PLEASE READ

It should be noted, due to the way the "VDOT DRAINAGE MANUAL" has been compiled to a single ".PDF" file, that it will not be possible to access the appendices of individual chapters by either (1) "clicking" on them from the table of contents for that chapter or (2) "clicking" on them from the navigation pane on the left side of the screen. All appendices for all chapters are included in this ".PDF" file, but they can only be accessed by first going to that chapter in which you are interested and then scrolling downwards until you find the particular appendix you want.

Virginia Department of Transportation

DRAINAGE MANUAL

1401 East Broad Street Richmond, Virginia 23219

(804) 786-2534 http://www.virginiadot.org

DRAINAGE MANUAL

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DRAINAGE MANUAL

Prepared by

LOCATION AND DESIGN DIVISION

HYDRAULICS SECTION

Adopted April 2002

Virginia Department of Transportation

Recommended for Approval: *Mohammad Mirshahi, P.E.*
Location and Design Engineer

Approved: *C. Frank Gee*
Acting Chief Engineer

Chapter 1 - Introduction

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Chapter 1 - Introduction

1.1 Background

The Virginia Department of Transportation (VDOT) has developed the 2001 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 fourth 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, with assistance from an engineering consultant.

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

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 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.

Table 1-1. Chapter Template and Contents

Sections	Contents
Introduction	Objectives
Policy	Define Course of Action for VDOT, State, Federal and Local Policy
Design Criteria	Specify Standards by which Policy is Carried Out
Design Concepts	Design Considerations/ Guidelines Theory and Equations Requirements Figures Necessary to Support Procedures or Examples
Design Procedures & Examples	Step-by-Step Procedures Specific Design Considerations Specific Software Solutions Figures Necessary to Support Procedures or Examples
References	Sources of Information / Bibliography
Appendices	All Figures, Forms and Design Aids Not Necessary to be in Concepts or Procedures Drainage Design Memoranda Definitions Checklists Symbology and Nomenclature

1.3 Drainage Design Memoranda

Most previous Location and Design Division (L&D) Instructional and Informational Memoranda (I&IM) regarding drainage design have been incorporated into this manual, as part of the text or as Drainage Design Memoranda (DDM). The drainage-related L&D I&IMs LD- (D) 121, Allowable Pipe Materials, and LD- (D) 11, Erosion and Sediment Control, will remain part of the Department's Location and Design Division. All previous Technical Supplements have been incorporated into the new manual text. The following Table 1-2 lists I&IMs that have been incorporated into Drainage Design Memoranda and Table 1-3 lists I&IMs that have been incorporated into manual text.

Table 1-2. I&IMs Incorporated into Drainage Design Memoranda

I&IM #	Subject	DDM #	CHAPTER
223	Drainage Instructions	1	15
195.3	Management of Stormwater	2	11
71.8	Minor Structure Excavation	3	15
140.6	Utility Conflicts	4	15
130.7	Underdrains	5	15
146.1	Board Policy	6	2

Table 1-3. I&IMs Incorporated into Manual Text

I&IM #	Subject	Chapter
73.3	Rip Rap	7
94.1	Removing Bridge Fill	12
166.3	Soil Stabilization Mat	7
173	Temp. Const. Causeways	12
182.4	Temp. Diversion Channels	10
214	Multi-barrel Culverts	15

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 Regulations (VR 215.02.00)
- Virginia Erosion and Sediment Control Regulations (VR 625.02.00)
- Virginia Erosion and Sediment Control Handbook
- Virginia Stormwater Management Handbook, Volumes I and II
- VDOT Survey Instruction Manual

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

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

1.5 User Instructions

This manual is divided into 15 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 as the entire document or by individual chapter.

The hardcopy or compact disc version of the Drainage Manual can be ordered from the VDOT Hydraulics Section (804) 786-2534.

The authors of this manual have strived to maintain the accuracy and reliability of the information and procedures presented herein. The execution of an engineering design; however, 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.

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 in Richmond, Virginia at (804) 786-2534. When revisions are available, the user will be notified via a "Revisions" file on the VDOT website at the location where the manual may be viewed and/or downloaded. This file will contain links that will take the user to the actual page (s) that have been revised. The file also briefly describes 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 Model Drainage Manual, Highway Drainage Guidelines, and other publications)
  - Executive Committee
  - Task Force on Hydrology and Hydraulics 1998
- Federal Highway Administration
- Federal Emergency Management Agency
- United States Geological Survey
- United States Army Corps of Engineers
- Virginia Department of Transportation –
  - Mr. Charles McIver, State Hydraulics Engineer
  - Mr. John Dewell, Assistant State Hydraulics Engineer
  - Mr. David LeGrande, Assistant State Hydraulics Engineer
  - Mr. Roy Mills, Assistant State Hydraulics Engineer and Member AASHTO Task Force on Hydrology and Hydraulics
  - Mr. Calvin Boles, Former State Hydraulics Engineer and Former AASHTO Task Force on Hydrology and Hydraulics, Chair
- Virginia Department of Conservation and Recreation - Division of Soil & Water Conservation
- Virginia Department of Environmental Quality
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- Research publications and materials furnished by the private sector
- Parsons Brinckerhoff Quade & Douglas, Inc.
  - Mr. Glenn Bottomley, P.E., Project Manager
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  - Mr. David Dee, P.E.
  - Mr. Peter Smith, P.E.
  - Jerome M. Normann & Associates
  - Mr. Jerome M. Normann, P.E.
- Photo on cover courtesy of Virginia Department of Game and Inland Fisheries. Photographed by Mr. Dwight Duke. 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.

Appendix 1A-1 Definitions and Abbreviations

Abbreviations:

AASHTO	American Association of State Highway and Transportation Officials
DDM	Drainage Design Memorandum
DEQ	Department of Environmental Quality
EPA	Environmental Protection Agency
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
I&IM	Instructional and Informational Memorandum
NPDES	National Pollutant Discharge Elimination System
RDM	Road Design Manual
USCOE/USACE	United States Corps of Engineers
USGS	United States Geological Survey
VDOT	Virginia Department of Transportation or the Department
VSMR	Virginia Stormwater Management Regulations

Chapter 2 - Policy

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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.

2.3 References

American Association of State Highway and Transportation Officials. 1999. *AASHTO Model Drainage Manual*.

Appendix 2A-1 Definitions and Abbreviations

Abbreviations:

CFR	Code of Federal Regulations
DOT	Department of Transportation
EPA	Environmental Protection Agency
FEMA	Federal Emergency Management Agency
FHPM	Federal-Aid Highway Program Manual
FHWA	Federal Highway Administration
FWPCA	Federal Water Pollution Control Act
FWS	Fish and Wildlife Service
NMFS	National Marine Fisheries Service
NPS	National Park Service
NRCS	National Resource Conservation Service; formerly Soil Conservation Service (SCS)
OCZM	Office of Coastal Zone Management
RCRA	Resource Conservation and Recovery Act
SIP	State Implementation Plan
TVA	Tennessee Valley Authority
USCOE/USACE	United States Army Corps of Engineers
USCG	United States Coast Guard
USFS	United States Forest Service

VIRGINIA DEPARTMENT OF TRANSPORTATION

LOCATION AND DESIGN DIVISION

DRAINAGE DESIGN MEMORANDUM

GENERAL SUBJECT: BOARD POLICIES ON PARTICIPATION BY TOWNS, CITIES AND COUNTIES	NUMBER: DDM # 6
SPECIFIC SUBJECT:	DATE: March 23, 1988
	SUPERSEDES: LD-83 (D) 146.1

Attached for your information and use is the policy revision, pertaining to state and local participation in the cost of right of way, sidewalks, utility adjustments, and storm sewers on improvements to the state's highway systems, that was adopted by the Commonwealth Transportation Board on February 18, 1998, and became effective upon adoption

Section 1.00 of this policy is applicable to the SECONDARY SYSTEM PROJECTS IN COUNTIES AND TOWNS OF UNDER 3,500 POPULATION. This revision provides the following principal changes on secondary system projects:

- Permits the right of way cost for sidewalks, where justified, to be borne by secondary construction allocations for the county.

- Where the Department determines the utilization of curb and gutter project's construction as the most economical design, the cost of the necessary storm sewer and appurtenances may be borne by secondary construction allocations for the county.

- Required right of way acquisition costs for projects within towns operating under Sections 33.1-79 and 33.1-82, Code of Virginia, may be provided from the secondary construction allocations for the county, in the same manner as previously permitted for projects situated beyond the corporate limits of such towns.

- All justification and/or design determination studies are to be documented in the project file.

Section 2.00 of this policy is applicable to the PRIMARY AND URBAN SYSTEMS WITHIN THE CORPORATE LIMITS OF CITIES AND TOWNS operating under the provisions of Sections 33.1-23.2, 33.1-23.3 and 33.1-44. Code of Virginia, as amended. The revision provides the following principal changes:

- It provides a uniform method for determining drainage participation costs within incorporated cities and towns without regard to population.

- Where storm sewer outfalls are constructed outside of the normal right of way limits and deemed necessary for adequate project drainage, the cost will be a part of the normal project rather than figured on a run-off ratio basis, provided none of the storm water conveyed is diverted from another watershed.

- Provides clarification that all storm sewers and outfalls constructed outside the normal right of way limits and deemed beyond that necessary to adequately drain the project will be financed on a run-off ratio basis.

Adopted by the Commonwealth Transportation Board of February 18, 1988.

POLICY FOR STATE PARTICIPATION IN THE COST OF RIGHT OF WAY, SIDEWALKS AND STORM SEWERS IN COUNTIES, TOWNS AND CITIES

WHEREAS, the Commonwealth Transportation Board has previously adopted a policy for state and local participation in the costs of right of way, sidewalks, utility adjustments, and storm sewers on projects in cities, towns, and counties; and

WHEREAS, changes in conditions, including revisions to the statutes of Virginia, make it advisable to amend certain provisions of this policy;

NOW, THEREFORE, BE IT RESOLVED, that the attached “Policy for State Participation in the Cost of Right of Way, Sidewalks and Storm Sewers in Counties, Towns and Cities” on Secondary System projects in counties and towns of under 3,500 population, and on Urban and Primary System projects within the corporate limits of cities and towns is hereby adopted; and

BE IT FURTHER RESOLVED, that the Board's policies – adopted on August 18, 1966 and September 21, 1978, be and hereby are rescinded

1.00 SECONDARY SYSTEM PROJECTS IN COUNTIES AND IN TOWNS OF UNDER 3,500 POPULATION.

1.01 The provisions of this section apply to the system of state highways in the several counties of the state as authorized by Section 33.1-67, Code of Virginia, as amended; and those within the corporate limits of towns of less than 3,500 population which operate under the provisions of Sections 33.1-79 and 33.1-82, Code of Virginia, as amended.

1.02 Where new sidewalks are desired and justified by traffic studies or otherwise determined by the Department as required for pedestrian safety, all right of way necessary for the construction of the sidewalks may be borne by secondary construction funds allocated for use in the county.

1.03 Where new sidewalks are desired and justified by traffic studies, one-half the construction cost of new sidewalks shall be borne by secondary construction funds allotted for use in the county and one-half from funds other than highway funds. However, where the contemplated improvement requires the relocation of existing sidewalks, these shall be replaced and the total cost shall be borne by secondary construction funds allocated for use in the county.

1.04 Existing storm sewers shall be relocated or replaced at no cost to others; secondary construction funds allocated for use in the county shall bear 100 percent of the cost.

1.05 Where the construction of new curb and gutter is determined by Department engineers to be the most economical design, the cost of new storm sewers and appurtenances such as drop inlets, manholes, etc., may be borne by secondary construction funds allocated for use in the county, provided none of the storm water to be conveyed is diverted from another watershed.

1.06 Where the construction of curb and gutter within the right of way limits is desired, or is necessary for the development of adjacent property, but is not deemed by Department engineers to be the most economical design, the cost of storm sewers and appurtenances (drop inlets, manholes and similar items) shall be financed from secondary construction funds and other sources on the basis of run-off ratios and percentages of participation as indicated below:

State: Run-off from within right of way, 100%. Run-off from areas outside the road right of way and within the watershed common to the project, 25%.

Others: Run-off from areas outside the road right of way and within the watershed common to the project, 75%.

1.07 Diverted drainage from water sheds not common to the project shall be financed from secondary construction funds and other sources on the run-off ratios and percentages of participation as indicated below:

State: Run-off from the state's right of way within the area of the diverted watershed, 100%

Others: Run-off from all areas in the diverted watershed, exclusion of state right of way, 100%

1.08 All storm sewer outfalls that are found necessary or desirable shall be financed from secondary construction funds and other sources on the run-off ratios and percentages of participation as indicated below:

State: Run-off from the state's right of way within the area being drained, 100%

Others: Run-off from all areas other than the state's right of way in the area being drained, 100%

1.09 Where, through zoning and development control ordinances, the local governing body requires participation in the off-site drainage and where their plans from an overall standpoint reasonably conform to the above-established policy, the local governing body's plan shall become the Transportation Board's policy for that locality.

1.10 The adjustment of utilities necessitated by the construction of sidewalk or storm sewer will be borne by secondary construction funds, except where the utilities are located on public property which has been dedicated or acquired for street or road purposes, including uses incidental thereto, or where there are franchise or other provisions where by the utility owner is required to bear the expense of such relocation of adjustment.

1.11 Unless otherwise specified by state statute or policy of the Commonwealth Transportation Board, all other right of way required for improvements to secondary system shall be acquired by purchase, gift, or power of eminent domain and cost thereof financed from secondary construction funds allocated for use in the county.

2.01 The provisions of this section apply to improvements in cities and towns for which construction funds, pursuant of Sections 33.1-23.2, 33.1-23.3 and 33.1-44, Code of Virginia, as amended, are allocated.

2.02 All storm sewers, both parallel and transverse and all appurtenances, such as drop inlets, manholes, etc., that fall within the right of way limits of urban improvement or construction projects on exiting or new locations and are considered necessary for adequate project drainage by Department engineers will be financed at the percentage required by law for the construction of the project; provided none of the storm water to be conveyed is diverted from another watershed.

2.03 All storm sewers and outfalls constructed outside of the normal right of way limits of urban projects that are considered by Department engineers as necessary for adequate project drainage will be financed at the percentage required by law for the construction of the project; provided none of the storm water to be conveyed is diverted from another watershed.

2.04 All storm sewers and outfalls constructed outside of the normal right of way limits of urban projects that are considered by Department engineers as beyond that needed to adequately drain the highway project shall be financed on a run-off ratio basis between federal and/or state funds and city or town funds.

2.05 Whenever parallel storm sewer, manholes, etc., within an urban project or outfalls beyond the right-of-way and project limits are utilized by a city or town for the conveyance of diverted storm drainage, then the cost of such storm sewers, outfalls, etc., shall be financed on a run-off ratio basis between federal and/or state funds and city or town funds.

Chapter 3 - Documentation

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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
    - 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.

3.2.2 Practices

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

1. 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

2. The designer should document all design assumptions and selected criteria including the decisions related thereto.

3. 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.

4. Documentation should be organized to be concise and complete, so that knowledgeable designers can understand years hence what predecessors did.

5. 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.

6. 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.

7 Documentation should include data and information from the conceptual stage of project development through service life to provide successors with all information.

8. Documentation should be organized to logically lead the reader from past history through the problem background, into the findings, and through the performance.

9. 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.

3.2.3 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 designer should refer to the VDOT Road Design Manual for required Hydrologic and Hydraulic Computation Report submittal times in advance of roadway milestones; such as, Field Inspection, Right-of-Way, etc.

3.4 References

American Association of State Highway, and Transportation Officials. 1982. *Highway Drainage Guidelines*

American Association of State Highway, and Transportation Officials. 1999. *AASHTO Model Drainage Manual*

Appendix 3A-1 Definitions and Abbreviations

Abbreviations:

FEMA	Federal Emergency Management Agency

Appendix 3B-1 Documentation Data Sheet for Hydrologic and Hydraulic Computations

SUBMITTAL:	***FIELD INSPECTION***	**SUBMITTAL DATE:**	***10/21/01***
Project:	*Route 33 West Point Bridges over Pamunkey and Mattaponi Rivers*	Scheduled Advertisement: Revised:	*~~May 2002~~* *May 2003*
Project Nos.	*0033-333-102, PE-101* *0033-333-103, PE-101*	Drainage Manual Revision:	*6/86*
		Agreement Date:	*10/23/99*
		Earliest Date of Calculations:	*11/99*
Location:	*New Kent County* *King William County* *Town of West Point* *King and Queen County*		
From:	*New Kent County*	Scheduled / actual milestones	
		Preliminary Field Review:	
To:	*King William County*	Field Inspection:	
		Public Hearing Plans:	
Other Info:			
		Public Hearing:	
Submitted	*Company/Agency*		
By:	*address*	Right-of-Way	
Submitted	*VDOT C.O.*	Pre-Final:	
To:	*Richmond, VA*		
		Final:	
Submitted			
To:		Construction Completion:	
Submitted			
To:			
Design Assignments			
	Project Manager	Hydraulics Task Leader	

Note: Sheet to be filled out and included in H&H Report. Blank sheet provided on next page.

