittal and includes revisions noted thereon by him. Any other revisions thereto, additions, or deletions require detailed written approval of each change

- Plans and computations submitted to the Department for review must specify the type of pipe used in the storm drain system and the storm drain system must be designed using the acceptable “n” value for that type. After plans are approved, no substitution or change in the type of pipe material will be allowed until the designer or contractor submits revised plans to the Department for review. The revised design cannot be implemented until approved by the Department.
14.3 Design Criteria

Where the local subdivision control ordinance requirements exceed VDOT requirements, the local ordinance should become the VDOT policy and govern when VDOT acts as an agent of the local governing body by the review and acceptance of subdivision streets. Drainage facilities, including off-site facilities when necessary to provide adequate drainage, must meet the minimum requirements for Maintenance, adequately pass the 10-year frequency runoff and comply with the following:

14.3.1 Hydrology

Peak discharge should be determined by methods appropriate for the size, location, and character of the watersheds involved. Where floodplain reports have been prepared for the area, they should be considered in the design. If these floodplains are affected by tides, tidal action reports should be included. Appropriate design storm frequencies should be utilized depending upon the risk of damage to both adjacent property and the roadway. Minimum design criteria applicable to the roadway may not be acceptable relative to the adjacent property damage potential, thus requiring higher design criteria.

Refer to Chapter 6 for more specific information relative to hydrology.

14.3.2 Hydraulic Design

No exact criteria for flood frequency or allowable headwater/ backwater values can be set which will apply generally to various locations. In the hydraulic design of drainage structures, the following risk evaluations should be considered.

- Damage to adjacent property
- Damage to the roadway and/or structure
- Traffic interruption
- Hazard to human life
- Damage to stream and floodplain environment
- Emergency access

Hydraulic design and analysis techniques should be appropriate for the type of structure or system of structures involved and may require flood profiles and water surface profile analyses. In areas involving floodplains, the Federal Flood Insurance requirements, relative to zoning and hydraulic design to accommodate the 100-year flood, should be fully considered.

The hydraulic design of drainage facilities for subdivisions should comply with or exceed the minimum requirements for maintenance as noted in other chapters in this manual and shall, in addition to the above, be designed to adequately pass the 10-year frequency runoff without interruption to traffic.
14.3.2.1 Culvert Hydraulics

The minimum design for culverts in a subdivision will accommodate the 10-year flood frequency runoff where the primary concern is the maintenance of traffic and convenience to the highway user.

For other culvert design considerations and a design procedure for the selection of highway culverts for use in subdivisions, refer to Chapter 8, Culverts.

For safety concerns in residential subdivisions, the minimum length of an entrance culvert should be the entrance width plus 8’. For example, a 16’ wide entrance would require a 24’ culvert. The length may be lessened when endwalls are used.

14.3.2.2 Storm Drain Hydraulics

Storm drains in subdivisions will be designed to accommodate the runoff from a 10-year frequency storm. Exceptions to this will be based on local conditions where potential damage to contiguous property is excessive or Federal or State regulations dictate the employment of a design storm of less frequency (greater intensity).

For other information concerning the design of storm drains and for design aids, see Chapter 9, Storm Drains.

14.3.3 Channels

Where open channels are used in lieu of closed storm drain systems, the minimum requirements should provide for a 10-year recurrence interval runoff without exceeding the banks of the channel. The dispersion of water from the termination of artificially constructed channels should be accomplished in such a manner as to avoid damage to adjacent properties. Where the combination of soil conditions and velocities will result in erosion, channel linings should be provided to prevent erosion. Where standard roadside ditches have insufficient capacity for the 10-year runoff, a storm drain system should be provided. Open channels may be considered if their construction can be accomplished without creating a hazard or condition detrimental to the appearance of the subdivision.

Additionally, the design of channels in subdivisions must adequately consider the protection of adjacent property, the roadway, the environment, and floodplains during floods of greater magnitude than the 10-year design storm, in accordance with Chapter 7, Ditches and Channels.

\*Rev. 7/14
14.3.4 Structural Design of Culverts, Storm Drains, and Bridges

Pipes for culverts and storm drains shall comply with the current VDOT Road and Bridge Specifications, and the current Road Designs Manual and the current Road and Bridge Standards, to the extent that they are respectively applicable to secondary roads and subdivision streets.

Bridges and box culverts shall be in accordance with the current bridge design specifications established by AASHTO. Calculations utilized in the design should be submitted with each bridge plan in order to expedite Department review.

14.3.5 Dams

Whenever dams are to be utilized as roadways, they shall be considered roadway dams and an alternate way of ingress and egress, which is open to the public, must be provided. Plans for dams which are designated for such use shall be reviewed and approved by the Hydraulics Section of the Department’s Location and Design Division prior to construction. A formal agreement must be executed between the developer and the Department regarding the relative responsibility of the maintenance of various elements of the dam prior to the Department’s acceptance of the roadway on the dam for maintenance. The agreement must absolve the Department of any responsibility for the maintenance of the dam and its control devices and for any damages claimed due to the existence or failure of the dam or its control devices. A sample agreement is found in “Guide for Additions, Abandonments, and Discontinuances – Secondary System of State Highways,” by the VDOT Maintenance Division.

Subdivision streets which cross a dam may be eligible for acceptance into the secondary system of state highways subject to the criteria listed in the Subdivision Street Requirements manual by the VDOT Maintenance Division. This manual defines dams as an embankment or structure intended or used to impound, retain, or store water, either as a permanent pond or as a temporary storage facility.

Dams shall comply with the applicable General Instructions and Criteria established in 4 VAC 50-20-10 and with the current applicable regulations of the State. Virginia Law, Dam Safety Act, Article 2, Chapter 6, Title 10.1, requires that dams be certified by the State Department of Conservation and Recreation (DCR), according to the information posted on their web site at http://www.dcr.virginia.gov.

*Rev. 9/09
A related situation is roadway embankments that cross impoundment areas upstream of the actual dam. The roadway embankment of these types of crossings typically functions as a causeway and exerts no influence over the function or control of the impoundment area. Increasingly, the Department is being requested to accept these causeway crossings into its maintained secondary system of roadways. In evaluating such request, the Department must consider future maintenance and liability issues regarding long term exposure of the embankment material to saturation and the inspection/repair/replacement of a drainage structure partially or fully inundated by a permanent water pool. In order to address these concerns, the guidelines included in Appendix 14 D-1 have been developed for use in the design of these “causeway” crossings and in evaluating their acceptability for inclusion into the VDOT maintained roadway system.

For additional information, see the DCR web site for Dam Safety Programs at http://www.dcr.virginia.gov.

14.3.6 Drainage Easements

Drainage easements should be provided from all drainage outfalls to extend to a natural watercourse, as defined in Chapter 8, or furnishes an acceptable agreement from county authorities to save the Department harmless from future claims.

In some counties, stormwater detention is required by County ordinances. This is recognized by VDOT as a viable stormwater management practice. However, stormwater detention, per se, is not an acceptable alternative to providing a drainage easement and outfall down to a natural watercourse, unless through agreement, the County assumes responsibility for maintenance of the detention facilities and the outfall and agrees to hold the Department harmless in case of damages claimed due to the existence or failure of the detention facilities or the outfall.

*Rev. 9/09
14.4 Design Procedures

14.4.1 Design Documentation

All design data and design considerations, including survey, hydraulic computations, floodplain studies, watershed and land use zones delineation, and other pertinent design data should be properly recorded.

The design documentation assembly should be submitted to the Department along with the subdivision plats and/or plans in order to facilitate the expeditious review of the plans and to minimize the turn-around time of the review process.

Some of the major items that should be addressed are as follows:

A. Perform a spot check of drainage calculations for:
   1. Proper/applicable design methods and procedures
   2. Completeness and accuracy
   3. Change in flow patterns and diversions

B. Review the drainage that would have a direct effect on the roadway.
   1. Check for adequate pavement drainage and proper placement of drainage structures
   2. Check the location and method by which pavement drainage is conveyed from the travelway. Ensure that drainage off of roadway does not flow into building sites/pads
   3. Review future driveway locations and driveway pipe sizes

C. Review drainage structures.
   1. Check existing structures (storm sewers, ditches, etc.) for adequacy to convey the runoff that will come to them in conformance with applicable criteria/requirements
   2. Check hydraulic design of proposed drainage facilities with applicable criteria/requirements

* Rev. 9/09
3. Check for proper treatment at ends of drainage facilities (riprap, paved ditches, etc.)

4. Check detention facilities for required hydraulic performance, proper outfall, and adequate roadway protection

D. Review erosion control*

1. Check for current and potential erosion and siltation problems

2. Check for impact of the development

3. Check for the adequate placement of erosion control devices

E. Check involvements with regulatory flood plains and/or the 100-year zone

F. Check to ensure that all necessary drainage easements have been designated

A sample subdivision review checklist that can be used in the plan review process is included as Appendix 14 B-1. The checklist is an indication of the pertinent data considered in the design and design review of subdivision plans.

* Rev. 9/09
14.5 References

Guide for Additions, Abandonments, and Discontinuances – Secondary system of State Highways, VDOT Maintenance Division

Virginia Law, Dam Safety Act, Article 2, Chapter 6, Title 10.1

DCR Dam Safety Req.

VDOT Subdivision Street Requirements (SSR)

VDOT Secondary Street Acceptance Requirements (SSAR)

VDOT Land Development Inspection Documentation Best Practices Manual

* Rev. 1/17
To be used by Citizens, Developers, Engineers, Surveyors, other Interested Parties, and VDOT

This checklist provides an itemized list of plans, documents, design calculations and other requirements for proposed subdivision roadway improvements to be submitted to VDOT for review and approval.
The following items should be shown or addressed in subdivision roadway plans and documents submitted to VDOT for approval. Check appropriate blank next to each item, sign last page, and submit checklist with plans. Right blank is for VDOT use only.

### A. GENERAL

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
<th>VDOT</th>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>1.</td>
<td>Three (3) copies of submittal letter attached outlining proposed development &amp; discussing any waivers or modifications from VDOT Standards either being requested or previously agreed upon.</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>2.</td>
<td>Three (3) copies of traffic study, erosion &amp; sediment control narrative and drainage calculations, pavement &amp; typical road section design calculations. Bound, pages numbered, no loose pages, table of contents. May combine in one report.</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>3.</td>
<td>Four (4) copies of plans (if rolled, please have print facing out)</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>4.</td>
<td>All plans are to be in accordance with VDOT Subdivision Street Requirements, Road &amp; Bridge Standards, Road &amp; Bridge Specifications, Minimum Standards of Entrances to State Highways, Road Design Manual, L&amp;D Instructional and Informational Memoranda, Drainage Manual, Hydraulic Design Advisories and other applicable VDOT and Federal polices.</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>5.</td>
<td>Plans should be self-explanatory with sufficient notes to explain the intent or purpose of the design.</td>
</tr>
<tr>
<td>☐</td>
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<td>☐</td>
<td>☐</td>
<td>6.</td>
<td>Title Sheet</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>7.</td>
<td>Subdivision name, phase, owner w/ address and phone number</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>8.</td>
<td>Designer with address, phone number, and professional stamp</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>9.</td>
<td>Tax Map number, Magisterial District, County, City or Town</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>10.</td>
<td>Master Plan (show which roads built, which roads in system)</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>11.</td>
<td>Plat, if available, showing rights-of-way, lots, &amp; easements.</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>12.</td>
<td>Type of development (i.e., industrial, commercial, single-family residential, etc.)</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>13.</td>
<td>Current and proposed zoning of property &amp; adjacent parcels</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>14.</td>
<td>Location map with scale</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>15.</td>
<td>General Notes including required VDOT general notes</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>16.</td>
<td>Date, revision dates</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>17.</td>
<td>Sheet Index with all sheets numbered and dated</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>18.</td>
<td>All lines and symbols clear &amp; labeled; all text legible</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>19.</td>
<td>Existing vs. proposed items easily distinguishable</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>20.</td>
<td>Plans must clearly indicate which roads are to be built for acceptance into VDOT Secondary System of Highways</td>
</tr>
</tbody>
</table>
## B. REVISIONS

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
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<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td></td>
<td></td>
<td>1.</td>
<td>Letter from designer must accompany revised plans submitted to VDOT for re-evaluation, describe changes made on revised plans, and provide dates of old &amp; revised plans. Letter should discuss any items that were not changed as requested and modifications that were made due to request of other agencies.</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>2.</td>
<td>Mark changed items with highlighter on 2 of the 5 sets of plans. Large revised areas need only be circled with a highlighter.</td>
</tr>
</tbody>
</table>

## C. TRAFFIC ANALYSIS

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
<th>VDOT</th>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.</td>
<td>Traffic Impact Analysis must be included with land development subdivision submitted. (completely replaces existing 1.)</td>
</tr>
<tr>
<td></td>
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<td>2.</td>
<td>Traffic Impact Analyses are to be prepared in accordance with VDOT Land Development Manual-Volume 1, dated December 1, 1995 Chapter 5 “Guidelines For a Traffic Impact Study” (or latest revision). Developer is responsible for roadway improvements to accommodate the acceptable level of service. (Developer responsible for supplying sufficient information to support designs shown.)</td>
</tr>
</tbody>
</table>
|     |    |     |      | 3.   | Detailed plans and studies may be required that address:  
- traffic analysis of existing and proposed conditions  
- intersection analysis including need for signalization / channelization / turn lanes & modification to existing signals  
- proposed roadway improvements to accommodate traffic generated by proposed development | |

## D. PLAN SHEETS

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
<th>VDOT</th>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.</td>
<td>North arrow, scale</td>
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<td>2.</td>
<td>Match lines clearly keyed to adjoining sheets w/ stationing</td>
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<td>3.</td>
<td>Limits of subdivision, limits of each phase</td>
</tr>
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<td>4.</td>
<td>Both edges of pavement (EP), shoulder width, and right-of-way (R/W) of connecting or adjacent streets along entire development plus 200' minimum each way. Show existing road spot elevations of both EPs and centerline @ 25' intervals near connection. Pavement design of existing streets.</td>
</tr>
</tbody>
</table>
**E. TYPICAL ROAD SECTIONS**

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
<th>VDOT</th>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.</td>
<td>Road and stations to which each applies</td>
</tr>
<tr>
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<td></td>
<td>2.</td>
<td>Proposed traffic count and design speed for each street(can be shown in a schedule)</td>
</tr>
<tr>
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<td>3.</td>
<td>Centerline and R/W width</td>
</tr>
<tr>
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<td></td>
<td>4.</td>
<td>Width &amp; slope of pavement, shoulder, ditch, etc.; type shoulder and cut and fill slopes.</td>
</tr>
<tr>
<td></td>
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<td>5.</td>
<td>Curb type, sidewalk, utility strip, etc., if applicable</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>7.</td>
<td>Show types, depths, and application rates of all pavement and aggregate layers and prime coats.</td>
</tr>
<tr>
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<td></td>
<td>8.</td>
<td>All aggregate layers are to extend 1' beyond EP or back of curb.</td>
</tr>
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<td>9.</td>
<td>The connections for intersections and commercial entrances (including Std. CG-11) shall be modified such that the street approach pavement is the same as the new roadway / entrance or mainline pavement, whichever has the highest structural value, or as determined by the District Materials Engineer.</td>
</tr>
</tbody>
</table>
### F. OTHER TYPICAL SECTIONS / DETAILS

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
<th>VDOT</th>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.</td>
<td>Where each applies</td>
</tr>
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<td>2.</td>
<td>Pavement widening or overlays of existing roads; crossovers</td>
</tr>
<tr>
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<td></td>
<td>3.</td>
<td>Special ditches: show shape, depth, slope, lining, min./max. grade. If paved, show details or reference VDOT Standard.</td>
</tr>
<tr>
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<td>4.</td>
<td>Entrances (internal, commercial, or private)</td>
</tr>
<tr>
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<td></td>
<td>5.</td>
<td>Special undercut or fill measures for unsuitable material, existing ponds, sinkholes, controlled fill, etc.</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>6.</td>
<td>Special drainage designs, structures, basins, berms, etc.</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>7.</td>
<td>Details of all items that are not a VDOT Standard or are a modification of a VDOT Standard.</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>8.</td>
<td>Cross sections of road, drainage, or other proposed construction may be required at areas of concern such as at connections to primary roads, when work is close to exterior property lines, at other constricted areas, etc.</td>
</tr>
</tbody>
</table>

### G. ROAD PROFILES

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
<th>VDOT</th>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.</td>
<td>Street name</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>2.</td>
<td>Horizontal and vertical scale and grid</td>
</tr>
<tr>
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<td></td>
<td>3.</td>
<td>Existing ground line (extended 100’ minimum beyond slope tie-in)</td>
</tr>
<tr>
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<td></td>
<td>4.</td>
<td>Proposed finished grade line</td>
</tr>
<tr>
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<td></td>
<td></td>
<td>5.</td>
<td>Percent grade, vertical curve data including K value (=L/A)</td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
<td>6.</td>
<td>Stations and elevations at begin / end, 50’ min. intervals high &amp; low points, PVC, PVI (CG), PVT, @ intersecting roads EPs and centerline (include super), &amp; at subdivision phase limits</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>7.</td>
<td>Provide adequate landing</td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
<td>8.</td>
<td>Intersection sight distances: eye = 3.5’, object = 3.5’</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>9.</td>
<td>Stopping sight distance (crest curves): eye = 3.5’, object =2.0’</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>10.</td>
<td>Culverts: size, type, invert, pipe number</td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
<td>11.</td>
<td>Storm sewer profiles and drainage structures (within R/W)</td>
</tr>
<tr>
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<td></td>
<td>12.</td>
<td>Ditch profile (where non-standard)</td>
</tr>
<tr>
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<td></td>
<td></td>
<td>13.</td>
<td>Water lines, sanitary sewer, and existing underground utilities</td>
</tr>
<tr>
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<td></td>
<td></td>
<td></td>
<td>14.</td>
<td>Special undercut or fill areas</td>
</tr>
</tbody>
</table>
H. OTHER Profiles

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
<th>VDOT</th>
<th>Item Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1. Special ditches, storm sewers, outfalls - extend ground line 100' minimum beyond tie-in</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>2. Entrances needing special attention, i.e. steep, constricted. (Tie proposed grade to edge of shoulder, not EP).</td>
</tr>
</tbody>
</table>

I. DRAINAGE (shown on plan & profile sheet, supplemental or detail sheet)

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
<th>VDOT</th>
<th>Item Description</th>
</tr>
</thead>
</table>
|     |    |     |      | 1. Contour plan of entire development and adjacent area:  
* every 5th contour highlighted & elevation clearly labeled  
* minimum contour interval usually two feet  
* shown on road plans or as separate sheet showing entire drainage system design  
* stationed centerline and R/W lines shown  
* drainage sub-areas outlined, labeled and areas shown  
* * showing topographic features, existing buildings, etc. |
|     |    |     |      | 2. Existing and relocated streams and drainage ways. |
|     |    |     |      | 3. Existing and proposed pipes, storm sewers, and drainage structures with location, size, type, lengths, inverts, design cover, and flow arrows. |
|     |    |     |      | 4. Proposed ditches (center of all shown graphically accurate by either flow arrows, finished contours, lining symbols or other methods). Show where linings begin and end. |
|     |    |     |      | 5. Std. CD-1 or CD-2 underdrains @ lower ends of cuts, vertical sags, and bridge approaches. |
|     |    |     |      | 6. Storm sewer system w/ VDOT standard structures. Show top, rim, height, grate, & invert elevations and throat lengths. |
|     |    |     |      | 7. Plan, profile and typical section of all ditches other than standard roadside ditches |
|     |    |     |      | 8. Proposed drainage easements to natural watercourses (usually 20' minimum width). |
|     |    |     |      | 9. Existing drainage facilities possibly affected by proposed development: location, size, inverts, etc. |
|     |    |     |      | 10. Erosion & sediment control measures |
|     |    |     |      | 11. Stormwater management plans and computations, where necessary. |
|     |    |     |      | 12. Stormwater management low impact development (LID) or other water quality techniques for the roadway are shown within the R/W detail sheets and computations. |
|     |    |     |      | 13. Have Maintenance agreements for LID or other water quality techniques, between the county, developer and VDOT been executed. |
J. **DRAINAGE COMPUTATIONS**

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
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<th>Description</th>
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<tbody>
<tr>
<td></td>
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<td></td>
<td></td>
<td>1.</td>
<td>Should be self-explanatory. (See Item A.2)</td>
</tr>
<tr>
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<td></td>
<td></td>
<td>2.</td>
<td>Subdivision name, date, author, professional stamp</td>
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<td>3.</td>
<td>In accordance with VDOT's current criteria including VDOT’s Drainage Manual. Discuss any methods or references used that are not generally used by VDOT.</td>
</tr>
<tr>
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<td></td>
<td>4.</td>
<td>Sufficient background, supporting information and summary of any computer printouts submitted</td>
</tr>
<tr>
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<td>5.</td>
<td>Copy of USGS topo map showing drainage patterns of area.</td>
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<td>6.</td>
<td>Discuss whether future sections are considered in design.</td>
</tr>
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<td></td>
<td>7.</td>
<td>Hydrology: drainage sub-areas to agree with contour plan, design discharge calculations, pre- &amp; post-development flows.</td>
</tr>
<tr>
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<td></td>
<td>9.</td>
<td>Outfall analysis (evaluation of receiving channel / structure)</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>10.</td>
<td>MS4 outfall data for the new street is provided to VDOT</td>
</tr>
</tbody>
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K. **UTILITIES**

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
<th>VDOT</th>
<th>Item</th>
<th>Description</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.</td>
<td>Show all existing underground and overhead utilities and easements, proposed water and sanitary mains, service laterals, types, sizes, and appurtenances.</td>
</tr>
<tr>
<td></td>
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<td>2.</td>
<td>Utilities should be located off R/W, where possible.</td>
</tr>
<tr>
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<td>3.</td>
<td>Utilities should be located out of pavement, where possible.</td>
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<td>4.</td>
<td>Adjustment of existing utilities, where needed.</td>
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<td></td>
<td>5.</td>
<td>Proposed utility crossings of existing roads: show location, alignment, size, type, encasements, lengths, crossing methods</td>
</tr>
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<td>6.</td>
<td>Route utilities under culverts where possible</td>
</tr>
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<td>7.</td>
<td>Set fire hydrants at R/W on lot lines, where possible.</td>
</tr>
<tr>
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<td>8.</td>
<td>Set manholes, valves, etc. in shoulder, utility strip or behind sidewalk, where possible.</td>
</tr>
<tr>
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<td>9.</td>
<td>Set streetlights at R/W line, outside clear zone.</td>
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<td>10.</td>
<td>Check for conflicts between utilities, road and drainage.</td>
</tr>
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</table>

L. **TEMPORARY CUL-DE-SACS / ROADS TO BE EXTENDED**

<table>
<thead>
<tr>
<th>Yes</th>
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<th>VDOT</th>
<th>Item</th>
<th>Description</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.</td>
<td>Traffic study must address ultimate projected traffic. If master plan of future area to be served is unavailable, give information on &amp; discuss acreage, access &amp; zoning of adjacent land. Discuss any County Comprehensive Plan available.</td>
</tr>
<tr>
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<td>2.</td>
<td>Indicate pavement design.</td>
</tr>
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<td>3.</td>
<td>Provide adequate temporary easement.</td>
</tr>
</tbody>
</table>
4. On profile, extend existing ground line and future grade line enough to show a satisfactory extension is possible.

### M. CURB & GUTTER STREETS

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Show street widths and radii to face of curb</td>
</tr>
<tr>
<td>2.</td>
<td>Show entrance type</td>
</tr>
<tr>
<td>3.</td>
<td>Show Std. CG-12's @ intersections &amp; other req'd. locations</td>
</tr>
<tr>
<td>4.</td>
<td>Tie standard CD-1 underdrains into drop inlets</td>
</tr>
<tr>
<td>5.</td>
<td>Intersection and cul-de-sac details are usually needed to show: type of intersection (i.e., Std. CG-11), how drainage is handled, top of curb and EP elevations around radii, etc.</td>
</tr>
<tr>
<td>6.</td>
<td>Provide necessary drainage computations</td>
</tr>
</tbody>
</table>

### N. MISCELLANEOUS

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>If subdivision identification sign is desired, provide for in easement off R/W</td>
</tr>
<tr>
<td>2.</td>
<td>If any special use of R/W is desired such as bike paths, landscaping, irrigation system, lighting, parking, retaining walls, etc., provide full details and technical specs. These may need to be shown on separate plan sheets.</td>
</tr>
<tr>
<td>3.</td>
<td>Details of any special entrance road design (i.e., one-way, islands, medians, etc. Details of cluster mailbox pull-offs.</td>
</tr>
</tbody>
</table>

### O. GEOTECHNICAL – General Information

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Soil Technician / Engineer qualification statement.</td>
</tr>
<tr>
<td>2.</td>
<td>Statement that investigation was completed under the direction of VDOT personnel.</td>
</tr>
<tr>
<td>3.</td>
<td>Contact information for developer, designer, soil testing laboratory, and soil technician.</td>
</tr>
<tr>
<td>4.</td>
<td>Site Map showing project location.</td>
</tr>
</tbody>
</table>
### P. PAVEMENT DESIGN

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Contact information for developer, designer, soil testing laboratory, and soil technician.</td>
</tr>
<tr>
<td>2.</td>
<td>Statement that samples were obtained according to the frequencies provided in the VDOT Pavement Design Guide for Subdivisions and Secondary Roads, Page 4, Section A.2.a.</td>
</tr>
<tr>
<td>3.</td>
<td>Sample Location Map showing borehole, test pit, and/or surface sample collection sites in reference to proposed alignment.</td>
</tr>
<tr>
<td>5.</td>
<td>Atterberg Limits Report in accordance with VTM-7 (for soils with more than 35% passing No. 200 sieve).</td>
</tr>
<tr>
<td>7.</td>
<td>CBR Report in accordance with VTM-8.</td>
</tr>
<tr>
<td>8.</td>
<td>Reports should include sample location, depth and natural water content.</td>
</tr>
<tr>
<td>9.</td>
<td>Documentation that the projected average daily traffic (ADT) volume to be used for design purposes follows the VDOT Road Design Manual, Appendix B, including %HCV and adjusted by Pavement Design Guide for Subdivision and Secondary Roads, Appendix IV.</td>
</tr>
<tr>
<td>11.</td>
<td>Please note that there are design, subgrade, and drainage considerations in addition to the procedure described in Appendix IV. Also, where locality requirements exceed the pavement design determined by Appendix IV, that locality’s design method governs. No checklist or worksheet will relieve the designer’s responsibility for the proper use and application of the design methods provided, or adherence to VDOT standards and specifications.</td>
</tr>
</tbody>
</table>

### Q. PIPE/BOX CULVERT FOUNDATION DESIGN REQUIREMENTS

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Contact information for developer, designer, testing laboratory, and soil technician.</td>
</tr>
<tr>
<td>2.</td>
<td>For box culverts or pipes with diameter 36” or greater, a minimum of one boring shall be advanced at each endwall and at 200-foot intervals along the alignment of pipe or culvert. Borings should extend at least one pipe diameter below the invert elevation, fully penetrating unsuitable material or fill and extending at least 5 feet into underlying natural soils.</td>
</tr>
<tr>
<td>3.</td>
<td>Sample Location Map showing borehole, test pit, and/or surface sample collection sites in reference to proposed box culvert location.</td>
</tr>
<tr>
<td>4.</td>
<td>Logs indicating sample location (station &amp; offset), SPT data, Unified Soil Classification System (USCS) description of subsurface materials, as well as natural water content.</td>
</tr>
</tbody>
</table>
5. Test reports should include soil pH and soil resistivity results.
6. Provide box culvert foundation design in accordance with VDOT Road & Bridge Standards and Specifications.