Appendix 3B-1 Documentation Data Sheet for Hydrologic and Hydraulic Computations

SUBMITTAL:	***FIELD INSPECTION***	**SUBMITTAL DATE:**	
Project:		Scheduled Advertisement: Revised:	
Project Nos.		Drainage Manual Revision:	
		Agreement Date:	
		Earliest Date of Calculations:	
Location:			
From:		Scheduled / actual milestones	
		Preliminary Field Review:	
To:		Field Inspection:	
		Public Hearing Plans:	
Other Info:			
		Public Hearing:	
Submitted			
By:		Right-of-Way	
Submitted		Pre-Final:	
To:			
		Final:	
Submitted			
To:		Construction Completion:	
Submitted			
To:			
Design Assignments			
	Project Manager	Hydraulics Task Leader	

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 - Hydrology

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

Section II - 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 III - Culverts Hydraulics

A. Criteria
B. Methodology
C. Culvert Computations and Summary Table
D. Data Gathering

Section IV - Storm Sewer Hydraulics

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

Section V - Stormwater Management

A. Criteria
B. Methodology
C. Stormwater Management Plan Summary
D. Detention Pond Computations
E. Data Gathering

Section VI – 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: Thisa suggested format and does not attempt to identify all the elements necessary for adequate analysis or documentation

Appendix 3B-3 Field Engineer's Hydraulic Report

Survey by ____________________

To :
Attention:
From :

Subject : Field Engineers
Hydraulic Report

Rte.________ Proj. ____________________

Stream Name ____________________

Sta.____________________

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.)

____

____

____

____

Appendix 3B-3 Field Engineer's Hydraulic Report

IV. State rip rap and/or scour protection recommendations and justification for these recommendations. (Dist. Bridge Engr.).

V. Provide a basic assessment of the environmental, ecological, historical and economic considerations, which may exert an influence on this site. (District Drainage Engineer)

VI. Make note of any flood plain zoning and/or flood plain studies in existence or eminently proposed. (Dist. Drainage Engr.)

VII. Other Special Considerations and Remarks (District Bridge and Drainage Engineers)

Chapter 4 – Legal

TABLE OF CONTENTS

List of Appendices

Chapter 4 - Legal

4.1 Overview

4.1.1 Introduction

Various drainage laws and rules applicable to highway facilities are discussed in this chapter. The intention is only 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
- The courts look with disfavor upon infliction of injury or damage that could reasonably 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 radical 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 where the Department has expressly agreed to abide by the local criteria. The Department recognizes its moral obligation to uphold and support 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:

- Highway Drainage Guidelines, American Association of State Highway and Transportation Officials, Washington, D. C., 1982. Chapter V - The Legal Aspects of Highway Drainage, which also includes a Glossary of legal definitions
- Legal Research Digest, Transportation Research Board
- Virginia Code ANN., tit. 21 (Drainage, Soil Conservation, Sanitation and Public Facilities Districts), §§ 21-292 to -432, (Michie 1995)
- 24 Virginia Administration Code § 30-70-90 (1989)
- 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 which 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. These include laws 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 National Flood Insurance Program

4.4.1 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 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.

4.4.2 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

4.4.3 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. Consistency with NFIP standards is a requirement for Federal-aid highway actions involving regulatory floodways. The community, by necessity, is the one who must submit proposals to Federal Emergency Management Agency (FEMA) for amendments to NFIP ordinances and maps in that community should it be necessary. The highway agency should deal directly with the community and, through them, deal with FEMA. Determination of the status of a community's participation in the NFIP and review of applicable NFIP maps and ordinances are, therefore, essential first steps in conducting location hydraulic studies and preparing environmental documents.

4.4.4 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. Three types of NFIP maps are published:

- Flood Hazard Boundary Map (FHBM)
- Flood Boundary and Floodway Map (FBFM)
- Flood Insurance Rate Map (FIRM)

An FHBM is generally not based on a detailed hydraulic study; therefore, the floodplain boundaries shown are approximate. An FBFM, on the other hand, is generally derived from a detailed hydraulic study and should provide reasonably accurate information. The hydraulic data from which the FBFM was derived are available through the regional office of FEMA. This is normally in the form of computer input data records for calculating water surface profiles. The FIRM is generally produced at the same time using the same hydraulic model and has appropriate rate zones and base flood elevations added.

Communities may or may not have published one or more of the above maps 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 semiannually for each State.

4.4.5 Coordination with FEMA

It is intended that there should be coordination 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
- 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, through 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 Consistent with Floodways

In many situations it is possible to design and construct highways in a cost-effective manner such that their components are excluded from the floodway. This is the simplest way to be consistent with the standards and should be the initial alternative evaluated. 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), the project may normally be considered as being consistent with the standards, if hydraulic conditions can be improved so that no water surface elevation increase is reflected in the hydraulic analysis of the new conditions.

4.4.7 Revisions of 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, the community 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. Such planning is best accomplished when the floodway is first established. However, where the community is willing to amend an established floodway to support this option, the floodway may be revised.

The responsibility for demonstrating that an alternative floodway configuration meets NFIP requirements rests with the community. However, this responsibility may be borne by the agency proposing to construct the highway crossing. Floodway revisions must be based on the hydraulic model that was used to develop the currently effective floodway but updated to reflect existing encroachment conditions. This will allow determination of the increase in the base flood elevation that has been caused by encroachments since the original floodway was established. Alternate floodway configurations may then be analyzed.

Base flood elevation increases are referenced to the profile obtained for existing conditions when the floodway was first established.

4.4.8 Data for Revisions

Data submitted to FEMA, through the community, in support of a floodway revision request should include the following:

- Copy of current regulatory Flood Boundary Floodway Map, showing existing conditions, proposed highway crossing and revised floodway limits
- Copy of computer printouts (input, computation, and output) for the current 100-year model and current 100-year floodway model
- Copy of computer printouts (input, computation, and output) for the revised 100-year floodway model. Any fill or development that has occurred in the existing flood fringe area must be incorporated into the revised 100-year floodway model
- Copy of engineering certification is required for work performed by private subcontractors

The revised and current computer data required above should extend far enough upstream and downstream of the floodway revision area in order to tie back into the original floodway and profiles using sound hydraulic engineering practices. This distance will vary depending on the magnitude of the requested floodway revision and the hydraulic characteristics of the stream.

If input data representing the original hydraulic model are unavailable, an approximation should be developed. A new model should be established using the original cross-section topographic information, where possible, and the discharges contained in the Flood Insurance Study, which established the original floodway. The model should then be run confining the effective flow area to the currently established floodway and calibrated to reproduce, within 0.10 foot, the "With Floodway" elevations provided in the Floodway Data Table for the current floodway. Floodway revisions may then be evaluated using the procedures outlined above.

4.4.9 Allowable Floodway 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 will 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 Federal-Aid Highway Program Manual (FHPM) 6-7-3-2, FHWA, "Location and Hydraulic Design of Encroachments on Floodplains" (23 CFR 650, Subpart A) 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 backwater greater than 1 foot.
- 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.
- Prior to initiating construction, the constructing agency provides FEMA with revised flood profiles, floodway and floodplain mapping, and background technical data necessary for FEMA to issue revised Flood Insurance Rate Maps and Flood Boundary and Floodway Maps for the affected area, upon completion of the structure.

4.4.9.1 Highway Encroachment on a Floodplain - Detailed Study (FIRM)
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 increase in the base flood elevation based on technical data from the flood insurance study. Technical data supporting the increased flood elevation shall be submitted to the local community and through them to FEMA for their files.

4.4.9.2 Highway Encroachment on a Floodplain Indicated on an FHBM
In communities where detailed flood insurance studies have not been performed, the highway agency must generate its own technical data to determine the base floodplain elevation and design encroachments in accordance with FHPM 6-7-3-2. Base floodplain elevations shall be furnished to the community, and coordination carried out with FEMA as outlined previously where the increase in base flood elevations in the vicinity of insurable buildings exceeds 1 foot.

4.4.9.3 Highway Encroachment on Unidentified Floodplains
Encroachments that are outside of NFIP communities or NFIP identified flood hazard areas should be designed in accordance with FHPM 6-7-3-2 of the Federal Highway Administration.

4.4.10 Levee System

For purposes of the National Flood Insurance Program (NFIP), FEMA will only recognize in its flood hazard and risk mapping effort those levee systems that meet, and continue to meet, minimum design, operation, and maintenance standards that are consistent with the level of protection sought through the comprehensive floodplain management criteria as outlined in the NFIP. The levee system must provide adequate protection from the base flood. Information supporting this must be supplied to FEMA by the community or other party seeking recognition of such a levee system at the time a flood risk study or restudy is conducted, when a map revision is sought based on a levee system, and upon request by the Administrator during the review of previously recognized structures. The FEMA review will be for the sole purpose of establishing appropriate risk zone determinations for NFIP maps and shall not constitute a determination by FEMA as to how a structure or system will perform in a flood event. For more information on the requirements related to levee systems see "National Flood Insurance Program and Related Regulations," Federal Emergency Management Agency, Revised October 1, 1986 and Amended June 30, 1987 (44 CFR 65.10).

4.5 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.

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 the Federal-Aid Highway Program Manual (FHPM), Volume 6, Chapter 7, Section 3, Subsection 2, and were published in the Federal Register, April 26, 1979 (44 FR 24678), and 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 Predominates

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. § 33.1-89 (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
    - 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.

  - 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
  - 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).
  - 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 anytime, 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.

- <u>Determine Facts About The Complaint:</u>
  - 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
- <u>Collect Facts About the Specific Flood Event(s) Involved:</u>
  - 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 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

American Association of State Highway and Transportation Officials. 1982. Highway Drainage Guidelines, Chapter 5.0 - Legal Considerations.

Federal Emergency Management Agency. 1987. National Flood Insurance Program and Related Regulations.

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 Flood, Chicago Illinois 60604.)

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An Engineer Looks at Drainage Law, Alfred R. Pagan, F ASCE, Engineering Issues, ASCE, October – 1972

Groundwater Law – The Riparian Problem, Alfred R. Pagan, F ASCE, American Water Works Association – 1978

Impoundment Safety Regulations, State Water Control Board, July – 1978

Chapter V – The Legal Aspect of Highway Drainage – Highway Drainage Guidelines – Published by the AASHTO - 1979

Appendix 4A-1 Definitions and Abbreviations

Definitions:

Code of Federal Regulations	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.
Common Enemy Doctrine	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,” and, 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.
Common Laws	The body of principles which developed from immemorial usage and custom and which receives judicial recognition and sanction through repeated application.
English Rule	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.
Executive Orders	Federal laws that are issued by the President of the United States.
Federal Register	A daily publication of the Federal Government making federal regulations, legal notices, Presidential Proclamations, Executive Orders, etc., known to the public as they are proposed and subsequently issued.
Flood Insurance Study	Established the original floodway.

Appendix 4A-1 Definitions and Abbreviations (Cont.)

Reasonable Use Rule	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.
Riparian Doctrine	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.
Statutory Laws	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.
Tort	A violation of a personal right guaranteed to the individual by law.

Abbreviations:

CFR	Code of Federal Regulations
EO	Executive Orders
NFIP	National Flood Insurance Program

Chapter 5 - Planning and Location

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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 stormwater management 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 stormwater management 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 stormwater management 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 stormwater management 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). 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 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, 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 (1982) 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.

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.

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

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 stormwater management 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.

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.

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 right-of-way 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 right-of-way? Are alternative sites available for storage of stormwater? Is property available outside the right-of-way? Does the project schedule allow time to acquire additional right-of-way?
- 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 stormwater management system?

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 stormwater management 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

5.4.3 Type of Data

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

5.4.3.1 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.

5.4.3.2 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.

5.4.3.3 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.

5.4.3.4 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.

5.4.3.5 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.

5.4.3.6 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.

5.4.3.7 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 an unusual debris or 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.

5.4.3.8 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.

5.4.3.9 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.

5.4.3.10 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.

5.4.3.11 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.

5.4.3.12 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.

5.5 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.

5.6 References

American Association of State Highway and Transportation Officials. 1982. Highway Drainage Guidelines

“Accuracy of Computer Water Surface Profiles”, Technical Paper No. 114, M. W. Burnham and D. W. Davis. 1986

Appendix 5A-1 Definitions and Abbreviations

Definitions:

eutrophication	The process of over-enrichment of water bodies by nutrients often typified by the presence of algal blooms.
karst topography	Irregular topography characterized by sinkholes, streamless valleys and streams that disappear into the underground, all developed by the action of surface and underground water in soluble rock such as limestone.

Abbreviations:

FEMA	Federal Emergency Management Agency
NEPA	National Environmental Protection Agency
NOAA	National Oceanic and Atmospheric Administration
USGS	United States Geological Survey

Chapter 6 – Hydrology

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Chapter 6 - Hydrology

6.1 Introduction

6.1.1 Objective

The analysis of the 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 more 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, or where flood routing through culverts is used, 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, Volume X.

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)

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 publication, :AASHTO Highway Drainage Guidelines".

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 highwater elevations, flood discharges, and locations of hydraulic features such as reservoirs, water projects, 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.

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. Refer to Section 6.5.1 Documentation Requirements and Chapter 3 of the "AASHTO Highway Drainage Guidelines" for more details.

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.

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 may be used for all routine designs provided there is at least 10 years of continuous or synthesized flow records for 10-year discharge estimates and 25 years for 100-year discharge estimates
- Suitable computer programs such as HYDRAIN's HYDRO, HEC-1, TR-55 and TR-20 may be used to facilitate tedious hydrologic calculations. The TR-55 method has been found best suited for drainage areas greater than 200 acres.
- Other methods may be approved as applicable upon submission to the VDOT State Hydraulics Engineer
- The 100-year discharges specified in the FEMA flood insurance study should be used to analyze the impacts of a proposed crossing on a regulatory floodway. However, if these discharges are deemed to be outdated, the discharges based on current methods may be used subject to receipt of necessary regulatory approvals.

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 should 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.

6.2.12 Review Frequency

All proposed structures designed to accommodate the selected design frequency should be reviewed using a base flood and a check storm of a higher design frequency 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 risks 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

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

Table 6-1 presents VDOT design frequencies for the various drainage facilities on streets and highways. For specific frequency criteria, the user is directed to the various design chapters for channels, culverts, storm drains, bridges, etc.

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. Minimum Criteria Recurrence Intervals (Flood Frequencies) for Use in Design

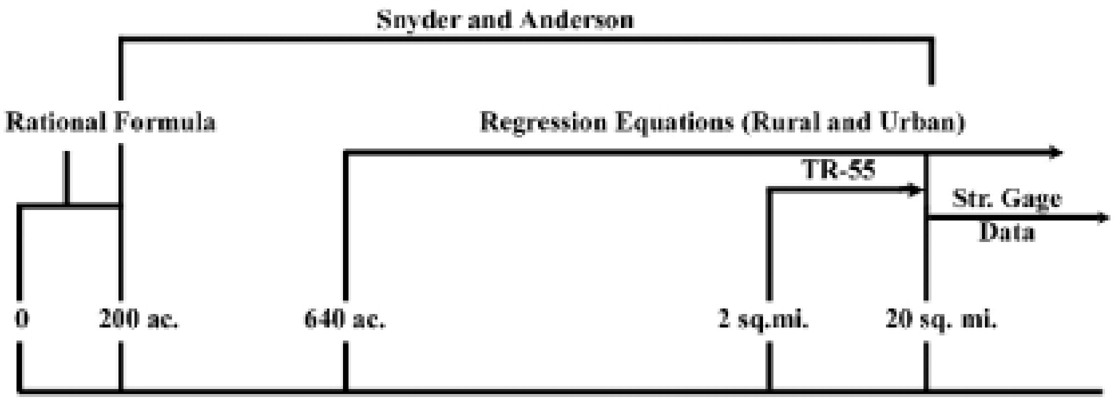
CHANNELS *Refer to Chapter 7*	
Roadside and Median Ditches	
Capacity	10-Year (10%)
Protective Lining	2-Year (50%)
Special Channels	Refer to Chapter 7
CULVERTS *Refer to Chapter 8*	
Interstate	50-Year (2%) Minimum for Non-depressed Roadways 100-Year (1%) for Depressed Roadways*
Principal Arterial	25-Year (4%) Minimum
Minor Arterial, Collector, Local Roads	5 to 10-Year (10-20%) Minimum
STORM DRAINAGE *Refer to Chapter 9*	
Interstate	50-Year (2%) for Non-depressed Roadways 100-Year (1%) for Depressed Roadways*
Principal Arterial	
With shoulder	25-Year (4%)
Without shoulder	10-Year (10%)
Minor Arterial, Collector, Local Roads	10-Year (10%)
CURB DROP INLETS *Refer to Chapter 9*	
Principal Arterial	
With shoulder (grade and sag)	10-Year (10%)
Without shoulder	
Grade	10-Year (10%)
Sag	50-Year (2%)
Minor Arterials, Collectors, and Local Roads	
With shoulder	No specific frequency (use a design intensity of 4 in/hr with check storm intensity of 6.5 in/hr)
Without shoulder	No specific frequency (use a design intensity of 4 in/hr with check storm intensity of 6.5 in/hr)

Note: (%) = Percentage Exceedance Probability

** A depressed roadway is defined as a roadway where water/runoff cannot escape the roadway such as underpasses, tunnels, low point of cut sections, etc. A typical 4-lane divided highway with depressed median is not necessarily depressed by this criteria unless water cannot overflow the travel lanes without ponding on the pavement.*

6.3.2 Peak Discharge Method Selection

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



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 Hydrologic Method Selection Based on Drainage Area

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.

<u>Travel Time</u>

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:

$$T_t = \frac{L}{3600V} \tag{6.1}$$

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

<u>Time of Concentration</u>

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.

$$t_c = T_{t1}+T_{t2}+...T_{tm} \tag{6.2}$$

Where:

t_c = Time of concentration, minutes (min)
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.

6.4.1.1 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:

$$V_w = \left(gD_m\right)^{0.5} \qquad (6.3)$$

Where:

V_w = The 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 ft/s).