Additional review may be required. Please contact the District Structure & Bridge, Environmental and Hydraulics Offices.

### R. BRIDGE FOUNDATION DESIGN REQUIREMENTS

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
<th>VDOT</th>
<th>Item</th>
<th>Description</th>
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<td></td>
<td>1.</td>
<td>Contact information for developer, designer, testing laboratory, and soil technician.</td>
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<tr>
<td></td>
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<td></td>
<td>2.</td>
<td>For bridges less than 100 feet wide, a minimum of two borings shall be advanced within the proposed footprint of each, abutment and pier. For bridges over 100 feet wide, advance three borings per each abutment and pier.</td>
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<tr>
<td></td>
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<td>3.</td>
<td>For shallow foundations, borings should be advanced to a depth at least twice the estimated width of the pier footing, or 4 times the width of the strip footing (L/B&gt;10). Borings shall fully penetrate unsuitable material or fill, and extend at least 10 feet into material with suitable bearing capacity. If rock is encountered, it shall be cored to a depth of at least 5 feet.</td>
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<tr>
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<td>4.</td>
<td>For deep foundations, borings should extend at least 15 feet below the anticipated pile or shaft tip elevation or a minimum of 2 times the maximum pile group dimension, whichever is greater. For piles bearing on rock, at least 10 feet of core shall be taken from each boring. For drilled shafts bearing on rock, at least 10 feet or 3 times the shaft diameter of rock core shall be taken from each boring.</td>
</tr>
<tr>
<td></td>
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<td>5.</td>
<td>Sample Location Map showing borehole locations in reference to footprints of proposed locations for bridge substructure units.</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>6.</td>
<td>Borehole logs indicating location (station &amp; offset, northing &amp; easting, and latitude &amp; longitude), SPT data, RQD for cored rock, USCS description of subsurface materials, initial and static groundwater elevations (if encountered), color digital photographs of individual rock cores, and any associated in-situ and lab test reports</td>
</tr>
<tr>
<td></td>
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<td>7.</td>
<td>Provide bridge foundation design for each bridge substructure unit. Include the estimated allowable bearing capacity of the materials encountered at the proposed foundation elevation.</td>
</tr>
</tbody>
</table>

### S. RETAINING WALL / SOUND WALL FOUNDATION DESIGN REQUIREMENTS

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
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<th>Item</th>
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<td></td>
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<td></td>
<td>1.</td>
<td>Contact information for developer, designer, testing laboratory, and soil technician.</td>
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</tbody>
</table>
|     |    |     |      | 2.   | Borings shall be advanced for each 100-200 feet along the proposed alignment over the full length of the wall, with a minimum of two borings. Borings shall be advanced to a depth of
twice the proposed wall height, should fully penetrate unsuitable material or fill, and extend 10 feet into competent material or 5 feet into rock.

3. Sample Location Map showing borehole locations in reference to wall alignment.

4. Borehole logs indicating location (station & offset), SPT data, RQD for cored rock, USCS description of subsurface materials, natural water content, and any associated in-situ and lab test reports.

5. Provide retaining wall foundation design in accordance with VDOT Standards and Specifications. Include the estimated allowable bearing capacity of the materials encountered at the proposed foundation elevation.

Additional review may be required. Please contact the District Structure & Bridge Office.

T. STORM WATER MANAGEMENT BASIN DESIGN REQUIREMENTS

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
<th>VDOT</th>
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<td>☐</td>
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<td>☐</td>
<td>☐</td>
<td>1.</td>
<td>Contact information for developer, designer, testing laboratory, and soil technician.</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>2.</td>
<td>A minimum of two borings shall be advanced for basins less than 2 acres in extent (one additional boring for each additional acre), one in the impoundment area and another in the dam. Borings shall be advanced 5 feet below the proposed bottom elevation of the impoundment area and to a depth twice the embankment height at the dam, should fully penetrate unsuitable material or fill, and extend 10 feet into competent material or 5 feet into rock. A groundwater observation well should be installed to monitor long-term groundwater levels.</td>
</tr>
<tr>
<td>☐</td>
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<td>☐</td>
<td>3.</td>
<td>Sample Location Map showing borehole locations in reference to basin layout and dam location</td>
</tr>
<tr>
<td>☐</td>
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<td>☐</td>
<td>☐</td>
<td>4.</td>
<td>Borehole logs indicating location (station &amp; offset), SPT data, RQD for cored rock, USCS description of subsurface materials, natural water content, and any associated in-situ and lab test reports.</td>
</tr>
<tr>
<td>☐</td>
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<td>☐</td>
<td>5.</td>
<td>Test reports to include gradation, Atterberg, USCS description and natural water content. A minimum of one sample from the impoundment subgrade should be tested for permeability</td>
</tr>
<tr>
<td>☐</td>
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<td>☐</td>
<td>6.</td>
<td>Provide stormwater management basin design in accordance with VDOT Standards and Specifications.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Additional review may be required. Please contact the District Environmental and Hydraulics Offices.</td>
</tr>
</tbody>
</table>
### U. SOIL SLOPE DESIGN REQUIREMENTS

<table>
<thead>
<tr>
<th>Yes</th>
<th>No</th>
<th>N/A</th>
<th>VDOT</th>
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<tbody>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>1.</td>
<td>Contact information for developer, designer, testing laboratory, and soil technician.</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>2.</td>
<td>In fill embankments, advance one boring every 200 feet along the toe of the proposed slope. Borings should be advanced to a depth twice the height of embankment for embankments over 15 feet in height; to a depth equal to the height of embankment for smaller embankments, but at least 5 feet below subgrade elevation.</td>
</tr>
<tr>
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<td>3.</td>
<td>In cut slopes, advance one boring every 200 feet along the top of the proposed slope. Borings should be advanced to a depth at least 10 feet below the proposed minimum elevation of cut for slopes greater than 15 feet in height; at least 5 feet below subgrade elevation for smaller slopes.</td>
</tr>
<tr>
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<td>☐</td>
<td>4.</td>
<td>Borings should fully penetrate unsuitable material or fill and extend at least 15 feet (for large slopes) or 5 feet (small slopes) into underlying suitable soils. At least one groundwater observation well should be installed to monitor long-term groundwater levels. If rock is encountered above design grade, it should be cored to the full depth of the planned cut.</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>5.</td>
<td>Sample Location Map showing borehole locations in reference to slope alignment</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>6.</td>
<td>Borehole logs indicating location (station &amp; offset), SPT data, RQD for cored rock, USCS description of subsurface materials, natural water content, and any associated in-situ and lab test reports.</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>7.</td>
<td>Test reports to include gradation, Atterberg Limits and USCS descriptions. May require advanced geotechnical tests to include direct and/or triaxial shear and consolidation testing.</td>
</tr>
<tr>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
<td>8.</td>
<td>Provided soil slope design in accordance with VDOT Standards and Specifications.</td>
</tr>
</tbody>
</table>

### CERTIFICATION

I hereby certify to the best of my knowledge that the above stated information is included in the submitted plans and attachments.

Designer’s Signature: ____________________________ Date: ____________

Designer’s Name (printed): ________________________________
Design Firm: ________________________________
Roadway Dams

In accordance with the provisions contained herein, VDOT may approve the use of highway embankments as dams.

Highway embankments as referred to herein shall include all of those roads and streets within the jurisdiction of VDOT.

The term “dam” as used herein shall mean a barrier to confine or raise water for storage, a diversion, or to create a hydraulic head.

In general when a permit is requested for use of an embankment as a dam, whether it be an existing or proposed embankment within the highway system or one ultimately to become a part of the system, it must be accompanied by plans and supporting data as outlined in the following paragraphs.

1. Purpose of Impoundment

2. Location

   A map of the vicinity with notations sufficient to accurately locate the project site will be required.

3. Plans

   The plans shall in general contain the following:

   a. Plan of reservoir area and dam site showing contours

   b. Sectional view of dam taken through control structure

   c. Details of control structures showing dimensions, types of materials, cutoff or antiseep collars, anti vortex devices, energy dissipators, and other pertinent details applicable to the particular project

   d. Where channel outlets are used for spillways, sufficient profile and cross sections shall be shown to permit checking the hydraulic characteristics

   e. Where the existing embankments are to be used, details will be given as to existing drainage structures and the materials and compaction used in the construction of the dam
4. Analyses – Computations
   
a. Hydrologic data used and its source

b. Hydrographs

c. Hydraulic computations for control structures, outlet channels and other applicable devices

5. Administrative Procedures

The plans shall be prepared by a licensed engineer or by a governmental agency whose engineers have previously prepared similar plans. The National Resources Conservation Service (NRCS) will generally assist in plan preparations when the impoundment is for conservation purposes.

Prints of plans and copies of supporting computation data shall be submitted in duplicate, one set to be reviewed by the Department and remain in the files of the Central Office, and the other to be returned with any pertinent notations. Prior to approval, for construction, revised prints of plans will be submitted in triplicate, one for each for the Central, District, and Residency offices.

All requests will be initiated through the Resident Engineer and be forwarded through proper channels to the Central Office. Where applicable, the petitioner will be required to furnish a performance bond or certified check to cover cost of work and any balance not expended by the Highway Commission will be returned to the petitioner.

All costs shall be borne by the petitioner and no permit will be granted for work which will result in additional expenditures by the Department. Where protective devices such as guardrails do not exist or would normally not be provided by the Department, such protective devices will be provided at the expense of the petitioner.

Under no circumstances shall the Department be committed to reconstruction, relocation, adjustment or protection of the highway at the expense of Highway funds without approval of the Commissioner.

Construction inspection under the supervision of VDOT may be required or certification by petitioner, obtained from a licensed professional.
6. Design Specifications & Criteria

a. **Watershed Area**: The area contributing to a reservoir shall be accurately determined. Delineation on dependable topographic maps or aerial photographs, when available, may be used for this purpose.

b. **Reservoir Area**: The area of the impoundment must be determined with sufficient accuracy at various elevations to permit the development of a storage curve. Where maps having a close contour interval (one or two foot) are available they may be used in lieu of field survey or reconnaissance.

c. **Dam**: (Roadway embankments) : The embankment will, in addition to being constructed to the Department’s specifications, have either a core or upstream blanket. If upstream blanket construction is used, the material will consist of a layer of highly impervious material placed on the reservoir floor and extended up the upstream slope of the embankment. In general a core will be required where the depth of impoundment is 15 feet or greater.

d. **Hydraulic Structure**: All structures conducting the effluent through highway fills shall be adequate to pass the design flood originating in the watershed. Generally, structures shall be so designed and constructed that the maximum high water stage from the design storm shall not be higher than eighteen inches below the outer edge of the shoulder of the highway at its lowest point adjacent to the reservoir.

The design storm for impoundments, wherein the only consideration is the highway, will generally be for a return period of 25-year or 50-year.

Where the failure of the dam would result in property damage or hazard to life, the criteria found on DCR’s web site at http://www.dcr.virginia.gov under Dam Safety should be followed.
There are many factors to be considered which may necessitate special consideration and, therefore, anyone contemplating the construction of a road as a dam wherein the Department would have an interest is advised to consult with the Hydraulic Section prior to development of the plans.

No moveable gates or valves will be permitted to serve as outlet control structures; however, gates will be provided to permit draining for management purposes. In general, no portion of the roadway will be permitted to serve as a spillway.

e. Landscaping: The shoreline shall be cleared of all weeds and stumps and maintained in a neat manner.

7. Legal Provisions

Where deemed necessary or desirable, by the Department, legal responsibilities and obligations shall be set forth as a condition in the permit or shall be provided for by a separate instrument.
Chapter 14– Subdivisions

Appendix 14D-1 Guidelines for the Design and Acceptance of Roadway Causeways

1) Definitions

For the purposes of this document, the following definitions apply:

a) The term “roadway dam” means an embankment designed to impound water, either temporarily or permanently, that also serves as a roadbed for motor vehicles.

b) The term “roadway causeway” means an earthen embankment intended to serve as a roadbed for motor vehicles across an area designated as a storm water impoundment area.

c) The term “stormwater impoundment area” means an area designed to be inundated by stormwater, either temporarily or permanently.

d) The term “permanent impoundment area” means the area within a stormwater impoundment area designed to be normally and permanently inundated by a pool of water.

e) The term “design impoundment area” means the total area designed to be temporarily inundated by storm water run-off resulting from a 10 year frequency design storm, inclusive of any permanent impoundment area.

f) The term “design flood area” means the area extending beyond the design impoundment area which will be inundated by storm water run-off resulting from a 100 year frequency design storm.

b) Design Criteria

a) Roadway Dams – Design criteria for roadway dams is found in Chapter 14 of the VDOT Drainage Manual. The criteria for accepting roads that cross dams as part of the secondary system of state highways is found in VDOT’s Subdivision Street Requirements.

b) Roadway Causeways Impacted By A Permanent Impoundment Area (See Figures 1 & 1A)

Because of the potential operational and maintenance issues associated with embankments and drainage structures permanently inundated by water, roadway causeways impacted by a permanent impoundment area shall be designed as roadway dams. The criteria for VDOT’s acceptance of a road on such a causeway as part of the VDOT maintained secondary system of state highways shall be the same as that for roadways crossing dams.
Figure 1
Permanent Impoundment Area abutting one side of a roadway causeway. Causeway is treated as a dam.

Figure 1A
Permanent Impoundment Area abutting both sides of a roadway causeway. Causeway is treated as a dam.
c) Roadway Causeways Crossing A Design Impoundment Area But Outside The Limits Of A Permanent Impoundment Area (if present) – See Figure 2

Roadway causeways crossing a design impoundment area but outside the limits of any permanent impoundment area shall not be treated as a roadway dam, provided the hydraulic capacity of the drainage facility under the roadway causeway equals or exceeds the hydraulic capacity of the principal spillway of the downstream dam. However, the embankment of such causeways shall, in addition to being constructed to the Department’s specifications, have all slopes within the design impoundment area protected by a blanket of highly impervious material (a layer of clay material with a one foot minimum thickness or a geosynthetic clay liner, as approved by the Department) extending from the floor of the impoundment area to an elevation not less than 2 feet above the surface elevation of the design impoundment area or to the edge of the roadway shoulder, whichever is less. The material used for the clay blanket must meet all of the following minimum specifications:

- 50% or more must pass the No. 200 sieve and,
- the Liquid Limit must be less than 50 and,
- the Plasticity Index must be greater than 7.

Example 2

Roadway causeway within a Design Impoundment Area but outside the limits of a Permanent Impoundment Area. Causeway is not treated as a dam provided the hydraulic capacity of the drainage facility under the causeway equals or exceeds the hydraulic capacity of the principal spillway of the downstream dam.
d) Roadway Causeways Crossing A Design Flood Area (See Figure 3)

Roadway causeways crossing a design flood area beyond the limits of the design impoundment area shall not be subject to the requirements of this document and shall only be subject to the Department’s standard specifications and criterion for roadway embankments and drainage structures.

Example 3
Roadway causeway within the Design Flood Area but outside the limits of the Design Impoundment Area. Causeway is not treated as a dam.
Chapter 15 – Drainage Design
Memoranda

TABLE OF CONTENTS

DDM1 Type of Structure Selection is now in VDM 8.3.1.1, Culvert End Treatment is in 8.3.3.4

DDM2 Basic Drainage Description Formats for Hydraulic Plan Items now are in the VDM 3.3.4

DDM3 Minor Structure Excavation is now in the VDM at 8.4.4.4 and 9.4.8.9

DDM4 Drainage Design at Railroads is now in the VDM at 8.3.8

DDM5 Underdrain is now in the VDM at 9.4.3.9

DDM6 Board Policies on Participation by Towns, Cities and Counties is now in IIM 146

DDM7 VDOT Procedures for Documentation and Notification of Activities in FEMA-Mapped Floodplains has been replaced and the requirements are now in the VDM Chapters 4, 8, 12, and 17

DDM8 VDOT Expanded Procedures for Estimating Bridge Scour Using a Variable D50 with Depth
EFFECTIVE DATE

- Unless identified otherwise within this DDM, the information contained in this DDM is effective upon receipt.

PURPOSE

- This DDM establishes minimum requirements for projects including roadway construction and maintenance that are located within Special Flood Hazard Areas (SFHA) Zone A, AE and VE as mapped on a Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM). These requirements require the engineer to fully document, to the extent necessary, compliance with FEMA practices and principles; VDOT standards, guidance and specifications; state and federal regulatory requirements; and all appropriate and necessary construction considerations. This DDM further references a Quality Assurance/Quality Control (QA/QC) process by which all work in the FEMA floodplain will be reviewed for consistency with this DDM and any applicable regulatory requirements.
REFERENCES

- The following editions apply when referenced in this DDM:
  - Virginia Code Section 10.1-602
  - Governors Executive Memorandum 2-97

POLICY

- This section amends and revises the existing VDOT procedures with respect to work in FEMA Designated Floodplains as discussed in Chapter 12 of the VDOT Drainage Manual.

PROCEDURE

- VDOT Hydraulics Staff will review all projects that will be submitted for environmental permits to assess the potential impact on flooding. At VDOT’s discretion, the assessment on flooding impact may be qualitative (Engineering Judgment) or quantitative (Engineering Analysis). Documentation regarding this assessment of the potential impacts will be included with the Joint Permit Application (JPA) developed by VDOT to manage the environmental permit process. The Department of Conservation & Recreation (DCR) is included in the distribution of the JPA.

- Any maintenance or construction activity that will restore or maintain the originally-designed hydraulic capacity of a VDOT asset that is present in the FEMA Mapped Floodplain, will not require additional coordination with DCR or the localities beyond what is included in the JPA. This would include, but is not limited to:
  - stream bank stabilization;
  - reconstruction and stabilization of roadway embankments;
  - bridge scour repairs and associated fill;
  - bridge deck replacements;
  - substructure repairs that may involve additional concrete or steel to reinforce the structure;
  - bridge and structure maintenance and repair; and
  - bridge and structure replacement with a Hydraulically Equivalent Replacement Structure (HERS)
When an Engineering Analysis of flooding impact will be performed on a project within a Zone AE, VDOT will request supporting hydraulic data from FEMA to use as a starting point for the analysis. VDOT will independently evaluate the hydrology for the purposes of comparison. The engineer will update the hydraulic model, as needed, to reflect the results of the VDOT survey and bridge design or construction. In the event that there is a discrepancy between the FEMA data and the Engineering Analysis, the VDOT data will take precedence. Any discrepancies will be noted and documented for future use but will not require a CLOMR submission. The hydraulic model and supporting documentation shall be forwarded to the Locality implementing the National Flood Insurance Program (NFIP) in that jurisdiction and DCR’s Division of Dam Safety & Floodplain Management, upon their request.

VDOT Design Policy is written such that it will support the goals of DCR to preserve the capacity of the floodplain to carry the 100-year flood and exceed the minimum requirements of the NFIP:

- VDOT will limit impacts to the 100-year flood elevation to a cumulative 1.0’ for work that is performed within FEMA Zone A areas, provided there is no adverse impact to offsite structures. Any impacts may require coordination with the community.

- VDOT will limit impacts to the 100-year flood elevation to 0.0’ for work that is performed within FEMA Zone AE areas.

If the impact limits noted above cannot be met, VDOT will coordinate with the NFIP community regarding the impact and determine the subsequent course of action. This may include supplemental surveys, a Conditional Letter of Map Revision (CLOMR), and as needed, a Letter of Map Revision (LOMR).

REPORTING AND DOCUMENTATION

For projects determined by using engineering judgement to have no adverse hydraulic impact as discussed above will require no additional coordination with the Locality or DCR except for the abbreviated Hydraulic Commentary provided in the JPA.

For projects located in a FEMA Designated A, AE, V or VE Zone, where a detailed hydraulic analysis has been performed, VDOT will:

- Modify the current documentation practices to include a comparison of the published and VDOT-determined discharges, including the published, revised existing and proposed condition flood elevations.
- Upon review and approval by the Hydraulics Staff the Preliminary Bridge Report based on the preliminary bridge design will be submitted to the Project Manager and the Central Office Hydraulics Section. This will include the LD293, FEMA FIRMette, the preliminary Bridge Front Sheet and preliminary Roadway Plan and Profile.

- The Central Office Hydraulics Staff will notify the Locality and DCR upon completion of the preliminary H&HA and provide that documentation for their use. In the event that the impact limits noted above are not met, there will be additional coordination with the locality regarding the need to engage in the CLOMR and LOMR process.

- If there is a difference between the VDOT and FEMA data and the existing versus proposed analyses shows that it meets the 100-year impact limits set above, this will not require a CLOMR / LOMR to update the FIRM and FIS. Upon receipt of the VDOT information, DCR and the Locality may coordinate with FEMA, as needed, regarding areas where the published FIS appears to be erroneous.

- In the event that there are changes in the final design that warrant modification of the analysis, the preliminary Bridge Report will be reissued to the PM and the Central Office Hydraulics Section. The Central Office will distribute copies to DCR and the Locality.

- Upon approval of the final bridge design and scour analyses by the Hydraulics Staff a Final Bridge Report will be submitted to the PM and the Central Office Hydraulics Section and will include the LD293, FEMA FIRMette, the Bridge Front Sheet, Roadway Plan and Profile, the hydraulic model, scour computations and scour plot.

- The Central Office Hydraulics Staff will distribute the final Bridge Report to the Locality and DCR. Survey data and engineering computations will be provided upon request. If required by this document, a CLOMR will be submitted at this time.

- These provisions are limited to projects where VDOT is the designated permittee. For VDOT-funded projects, where the environmental permits are held by others (i.e., Locally Administered Projects, Design Build Contractors, PPTAs, etc.), they will be responsible for independent coordination with the community with regards to Floodplain Development.

- VDOT will establish a centralized repository for the hydraulic analyses to facilitate requests made by DCR or FEMA for specific studies.

- In the event that CLOMR is submitted it will be necessary that the project schedule and budget be modified to accommodate the submission of a LOMR at project completion. This will include an as-built survey and possible modification to the hydraulic modeling as needed.
**GENERAL SUBJECT:** DRAINAGE INSTRUCTIONS  
**NUMBER:** DDM-8  

<table>
<thead>
<tr>
<th>SPECIFIC SUBJECT:</th>
<th>DATE:</th>
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<tbody>
<tr>
<td>VDOTS EXPANDED PROCEDURE FOR ESTIMATING BRIDGE SCOUR USING A VARIABLE D50 WITH DEPTH AND ESTIMATED D50 BASED ON BEDROCK CORE MATERIAL</td>
<td></td>
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<tr>
<td>SUPERSEDES:</td>
<td>N/A</td>
</tr>
</tbody>
</table>

**APPROVAL:** Jeffrey S. Bragdon, P.E.  
State Hydraulics and Utilities Engineer

---

**EFFECTIVE DATE**

- Unless identified otherwise within this DDM, the information contained in this DDM is effective upon receipt.

---

**PURPOSE**

- This DDM applies to a hydraulic evaluation of a steady state flow condition that will provide the maximum theoretical scour. At this time, VDOT does not provide general guidance for the use of the scour computations that take into account long term variable flow as it applies to cohesion or abrasion. These approaches may be considered on a case-by-case basis at the discretion of VDOT staff.
- The methods provided in HEC-18, 5th edition, were largely developed in isolation to one another and there are significant gaps between the theory as presented, and the hydraulic and geotechnical conditions found in the field.
- The processes and procedures within HEC-18, 5th edition, do not describe how to incorporate the computational process when the geotechnical report returns results that provide an increasing D50 with depth.
- The processes and procedures within HEC-18, 5th edition, do not include discussion or practices to evaluate the scour potential in material that can be cored, but is fractured and does not meet an RQD of 50%.
• In general, the procedures in HEC-18, 5th edition, treat live-bed scour and clear-
water scour as a binary choice determined based on the flood condition. However,
the live-bed scour equations alone are not capable of incorporating the effects of a
coarser material overlain by a finer material. This procedure will also look at both
conditions to evaluate which condition controls.
• This DDM establishes an expanded procedure for the estimation of bridge scour
using the existing equations available within HEC-18, 5th edition. In the event that
supplemental guidance is issued by the FHWA, these processes will need to be
reevaluated for applicability.
• The procedure described was developed by VDOT staff for use on VDOT projects. If
this process is to be utilized by other agencies or in other areas, it should be
thoroughly evaluated for applicability to that alternate use.

• Quarrying and plucking scour (Annandale method) for fractured material is
applicable to pier scour only.

REFERENCES

• The following editions apply when referenced in this DDM:
  o FHWA HEC-18, 5th edition
  o Materials Manual of Instruction rev May 2016
  o VDOT Drainage Manual Chapter 12

POLICY

• This section expands the existing VDOT recommendations with respect to scour as
discussed in Chapter 12 of the VDOT Drainage Manual.

PROCEDURE

• The Materials Division has modified the Geotechnical Manual of Instruction to
expand upon the requirements for coring and reporting of D50 values:
  o Report more frequent D50 values for soil data, especially when there is a change
    in character including larger components
  o Report the median core size for those samples of bed rock that do not display
    RQD values ≥ 50% and classified as moderately hard to very hard and
    moderately weathered to unweathered.
  o Includes the position that until more actionable guidance on the scour resistance
    of rock is provided by the FHWA, that VDOT will consider rock with an RQD ≥ 
    50% to be scour resistant.
General
Scour estimation processes were developed, based upon data collected in controlled laboratory conditions. Unfortunately, these processes do not cover many of the conditions typically encountered in the field. To accommodate these conditions, the processes described below will focus on a multipronged approach to evaluating both live-bed and clear-water scour based on a variety of materials encountered, to establish what is likely to be the controlling condition.

In addition, individual site characteristics should also be taken into account in the determination of live-bed vs. clear-water scour. The conditions that could limit the potential Live Bed condition are:

- A dam located in close proximity upstream, capturing the sediments upstream of the crossing
- The crossing is within a normal/flood pool of a lake/reservoir, and the sediments would be expected to fall out upstream.
- An overbank area functioning as an independent scour zone, located downstream of an area that is well-vegetated.