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 percent open water), Manning's equation is more appropriate.

For additional discussion of Equation 6.3 and travel time in lakes and reservoirs, see King's Handbook of Hydraulics, fourth edition, page 8-50, or Elementary Mechanics of Fluids, by Hunter Rouse, John Wiley and Sons, Inc., 1946, page 142.

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 percent 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 $100 \div \text{RI}$.

The designer should note that the 5-year flood is not one that will necessarily be equaled or exceeded in any given five years. There is a 20 percent chance that the flood will be equaled or exceeded in any year; therefore, the 5-year flood could conceivably occur in several consecutive years, or it may not occur for many years. The same reasoning applies to floods with other return periods.

6.4.2.2 Design Frequency

Cross Drainage: 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
- Overtop the highway
- 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 is designed to pass the 25-year flow, it would not be uncommon for a larger event storm (such as the 100-year event) to overtop the roadway. In this scenario, the larger event storm should be used as the “check” or review frequency in the hydraulic analysis. Refer to Chapter 8 for additional details.

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. If a storm drain system is designed to pass the 25-year flow, it would not be uncommon for a larger event storm (such as the 100-year event) to cause inundation of the roadway. In this scenario, the larger event storm should be used as the “check” or review frequency in the hydraulic analysis.

6.4.2.3 Review Frequency

After sizing a drainage facility, it will be necessary to review this proposed facility with a base discharge. This is done to ensure that there are no unexpected flood hazards inherent in the proposed facilities. The review flood is usually the 100-year event. In some cases, a flood event larger than the 100-year flood is used for analysis to ensure the safety of the drainage structure and nearby development.

6.4.2.4 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-year rainfall will produce the 10-year flood. Depending on antecedent soil moisture conditions, and other hydrologic parameters this may be true or there may not be a direct relationship between rainfall and flood frequency.

6.4.2.5 Intensity-Duration-Frequency Curves (IDF)

Rainfall data are available for many geographic areas. From these data, rainfall intensity-duration-frequency curves (IDF) have been developed for the commonly used design frequencies.

Figure 6-2 shows a sample IDF curve for Richmond, VA. There are different IDF curves for different areas of the state. Choose the IDF curve that is the closest to your project site. The charts provide rainfall intensities for use in the Rational Method and other hydrologic analyses based on a storm duration (minutes) and frequency (return period). Appendices 6B-3 through 6B-18, at the end of this chapter, contain IDF curves available in Virginia.

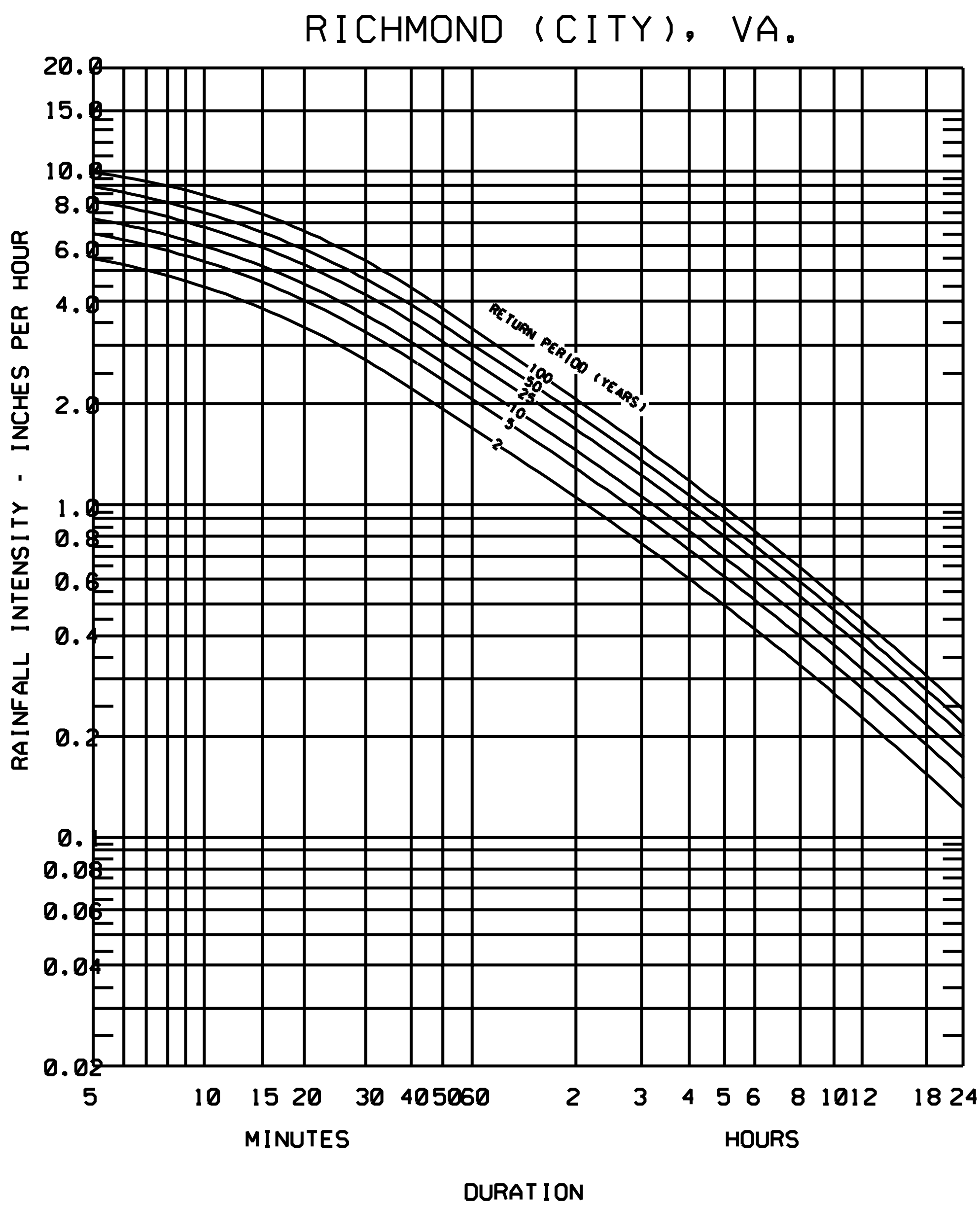


Figure 6-2. Intensity-Duration-Frequency Curve for Richmond

6.4.2.6 Discharge Determination

Estimating peak discharges of various recurrence intervals is one of the most common engineering challenges faced by drainage facility designers. The problem 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.

A listing of gaged sites can be found at the following website: >http://www.afws.net/gauges/start.html<.

- 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 Hydrologic Analysis Procedure Flowchart

The hydrologic analysis procedure flowchart, Figure 6-3 shows the steps needed for the hydrologic analysis, and the designs that will use the hydrologic estimates.

6.4.4 Peak Discharge Methods

6.4.4.1 Rational Method

6.4.4.1.1 Introduction

The Rational Method is recommended for estimating the design storm peak runoff for areas as large as 200 acres. In low-lying tidewater areas, the Rational Method can be considered for areas up to 300 acres. While the Rational Method is relatively straightforward to apply, its concepts are quite sophisticated. Considerable engineering judgment is required to reflect representative hydrologic characteristics, site conditions, and a reasonable time of concentration (t_c). Its widespread use in the engineering community represents its acceptance as a standard of care in engineering design.

6.4.4.1.2 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. Also, the effects of upstream detention facilities may be taken into account.
- Restrictions to the natural flow such as highway crossings and dams that exist in the drainage area should be investigated to see how they affect the design flows
- Charts, graphs, and tables included in this chapter are not intended to replace reasonable and prudent engineering judgment in the design process

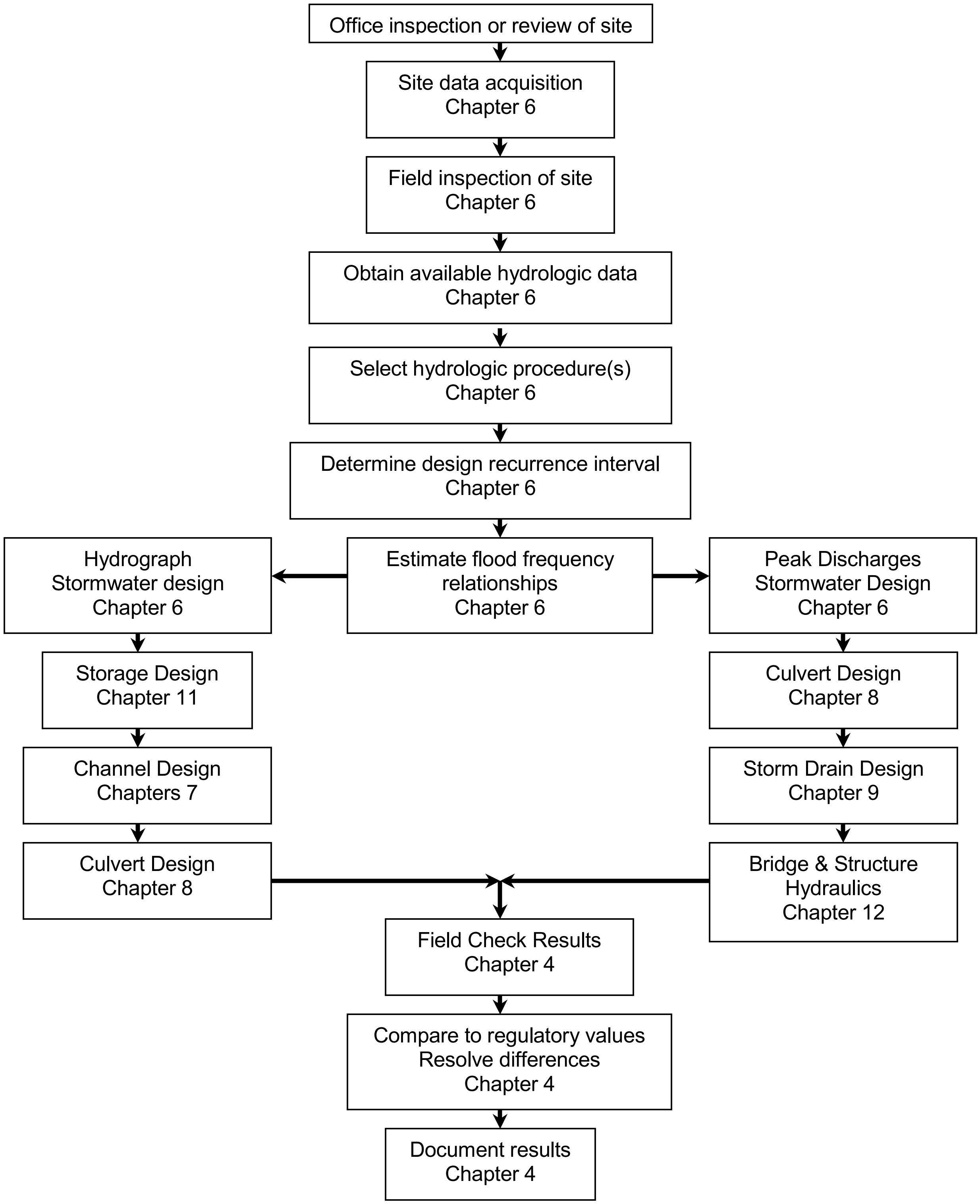


Figure 6-3. Hydrologic Analysis Procedure Flowchart

6.4.4.1.3 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 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.

   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 6.4.5.1)*

6.4.4.1.4 Equations

The rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and 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 location being analyzed).

The Rational Method Formula is expressed as follows:

$$Q = C_f CiA \qquad (6.4)$$

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 design location, acres (ac)

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

6.4.4.1.5 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 be greater than 1.0. Table 6-2 lists the saturation factors for the Rational Method.

Table 6-2. Saturation Factors For Rational Formula

Recurrence Interval (Years)	C_f
2, 5, and 10	1.0
25	1.1
50	1.2
100	1.25

Note: C_f multiplied by C must be less than or equal to 1.0

6.4.4.1.6 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 under investigation. 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 rainfall intensity-duration-frequency (IDF) curve for the area of study. IDF curves for Virginia are found in Appendices 6B-3 through 6B-18.

Time of concentration (t_c) for most drainage areas less than about 200 acres will normally be comprised of overland flow (OLF), channel flow or concentrated flow (CF), and conveyance flow in pipes. 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 feet.

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 properly applied within the limits shown on the nomograph. Refer to Appendix 6D-1 for the Seelye chart. The Seelye method is the preferred VDOT method for computing overland flow.

Kinematic Wave Method

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

$$t_c = 0.93\frac{L^{0.6}n^{0.6}}{i^{0.4}s_o^{\ 0.3}} \tag{6.5}$$

Where:

L = Length of strip feet (ft)
n = Manning's roughness coefficient
i = Rainfall intensity (determined iteratively), inches per hour (in/hr)
S_o = Slope, feet/foot (ft/ft)

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 reliable **only for impervious surfaces** with n=0.05 or less and a maximum length of 300 feet. The kinematic wave method should not be used beyond these parameters. 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 inches. The "n-values" normally associated with channel or ditch flow do not apply to the kinematic wave calculations for overland flow time.

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.

When the total time of concentration has been calculated for a point of study (i.e.: culvert) 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.

6.4.4.1.7 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 percent. The average range of C-values should be used for intermediate areas where the majority of grades and slopes range from 2 to 5 percent. The highest range of C-values should be used for steep areas (grades greater than 5 percent), 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.

6.4.4.1.8 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 feet in urban areas and 400 feet in rural areas.

6.4.4.2 Anderson Method
6.4.4.2.1 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.

6.4.4.2.2 Application
This method was developed from analysis of drainage basins in Northern Virginia with drainage area sizes up to 570 square miles.

6.4.4.2.3 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 ($L/\sqrt{S}$)) were selected as more appropriate for use on the basis of independent work by Snyder (1958) and theoretical considerations.

A graphical description of the lag-time relationships for the extreme conditions, the natural channels or completely sewered smooth channels, is shown in Appendix 6F-2.

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 Appendix 6F-2 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 Appendix 6F-2. 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.

6.4.4.2.4 Equations

The equation for the Anderson Method is as follows:

$$Q = R(230)KA^{0.82}T^{-0.48} \tag{6.6}$$

Where:

- Q = Maximum rate of runoff, cubic feet per second (cfs)
- R = Flood frequency ratio (Refer to Appendix 6F-1)
- K = Coefficient of imperviousness = 1.00 + 0.015(I)
  where I is the percentage of basin covered with impervious surface
- A = Basin area, square miles (sq. mi.)
- T = Time lag, hours (Refer to Table 6-3 or Appendix 6F-2)

Table 6-3. Anderson Time Lag Computation

Time Lag, T	Basin Description
$4.64\left(\frac{L}{\sqrt{S}}\right)^{0.42}$	For natural rural basins
$0.90\left(\frac{L}{\sqrt{S}}\right)^{0.50}$	For developed basins partially channelized
$0.56\left(\frac{L}{\sqrt{S}}\right)^{0.52}$	For completely developed and sewered basins

L = Basin length in miles along watercourse from site to basin boundary
S = Index of basin slope in feet per mile based on slope between points 10 and 85 percent of L

6.4.4.3 Snyder Method

6.4.4.3.1 Introduction

The Snyder Method was developed as the “Synthetic Flood Frequency Method” by Franklin F. Snyder. This method was originally presented in the “ASCE Proceedings, Vol. 84 No. HYS) in October 1958.

6.4.4.3.2 Applications

The Snyder Method has been found to produce acceptable results when properly applied to drainage areas between 200 acres and 20 square miles. This method provides the user with an adjustment factor for partly developed basins by the use of percentage factors for the length of channel storm sewered and/or improved.

6.4.4.3.3 Equations

The Snyder Method can be used to determine peak discharges based on runoff, time of concentration, and drainage area. The Snyder Equation can be used for natural basins, partially developed basins, and completely sewered areas. The following is the Snyder Equation:

$$Q_p = 500AI_R \tag{6.7}$$

Where:

Q_p = Peak discharge, cubic feet per second (cfs)
A = Basin area, square miles (sq. mi.)
I_R = $\left[\frac{\text{Runoff}}{T_c}\right]$, inches per hour (in/hr)
T_c = Time of concentration, hours (hrs)

Runoff can be determined from charts in Appendix 6G-4 through 6G-12.

Refer to the Design Procedure and Sample Problem, Section 6.5.2.3, for the computation of I_R and T_c.

6.4.4.4 Rural Regression Method

6.4.4.4.1 Introduction

Regional regression equations are a commonly accepted method for estimating peak flows at ungaged sites or sites with insufficient data. Also, they have been shown to be accurate, reliable, and easy to use as well as providing consistent findings when applied by different hydraulic engineers (Newton and Herrin, 1982). 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 meteorologic characteristics.

For details on the application of Rural Regression Equations in Virginia, the user is directed to the following publication: "Methods for Estimating the Magnitude and Frequency of Peak Discharges of Rural, Unregulated Streams in Virginia," U.S.G.S. Water Resource Investigations Report 94-4148 (1995).

6.4.4.4.2 Application

The regression equations should be used routinely in design for drainage areas greater than one square mile. Where there is stream gage data, the findings from a Log Pearson III method should govern if there is significant variance $\pm$10% from those obtained using the rural regression equations, and provided there is at least 10 years of continuous or synthesized stream gage record. Where there is less stream gage record, reasonable and prudent judgment along with consideration of the standard regression error should be used in reaching a design decision.