Contraction Scour
If there is the potential for live-bed scour, determine via HEC-18 (Section 6.2.1 Eq. 6.1), then the material present in the stream bed is in motion. If the live-bed conditions exist, then both live-bed and clear-water scour are to be computed in the channel. If live-bed conditions are not indicated, only clear-water scour is to be computed and included below.

The recommended equations to consider are the Live Bed Scour Equation (Section 6.3 Eq. 6.2), the Clear Water Scour Equation (Section 6.4, Eq. 6.4) and if applicable, the Vertical Contraction Scour Equation (Section 6.10 Eq. 6.16).

Compute all three using the D50 of the stream bed material, as applicable to the specific equations (minimum D50 is 0.2mm). Combine the vertical scour to both the live-bed and clear-water scour, and compare the results to each other. The estimate with the least total scour will be considered the controlling condition. Once the controlling condition is determined, evaluate the soil boring data to see if the scour is contained within the limits of the material defined by the D50 selected. If the controlling scour is confined to this layer (Layer 1), then the contraction scour is arrested at this elevation and this portion of the computations is complete. However, if the controlling scour terminates in material that is below Layer 1, then additional computations are needed.
In this case it can be concluded that, at minimum, the scour will reach the point where the material changes between Layer 1 and Layer 2. Clear water scour computations are repeated using the same hydraulic parameters as in the analysis above, assuming that the subsequent D50 material is present in the entire soil column. Combine this result with the vertical scour and compare to the previously computed live-bed and vertical scour, then estimate with the least total scour will be considered the controlling condition.

If the controlling scour is above this layer, then the scour is arrested at the interface between the two layers. If the controlling scour confined to this layer, then the contraction scour is arrested at this elevation. However, if the controlling scour terminates in material that is below the layer characterized by the D50 selected, then additional computations are needed. Repeat these last two steps until a controlling scour is reached, or scour resistant material is encountered.

**Pier Scour**
Use the most applicable method that would apply to the project for the pier configuration, where there is little distinction between live-bed and clear-water scour. Combine this with the estimated contraction scour. Assess the material at the estimated bottom of the scour hole. If the material in the soil column affected by the combined contraction and pier scour has a D50 > 20mm, then the method described by Pier Scour in Coarse Bed Material (Section 7.11 Eq. 7.34) may be used. This method is reported to be applicable to clear-water conditions only. However, in the event that the coarse layer is overlain by the finer layer, this equation would be applicable. The process in evaluating successive layers is similar to what is considered under contraction scour. In this case, any material with a D50 smaller than 20mm will have been removed above the originally estimated elevation.

**Abutment Scour**
Abutment scour is much more complex than the other scour modes, and the equations are more often difficult to apply. Both live-bed and clear-water scour are to be computed:
(1) If the abutments are significantly close to the channel, not meeting the setback limits,
(2) If there is the potential for live-bed scour, as determined by HEC-18 (Section 6.2.1 Eq. 6.1) and
(3) The other live-bed criteria are present.
If live-bed conditions are not indicated, only clear-water Scour is to be considered.

The recommended equations to consider are the Froelich’s Equation (Section 8.6 Eq. 8.1), the HIRE Equation (Section 8.6.2, Eq. 8.2), and the NCHRP 24-20 Method for both clear-water and live-bed (Section 8.6.3 Eq. 8.5 and Eq. 8.6).
Compute all four (4) equations using the D50 of the stream bed material as applicable to the specific equations (minimum D50 is 0.2mm). Combine the total contraction scour as computed above to Froelich’s and HIRE results. The NCHRP 24-20 Method already incorporates the scour based on lateral contraction. However, this should be combined with the vertical contraction component to both the clear-water and live-bed results.

Compare the results of the four (4) methods to each other, and the estimate with the least total scour will be considered the controlling condition. Once the controlling condition is determined, evaluate the soil boring data to see if the scour is contained within the limits of the material defined by the D50 selected. If the controlling scour is confined to this layer (Layer 1), then the scour is arrested at this elevation and this portion of the computations is complete. However, if the controlling scour terminates in material that is below Layer 1, then additional computations are needed.

In this case it can be concluded that, at minimum, the scour will reach the point where the material changes between Layer 1 and Layer 2. Clear water scour computations are repeated using the same hydraulic parameters as in the analysis above, assuming that the subsequent D50 material is present in the entire soil column. Combine the results as described above, the estimate with the least total scour will be considered the controlling condition.

If the controlling scour is above this layer, then the scour is arrested at the interface between the two layers. If the controlling scour is confined to this layer, then the scour is arrested at this elevation. However if the controlling scour terminates in material that is below the layer characterized by the D50 utilized, then additional computations are needed. Repeat this cycle until a controlling scour is reached, or scour resistant material is encountered.

REPORTING AND DOCUMENTATION

- The submission requirements for analysis using this procedure should include the following:
  - Excerpts from the Geotechnical Report documenting the D50 and RQD values of the materials
  - Detailed computations identifying the controlling scour determination at each substructure
  - Scour Plot to scale

Worked example
Chapter 16 – Engineering Software*

TABLE OF CONTENTS

CHAPTER 16 – ENGINEERING SOFTWARE ............................................................... 1
16.1 Software Utilized in the Electronic Development & Delivery of Plans .......... 1
16.2 Hydraulic/Hydrologic Engineering Software in Use by The Department ...... 2
  16.2.1 Introduction and Disclaimer ............................................................... 2
  16.2.2 Link to Appendix 16A-1 ................................................................. 2
16.3 VDOT Web-Based Hydrologic/Hydraulic Applications ............................... 3
  16.3.1 Introduction and Disclaimer ............................................................... 3
  16.3.2 Link to Appendix 16A-2 ................................................................. 3
  16.3.3 Link to VDOT Web Applications Usage Agreement Form ................. 3
  16.3.4 Link to VDOT Web Applications Sign-In Page ............................... 3

List of Appendices

Appendix 16A-1 Hydrologic/Hydraulic Engineering Computer Software in Use by the Department (revised 2/2016)

Appendix 16A-2 VDOT Web-Based Hydrologic/Hydraulic Applications (revised 2/2016)

* Chapter 16 added to Drainage Manual July 2012
Chapter 16 – Engineering Software

16.1 Software Utilized in the Electronic Development & Delivery of Plans

See VDOT CADD Manual For Software Requirements*
16.2 Hydraulic/Hydrologic Engineering Software in Use by The Department

16.2.1 Introduction and Disclaimer

The following section provides a link to the current list of all hydrologic and hydraulic engineering software generally utilized by the Department.

It should be noted that the Department does not necessarily prefer all software that is included on the list for a given application, nor does it reject software that is not included. The list is intended only to represent such hydrologic and/or hydraulic engineering software that the Department either currently uses or has at least summarily tested. It serves as a recommendation, not a requirement. If there is any question as to the application of hydrologic and/or hydraulic engineering software either on Department projects or those projects that will ultimately come under the Department's jurisdiction, an inquiry should be made to the Department's Central Office Hydraulics/Utilities Program.

16.2.2 Link to Appendix 16A-1

Appendix 16A-1 Hydrologic/Hydraulic Engineering Microcomputer Software in Use by the Department (revised 2/2016)
16.3 VDOT Web-Based Hydrologic/Hydraulic Applications

16.3.1 Introduction and Disclaimer

The following section provides a link to a list of all the current web-based hydrologic and hydraulic applications in use by the Department. A User’s Guide for all of the applications can be accessed on the Sign-In page.

VDOT assumes no responsibility for the use/misuse of these software products. The application of these software products is the sole responsibility of the user. There are no expressed or implied warranties. No user support for this software will be provided by VDOT.

Most of these web-based WINDOWS software modules were created to replace older DOS-based programs that will no longer function in the latest MICROSOFT WINDOWS environments. The Department no longer supports or distributes these DOS-based programs.

16.3.2 Link to Appendix 16A-2

Appendix 16A-2 VDOT Web-Based Hydrologic/Hydraulic Applications (revised 2/2016)

16.3.3 Link to VDOT Web Applications Usage Agreement Form

The following is a link to the VDOT Web Applications Usage Agreement Form for External Customers (Consultant):

VDOT Web Applications Usage Agreement Form

External Customers will be required to complete and submit this form, before being issued a sign-in ID and Password.

16.3.4 Link to VDOT Web Applications Sign-In Page

The following is a link to the VDOT Web Applications Sign-In Page for External Customers (Consultant):

VDOT Web Applications Sign-In Page (External)

VDOT Internal Customers can access the applications through this link:

VDOT Web Applications Sign-In Page (Internal)

* Rev. 7/16
Appendix 16A-1 Hydrologic/Hydraulic Engineering Software In Use By The Department

HYDROLOGIC/HYDRAULIC ENGINEERING SOFTWARE IN USE BY THE DEPARTMENT

HYDROGRAPH/FLOOD ROUTING

(1) HEC-HMS*
   - U.S. Army Corps of Engineers' Hydrologic Modeling System
   - Computer requirements: WINDOWS-based.

   Source: U.S. Army Corps of Engineers – Hydrologic Engineering Center

(2) WIN TR-55
   - An interactive package for calculating peak flows and hydrographs using the N.R.C.S. TR-55 procedures. Routing provisions are included.
   - Computer requirements: WINDOWS-based.

   Source: USDA - National Resource Conservation Service

(3) WIN TR-20
   - A program for performing hydrographic analyses & flood routing using N.R.C.S.' procedures described in their "NEH-4" publication.
   - Computer requirements: WINDOWS-based.

   Source: USDA - National Resource Conservation Service
   http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/null/?cid=stelprdb1042793

* Rev. 7/16
Appendix 16A-1

(4) CRITSTRM
• Actually "Critical Storm Duration", the program will determine the ordinates of a hydrograph for the storm even that will produce the largest volume of water for a given frequency using the Rational Formula as a basis for the calculation. This is a module of the Department’s “Web-Based Hydraulics Applications”.
• Computer requirements: Internet access, Microsoft WINDOWS, Microsoft “Internet Explorer” (or fully compatible web browser), and “Adobe Reader” for viewing, saving, and/or generating hardcopy printouts

Source: VDOT – Location & Design Web Section
http://www.virginiadot.org/business/locdes/notification.asp

(5) PONDPACK
• WINDOWS based hydrologic modeling/routing program.
• Analyzes pre and post-developed watershed conditions and pond sizes
• Computes outlet rating curves with tailwater effects, pond infiltration, pond detention times, and analyzes channels
• Computes interconnected pond routing with divergent (multiple) outfalls
• Can use any rainfall duration or distribution to compute hydrographs
• Computes hydrographs for multiple events, adds them at junctions, and routes them through multiple reaches and ponds
• Computer requirements: WINDOWS-based.

Source: Bentley http://www.bentley.com/en-US/Products/PondPack/

(6) SWMS SoftVA
• WINDOWS based hydrologic modeling/routing/analysis program.
• Computes Water Quality and Water Quantity requirements in accordance with the VSMP Part IIB and Part IIC regulations.
• Analyzes pre and post-developed watershed conditions
• Designs BMP’s in accordance with the VDOT BMP Manual and DEQ BMP Clearinghouse
• Computes hydrographs for multiple drainage areas and combines them at defined outfalls
• Computer requirements: WINDOWS-based.

Source: ENSOFTEC, INC.
P.O. Box 3009
Gaithersburg, MD 20885-3009
www.ensoftec.com

* Rev. 7/16
Appendix 16A-1

PEAK DISCHARGE HYDROLOGY

(1) NSS (National Streamflow Statistics Program)
   • NSS is a Windows program for estimating the magnitude and probability of peak discharges for unregulated rural and urban watersheds and for estimating other streamflow statistics for unregulated rural watersheds. NSS replaced NFF (National Flood Frequency Program) in 2004.
   • Computer requirements: WINDOWS- based.

Source: U.S. Geological Survey
http://water.usgs.gov/software/NSS/

(2) PEAKFQWIN
   • A program for determining design peak discharges from stream gaging records (downloadable from USGS’ Internet site) using the Log-Pearson Type III frequency distribution method in accordance with WRC Bulletin 17-B guidelines.
   • Computer requirements: WINDOWS- based.

Source: U.S. Geological Survey
http://water.usgs.gov/software/peakfq.html

(3) EPSON
   • A program that projects design peak flows based on analysis of annual gaged peak flows. Gage records are available on for most all gaging stations in Va. This is a module of the Department’s “Web-Based Hydraulics Applications.”
   • Computer requirements: Internet access, Microsoft WINDOWS, Microsoft “Internet Explorer” (or fully compatible web browser), and “Adobe Reader” for viewing, saving, and/or generating hardcopy printouts

Source: VDOT – Location & Design Web Section
http://www.virginiadot.org/business/locdes/notification.asp

* Rev. 7/16
Appendix 16A-1

(4) DISCHARGE

- A program for estimating the 2, 5, 10, 25, 50, 100 and 500-yr peak flows using the Daniel G. Anderson Method ("MAGNITUDE AND FREQUENCY OF FLOODS IN NORTHERN VIRGINIA") and the Franklin Snyder Method (A.S.C.E. Journal – Hydraulics Division - October, 1958. One hundred point rainfall curves, in the form of external data files, are supplied with the program for use with the Franklin Snyder Method. This is a module of the Department’s “Web-Based Hydraulics Applications”.

- Computer requirements: Internet access, Microsoft WINDOWS, Microsoft “Internet Explorer” (or fully compatible web browser), and “Adobe Reader” for viewing, saving, and/or generating hardcopy printouts.

Source: VDOT – Location & Design Web Section
http://www.virginiadot.org/business/locdes/notification.asp

(5) PQTRANS

- A program for estimating the peak discharges at an ungaged location from a nearby gaging station using both the U.S.G.S. and N.R.C.S. peak discharge transfer formulae. This is a module of the Department’s “Web-Based Hydraulics Applications”.

- Computer requirements: Internet access, Microsoft WINDOWS, Microsoft “Internet Explorer” (or fully compatible web browser), and “Adobe Reader” for viewing, saving, and/or generating hardcopy printouts.

Source: VDOT – Location & Design Web Section
http://www.virginiadot.org/business/locdes/notification.asp

(6) HydrologyVA

- HydrologyVA is an engineering tool which provides computations for the different hydrologic methods used in Virginia.

- The program will calculate the following methods: Ration Method, NRCS TR-55, Anderson Method, Snyder Method, USGS Rural and Urban Regression Method, Log Pearson III

- Computer requirements: WINDOWS-based.

Source: ENSOFTEC, INC.
P.O. Box 3009
Gaithersburg, MD 20885-3009
www.ensoftec.com

* Rev. 7/16
(7) VIRTOC

- A program for determining peak discharges using the Rational Formula. Program has several options for calculating both overland and channel flow time. The program uses rainfall data based on “B, D, & E” factors derived from the NOAA’s “Atlas-14” publication. This is a module of the Department’s “Web-Based Hydraulics Applications.”
- Computer requirements: Internet access, Microsoft WINDOWS, Microsoft “Internet Explorer” (or fully compatible web browser), and “Adobe Reader” for viewing, saving, and/or generating hardcopy printouts

Source: VDOT – Location & Design Web Section
http://www.virginiadot.org/business/locdes/notification.asp

OPEN CHANNEL FLOW

(1) HY-15

- A program for use in designing stable linings for open channels in accordance with the FHWA "HEC-15" publication. The program was originally developed by the FHWA but the Department has re-written as a WINDOWS application and is a module of the Departments “Web-Based Hydraulics Applications”.
- Computer requirements: Internet access, Microsoft WINDOWS, Microsoft “Internet Explorer” (or fully compatible web browser), and “Adobe Reader” for viewing, saving, and/or generating hardcopy printouts

Source: VDOT – Location & Design Web Section
http://www.virginiadot.org/business/locdes/notification.asp

(2) RDDITCH

- A program for use in determining depth and velocity for the 2-yr and 10-yr peak flows in roadside and median ditches. Flow characteristics are calculated for Manning's "n" values of 0.03, 0.05 and 0.015. The program uses rainfall data based on “B, D, & E” factors derived from the NOAA’s “Atlas-14” publication. This is a module of the Department’s “Web-Based Hydraulics Applications.”
- Computer requirements: Internet access, Microsoft WINDOWS, Microsoft “Internet Explorer” (or fully compatible web browser), and “Adobe Reader” for viewing, saving, and/or generating hardcopy printouts

Source: VDOT – Location & Design Web Section
http://www.virginiadot.org/business/locdes/notification.asp

* Rev. 7/16
Appendix 16A-1

(3) RIPRAP

- A program for designing riprap slope protection in accordance with the FHWA's "HEC-11" publication. It considers channel side slopes, bottoms, slope stability by tractive force procedures and riprap slope protection for wave action. This is a module of the Department’s “Web-Based Hydraulics Applications.”
- Computer requirements: Internet access, Microsoft WINDOWS, Microsoft “Internet Explorer” (or fully compatible web browser), and “Adobe Reader” for viewing, saving, and/or generating hardcopy printouts.

Source: VDOT – Location & Design Web Section
http://www.virginiadot.org/business/locdes/notification.asp

(4) DitchSoftVA

- A WINDOWS-based application for use in designing, analyzing, and checking allowable flow velocities and depths of roadside and median ditches in accordance with Chapter 7 of the VDOT DRAINAGE MANUAL. Also allows the user to determine to test different flexible and concrete linings in accordance with the latest version of the FHWA's "HEC-25" publication. Works either as a stand-alone application or, as appropriate, in conjunction with the other modules of Ensoftec's "ENSOFT HYDRO" software suite.
- Computer requirements: Microsoft's EXCEL spreadsheet (required only to generate output using the Department's standard LD-268 form).

Source: ENSOFTEC, INC.
P.O. Box 3009
Gaithersburg, MD 20885-3009
www.ensoftec.com

PIPE FLOW/CULVERT HYDRAULICS

(1) HY-8

- A program for designing and/or analyzing round culvert pipes and box culverts. HY-8 automates culvert hydraulic computations utilizing a number of essential features that make culvert analysis and design easier.
- Computer requirements: Internet access, Microsoft WINDOWS, Microsoft “Internet Explorer” (or fully compatible web browser), and “Adobe Reader” for viewing, saving, and/or generating hardcopy printouts.

Appendix 16A-1

(2) CulvertSoftVA
- A WINDOWS-based application for use in designing/analyzing culverts using the FHWA’s “HDS-5” procedures. The software also includes provisions for designing outlet protection/energy dissipators using VDOT, FHWA, & DEQ procedures.
- Computer requirements: Microsoft’s EXCEL spreadsheet (Required only to generate the Department’s Standard LD-269 form and other basic reports associated with the program).

Source: ENSOFTEC, INC.
P.O. Box 3009
Gaithersburg, MD 20885-3009
www.ensoftec.com

(3) CulvertMaster
- CulvertMaster is an easy-to-use calculator product that designs new culverts and analyze existing culvert hydraulics. It can also be used to analyze: single-barrel crossings, complex embankment cross-drain systems, different shapes and sized culverts, special tailwater considerations, roadway overtopping considering watershed data, culvert characteristics, and weir geometry.
- Computer requirements: WINDOWS-based.

Source: Bentley http://www.bentley.com/en-US/Products/CulvertMaster/

DROP INLET/STORM SEWER DESIGN

(1) InletSoftVA
- A WINDOWS-based application for use in designing/analyzing all types of drop inlets in accordance procedures presented in the VDOT DRAINAGE MANUAL and the FHWA’s HEC-22 publication. Works either as a stand-alone application or in conjunction with the PipeSoftVA software package shown below.
- Computer requirements: Microsoft’s EXCEL spreadsheet (Required only to generate the Department’s Standard LD-204 form).

Source: ENSOFTEC, INC.
P.O. Box 3009
Gaithersburg, MD 20885-3009
www.ensoftec.com

* Rev. 7/16
(2) PipeSoftVA

- A WINDOWS-based application for use in designing/analyzing storm sewers in accordance with procedures described in the VDOT DRAINAGE MANUAL. The software will also generate a hydraulic grade line utilizing the VDOT method. Works either as a stand-alone application or in conjunction with Inletsoft software package described above.

- Computer requirements: Microsoft’s EXCEL spreadsheet (Required only to generate the Department’s Standard LD-229 form and other basic reports associated with the program).

Source: ENSOFTEC, INC.

* P.O. Box 3009
Gaithersburg, MD 20885-3009
www.ensoftec.com

(3) PipeProfilerVA

- A WINDOWS-based application that plots and/or displays storm sewer pipes and appurtenances (in plan profile view) using data files created by the “InletSoftVA” and “PipeSoftVA” program modules from the “Ensoft Hydro” hydraulic design software suite. Calculated hydraulic grade lines may be plotted and/or displayed. The program can also optionally generate separate storm sewer pipe and appurtenance summaries using the same data. The plots and summaries may be viewed and/or printed from with “PipeProfilerVA” but, to be included as part of an electronic plan assembly, must be used in conjunction with CADD software such as “AUTOCAD”, MICROSTATION”, etc. Pipe and structure summaries may also be exported to an “EXCEL” spreadsheet.

- Computer requirements: CADD software such as “AUTOCAD”, “MICROSTATION”, etc. is required in order to import the program’s output into a standard CADD file format. Microsoft “EXCEL” is also required if it is desired to export the summaries to a spreadsheet.

Source: ENSOFTEC, INC.

* P.O. Box 3009
Gaithersburg, MD 20885-3009
www.ensoftec.com

* Rev. 7/16
Appendix 16A-1

(4) **PFLOW**

- A WINDOWS-based application for use in determining flow characteristics in round pipe based on Manning’s equation. This is a module of the Department’s “Web-Based Hydraulics Applications.
- Computer requirements: Internet access, Microsoft WINDOWS, Microsoft “Internet Explorer” (or fully compatible web browser), and “Adobe Reader” for viewing, saving, and/or generating hardcopy printouts

Source: ENSOFTEC, INC.
P.O. Box 3009
Gaithersburg, MD 20885-3009
www.ensoftec.com

(5) **GEOPAK Drainage**

- A module within the GEOPAK Design Software Package used primarily for the design of roadway drainage systems and the production of storm sewer profiles.
- Computer requirements: MICROSTATION CADD software. Program operates within the MICROSTATION environment.

Source: Bentley
http://www.bentley.com/en-US/Products/GEOPAK+Civil+Engineering+Suite/

(6) **StormCAD**

- StormCAD provides comprehensive modeling for the design and analysis of storm sewer systems. StormCAD also provides calculations for catchment runoff, gutters, inlets, junctions, pipe networks, and outfalls, and its intuitive interface makes the design and analysis of storm sewer systems.
- StormCAD includes automated constraint-based design, scenario and data management, and reporting capabilities.
- StormCAD provides comprehensive modeling for the design and analysis of storm sewer systems using a peak flow (Rational Method) approach.
- Computer requirements: WINDOWS-based

Source: Bentley http://www.bentley.com/en-US/Products/StormCAD/

* Rev. 7/16
WATER SURFACE PROFILES / BRIDGE HYDRAULICS

(1) HEC-RAS
- (Hydrologic Engineering Center - River Analysis System) - The U.S. Army Corps of Engineers new software package for the analysis of floodplains and bridged waterways. Full graphics package for viewing x-sections, profiles, rating curves, and 3-D floodplain views.
- Computer requirements: WINDOWS-based.

Source: U.S. Army Corps of Engineers – Hydrologic Engineering Center

(2) FESWMS-2DH
- (Finite Element Surface Water Modeling System) is a two dimensional stream flow model which employs finite element analysis techniques.
- Computer requirements: MS-DOS 3.1 or greater operating system, 640K RAM (minimum), a 10-MEGABYTE hard disk (minimum), a math coprocessor.

Source: USGS http://water.usgs.gov/software/FESWMS-2DH/

(3) BRRIPRAP
- A program that calculates the size of riprap necessary to protect bridge abutments based on the FHWA's "HEC-23" publication (2009). This is a module of the Department's "Web-Based Hydraulics Applications".
- Computer requirements: Internet access, Microsoft WINDOWS, Microsoft "Internet Explorer" (or fully compatible web browser), and "Adobe Reader" for viewing, saving, and/or generating hardcopy printouts

Source: VDOT – Location & Design Web Section
http://www.virginiadot.org/business/locdes/notification.asp

* Rev. 7/16
Appendix 16A-1

(4) CHECKRAS
• A software package developed by the Federal Emergency Management Agency (FEMA) specifically for checking HEC-RAS data sets for compliance with FEMA modeling practices. Note: this software works in conjunction with HEC-RAS so it must be installed on the user’s computer.
• Computer requirements: WINDOWS-based.

Source: VDOT – Location & Design Web Section
http://www.virginiadot.org/business/locdes/notification.asp

(5) RASPLOT
• A software package developed by the Federal Emergency Management Agency (FEMA) specifically generating water surface profile plots in FEMA’s preferred format as extracted from HEC-RAS. Note: this software works in conjunction with HEC-RAS so it must be installed on the user’s computer.
• Computer requirements: WINDOWS-based.

Source: Federal Emergency Management Agency (FEMA)
https://www.fema.gov/rasplot-version-30

INTERACTIVE HYDROLOGIC/HYDRAULIC ENGINEERING PACKAGE

(1) HYDRAIN
• (Also known as POOL FUND PROJECT) An interactive package of programs that perform most hydrologic/hydraulic engineering functions. A master program supervisor and data input shells are included to facilitate using the individual programs. The package currently includes HYDRO (a program to develop peak flows, inflow hydrographs, and analyze gaging data), HYCULV & HY-8 (programs for the design and analysis of culverts), HYDRA (a program for the design and analysis of storm sewers, sanitary sewers, and combination sewers), HY-7/WSPRO (water surface profiles) and HYCHANL (a program for designing channels, ditches & linings).
• Computer requirements: Developed for MS-DOS but will run under WINDOWS.

Source: The Federal Highway Administration, though it apparently is no longer available as a download option on their web site. You can contact Joe Krolak either by e-mail at joseph.krolak@fhwa.dot.gov or by phone at (202) 366-4611.

* Rev. 7/16
MISCELLANEOUS

(1) FISHXING
- Assists in designing and analyzing highway culvert pipes to facilitate the passage of various fish species.
- Computer requirements: WINDOWS-based.

Source: [http://www.stream.fs.fed.us/fishxing/](http://www.stream.fs.fed.us/fishxing/)

(2) HYDRAULIC TOOLBOX
- The Hydraulic Toolbox is a computer program containing calculators that perform many of the routine hydrologic and hydraulic computations.

The following calculators are included: roadway hydrology, open channel flow, weir flow, pavement drainage, inlet capture/bypass, ditch inlet capture/bypass, detention basin routing, channel lining design (vegetation, rolled erosion control products, and rock), multiple riprap sizing applications (channel bank revetments; bridge piers, abutments, and guide banks; spur dikes; embankment overtopping; culvert outlets; open-bottom culverts; and wave attack), riprap filter design, gradation analyses via pebble count or digital image, ditch inlet capture/bypass calculator, culvert assessment tool, a profile system that allows a user defined riprap classification system, culvert assessment profiles, bridge scour calculator and the horizontal grade inlet analysis, and new map or plan view feature, which will allow the user to define a location for their calculators and visually represent their project.
- Computer requirements: WINDOWS-based.