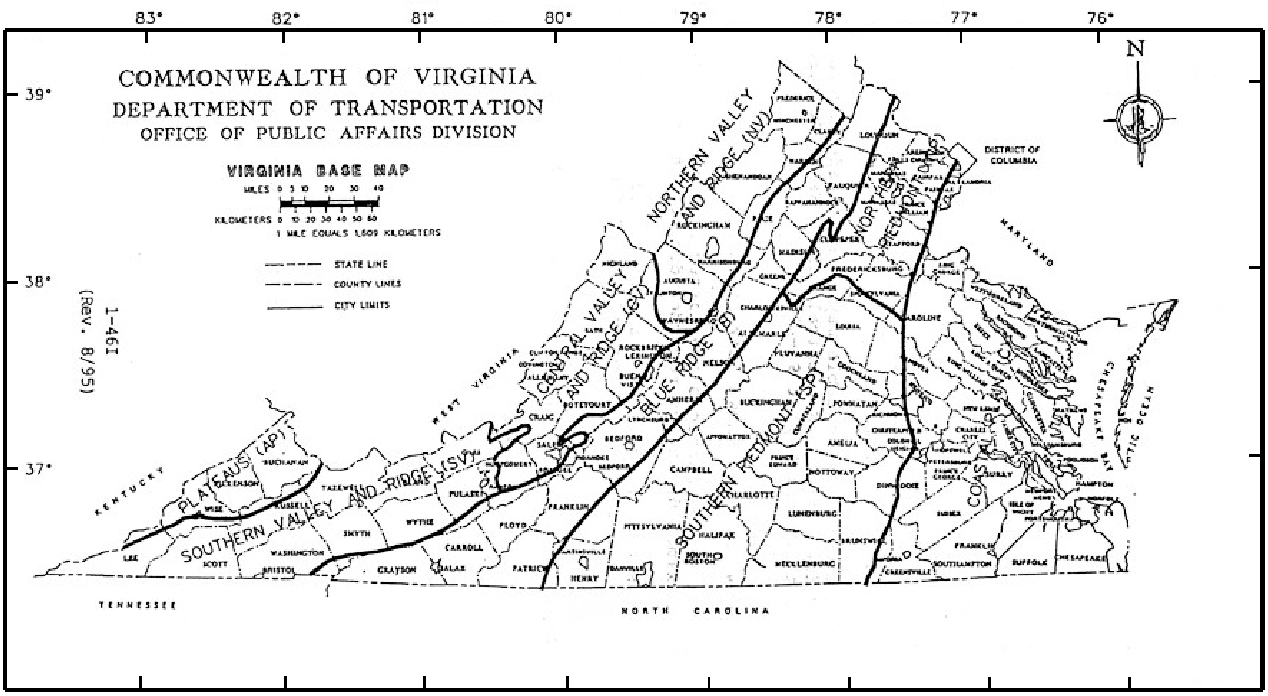
6.4.4.4.3 Characteristics

Watershed Characteristics Method: The primary characteristics commonly include the drainage area above the point of interest as an independent variable. The remaining watershed characteristics are much more varied and depend upon the statistical significance of such variables as channel slopes and length as well as the watershed shape factors, perimeter, basin fall, basin orientation, natural storage, and land use. Meteorological characteristics considered as independent variables include rainfall parameters, snow melt, evaporation, temperature, and wind.

Channel Geometry Characteristics Method: The primary characteristics commonly include a uniquely measured channel width, depth or combination thereof. These geometric features are measured at the most stable point on a channel, usually at the low flow crossover point above or below a bendway. Sometimes this stable reach is considered to be about five times the channel width. The top flow width and corresponding depth are those corresponding to the dominant discharge with about a 2 to 3-year recurrence interval. Additionally, channel geometry measurements must always be obtained beyond the influence of any man-made facilities as well as within a reach that has not incurred major damage from a recent flood; i.e., no extreme bank erosion or "head cuts".

6.4.4.4.4 Hydrologic Regions

Regression analyses use stream gage data to define hydrologic regions. These are geographic regions having very similar flood frequency relationships and, as such, commonly display similar watershed, channel and meteorological characteristics; often termed hydrologically homogeneous geographic areas. Because of the distance between stream gages and sometimes due to the foregoing mixed flood population events, the regional boundaries cannot be considered as precise. Figure 6-4 shows the hydrologic regional boundaries for Virginia.



PEAK - DISCHARGE REGIONS
(modified from Feeneman, 1938)

Figure 6-4. Peak Discharge Regions

Problems related to hydrologic boundaries may occur in selecting the appropriate regression equation. First, the watershed of interest may lie partly within two or more hydrologic regions or a problem may occur when a watershed lies totally within a hydrologic region but close to a hydrologic region boundary. Another problem may occur when the watershed is partly or totally within an area subject to mixed population floods. In these instances, care must be exercised in using regression equations. A field visit is recommended to first collect all available historical flood data, as well as to compare the project's watershed characteristics with those of the abutting hydrologic regions.

6.4.4.4.5 Equations

Tables 6-4 and 6-5 contain the multi-parameter and drainage-area-only regression equations for estimating peak discharges in Virginia.

Table 6-4. Multiple-Parameter Regional Regression Equations for Estimating Peak Discharges of Streams in Virginia

Regression Equation	Standard Error of Prediction (percent)	Equivalent Years of Record	Regression Equation	Standard Error of Prediction (percent)	Equivalent Years of Record
Coastal Plain (C) – 29 Sites			**Northern Valley and Ridge (NV) – 29 Sites**		
$Q_{(2)} = 2.4(A)^{1.005}(SL)^{0.852}$	57.1	1.4	$Q_{(2)} = 73.0(A)^{0.955}(L)^{-0.307}(F)^{0.041}$	37.8	3.6
$Q_{(5)} = 4.0(A)^{0.999}(SL)^{0.884}$	59.7	2.5	$Q_{(5)} = 119(A)^{0.953}(L)^{-0.290}(F)^{0.063}$	33.5	7.4
$Q_{(10)} = 4.9(A)^{1.005}(SL)^{0.932}$	59.4	3.8	$Q_{(10)} = 153(A)^{0.944}(L)^{-0.273}(F)^{0.081}$	31.4	12.2
$Q_{(25)} = 6.0(A)^{1.016}(SL)^{0.998}$	61.0	5.6	$Q_{(25)} = 196(A)^{0.931}(L)^{-0.251}(F)^{0.107}$	30.9	18.5
$Q_{(50)} = 6.8(A)^{1.024}(SL)^{1.044}$	64.1	6.7	$Q_{(50)} = 228(A)^{0.926}(L)^{-0.241}(F)^{0.124}$	31.9	22.2
$Q_{(100)} = 7.6(A)^{1.033}(SL)^{1.088}$	68.5	7.5	$Q_{(100)} = 263(A)^{0.925}(L)^{-0.237}(F)^{0.138}$	33.8	24.4
$Q_{(200)} = 8.3(A)^{1.042}(SL)^{1.130}$	73.9	8.0	$Q_{(200)} = 300(A)^{0.928}(L)^{-0.239}(F)^{0.149}$	36.3	25.3
$Q_{(500)} = 9.2(A)^{1.055}(SL)^{1.185}$	82.7	8.5	$Q_{(500)} = 356(A)^{0.936}(L)^{-0.247}(F)^{0.161}$	40.8	25.1
Northern Piedmont (NP) – 19 Sites			**Central Valley and Ridge (CV) – 34 Sites**		
$Q_{(2)} = 179(A)^{0.655}$	51.1	1.6	$Q_{(2)} = 89.2(A)^{0.788}$	31.0	4.8
$Q_{(5)} = 317(A)^{0.644}$	49.3	3.3	$Q_{(5)} = 222(A)^{0.712}$	29.3	8.7
$Q_{(10)} = 438(A)^{0.641}$	50.2	4.9	$Q_{(10)} = 372(A)^{0.668}$	28.6	12.9
$Q_{(25)} = 626(A)^{0.640}$	53.8	6.7	$Q_{(25)} = 647(A)^{0.620}$	29.5	17.5
$Q_{(50)} = 793(A)^{0.640}$	58.0	7.7	$Q_{(50)} = 918(A)^{0.591}$	31.4	19.4
$Q_{(100)} = 984(A)^{0.641}$	63.5	8.2	$Q_{(100)} = 1{,}254(A)^{0.565}$	34.1	20.2
$Q_{(200)} = 1{,}200(A)^{0.643}$	70.1	8.5	$Q_{(200)} = 1{,}665(A)^{0.542}$	37.4	20.2
$Q_{(500)} = 1{,}535(A)^{0.646}$	80.4	8.6	$Q_{(500)} = 2{,}354(A)^{0.514}$	42.6	19.5
Southern Piedmont (SP) – 67 Sites			**Southern Valley and Ridge (SV) – 35 Sites**		
$Q_{(2)} = 21.6(A)^{0.881}(E)^{0.310}(L)^{-0.423}$	40.2	2.8	$Q_{(2)} = 45.7(A)^{0.880}$	45.0	1.7
$Q_{(5)} = 31.9(A)^{0.854}(E)^{0.351}(L)^{-0.417}$	35.7	6.2	$Q_{(5)} = 89.5(A)^{0.825}$	43.4	2.6
$Q_{(10)} = 38.8(A)^{0.848}(E)^{0.379}(L)^{-0.430}$	35.5	9.3	$Q_{(10)} = 127(A)^{0.800}$	44.2	3.3
$Q_{(25)} = 54.8(A)^{0.852}(E)^{0.392}(L)^{-0.463}$	38.0	12.3	$Q_{(25)} = 181(A)^{0.774}$	46.6	4.2
$Q_{(50)} = 74.3(A)^{0.860}(E)^{0.390}(L)^{-0.495}$	41.4	13.6	$Q_{(50)} = 228(A)^{0.759}$	49.1	4.7
$Q_{(100)} = 101(A)^{0.869}(E)^{0.382}(L)^{-0.529}$	45.7	14.2	$Q_{(100)} = 281(A)^{0.745}$	52.0	5.2
$Q_{(200)} = 136(A)^{0.879}(E)^{0.373}(E)^{-0.561}$	50.6	14.4	$Q_{(200)} = 339(A)^{0.733}$	55.3	5.5
$Q_{(500)} = 197(A)^{0.893}(E)^{0.361}(L)^{-0.602}$	58.0	14.2	$Q_{(500)} = 425(A)^{0.718}$	60.2	5.7
Blue Ridge (B) – 54 Sites			**Appalachian Plateaus (AP) – 17 Sites**		
$Q_{(2)} = 95.4(A)^{0.760}$	33.4	4.0	$Q_{(2)} = 262(A)^{0.749}(SL)^{-0.175}$	33.6	3.5
$Q_{(5)} = 201(A)^{0.726}$	34.1	6.5	$Q_{(5)} = 134(A)^{0.844}(SL)^{0.032}$	21.3	12.2
$Q_{(10)} = 298(A)^{0.710}$	35.5	8.8	$Q_{(10)} = 103(A)^{0.880}(SL)^{0.136}$	18.1	23.5
$Q_{(25)} = 450(A)^{0.695}$	38.8	11.0	$Q_{(25)} = 90.4(A)^{0.902}(SL)^{0.227}$	19.3	31.5
$Q_{(50)} = 584(A)^{0.687}$	42.2	12.0	$Q_{(50)} = 87.0(A)^{0.910}(SL)^{0.280}$	21.9	33.0
$Q_{(100)} = 735(A)^{0.680}$	46.2	12.5	$Q_{(100)} = 85.7(A)^{0.916}(SL)^{0.324}$	24.7	33.4
$Q_{(200)} = 907(A)^{0.674}$	50.7	12.6	$Q_{(200)} = 85.0(A)^{0.920}(SL)^{0.365}$	27.9	33.5
$Q_{(500)} = 1{,}165(A)^{0.667}$	56.7	12.8	$Q_{(500)} = 85.5(A)^{0.923}(SL)^{0.411}$	31.9	33.5

A=Drainage area, square miles (sq.mi.); E= Average basin elevation, feet above sea level (ft); F=Forest, percent (%); L=Main channel length, miles (mi); SL=Main channel slope, feet per mile (ft/mi). Peak discharge regions are shown in Figure 6-4.

Table 6-5. Drainage-Area-Only Regional Regression Equations for Estimating Peak Discharges of Streams in Virginia

Regression Equation	Standard Error of Prediction (percent)	Equivalent Years of Record	Regression Equation	Standard Error of Prediction (percent)	Equivalent Years of record
Coastal Plain (C) – 29 Sites			**Northern Valley and Ridge (NV) – 29 Sites**		
$Q_{(2)} = 57(A)^{0.589}$	55.8	1.4	$Q_{(2)} = 72(A)^{0.785}$	39.2	3.4
$Q_{(5)} = 106(A)^{0.569}$	58.3	2.5	$Q_{(5)} = 128(A)^{0.794}$	35.0	6.9
$Q_{(10)} = 153(A)^{0.555}$	62.1	3.5	$Q_{(10)} = 178(A)^{0.796}$	32.7	11.4
$Q_{(25)} = 230(A)^{0.539}$	68.6	4.5	$Q_{(25)} = 254(A)^{0.797}$	32.1	17.6
$Q_{(50)} = 302(A)^{0.528}$	74.1	5.2	$Q_{(50)} = 317(A)^{0.798}$	33.2	21.1
$Q_{(100)} = 388(A)^{0.518}$	80.2	5.7	$Q_{(100)} = 386(A)^{0.800}$	35.3	23.1
$Q_{(200)} = 489(A)^{0.509}$	86.7	6.2	$Q_{(200)} = 461(A)^{0.802}$	38.2	23.7
$Q_{(500)} = 652(A)^{0.497}$	96.1	6.7	$Q_{(500)} = 569(A)^{0.805}$	43.2	23.2
Northern Piedmont (NP) – 19 Sites			**Central Valley and Ridge (CV) – 34 Sites**		
$Q_{(2)} = 179(A)^{0.655}$	51.1	1.6	$Q_{(2)} = 89 (A)^{0.788}$	31.0	4.8
$Q_{(5)} = 317(A)^{0.644}$	49.3	3.3	$Q_{(5)} = 222(A)^{0.712}$	29.3	8.7
$Q_{(10)} = 438(A)^{0.641}$	50.2	4.9	$Q_{(10)} = 372(A)^{0.668}$	28.6	12.9
$Q_{(25)} = 626(A)^{0.640}$	53.8	6.7	$Q_{(25)} = 647(A)^{0.620}$	29.5	17.5
$Q_{(50)} = 793(A)^{0.640}$	58.0	7.7	$Q_{(50)} = 918(A)^{0.591}$	31.4	19.4
$Q_{(100)} = 983(A)^{0.641}$	63.5	8.2	$Q_{(100)} = 1{,}254(A)^{0.565}$	34.1	20.2
$Q_{(200)} = 1{,}200(A)^{0.643}$	70.1	8.5	$Q_{(200)} = 1{,}665(A)^{0.542}$	37.4	20.2
$Q_{(500)} = 1{,}535(A)^{0.646}$	80.4	8.6	$Q_{(500)} = 2{,}354(A)^{0.514}$	42.6	19.5
Southern Piedmont (SP) – 67 Sites			**Southern Valley and Ridge (SV) – 35 Sites**		
$Q_{(2)} = 122(A)^{0.635}$	40.2	2.8	$Q_{(2)} = 46(A)^{0.880}$	45.0	1.7
$Q_{(5)} = 233(A)^{0.610}$	38.7	5.4	$Q_{(5)} = 90(A)^{0.825}$	43.4	2.6
$Q_{(10)} = 335(A)^{0.596}$	38.5	8.0	$Q_{(10)} = 127(A)^{0.800}$	44.2	3.3
$Q_{(25)} = 504(A)^{0.581}$	40.8	10.9	$Q_{(25)} = 181(A)^{0.774}$	46.6	4.2
$Q_{(50)} = 661(A)^{0.570}$	43.8	12.3	$Q_{(50)} = 228(A)^{0.759}$	49.1	4.7
$Q_{(100)} = 849(A)^{0.559}$	47.7	13.2	$Q_{(100)} = 281(A)^{0.745}$	52.0	5.2
$Q_{(200)} = 1{,}070(A)^{0.549}$	52.2	13.7	$Q_{(200)} = 339(A)^{0.733}$	55.3	5.5
$Q_{(500)} = 1{,}418(A)^{0.538}$	59.0	13.9	$Q_{(500)} = 425(A)^{0.718}$	60.2	5.7
Blue Ridge (B) – 54 Sites			**Appalachian Plateaus (AP) – 17 Sites**		
$Q_{(2)} = 95(A)^{0.760}$	33.4	4.0	$Q_{(2)} = 93(A)^{0.840}$	32.7	3.7
$Q_{(5)} = 201(A)^{0.726}$	34.1	6.5	$Q_{(5)} = 162(A)^{0.828}$	19.9	14.0
$Q_{(10)} = 298(A)^{0.710}$	35.5	8.8	$Q_{(10)} = 230(A)^{0.809}$	17.8	24.3
$Q_{(25)} = 450(A)^{0.695}$	38.8	11.0	$Q_{(25)} = 341(A)^{0.784}$	20.7	27.5
$Q_{(50)} = 584(A)^{0.687}$	42.2	12.0	$Q_{(50)} = 441(A)^{0.767}$	24.0	26.5
$Q_{(100)} = 735(A)^{0.680}$	46.2	12.5	$Q_{(100)} = 557(A)^{0.751}$	27.8	25.2
$Q_{(200)} = 907(A)^{0.674}$	50.7	12.6	$Q_{(200)} = 691(A)^{0.736}$	31.4	24.2
$Q_{(500)} = 1{,}165(A)^{0.667}$	56.7	12.8	$Q_{(500)} = 902(A)^{0.717}$	36.3	23.1

A=Drainage area, square miles (sq. mi.) Peak-discharge regions are shown in Figure 6-4.