(3) Terrain Navigator Pro
- Mapping software that contains high resolution scans of USGS topographic maps as well as current aerial photographs overlaid with a current street layer. These maps and photos can be customized with labels, marks, symbols, lines, routes, tracks, area fills, GIS data sets, and notes. Layers, maps, and photos can be exported to be used in other GIS and CAD software or image editor.
- Computer requirements: WINDOWS-based.

Source: MyTopo [https://www.terrainnavigator.com/](https://www.terrainnavigator.com/)

* Rev. 7/16
(4) Flowmaster

- FlowMaster quickly performs hydraulic calculations for dozens of element types, from pipes and open channels to drop inlets and weirs.
- Computer requirements: WINDOWS-based.


**DISCLAIMER**

It should be noted that the Department does not necessarily prefer all software that is included on the above list for a given application nor does it necessarily reject software that is not included. The list is intended only to represent such hydrologic and/or hydraulic engineering software that the Department either currently uses or has at least summarily tested. It serves as a recommendation, not a requirement. If there is any question as to the application of hydrologic and/or hydraulic engineering software either on Department projects or those projects that will ultimately come under the Department's jurisdiction, an inquiry should be made to the Department's Central Office Hydraulics/Utilities Program.

As mentioned, this chapter is used to represent the most up-to-date software in use by the Department. The information shown should take precedence over older software that may be listed in previous chapters.

* Rev. 7/16
WEB-BASED HYDRAULIC/HYDROLOGIC APPLICATIONS IN USE BY THE DEPARTMENT

PEAK DISCHARGE HYDROLOGY

DISCHAR
This module is intended for use in computing peak discharges (2, 5, 10, 25, 50, 100 and 500 yr.) for watersheds of 200 acres or more. The module uses Daniel G. Anderson's method and Franklin F. Snyder's method. Anderson's method was developed from test sites up to 570 square miles in northern Virginia. This method applies to an area of 200 acres or more. Anderson's method, entitled *Effects of Urban Development on Floods in Northern Virginia*, was published in 1968. A copy of the original study can be obtained from the U.S. Geological Survey by contacting:

U.S. Geological Survey  
U.S. Books & Reports Sales  
Federal Center  
Box 25425  
Denver, Colorado 80225  
Phone: (303) 236-7476

Snyder's method was published in the October 1958 in the A.S.C.E. *Journal of the Hydraulics Division*. Refer to that publication for detailed explanation of this method. Application of the Snyder Method would be as indicated in Chapter VI of the [VDOT DRAINAGE MANUAL](#).

EPSON - LOG PEARSON TYPE III FREQUENCY CALCULATIONS
This module is based on “*Guidelines for Determining Flood Flow Frequency, Bulletin 17B*” from the US Department of the Interior. It is used as an alternative Log-Pearson type III analysis to LP3SHELL.

This module provides a statistical analysis of stream gauge records in order to establish the discharge - frequency relationship. While this module will function with minimum of four (4) annual gauge flows, it is recognized that approximately twenty years of continuous records is required to establish a reliable gauge rating. Further, the reliability of the discharge - frequency relationship is restricted to approximately 2.5 times the length of record.
Appendix 16A-2

**PQTRANS – PEAK DISCHARGE TRANSFER**

This module allows the user to employ peak discharges from a site, for which they are known (i.e. gauging records, etc.), and utilize them as a basis for estimating peak discharges at another site, on the same or similar nearby watershed. This is done by prorating the known discharges using two nationally recognized formulas developed for this purpose:


2) NRCS (National Resource Conservation Service) Transfer Formula from their NEH-4 publication

**VIRTOC – VIRGINIA RATIONAL METHOD AND TIME OF CONCENTRATION**

This module determines peak discharges using the Rational Formula. The program has several options for calculating both overland and channel flow time. The program utilizes NOAA ATLAS-14 Rainfall Precipitation Frequency Data for every county, and most cities, within the State.

This module was designed to be a user-friendly tool that allows the user to quickly and accurately calculate the peak flow for a given watershed. The VIRTOC module is designed to collect input and present output in English units. It allows the user to make choices in determining the variables used in the Rational formula. The user may enter all of the required variables or choose to calculate the Rational runoff coefficient, time of concentration or intensity. The format also allows the user to make changes in previous input values and recalculate the peak flow without leaving the program.

The user is advised that the use of VIRTOC is constrained by the assumptions of the Rational method and thus the program should not be used for watersheds over 200 ac in size.
OPEN CHANNEL FLOW

RDDITCH - FLOW IN MEDIAN AND SIDE DITCHES
This module was developed for use in determining the average velocity and depth of flow in highway roadside and median ditches. It is particularly useful in ascertaining locations where some sort of ditch lining (i.e. EC-2, EC-3, or paving) is needed. This module can handle multiple reaches of ditch and multiple cross sections (or stations) per reach. Either triangular or trapezoidal shapes can be considered and ditch side slopes and/or bottom width may vary from cross section to cross section. Depth and velocity of flow are calculated for the 2 yr. (50%) and 10 yr. (10%) peak flows for the following Manning's "n" values:

- 0.03 (assumed for natural, earth linings)
- 0.05 (assumed for protective linings, i.e. EC-2, EC-3, etc.)
- 0.015 (assumed for paved linings).

RIPRAP – BASED ON PROCEDURES PRESENTED IN FHWA'S "HEC-11" AND "HIGHWAYS IN THE RIVER ENVIRONMENT" PUBLICATIONS
This module is used for designing rip rap slope protection in accordance with the FHWA's HEC-11 publication. It considers channel side slopes, bottoms, and slope stability by tractive force procedures and rip-rap slope protection for wave action.

The Rip-Rap module is really three (3) separate modules in one: Channel Rip-Rap Design, Wave Action Rip-Rap Design, and Tractive Force. These modules sections will be additionally segmented by these options.

HY-15 – DESIGN OF CHANNELS WITH FLEXIBLE LININGS
Originally developed by SIMONS, LI & ASSOCIATES, INC., this module analyzes flexible and concrete linings for trapezoidal or triangular channels in straight reaches. The module uses the design procedures of Hydraulic Engineering Circular No. 15 (1988). The Manning's "n" value and normal depth calculated may be different from values obtained by use of charts and tables. Manning's "n" varies with the depth and is more accurately calculated by this process. The user has the option to have the module calculate the maximum Discharge (Q) for a given lining

IRRCHANL – IRREGULAR CHANNEL, STAGE-DISCHARGE
This module performs normal depth calculations in irregular shaped (natural) channels using the Manning's equation.
Appendix 16A-2

**PIPE FLOW/CULVERT HYDRAULICS**

**PFLOW – PIPE FLOW (IN CIRCULAR PIPES)**

This module will determine normal depth, discharge, and velocity in circular pipes. Both English and SI metric versions are available. This module function performs similar to the "Field’s Wheel". It will calculate Velocity and:

- "Q" for a given Depth
- Depth for a given "Q"
- Friction Slope for a given Diameter
- Diameter for a given Friction Slope

**BRIDGE HYDRAULICS**

**BRRIPRAP – SIZING RIPRAP FOR BRIDGE ABUTMENTS**

This module is used to calculate the size of riprap necessary to protect bridge abutments. This module was developed using equations and procedures described in the Federal Highway Administration's publication entitled "Bridge Scour and Stream Instability Countermeasures" as revised in 2009. The publication is more popularly known as "Hydraulic Engineering Circular (HEC) No. 23". It is publication # FHWA NHI 01-003 and can be obtained at the following web address: [http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec23.pdf](http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec23.pdf)

**HYDROGRAPH/FLOOD ROUTING**

**CRITSTRM – CRITICAL STORM DURATION (UTILIZING THE RATIONAL METHOD)**

Actually "Critical Storm Duration", the module will determine the storm duration that will produce the largest volume of water for a given frequency using the Rational Formula as a basis for the calculation. The module has been modified from the original equations to incorporate the NOAA Atlas 14 rainfall data, using the B, D & E coefficients.

The need for and the process of determining the Critical Storm Duration is describe in chapter 6 (Hydrology), section 6.4.5.1.4 of the VDOT Drainage Manual.

**DISCLAIMER**

**NOTE:** Most of these web-based WINDOWS software modules were created to replace older DOS-based programs that will no longer function in the latest MICROSOFT WINDOWS environments. The Department longer supports or distributes these DOS-based programs.
Chapter 17 - Flood Plain Hydraulic Studies

TABLE OF CONTENTS

CHAPTER 17 - FLOOD PLAIN HYDRAULIC STUDIES................................................................. 17-1

17.1 Introduction .................................................................................................................. 17-1
17.1.1 Purpose .............................................................................................................. 17-1
17.1.2 Definitions ......................................................................................................... 17-1
17.1.3 Analysis and Design ......................................................................................... 17-2
17.1.4 Hydraulic Studies .............................................................................................. 17-3
  17.1.4.1 Location Hydraulic Study .................................................................. 17-3
  17.1.4.2 Design Hydraulic Study .................................................................... 17-3

17.2 Design Policy/Criteria ............................................................................................... 17-4
  17.2.1 FHWA .......................................................................................................... 17-4
    17.2.1.1 Code of Federal Regulations ............................................................ 17-4
    17.2.1.2 Hydraulic Studies .............................................................................. 17-4
    17.2.1.3 New Construction Criteria ................................................................. 17-4
  17.2.2 AASHTO .......................................................................................................... 17-5
  17.2.3 VDOT .......................................................................................................... 17-5
    17.2.3.1 Design Storm .................................................................................... 17-5
    17.2.3.2 Allowable Backwater ......................................................................... 17-5
    17.2.3.3 Flow Distribution ............................................................................... 17-6
    17.2.3.4 Stream Restoration ........................................................................... 17-6
    17.2.3.5 Flood Control Structures ................................................................... 17-6
    17.2.3.6 Temporary Measures ......................................................................... 17-6
    17.2.3.7 Hydrologic Methods .......................................................................... 17-6
    17.2.3.8 Coastal .............................................................................................. 17-6

17.3 Design Concepts .......................................................................................................... 17-7

17.4 Location Hydraulic Study ............................................................................................ 17-7

17.5 Design Hydraulic Study ............................................................................................... 17-8

17.6 Coordination ............................................................................................................... 17-9
  17.6.1 FEMA ........................................................................................................ 17-9
  17.6.2 DCR Flood Plains ....................................................................................... 17-9
  17.6.3 Locality ......................................................................................................... 17-9

17.7 References .................................................................................................................... 17-9

List of Tables
Table 17-1. Allowable Base Flood Elevation Increases......................................................... 17-5

List of Appendices
Appendix 17A Sample Location Hydraulic Study
Appendix 17B Sample Design Hydraulic Study
Chapter 17 - Flood Plain Hydraulic Studies

17.1 Introduction

17.1.1 Purpose

The purpose of this chapter is to consolidate discussion of activities in flood plains into a single chapter that is consistent with the FHWA Guidance as outlined in Chapter 4. It includes language previously presented in Chapters 8 and 12.

This chapter addresses procedures for compliance with waterway and flood plain management requirements or regulations. The procedure described herein allows for the Department to obtain approvals from various regulatory agencies while fulfilling the applicable requirements and regulations discussed in Chapter 4.

17.1.2 Definitions

Action – Shall mean any highway construction, reconstruction, rehabilitation, repair or improvement undertaken by the Department.

Base Flood – This is the common description of the 1 percent chance flood as used in the Federal Documentation for both FEMA and FHWA. This is also commonly referred to as the 100 year flood, which describes the recurrence interval for the base flood event. See Chapter 6 for means and methods to estimate this value.

Base Flood Plain – That area subject to flooding by the base flood.

Base Flow – This is the flow that can be typically expected in the stream under normal flow conditions. See Chapter 6 for means and methods to estimate this value.

Design Flood – The storm event based on the criteria described in Chapter 6 for roadway safety; typically it is not to be less than 18” clearance to the low shoulder.

Design Hydraulic Study – Study performed via detailed methods in support of projects that are determined by VDOT Hydraulics Staff to have higher risk of adverse impact to the base flood elevation.

Encroachment – An action within the limits of the base flood plain.

Flood Insurance Rate Map (FIRM) – FEMA published map showing flood plain zones, floodways, and other features used for implementation of the National Flood Insurance Program (NFIP).

Insurable Structure - For floodplain management purposes, a structure is a walled and roofed building, including a gas or liquid storage tank, principally above ground, as well as a manufactured home. The terms "structure" and "building" are interchangeable in the National Flood Insurance Program (NFIP). Residential and non-residential structures are treated differently. A residential building built in a floodplain must be elevated above the Base Flood Elevation (BFE). Non-residential buildings may be elevated or floodproofed.
Regulatory Flood Plain – A delineated flood plain presented on the FEMA FIRMs that may or may not have a flood way and may or may not be based on detailed computations.

Regulatory Floodway – A delineated zone located in some regulatory flood plains identifying an area of greater development restriction.

Location Hydraulic Study – Study based on engineering evaluation of the site conditions. A Location Hydraulic Study may be supported by approximate methods or abbreviated computations if needed.

Longitudinal Encroachment – An action that involves placement of fill within a base flood plain that is not directly in support of a stream crossing. This is often associated with a road widening project for an existing roadway adjacent to a stream or a new roadway.

Maintenance – Any action that is necessary to maintain the serviceability or function of an existing roadway, stream crossing, longitudinal encroachment, stream element or flow conveyance.