6.4.4.4.5 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.

6.4.4.5 Urban Regression Method
6.4.4.5.1 Introduction
Regression equations developed by the USGS (Sauer et al., 1983) as part of a nationwide project can be used to estimate peak runoff for urban watershed conditions.

Sauer et al. (1983) provide two sets of seven-parameter equations and a third set based on three parameters. The three-parameter equations display almost as high a degree of reliability as the seven-parameter equations and are easier to use. Therefore, only the three-parameter equations will be discussed in this manual. The equations account for regional runoff variations through the use of the equivalent rural peak runoff rate (RQ). The equations adjust RQ to an urban condition using the basin development factor (BDF) and the percentage of impervious area (IA).

6.4.4.5.2 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, predominately urban in nature.

6.4.4.5.3 Characteristics
The basin development factor (BDF) is an index of the prevalence of urban drainage characteristics. The BDF can range from 0-12. A value of zero (0) for BDF indicates that urban drainage characteristics are not prevalent, but does not necessarily mean the basin is non-urban. A value of 12 indicates full development throughout the basin. Before a BDF factor is assigned, the drainage area is subdivided into three sections as shown in Figure 6-5. Each section is then assigned a code of either zero (0) or one (1) based upon the following four characteristics: channel improvement, channel lining, storm drain, and curb and gutter. The total of the 12 codes provides the BDF factor.

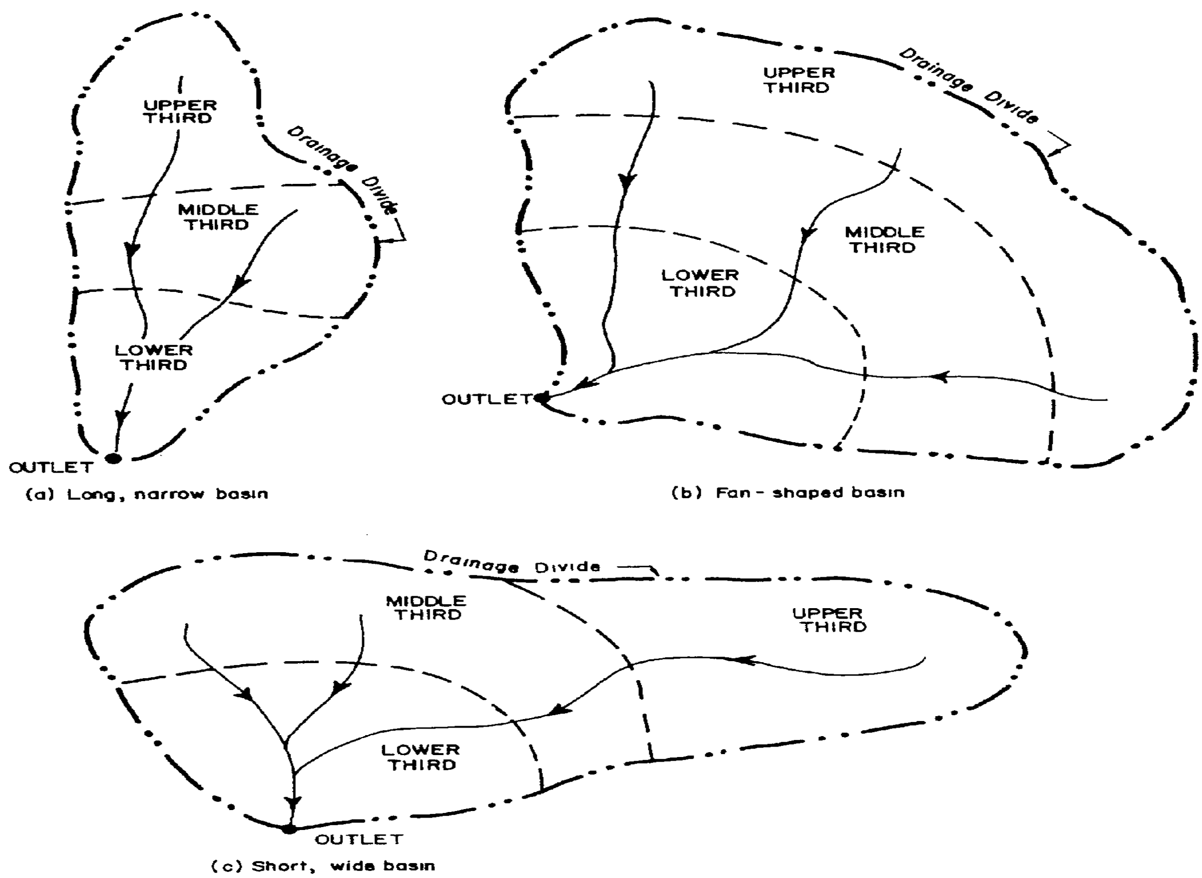


Figure 6-5. Basin Subdivisions

Channel Improvement

If channel improvements such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channels and principal tributaries (those that drain directly into the main channel), then a BDF code of one (1) is assigned. To be considered prevalent, at least 50 percent of the main drainage channels and principal tributaries must be improved to some degree over natural conditions. If channel improvements are not prevalent, then a BDF code of zero (0) is assigned.

Channel Linings

If more than 50 percent of the length of the main drainage and principal tributaries has been lined with an impervious material, such as concrete, then a BDF code of one (1) is assigned to this aspect. If less than 50 percent of these channels are lined, then a BDF code of zero (0) is assigned. The presence of channel linings is a good indication that channel improvements have been performed and signifies a more highly developed drainage system.

Storm Drain

Storm drains are enclosed drainage structures (usually pipes) frequently used on the secondary tributaries which receive drainage directly from streets or parking lots. Many of these drains empty into open channels; in some basins, however, they empty into channels enclosed as box or pipe culverts. When more than 50 percent of the secondary tributaries within a sub-basin consists of storm drains, a BDF code of one (1) is assigned to this aspect; if less than 50 percent, then a BDF code of zero (0). Note that if 50 percent or more of the main drainage channels and principal tributaries are enclosed, the aspects of channel improvements and channel linings would also be assigned a BDF code of one (1).

Curb and Gutter

If more than 50 percent of a sub-basin is urbanized, and if more than 50 percent of the streets and highways in the sub-basin are constructed with curbs and gutters, then a BDF code of one (1) would be assigned to this aspect. Otherwise, it would receive a BDF code of zero (0). Drainage from curb and gutter streets frequently empties into storm drains.

6.4.4.5.4 Equations

The nationwide equations for urban conditions take the following general form:

$$UQ_T = CA^{b1}(13\text{-}BDF)^{b2}RQ_T^{\;b3} \qquad (6.8)$$

Where:

- UQ_T = Peak discharge for the urban watershed for recurrence interval T, cubic feet per second (cfs)
- C = Regression constant (dimensionless)
- A = Contributing drainage area, square miles (sq. mi.)
- BDF = Basin development factor (dimensionless)
- RQ_T = Peak discharge for an equivalent rural drainage basin in the same hydrologic area as the urban basin and for recurrence interval T, cubic feet per second (cfs)
- b1, b2, b3= Regression exponents

Table 6-6 lists the 3-parameter equations for various storm frequencies.

Table 6-6. 3-Parameter Urban Equations

$UQ_2 = 13.2A^{0.21} (13\text{-}BDF)^{-0.43} RQ_2^{\ 0.73}$
$UQ_5 = 10.6A^{0.17} (13\text{-}BDF)^{-0.39} RQ_5^{\ 0.77}$
$UQ_{10} = 9.51A^{0.16} (13\text{-}BDF)^{-0.36} RQ_{10}^{\ 0.79}$
$UQ_{25} = 8.68A^{0.15} (13\text{-}BDF)^{-0.34} RQ_{25}^{\ 0.80}$
$UQ_{50} = 8.04A^{0.15} (13\text{-}BDF)^{-0.32} RQ_{50}^{\ 0.81}$
$UQ_{100} = 7.70A^{0.15} (13\text{-}BDF)^{-0.32} RQ_{100}^{\ 0.82}$
$UQ_{500} = 7.47A^{0.16} (13\text{-}BDF)^{-0.32} RQ_{500}^{\ 0.83}$

6.4.4.6 Analysis of Stream Gage Data

6.4.4.6.1 Introduction

Many gaging 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 nearby or representative of a watershed with similar hydrologic characteristics, transposition of frequency discharges is possible.
- If the flood-frequency curve is from one of a group of several gaging stations comprising a hydrologic region, then regional regression relations may be derived. Regional regression relations 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.

6.4.4.6.2 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 should be used for analysis and the Gumbel graphical method used as a check to ensure errors are not made – especially in the estimation of larger floods. Where serious discrepancies (20%+) are encountered in the findings between the two methods, special studies may be required. These special studies should consist of comparison with regression equations, application of other flood-frequency methods, and the collection and analysis of historical data. Outliers should be examined using the procedure found in Water Resources Council Bulletin 17B.

Gage data may be obtained from various publications including “Annual Maximum Stages And Discharges of Selected Streams In Virginia”, prepared by the U.S.

Geological Survey. For additional information contact: Chief, Virginia District U.S. Geological Survey, Richmond, Virginia, phone (804) 261-2639. Historical NWIS-W gage data for Virginia (currently over 550 gaging locations) can also be obtained from the USGS world wide web site: <http://water.usgs.gov/nwis>

Copies of reports may also be purchased from: U.S. Geological Survey, Books and Open – File Reports Section, Federal Center, Box 25425 Denver, Colorado 80225.

6.4.4.6.3 Skews

Skewness is a measure of asymmetry or lop-sidedness of a statistical distribution. The skew coefficient is defined as the skewness divided by the cube of the standard deviation. Skew coefficients play an integral role in the Log-Pearson analysis.

There are two alternative methods for determining the value of the skew coefficient to be used in calculating the Log-Pearson curve fit. The value of skew that is calculated directly from the gage data is called the station skew. This value may not be a true representation of the actual skew of the data if the period of record is short or if there are extreme events in the period of record. WRC Bulletin 17B contains a map of generalized skew coefficients of the logarithms of annual maximum streamflows throughout the United States and average skew coefficients by one degree quadrangles over most of the country.

Often, the station skew and the generalized skew can be combined to provide a better estimate for a given sample of flood data. Bulletin 17B outlines a procedure for combining the station skew and the generalized skew to provide a weighted skew.

6.4.4.6.4 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 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. The following example uses an exponent of 0.8.

Table 6-7. Transposition of Data Sample Problem

Watershed	Q_{25}, cfs	Area, sq. mi
Gaged Watershed A	62,000	737.0
Gaged Watershed B	38,000	734.0
Gaged Watershed C	45,000	971.0
Ungaged Watershed D	Find Q_{25}	449.8

Adjust Q_{25} for each subshed by area ratio:

$$\text{A: } 62{,}000\left(\frac{449.8}{737.0}\right)^{0.8} = 41{,}766 \text{ cfs } (41{,}800 \text{ cfs})$$

$$\text{B: } 38{,}000\left(\frac{449.8}{734.0}\right)^{0.8} = 25{,}682 \text{ cfs } (25{,}700 \text{ cfs})$$

$$\text{C: } 45{,}000\left(\frac{449.8}{971.0}\right)^{0.8} = 24{,}314 \text{ cfs } (24{,}300 \text{ cfs})$$

Average the Q_{25} for subsheds A, B, and C to Obtain Q_{25} for subshed D:

$$\text{D: } Q_{25} = \frac{(41{,}766+25{,}682+24{,}314)}{3} = 30{,}587 \ (30{,}600 \text{ cfs})$$

6.4.5 Hydrograph Methods

6.4.5.1 Modified Rational Method

6.4.5.1.1 Introduction

The Modified Rational Method provides hydrographs for small drainage areas where the peak, Q, is normally calculated by the Rational Method.

6.4.5.1.2 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 acres.

6.4.5.1.3 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 found on the rainfall IDF curve for the selected duration and frequency.
- Hydrographs are normally calculated for durations of:
  1. t_c
  2. $1.5t_c$
  3. $2t_c$
  4. $3t_c$
- 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.

6.4.5.1.4 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.

6.4.5.1.5 Estimating the Critical Duration Storm

The Virginia Department of Conservation and Recreation (DCR) has developed a method to estimate the critical duration storm. This method should be used with the following notes:

- 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 the equation are listed in Appendices 6B-2 through 6B-18

6.4.5.1.6 Equations

The approximate length of the critical storm duration can be estimated by the following equation:

$$T_c = \sqrt{\frac{2CAa(b-\frac{t_c}{4})}{q_o}}-b \qquad (6.9)$$

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 6B-3 through 6B-18

6.4.5.2 SCS Unit Hydrograph

6.4.5.2.1 Introduction

Techniques developed by the former United States Department of Agriculture, Soil Conservation Service (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 SCS has recently been renamed the National Resources Conservation Service or NRCS. Because this method has been traditionally called the SCS method, this manual will continue to use this terminology. The SCS 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 SCS method, the direct runoff can be calculated for any storm, either real or synthetic, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess. Details of the methodology can be found in the SCS National Engineering Handbook, Section 4.

6.4.5.2.2 Application

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

6.4.5.2.3 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.

6.4.5.2.4 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 SCS derived the following empirical relationship between lag time and time of concentration:

$$L = 0.6t_c \tag{6.10}$$

6.4.5.2.5 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 SCS 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, the higher is the runoff potential. Runoff curve numbers are provided in Appendices 6H-3, 6H-4, and 6H-5. Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The SCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C and D). Soil type A has the highest infiltration and soil type D has the least amount of infiltration. Soil surveys are available from your local NRCS office or:

NRCS State Conservationist
Culpeper Building, Suite 209
1606 Santa Rosa Road
Richmond, VA 23229-5014
Phone: 804-287-1687
FAX: 804-287-1737

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.

6.4.5.2.6 Equations

The following discussion outlines the equations and basic concepts utilized in the SCS 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 affects 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 SCS method is based on a 24-hour storm event, which has a Type II (or Type III) time distribution. Virginia uses the Type II storm distribution (Appendix 6H-1), which is a typical time distribution, that the Soil Conservation Service (SCS) has prepared from rainfall records from the United States. Some areas in southeast Virginia use a Type III storm. To use this distribution it is necessary for the user to obtain the 24-hour rainfall value (refer to Chapter 11) for the frequency of the design storm desired. Then multiply this value by 24 to obtain the total 24-hour storm volume in inches.

Rainfall-Runoff Equation - A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. Data for land treatment measures, such as contouring and terracing, from experimental watersheds were included. The equation was developed mainly for small watersheds 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 SCS runoff equation is therefore a method of estimating direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$Q = \frac{\left(P\text{-}I_a\right)^2}{\left(P\text{-}I_a\right)} + S \qquad (6.11)$$

Where:

Q = Direct runoff, inches (in)
P = Precipitation, inches (in)
I_a = Initial abstractions, inches (in)

$$I_a = 0.2S \qquad (6.12)$$

S = Potential maximum retention after runoff begins, inches (in)

$$S = \frac{1000}{CN} - 10 \tag{6.13}$$

CN = SCS Runoff curve number

6.5 Design Procedures and Sample Problems

6.5.1 Documentation Requirements

These items establish a minimum requirement. Also, see Chapter 3, Documentation, Section 3.3.1. The following items used in the design or analysis should be included in the documentation file:

- Contributing watershed area size and identification of source (map name, etc.)
- Design frequency and decision for selection
- Hydrologic discharge and hydrograph estimating method and findings
- Flood frequency curves to include design, 100-year flood, discharge hydrograph and any historical floods
- Expected level of development in upstream watershed over the anticipated life of the facility (include sources of and basis for these development projections)

6.5.2 Peak Discharge Procedures and Sample Problems

6.5.2.1 Rational Method Procedure

The results of using the rational formula to estimate peak discharges is very sensitive to the parameters that are used. The designer must use good engineering judgment in estimating values that are used in the method. The following procedure should be used for the Rational Method:

Step 1: Gather background information such as topographic mapping, land use data, precipitation information, etc. and determine point of analysis

Step 2: Delineate drainage area and determine the various land use characteristics

Step 3: Determine areas of overland flow, channel flow, and pipe conveyance and check results to verify reasonable flow time and velocity

Step 4: Compute total time of concentration (t_c)

Step 5: Determine rainfall intensity, I, using the IDF curves in Appendices 6B-3 through 6B-18 or the regression constants.

Step 6: Determine runoff coefficient(s), C, from Appendix 6E-1

Step 7: Compute peak discharge ($Q_{p.}$)

6.5.2.1.1 Rational Method Sample Problem

Preliminary estimates of the maximum rate of runoff are needed at the inlet to a culvert for a 10-year and 100-year return period.

Step 1: Gather background information such as topographic mapping, land use data, precipitation information, etc. and determine point of analysis

Site Data: Richmond Area

Step 2: Delineate drainage area and determine the various land use characteristics

Drainage Area:
From a topographic map and field survey, the area of a drainage basin upstream from the point in question is found to be 90 acres.

Land Use:
From existing land use maps, land use for the drainage basin was estimated to be:

Residential (1/2 acre lots)	80% of total drainage area
Undeveloped (2% slope)	20% of total drainage area

Step 3: Determine areas of overland flow, channel flow, and pipe conveyance and check results to verify reasonable flow time and velocity

Overland Surface = Average Grass
Length of Overland Flow = 150 feet
Average Overland Slope = 2.0%
Length of Main Basin Channel = 2300 feet
Slope of Channel = 1.8%

Step 4: Compute total time of concentration (t_c)

Overland Flow:
A runoff coefficient (C) for the overland flow area is determined from Appendix 6E-1.