New Construction – Any action involving a new roadway or modification of an existing roadway by substantially changing the alignment or grade.

Risk – In the context of the discussion in this chapter shall mean the consequences associated with probability of flooding during the base flood event attributed to an encroachment.

Stream Crossing – A VDOT asset that passes from one side of a waterway to the other, with the expressed purpose of conveying the traveling public.

Stream Realignment – Actions taken to relocate a portion of a stream as needed to facilitate other construction activities, improve approach angle or repairing damage to the roadway.

Stream Restoration – Actions taken within the degraded stream and flood plain with the intent to restore sediment transport balance and improve riparian habitat.

Temporary Measures – Actions within the base flood plain that are necessary to facilitate construction, or as a condition of an environmental permit, which will be removed prior to the end of the project.

17.1.3 Analysis and Design

When designing, constructing and maintaining VDOT assets within the base flood plain proper care should be taken to:

- Provide desired level of hydraulic performance up to an acceptable risk level
- Mitigate impacts to the stream environment
- Limit increases to base flood elevation
- Limit adverse impacts to offsite areas affected by the base flood
- Comply with the relevant recommendations of 23 CFR 650
17.1.4 Hydraulic Studies

The level of analysis necessary to make hydraulic determinations referenced in 17.1.3 will be commensurate with the level of risk associated with the scale, scope and location of the project.

For practical purposes only those actions in the base flood plain, which are likely to cause a noticeable adverse effect, or those that otherwise require environmental permits, would prompt a Location Study. The exempted actions would include, but are not limited to, painting, pothole repair, signage, guardrail repair, bridge maintenance activities, temporary measures, etc.

Evaluations of work being taken in base flood plains may be divided into 2 categories, Location Hydraulic Study and Design Hydraulic Study.

17.1.4.1 Location Hydraulic Study
All new construction projects and those few maintenance projects subject to detailed environmental permits shall have a Location Hydraulic Study performed. This is to assess the potential for the project to impact the base flood plain elevation based on an appropriate prior condition. If it cannot be determined using sound engineering judgement that the project will not increase the base flood elevation, then a Design Hydraulic Study may be necessary. In addition there may be other factors that would dictate the need for a detailed study, primary among these would be scour computations.

17.1.4.2 Design Hydraulic Study
The Location Study, may conclude that a detailed engineering analysis is necessary to document the base flood elevation and aid in the design process for the purposes of assessing compliance with the impact limits. The Design Hydraulic Study will document the analysis using the appropriate prior condition and proposed condition flood elevations for comparison. The appropriate analysis tool will be selected based on the risk associated with the location and the complexity of the project.
17.2 Design Policy/Criteria

17.2.1 FHWA

17.2.1.1 Code of Federal Regulations

VDOT is subject to the federal guidance provided in 23 CFR 650 with respect to proposed actions and coordination with FEMA and Localities. VDOT is to apply the guidance to all base flood plains mapped or otherwise.

- Avoid longitudinal encroachments where practical
- Avoid significant encroachments where practical
- Limit impacts to the base flood elevation to no more than a 1’ rise
- Be consistent with the intent of the NFIP where appropriate
- Apply these standards to all construction, repair, rehab and maintenance actions
- Location Studies shall be commensurate with the associated level of risk
- Design Studies, when necessary, shall be commensurate with the associated level of risk

In cases where a project impacts a FEMA mapped flood plain the following additional requirements are to be adhered to consistent with the correspondence between FHWA and FEMA in 1982 establishing the working relationship between the two Federal Agencies. Coordination with FEMA is required when:

- An encroachment into a regulatory floodway would cause an amendment to the floodway map
- An encroachment into a detailed study area without a floodway or a mapped area without base flood elevations (BFE) would cause more than a 1’ rise.

17.2.1.2 Hydraulic Studies

A Location Hydraulic Study shall include the NFIP data if available, discussion of alternatives, risks to the base flood elevation, measures taken to minimize base flood impacts, and involve NFIP community coordination.

A Design Hydraulic Study shall be commensurate with the associated level of risk, detailed engineering analyses utilizing the NFIP data if available and appropriate, use of appropriate design storms, and consistent with the NFIP as practical.

No highway structure should be evaluated as a flood control element unless specifically designed as such and approved by the relevant state and federal agencies.

17.2.1.3 New Construction Criteria

The design hydraulic study shall include the following items for new construction not associated with an existing roadway.

- evaluation and discussion of the practicability of alternatives
- risks associated with implementation of the action/alternative
- impacts on natural and beneficial flood-plain values
- support of probable incompatible flood-plain development
- measures to minimize flood-plain impacts associated with the action
- measures to restore and preserve the natural and beneficial flood-plain values impacted
- evaluation and discussion of any support of incompatible flood-plain development
For lateral encroachments also include

- reasons why the proposed action must be located in the flood plain
- alternatives considered and why they were not practicable

17.2.2 AASHTO

AASHTO guidance on flood plains suggests that increases in the flood elevation due to a transportation project will not significantly increase flood damage to property upstream of the project area and maintain the existing flood distribution to the extent practical.

17.2.3 VDOT

17.2.3.1 Design Storm

Inundation of the travelway and clearance below the low shoulder dictates the level of traffic services provided by the facility and would apply to all roadways located within the base flood plain (stream crossings and longitudinal encroachments). New construction and projects that significantly change the alignment or grade of the roadway shall have an 18” freeboard from the low shoulder to the design storm as determined by the functional classifications of roadways presented in Chapter 6, Hydrology.

17.2.3.2 Allowable Backwater

To protect the public from adverse impacts of flooding due to VDOT projects and meet the federal guidance, designers shall limit the increase to the base flood elevation (BFE) outside of the VDOT ROW to the values reported in Table 17-1.

<table>
<thead>
<tr>
<th>Situation</th>
<th>Increase in BFE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Insurable structure within the base flood plain</td>
<td>0.0’</td>
</tr>
<tr>
<td>FEMA Zone A Area</td>
<td>1.0*</td>
</tr>
<tr>
<td>FEMA Zone AE or A# but not within a floodway</td>
<td>1.0’</td>
</tr>
<tr>
<td>FEMA Zone AE floodway</td>
<td>0.0’</td>
</tr>
<tr>
<td>FEMA Detailed Study Stream with a Floodway</td>
<td>0.0’</td>
</tr>
<tr>
<td>Unmapped or undeveloped area</td>
<td>1.0’</td>
</tr>
<tr>
<td>Other zone designations not considered</td>
<td>Not applicable</td>
</tr>
</tbody>
</table>

* When data provided by the locality determines that the cumulative impact is no greater than 1.0’

In the event that the limitations in Table 17-1 cannot be met there will be additional coordination measures required such as pursuing a CLOMR and LOMR with FEMA or the acquisition of flood easements.
17.2.3.3 Flow Distribution
The conveyance of the proposed stream crossing should be calculated to determine the flow distribution and to establish the location of bridge opening(s). The proposed facility should not cause any significant change in the existing flow distribution. Relief openings in the approach roadway embankment or other appropriate measures should be investigated if there is more than a 10% redistribution of flow provided such openings do not create a concentration of flow which could damage downstream properties.

17.2.3.4 Stream Restoration
To meet environmental regulations VDOT is engaged in a number of projects to improve impaired streams through restoring sediment transport and creating wildlife habitats. The filling of oversized channels and the planting of riparian vegetation has the potential to adversely impact flood elevations. The analytical methods and impact limits described in this chapter shall be applied to projects of this nature. Documentation shall conform to the requirements of this chapter and shall include base flood elevation comparison tables for the reach impacted by the stream restoration.

17.2.3.5 Flood Control Structures
A structure that is not designed as an impounding structure, dam or levee shall not be evaluated as such. VDOT does not permit the use of roadways as dams or levees. It is VDOT practice to not accept responsibility for portions of new roadways that have been designed as dams. Where there are existing roadways that are located on dams, it may be necessary to address current DCR Dam Safety Regulations prior to making any improvements to the roadway or any outfall structure.

17.2.3.6 Temporary Measures
The use of cofferdams, causeways or other measures necessary to meet environmental permit requirements or facilitate construction shall be minimized to the extent practical. However they will not be assessed for impacts to the base flood event, except at the discretion of the Engineer.

17.2.3.7 Hydrologic Methods
The methods as described in Chapter 6 may be used as appropriate. Alternate methods may be used at the discretion of VDOT. If it is determined that the NFIP Published discharges are inappropriate for use, this will be documented in the Design Study and computational methods used consistent with Chapter 6 to determine the discharges to be used for analysis and design.

17.2.3.8 Coastal
When the base flood plain is determined to be due to storm surge and not riverine flow, it is not expected that a VDOT project will affect the base flood elevations. As such activities in coastal areas will typically be limited to a Location Hydraulics Study. However there may be cases where a Design Hydraulic Study is required and they will be addressed on a case by case basis.
17.3 Design Concepts

If the Location Hydraulic Study determines that a Design Hydraulic Study is warranted such studies should reference the appropriate chapter of the Drainage Manual for detailed engineering means and methods based upon the nature of the work being performed. (See Chapter 8 for minor culverts, and see Chapter 12 for all other conditions.)

The following are guidelines governing the level of accuracy to show in the Location Hydraulic Study and Design Hydraulic Study:

- Elevations and distances from surveys are to be shown one decimal places.
- Elevations, distances, from plans, are to be shown up to two decimal places.
- The magnitude of peak discharges are to be shown to three significant figures.
- Velocities are to be shown to the nearest 0.1 fps.
- Calculated water surface elevations are to be shown to the nearest 0.1’.
- Changes in calculated water surface elevations are to be shown to the nearest 0.1’.
- Watershed areas are to be shown to the nearest square mile (sq.mi.) or 3 significant figures if less than 100 sq.mi.

17.4 Location Hydraulic Study

All design projects and those maintenance projects required to have detailed environmental permits are subject to a Location Hydraulic Study. The scope and scale of the study is determined by the nature of the work being performed. The study may be based upon best available data or may require a field visit. It may be appropriate to include computations to support the engineer’s assessment of the conditions but they are not always necessary. Sample Location Hydraulic Study can be found in Appendix 17.A.

The Location Hydraulic Study shall include:

- Project Identifying information
- Project Description: general description and actions being performed within the base flood plain (encroachment)
- Project Type: Maintenance, replacement/rehab, improvement, new construction
- Drainage Area
- Published Regulatory Flood Plain: as applicable
  - Published Mapping
  - Zone Descriptor
  - Provide published base flood elevation if available
  - Note actions in Floodway as applicable *
  - Any other notations as applicable
- Engineers assessment and evaluation criteria
- Conclusion, No further study is required. or: Design Hydraulic Study is required to determine.

The Location Hydraulic Study shall be included in the documentation to be submitted to the Project Manager and provided to the Environmental Division for their use as needed in permitting.

* Action in a Floodway is NOT the sole factor determining that a Design Study is required.
17.5  Design Hydraulic Study

Once a Design Hydraulic Study has been deemed necessary by the Location Hydraulic Study or other engineering criteria, it shall follow the technical guidance provided in the appropriate section of the drainage manual for means and methods. Sample Location Hydraulic Study can be found in Appendix 17.B.

The Design Hydraulic Study shall be appended to the Location Hydraulic Study and shall include:

- Brief Description of the means and methods used in the analysis
- Report the Historical Flood data as available
- Study conclusion Project impacts, success criteria relative to design flood or results as dictated by the nature of the study in CH 8 or CH 12. Include New Construction discussion if applicable.
- Additional actions needed Structure Types evaluated for design and pertinent data (Culvert, Bridge, Other)
- Summary Results as dictated by the nature of the project as per in CH 8 or CH 12.
- Table of Existing and Proposed Flood elevations of all storms evaluated at the VDOT ROW as dictated by the nature of the analysis.
- Published Regulatory Flood Plain: as applicable
  - Comparison of published data to VDOT determined data
    - Hydrology <Published, VDOT, Discussion>
    - Hydraulics <Published, Revised Existing, Proposed, Discussion>
  - Impact to base flood / floodway elevations <Revised Existing, Proposed>
  - Impact to flood / floodway boundary <Revised Existing, Proposed>
  - Is a CLOMR or LOMR necessary? No: The work is consistent with the intent of the NFIP as the documented project impacts do not increase the flood risk to the community. Yes: The project impacts outside the VDOT ROW are such that coordination with FEMA is required or there is modification to the flood boundary or floodway due to the project impacts.
- Electronic Files containing the analysis/modeling on which the conclusions were based. The analysis should be sufficiently annotated to clearly identify the relevant components.
- Existing and Proposed Design Drawings (by reference as applicable):
  - Roadway Plan and Profiles
  - Bridge Front Sheet
  - Survey Drawings

The Design Hydraulic Study shall be included in the documentation to be submitted to the Project Manager and provided to the Environmental Division for their use as needed in permitting. The supporting analyses shall be archived permanently on the VDOT System and entered into a database.
17.6 Coordination

17.6.1 FEMA

In the event that the Design Hydraulic Study determines that a revision to the flood maps is warranted, VDOT will complete the MT-2 Documentation and submit to FEMA for review and approval. The community coordination portion should be completed by the locality and included in the submission.

17.6.2 DCR Flood Plains

DCR Flood Plains will be included in the permit distributions through the IACM process and provided an opportunity to review and comment on any general permit applications.

17.6.3 Locality

When a Location or Design Hydraulic Study documents a project within a Regulatory Flood Plain, after the permit package has been submitted before the IACM for review and comment the locality will be provided with a copy of the VDOT Joint Permit Application which will include the Hydraulic Study for their use. Detailed supporting computations performed are available upon request.

17.7 References