C = 0.30

Analyze overland flow (OLF) and channel flow (CF) time by all applicable methods.

Overland flow time by:

Appendix 6D-1 (Seelye) = 13 minutes
Appendix 6D-3 = 15 minutes

The two methods essentially agree, therefore, use OLF = 14 minutes (*average*)

Channel Flow:

Channel flow by:

Appendix 6D-5(Kirpich) = 14 minutes

Use CF = 14 minutes

Time of Concentration:
Total t_c = 14 minutes (OLF) + 14 minutes (CF) = 28 minutes

Step 5: Determine rainfall intensity (i) using the IDF curves in Appendices 6B-3 through 6B-18 or the regression constants

From Appendix 6B-12 (Richmond IDF curve) with duration equal to 28 minutes, determine rainfall intensity, (i), graphically or by using regression constants, and solving Equation 6.14.

$$i = \frac{a}{b+t_c} \qquad (6.14)$$

$$I_{10} \text{ (10-year return period)} = \frac{185.51}{21.13+28} = 3.8 \text{ in/hr}$$

$$I_{100} \text{ (100-year return period)} = \frac{278.85}{23.60+28} = 5.4 \text{ in/hr}$$

Step 6: Determine runoff coefficients (C) from Appendix 6E-1

The weighted runoff coefficient (C) for the total drainage area is computed.

Land Use	**Column 1 % of Total Land Area**	**Column 2 Runoff Coefficient**	**Column 3 Weighted Runoff Coefficient***
Residential (1/2 Acre lots)	0.80	0.35	0.28
Undeveloped	0.20	0.30	0.06
Total Weighted Runoff Coefficient			0.34

*Column 3 = Column1 x Column 2

Step 7: Compute peak discharge (Q_p)

$$Q = C_f CiA$$

Determine Coefficient of Saturation, C_f , from Table 6-2:

Q_{10}; C_f = 1.0
Q_{100}; C_f = 1.25

Check $C_f \times C \leq 1.0$:

Q_{10}; C x C_f = 0.34 x 1.0= 0.34 < 1.0, therefore, OK
Q_{100}; C x C_f = 0.34 x1.25 = 0.43 < 1.0, therefore, OK

Determine Discharges:

Q_{10} = 0.34 x 3.8 x 90 = 116 cfs
Q_{100} = 1.25 x 0.34 x 5.4 x 90 = 207 cfs

These are the estimates of peak runoff for a 10-year and 100-year design storm for the given drainage area.

6.5.2.2 Anderson Method Procedure

6.5.2.2.1 Anderson Method Sample Problem

Estimate the 25-year peak discharge on Rabbit Branch near Burke, Virginia, for an expected future development consisting of 40% impervious surface and a drainage system of storm sewers for tributaries but a natural main channel.

Step 1: From topographic maps, determine the following data

A = 3.81 sq. miles
L = 0.34 mile to 10% distance to rim
2.90 miles to 85% distance to rim
3.40 miles to rim

Step 2: Compute the average channel slope

Elevation = 282 feet at 10% distance to rim
395 feet at 85% distance to rim

$$\text{Slope} = \frac{(395\text{-}282)}{(2.90\text{-}0.34)} = \frac{113}{2.56} = 44.2 \text{ ft/mi}$$

Step 3: Compute time (T)

$$T = 0.9\left(\frac{L}{\sqrt{S}}\right)^{0.5} \text{ for Developed Basin, Partly Channeled} \qquad (6.15)$$

$$= 0.9\left(\frac{L}{\sqrt{S}}\right)^{0.5} = 0.9\left(\frac{3.4}{\sqrt{44.2}}\right)^{0.5} = 0.64 \text{ hrs}$$

Step 4: Compute K

$$K = 1.00 + 0.015\mathbf{I}, \text{ where } \mathbf{I} = 40\% \text{ for impervious surface}$$
$$= 1.00 + 0.015(40)$$
$$= 1.60$$

Step 5: Compute Flood Ratio (R) using Appendix 6F-1

From Appendix 6F-1, Flood Ratio, R Values, determine R = 2.34 for 40% impervious surface and a 25-year recurrence interval.

Step 6: Compute peak discharge (Q)

$$Q = R(230)KA^{0.82}T^{-0.48}$$

$$Q_{25} = 2.34(230)(1.60)(3.81)^{0.82}(0.64)^{-0.48}$$
$$= 3195 \text{ cfs (Say 3200 cfs)}$$

6.5.2.3 Snyder Method Procedure

The procedure to develop peak discharges using the Snyder Method is provided below.

Step 1: Obtain the following data from topographic maps.

DA = Watershed area, square miles
L = Length of principal channel, miles
S = Weighted slope of channel (mean height in feet of channel profile divided by L ÷ 2), percent
N = Friction factor, Manning's "n"
Mean height = (elevation at upper end of channel – elevation of lower end of channel) ÷ 2

% of watershed sewered
% of natural channel eliminated
% of impervious area

Step 2: Compute T_c

$$T_c = C_t L'^{0.6} \qquad (6.16)$$

Where:

C_t = Adjustment factor defined by the development condition of the drainage area

C_t is defined in Table 6-8. This table is for natural basins, overland flow, and completely sewered areas. If the drainage area is partially sewered, determine C_t from Table 6-8 and then compute Adj. C_t from Equation 6.17.

Partially Sewered Area Adjustment:

$$\text{Adj } C_t = 1.7 - \frac{(\%\text{ Sewered} + \%\text{ Nat. Chan. Elim.})(1.7-0.42)}{200} \qquad (6.17)$$

Table 6-8. C_t Values for the Snyder Method

Type of Areas	C_t, Hours/mile
Natural Basins	1.7
Overland Flow	0.85
Sewered Areas	0.42

L' = Equivalent length of channel with slope of 1% and friction factor equal to 0.1 and is defined by Equation 6.18.

$$L' = \frac{10Ln}{\sqrt{S}} \qquad (6.18)$$

Step 3: Compute peak discharge, (Q_p) using the following steps:

Step 3a: Determine storm frequency (years)

Step 3b: Using Appendix 6G-4 through 6G-12, determine precipitation (inches) for each frequency

Step 3c: Using Appendix 6G-1, determine percent natural runoff for each frequency

Step 3d: Using Appendix 6G-2, determine percent runoff adjusted for impervious area

Step 3e: Compute I_R

$$I_R = \left[\frac{\text{Runoff}}{T_c}\right] \tag{6.19}$$

Step 3f: Compute Q_p

$$Q_p = 500AI_R$$

6.5.2.3.1 Snyder Method Sample Problem

Find the 10- and 100-yr peak discharges for a tributary draining into the James River near Richmond, VA. The tributary is primarily undeveloped; however, several developments within the drainage basin have storm drains.

Step 1: Obtain the following data from topographic maps.

Site: A tributary of the James River near Richmond, VA.

DA = 4.11 sq. mi.
L = 2.75 mi.
S = 53 ft./mi.=l.0%
n = 0.045.

DA is 22% impervious
10% Natural Channel Eliminated
30% Storm Sewered

Step 2: Compute T_c

$$T_c = C_t L'^{0.6}$$

$$L' = \frac{10Ln}{\sqrt{S}} \qquad \text{Adj } C_t = 1.7 - \frac{(\%\,\text{Sewered} + \%\,\text{Nat. Chan. Elim.})(1.7\text{-}0.42)}{200}$$

Substitute Equations 6.17 and 6.18 into Equation 6.16.

$$T_c = \left[1.7 - \frac{(\%\,\text{Sewered} + \%\,\text{Nat. Chan. Elim.})(1.7\text{-}0.42)}{200}\right]\left[\frac{10Ln}{\sqrt{S}}\right]^{0.6}$$

$$= \left[1.7 - \frac{(30+10)(1.7\text{-}0.42)}{200}\right]\left[\frac{10(2.75)(0.045)}{\sqrt{1.0}}\right]^{0.6}$$

$$= 1.64 \text{ hrs}$$

Step 3: Compute peak discharge (Q_p) using the following steps:

The results of Steps 3a through 3g are summarized in Step 3g.

Step 3a: Determine storm frequency (years)

Step 3b: Using Appendix 6G-8 (Richmond), determine precipitation (inches) for each frequency

Step 3c: Using Appendix 6G-1, determine percent natural runoff for each frequency

Step 3d: Using Appendix 6G-2, determine percent runoff adjusted for impervious area

Step 3e: Compute I_R

$$I_R = \left[\frac{\text{Runoff}}{T_c}\right]$$

Step 3g: Compute Q_p

$$Q_p = 500AI_R$$

Frequency (yrs.)	10	100
Rainfall (inches) (Appendix 6G-8)	2.7	4.0
Percent Runoff Natural (%) (Appendix . 6G-1)	31	42
Percent Runoff Adjusted (%) (Appendix 6G-2)	46	54
Runoff (Runoff Adj. x Rainfall)	1.24	2.16
I_R = [Runoff ÷ T_c] (in/hr)	0.76	1.32
Q_p = 500 AI_R, (cfs)	**1562 (1560 cfs)**	**2713 (2710 cfs)**

6.5.2.4 Rural Regression Method Procedure

The rural regression method procedure is presented in the urban regression method procedure and sample problem discussed in Section 6.5.2.5.

6.5.2.5 Urban Regression Method Procedure

This procedure is not intended to require precise measurements. A certain amount of subjectivity is involved, and field checking should be performed to obtain the best estimate. The BDF is the sum of four assigned codes; therefore, with three sub-basins per basin, and four drainage characteristics to which codes are assigned in each sub-basin, the maximum value for a fully developed drainage system would be 12. Conversely, a totally undeveloped drainage system would receive a BDF of zero (0). This rating does not necessarily mean that the basin is unaffected by urbanization. In fact, a basin could be partially urbanized, have some impervious area, and have some improvement of secondary tributaries, and still have an assigned BDF of zero (0). The following steps are used to apply the nationwide equations:

Step 1: Determine the following input parameters: drainage area (A), main channel slope (SL), rainfall intensity (RI2), basin storage (ST), and impervious area (IA).

Step 2: Using the USGS regression equations, estimate peak runoff for natural flow conditions for an equivalent rural drainage basin (RQ_T) for a specific design frequency. Depending on the amount of data available, use Tables 6-4 and 6-5.

Step 3: Determine the basin development factor (BDF)

Step 4: Calculate peak runoff rates for desired return periods using the urban regression equations given in Table 6-6

6.5.2.5.1 Urban Regression Method Sample Problem

Determine the peak discharge for the 10- and 100-yr design storms using the Urban Regression equations for a drainage area of one (1) square mile in northern Spotsylvania County, VA. The main channel slope is 52 ft/mi and the channel length is 1.3 miles. The current land use is a mixture of rural cultivation and woods.

A plan is offered that will provide single family and commercial development within portions of this watershed.

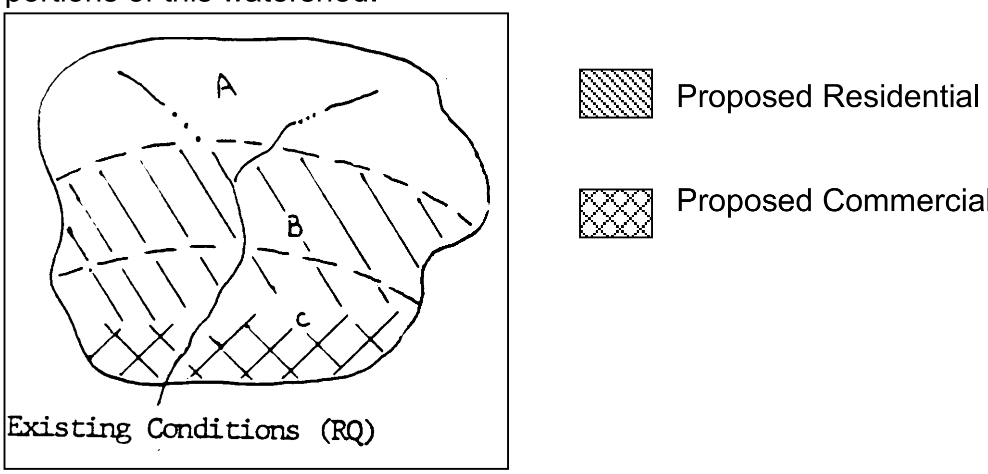


Figure 6-6. Basin Sketch

Step 1: Determine the following input parameters: drainage area (A), main channel slope (SL), rainfall intensity (RI2), basin storage (ST), and impervious area (IA).

Step 2: Using the USGS regression equations, estimate peak runoff for natural flow conditions for an equivalent rural drainage basin (RQ_T) for a specific design frequency.

From Table 6-5, obtain the USGS Rural Regression Equations for the Northern Piedmont and compute the 10-yr and 100-yr peak discharges for the existing conditions.

$Q_{10} = 438\ (1^{.641}) = 438$ cfs
$Q_{100} = 984\ (1^{.641}) = 984$ cfs

Step 3: Determine the basin development factor (BDF) by subdividing the drainage area into thirds, illustrated in Figure 6-6, and assign a BDF code (0 or 1) based on the presence of four characteristics of the drainage system within each third.

The drainage area is subdivided into thirds as shown in Figure 6-6.

Proposed Conditions BDF:

	A	B	C
Storm drains	0	0	1
Channel improvements	0	0	1
Impervious channel linings	0	1	1
Curb & gutter	0	1	1
	0	2	4

Total BDF =6

Step 4: Calculate peak runoff rates for desired return periods using the urban regression equations given in Table 6-6

$UQ_{10} = 9.51(1)^{0.16}\ (13\text{-}6)^{-0.36}\ (438)^{0.79} = 576$ cfs
$UQ_{100} = 7.7(1)^{0.15}\ (13\text{-}6)^{-0.32}\ (984)^{0.82} = 1{,}176$ cfs

6.5.2.6 Analysis of Stream Gage Data Procedure

6.5.2.6.1 Statistical Method for Analyzing Stream Gage Data

The log-Pearson Type III distribution is the recommended statistical method. This method is defined by three standard statistical parameters: the mean, standard deviation and coefficient of skew. These parameters are determined from the data sample, which normally consists of the peak annual flows for a period of record. Formulas for the computation of these parameters are given below:

$$Q = \frac{(\sum X)}{N} \text{ (mean of logs)} \qquad (6.20)$$

Where:

N = Number of observations and X is the logarithm of the annual peak

The standard deviation of logs is:

$$S_L = \left(\frac{\left[\Sigma X^2 - \frac{(\Sigma X)^2}{N} \right]}{[N-1]} \right)^{\frac{1}{2}} \tag{6.21}$$

The coefficient of skew of logs is:

$$G = \frac{\left[N^2 \left(\Sigma X^3 \right) - 3N(\Sigma X)\left(\Sigma X^2 \right) + 2(\Sigma X)^3 \right]}{\left[N(N-1)(N-2) S_L{}^3 \right]} \tag{6.22}$$

Using these three parameters, the magnitude of the flood of the desired frequency can be determined from the equation.

$$\log Q = Q_L + KS_L \tag{6.23}$$

Where:

- $\log Q$ = Logarithm of the flood magnitude
- Q_L = Mean of the logarithms of the peak annual floods
- K = Frequency factor for a particular return period and coefficient of skew (Values of K for different coefficients of skew and return periods are given in WRC Bulletin 17B)
- S_L = Standard deviation of the logarithms of the peak annual flood

If a flood frequency curve is necessary, then by computing several values of Q for different return periods, the log-Pearson fit to the data can be plotted on standard log probability paper. If the skew of the sample data happens to be equal to zero, the plot of the log-Pearson fit to the data will be a straight line. If the skew is negative the plot will be a curve with a downward concavity. If the skew is positive, the plot will be a curve with upward concavity.

6.5.3 Hydrograph Procedures and Sample Problems

6.5.3.1 Modified Rational Method Hydrograph Procedure

Use of the Rational Method for calculation of peak Q is based upon the assumption that the highest Q from a small drainage area occurs when the duration (D_e) of the storm is equal to the time of concentration (t_c) or $D_e = t_c$.

Step 1: Obtain all of the basic background information that is required for the Rational Method. Refer to section 6.5.2.1, Rational Method procedures

Step 2: Compute the time of concentration (t_c) as described in the Rational Method

Step 3: Develop a range of hydrographs for various durations (D_e):

a. t_c
b. $1.5t_c$
c. $2t_c$
d. $3t_c$

Note: 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.

Step 4: Determine storm duration (D_e) for each scenario

Step 5: Compute Q_p, T_b, and D_e for each scenario

6.5.3.1.1 Modified Rational Method Hydrograph Sample Problem

For a 5 acre drainage site with a post development C-value of 0.7 and a time of concentration of 20 min, develop hydrographs (HYG) for a 2-year frequency storm in the Richmond area.

Step 1: Obtain all of the basic background information that is required for the Rational Method

DA = 5 acres
C = 0.7 (after development)

Step 2: Compute the time of concentration (t_c) as described in the Rational Method

Time of concentration (t_c) = 20 minutes

Step 3: Develop a range of hydrographs for various durations (D_e)

a. t_c
b. $1.5t_c$
c. $2t_c$
d. $3t_c$

The results of steps 3 through 3b are summarized after Step 3b and shown graphically in Figure 6-7.

Step 3a: Determine storm duration (D_e) and time base (T_b) for each scenario

All hydrographs for this drainage area will have T_p = 20 minutes and also T_r = 20 minutes because t_c = 20 minutes for this drainage area.

Step 3b: Compute Q_p for each scenario

HYG #1 Duration (D_e) = 20 minutes
Intensity (I) = 3.3 in/hr (from IDF curve)
Peak Q = 0.7 x 3.3 x 5 = 11.6 or 12 cfs
T_b = 20(or D_e) + 20(or T_r) = 40 minutes

HYG #2 $D_e = 1.5 \times t_c = 1.5 \times 20 = 30$ minutes
Intensity (I) = 2.7 in/hr (from IDF curve for a 30 minute storm)
Peak Q = 0.7 x 2.7 x 5 = 9.5 or 10 cfs
T_b = 30(or D_e) + 20(or T_r) = 50 minutes

HYG #3 $D_e = 2 \times t_c = 2 \times 20 = 40$ minutes
Intensity (I) = 2.2 in/hr (from IDF curve for a 40 minute storm)
Peak Q = 0.7 x 2.2 x 5 = 7.7 or 8 cfs
T_b = 40(or D_e) + 20(or T_r) = 60 minutes

HYG #4 $D_e = 3 \times t_c = 3 \times 20 = 60$ minutes
Intensity (I) = 1.7 in/hr (from IDF curve for a 60 minute storm)
Peak Q = 0.7 x 1.7 x 5 = 6 cfs
T_b = 60(or D_e) + 20(or T_r) = 80 minutes

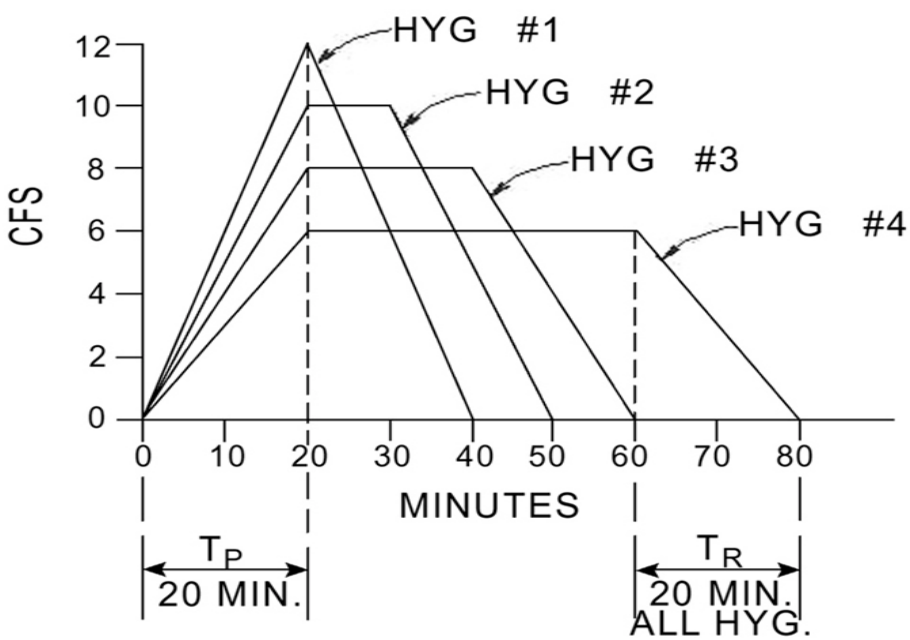


Figure 6-7. Example of Hydrographs for 2-year Frequency Storm Using the Modified Rational Method

6.6 References

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Appendix 6A-1 Definitions and Abbreviations

Abbreviations:

AASHTO	American Association of State Highway and Transportation Officials
BDF	Basin Development Factor
CF	Channel Flow
DCR	Department of Conservation and Recreation
FEMA	Federal Emergency Management Agency
HYG	Hydrograph
IDF	Intensity Duration Frequency
NEH	National Engineering Handbook
NRCS	National Resource Conservation Service; formerly Soil Conservation Service (SCS)
OLF	Overland Flow
TR	Technical Release
USGS	United States Geological Survey
VDOT	Virginia Department of Transportation

Appendix 6A-2 Symbols

Symbol	Definition	Units
A	Drainage Area	acres, sq. mi.
a	Rainfall regression constant	-
b	Rainfall regression constant	-
b1	Urban Regression Method exponent	-
b2	Urban Regression Method exponent	-
b3	Urban Regression Method exponent	-
BDF	Basin Development Factor	%
C	Runoff coefficient	-
C	Urban Regression Method constant	-
C_f	Frequency factor	-
CN	SCS-runoff curve number	-
C_t, C_p	Physiographic coefficients	-
D_m	Mean depth of lake or reservoir	ft
D_e	Storm duration	min
G	Coefficient of Skew	ft
g	Acceleration due to gravity	ft/s^2
i	Average Rainfall intensity	in/hr
IA	Percentage of impervious area	%
I_a	Initial abstraction from total rainfall	in
K	Statistical Method Frequency Factor	-
K	Anderson Method Coefficient of Imperviousness	-
L	Flow Length or Length of Strip	ft
L'	Equivalent length of channel	mi
L	Anderson Method Basin Length or Snyder Method Channel Length	mi
l	Length of mainstream to furthest divide	ft
L_{ca}	Length along main channel to a point opposite the watershed centroid	mi
M	Rank of a flood within a long record	-
m	Number of flow segments	-
n	Manning's roughness coefficient	-
N	Number of years of flood record	yrs
P	Precipitation	in
q	Storm runoff during a time interval	in
R	Flood frequency ratio	-
R	Hydraulic radius	ft
RI2	Rainfall intensity for the 2-hr, 2-year occurrence	in
RQ_T	Rural Regression for Return Period T Peak Discharge	cfs
RC	Regression constant	-
RQ	Equivalent rural peak runoff rate	cfs
S or Y	Ground slope	ft/ft or %
S	Anderson Method Index of Basin Slope	ft/mi
S	SCS Method Potential maximum retention storage	in

Appendix 6A-2 Symbols

S_o	Channel slope	ft/ft
SL	Urban Regression Method Main Channel Slope	ft/mi
S_L	Standard Deviation	-
ST	Basin storage factor	%
T_b	Time base of hydrograph	min or hrs
T_c	Time of concentration	min
T_c	Modified Critical Storm Duration	min
T_r	Time to Recede	min
T_t	Travel time	hrs
T_p	Time to Peak	min
T	Anderson Method Lag Time	hrs
Q_L	Mean of the logarithms of the peak annual floods	cfs
q_o	Allowable outflow rate	cfs
Q, Q_p	Maximum rate of runoff or Peak Discharge	cfs
Q	SCS Direct Runoff	in
Q	Statistical Method Mean of Logs	-
UQ	Urban Regression Method peak runoff rate	cfs
UQ_T	Peak runoff rate for Urban Watershed for Return Period T	cfs
V_w	Wave velocity	ft/s
V	Velocity or Average velocity	ft/s
X	Logarithm of the annual peak	-

Appendix 6B-1 A and B Factors that Define Intensity-Duration-Frequency (IDF) Curves for Use with the Rational Method and the Modified Rational Method

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 also used in Equation 6.9 that estimates the “Critical Storm Duration” (T_c).

Appendix 6B-2 Regression Constants a and b for Virginia

		2 YEAR		10 YEAR		100 YEAR	
COUNTY	**#**	**A**	**B**	**A**	**B**	**A**	**B**
Arlington	00	119.34	17.86	178.78	20.66	267.54	22.32
Accomack	01	107.75	14.69	175.90	20.64	277.44	24.82
Albemarle	02	106.02	15.51	161.60	18.73	244.82	20.81
Allegheny	03	95.47	13.98	145.89	17.27	220.94	19.29
Amelia	04	112.68	15.11	173.16	18.81	266.77	22.13
Amherst	05	106.72	15.39	162.75	18.83	245.52	21.02
Appomattox	06	109.11	15.39	167.44	19.12	254.03	21.61
Augusta	07	84.21	10.44	135.74	14.54	210.02	16.99
Bedford	09	114.59	17.21	171.51	20.47	258.17	22.80
Bland	10	105.33	16.56	162.75	20.41	247.84	22.87
Botetourt	11	110.32	16.95	164.94	20.01	247.92	22.16
Brunswick	12	126.74	17.27	190.73	21.52	287.02	24.46
Buchanan	13	87.14	13.22	128.51	15.15	189.98	16.22
Buckingham	14	109.95	15.41	168.28	19.11	254.59	21.47
Campbell	15	110.26	15.76	167.27	19.18	252.65	21.56
Caroline	16	121.21	17.33	182.56	20.88	275.65	23.30
Carroll	17	119.79	18.65	188.13	23.81	288.94	27.06
Charles City	18	124.23	17.14	186.52	21.05	281.04	23.85
Charlotte	19	109.87	14.71	171.75	19.25	265.18	22.56
Chesterfield	20	124.66	17.55	186.15	21.03	277.94	23.26
Clarke	21	94.13	12.88	141.03	15.39	210.66	16.85
Craig	22	106.67	16.54	166.19	20.94	251.27	22.95
Culpeper	23	111.90	16.25	169.78	19.51	255.26	21.52
Cumberland	24	111.34	15.29	172.73	19.29	271.55	24.02
Dickenson	25	87.03	13.10	128.09	14.82	190.08	15.98
Dinwiddie	26	125.08	17.29	189.77	21.51	284.68	24.02
Essex	28	119.70	16.76	180.50	20.18	271.79	22.58
Fairfax	29	117.06	17.34	178.32	20.49	269.23	22.40
Fauquier	30	116.55	17.52	172.47	20.02	255.06	21.38
Floyd	31	121.22	19.16	185.59	23.38	281.91	26.26

Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999.

Appendix 6B-2 Regression Constants a and b for Virginia

COUNTY	#	2 YEAR A	2 YEAR B	10 YEAR A	10 YEAR B	100 YEAR A	100 YEAR B
Frederick	34	93.79	13.15	141.02	15.77	211.40	17.42
Giles	35	106.14	16.72	165.04	20.80	252.79	23.46
Gloucester	36	119.62	16.09	182.54	20.40	276.43	23.35
Goochland	37	114.42	15.95	177.24	19.93	269.07	22.27
Grayson	38	119.29	18.94	176.02	22.06	262.24	24.25
Green	39	105.71	15.10	159.92	18.20	241.18	20.34
Greensville	40	129.97	17.80	194.08	22.01	291.37	24.83
Halifax	41	111.92	15.14	173.81	19.52	267.09	22.70
Hanover	42	122.80	17.29	185.01	20.91	278.40	23.40
Henrico	43	123.51	17.35	185.51	21.13	277.61	23.44
Henry	44	116.19	17.33	177.84	21.34	270.32	24.01
Highland	45	90.13	12.61	134.38	15.02	199.74	16.50
Isle of Wight	46	125.69	17.02	190.34	21.71	287.14	24.73
James City	47	121.86	16.58	185.06	20.81	279.14	23.67
King George	48	120.31	17.28	181.05	20.50	273.29	22.83
King & Queen	49	113.84	15.29	179.09	19.95	275.98	23.15
King William	50	114.92	15.58	180.36	20.13	277.03	23.26
Lancaster	51	109.80	14.49	170.27	18.72	259.78	21.41
Lee	52	93.78	14.40	143.28	17.58	215.10	19.22
Loudoun	53	104.05	14.91	157.67	17.71	237.83	19.65
Louisa	54	112.63	15.89	174.35	19.72	265.20	22.11
Lunenberg	55	122.01	16.82	184.70	20.80	278.38	23.48
Madison	56	106.87	15.33	161.43	18.49	242.78	20.62
Mathews	57	118.61	15.83	180.56	20.17	274.12	23.29
Mecklenberg	58	121.77	16.55	184.54	20.74	278.33	23.48
Middlesex	59	110.72	14.57	172.76	19.15	264.49	22.13
Montgomery	60	118.78	19.21	176.95	22.39	262.93	24.17
Nelson	62	103.46	14.52	160.23	18.36	245.04	20.89
New Kent	63	121.03	16.58	183.93	20.72	277.89	23.51
Norfolk	64	124.88	17.02	190.64	22.14	288.73	25.60

Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999.

Appendix 6B-2 Regression Constants a and b for Virginia

COUNTY	#	2 YEAR A	2 YEAR B	10 YEAR A	10 YEAR B	100 YEAR A	100 YEAR B
Northampton	65	111.07	14.78	173.72	19.63	267.48	23.04
Northumberland	66	111.20	14.99	171.55	19.00	260.59	21.63
Nottoway	67	122.38	17.06	183.97	20.87	275.78	23.19
Orange	68	116.77	16.63	178.14	20.19	270.55	22.72
Page	69	84.19	10.29	135.43	14.29	209.57	16.86
Patrick	70	123.68	19.26	189.08	23.60	284.78	26.12
Powhatan	72	114.14	15.64	175.93	19.65	266.86	22.15
Pittsylvania	71	112.30	16.02	173.58	20.27	263.51	22.98
Prince Edward	73	111.01	15.06	172.73	19.29	264.28	22.20
Prince George	74	126.22	17.46	188.62	21.39	283.12	24.09
Virginia Beach	75	129.20	17.84	196.25	22.74	294.74	26.33
Prince William	76	116.04	17.08	176.18	20.19	266.75	22.36
Pulaski	77	117.44	18.71	182.33	23.39	279.39	26.49
Rappahannock	78	104.86	15.05	159.40	18.34	239.30	20.19
Richmond	79	117.41	16.23	177.35	19.85	267.20	22.24
Roanoke	80	117.53	18.79	174.97	21.80	261.95	23.81
Rockbridge	81	84.23	10.46	143.41	15.89	229.43	19.56
Rockingham	82	83.83	10.55	128.80	13.37	195.24	15.29
Russell	83	92.64	14.17	143.00	17.32	216.40	19.36
Scott	84	92.64	14.17	143.00	17.32	216.40	19.36
Smyth	86	106.19	16.57	169.30	21.37	262.49	24.57
Southampton	87	129.91	17.77	195.84	22.34	294.40	25.43
Spotsylvania	88	117.31	16.86	179.21	20.48	269.84	22.55
Stafford	89	118.72	17.34	179.62	20.64	270.74	22.79
Surry	90	124.79	16.97	188.62	21.39	283.36	24.16
Sussex	91	130.37	18.03	193.23	21.91	287.99	24.56
Tazewell	92	91.25	13.56	141.61	17.04	217.59	19.48
Warren	93	89.03	11.53	137.69	14.73	210.46	16.87
Washington	95	106.65	16.86	162.19	20.02	244.60	21.98
Westmoreland	96	114.40	15.76	174.96	19.47	266.16	22.12

Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999.

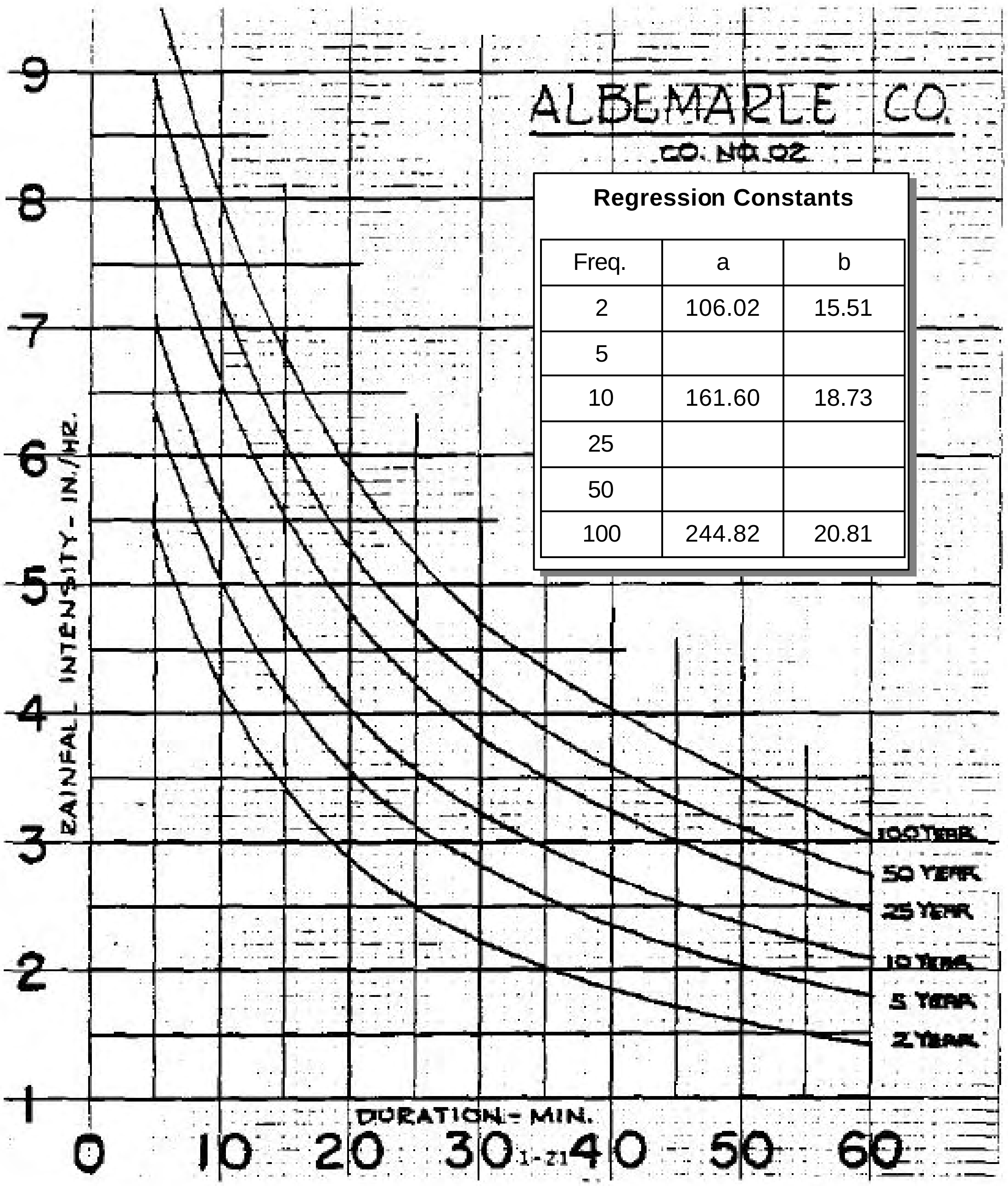
Appendix 6B-2 Regression Constants a and b for Virginia

COUNTY	#	2 YEAR A	2 YEAR B	10 YEAR A	10 YEAR B	100 YEAR A	100 YEAR B
Wise	97	89.83	13.49	132.05	15.44	194.10	16.35
Wythe	98	116.78	18.83	174.91	22.13	261.68	24.25
York	99	122.93	16.72	186.78	21.22	282.80	24.39

CITIES	#'s	2 YEAR A	2 YEAR B	10 YEAR A	10 YEAR B	100 YEAR A	100 YEAR B
Richmond	127/43	122.47	17.10	185.51	21.13	278.85	23.60
Hampton	114/27	123.93	16.94	186.78	21.22	283.18	24.56
Lynchburg	118/15	107.39	15.15	166.87	19.37	255.02	22.08
Suffolk	133/61	129.97	17.80	196.63	22.61	298.69	26.35
Newport News	121/94	126.11	17.37	189.27	21.62	285.24	24.71

Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999.

Appendix 6B-3 Rainfall Intensity Inches Per Hour
Albemarle, Co.

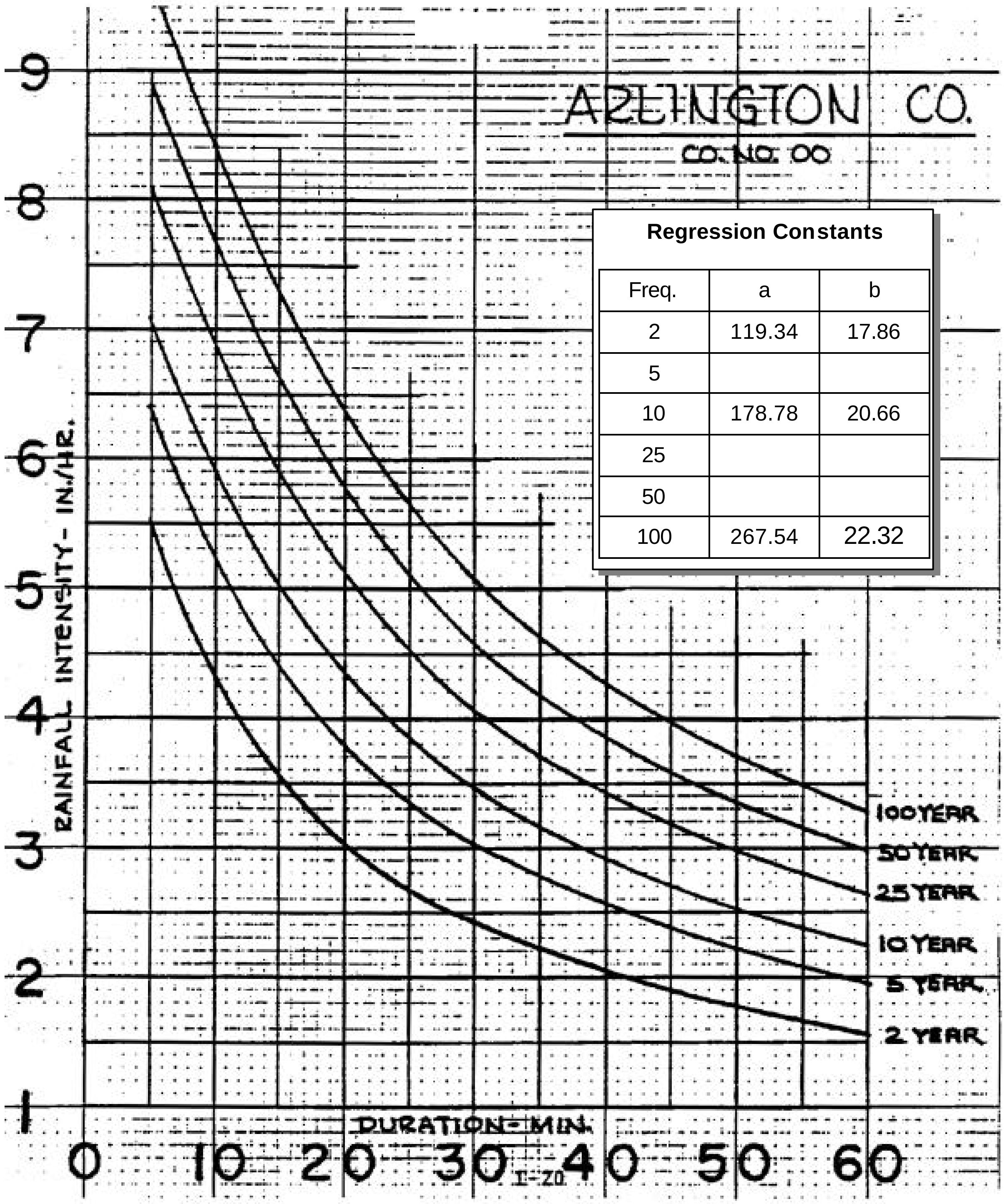


Regression Constants

Freq.	a	b
2	106.02	15.51
5		
10	161.60	18.73
25		
50		
100	244.82	20.81

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6B-4 Rainfall Intensity Inches Per Hour Arlington Co.

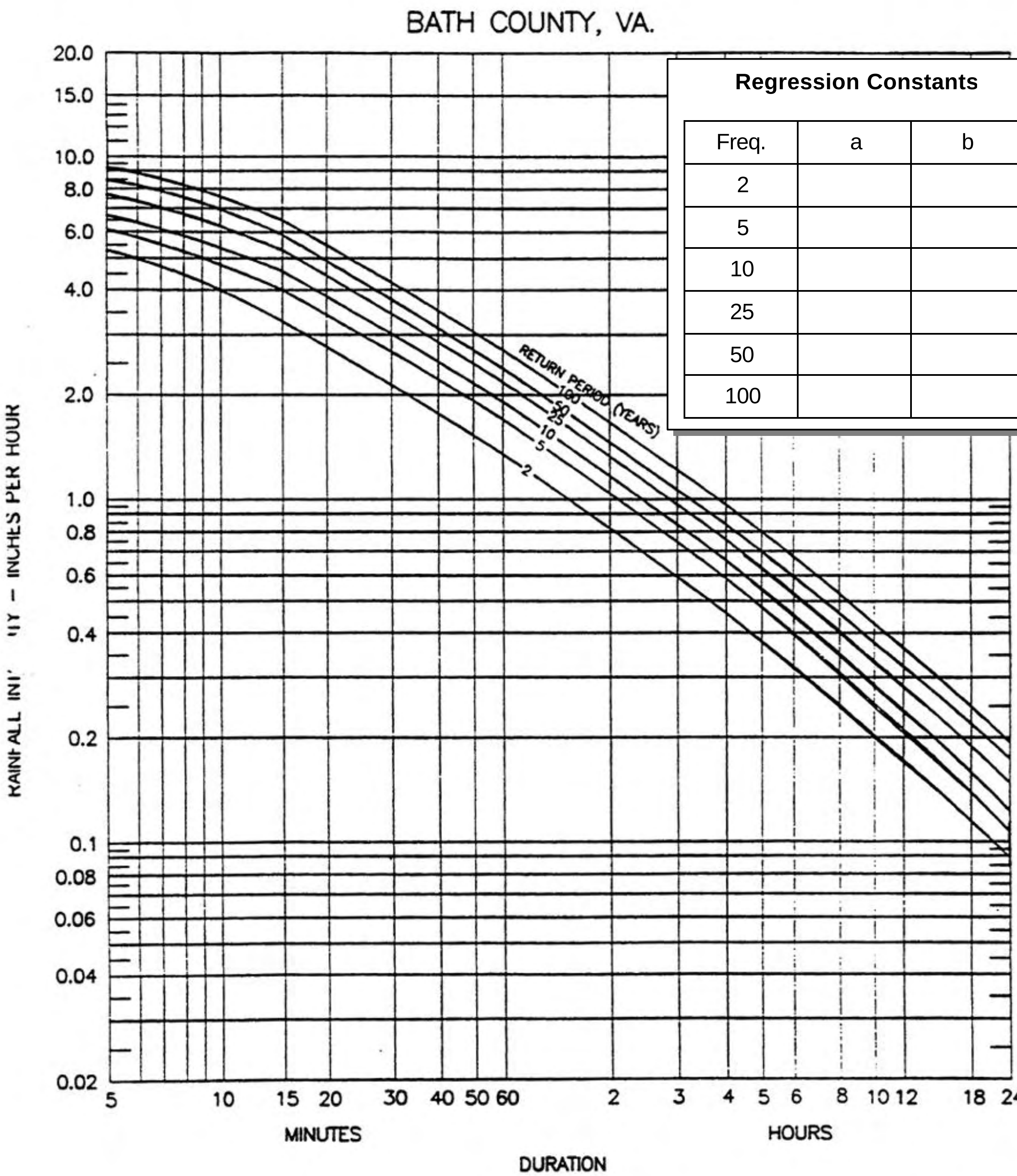


Regression Constants

Freq.	a	b
2	119.34	17.86
5		
10	178.78	20.66
25		
50		
100	267.54	22.32

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6B-5 Rainfall Intensity Inches Per Hour Bath Co.



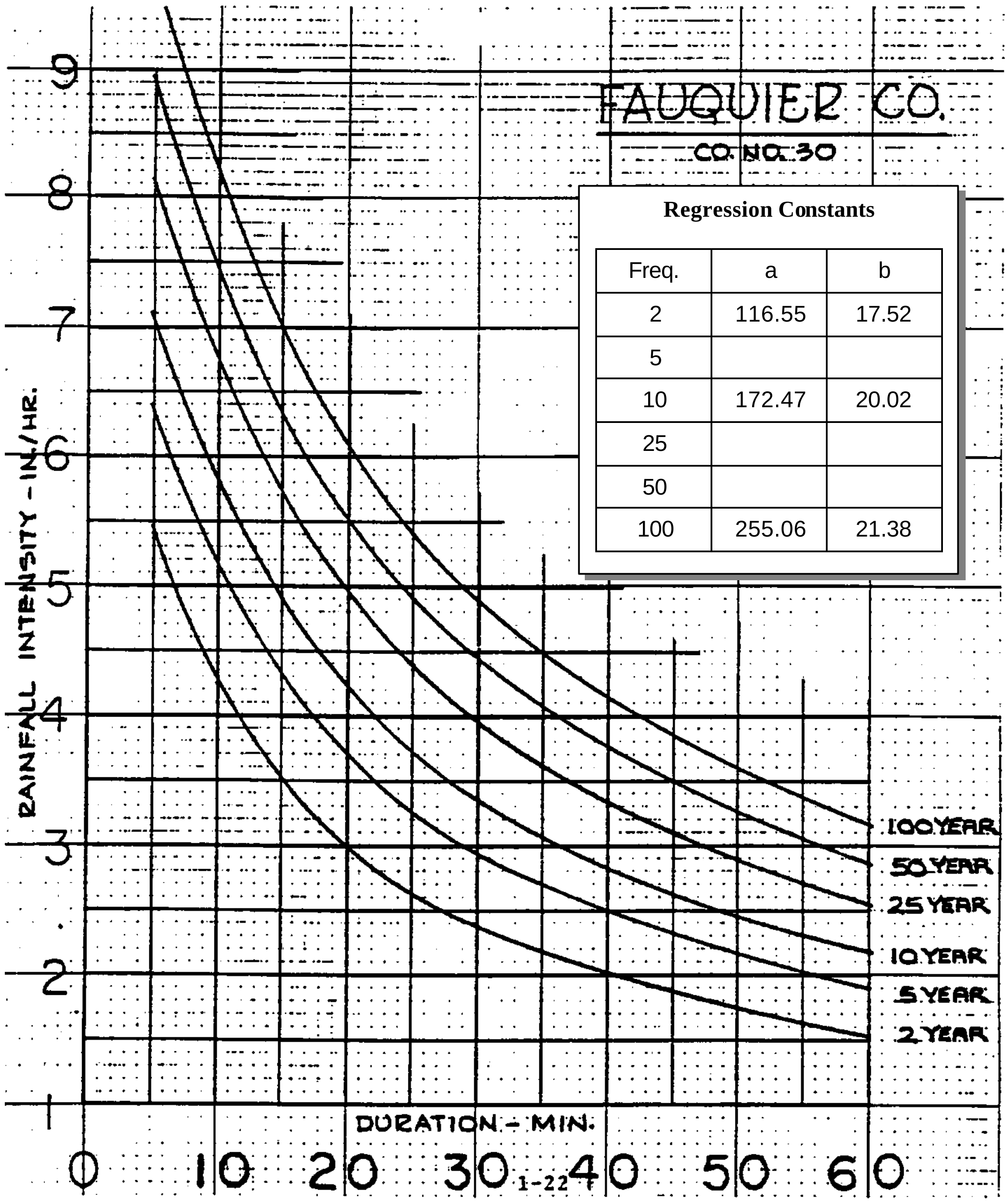
Regression Constants

Freq.	a	b
2		
5		
10		
25		
50		
100		

Rev. 6/92

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6B-6 Rainfall Intensity Inches Per Hour Fauquier Co.

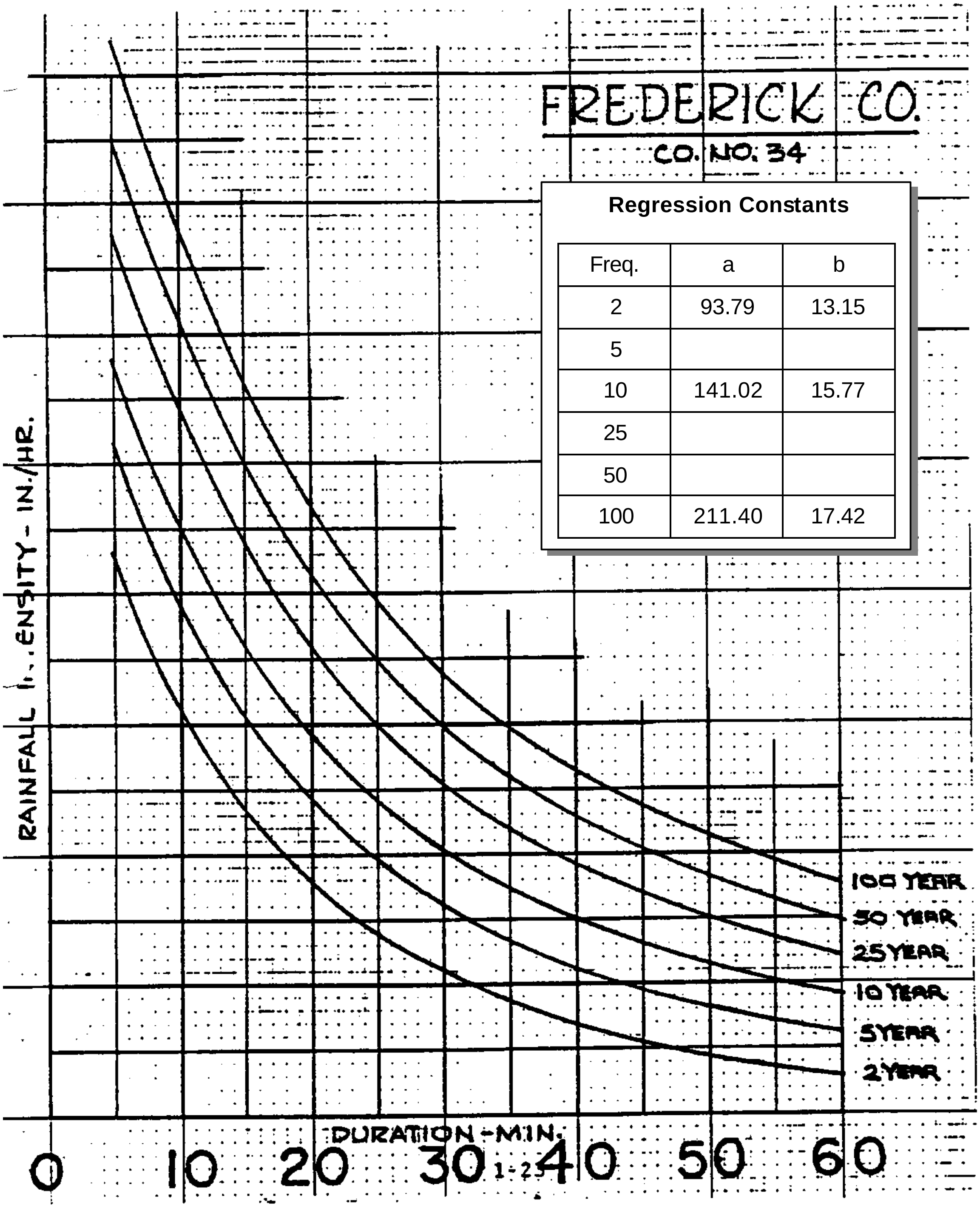


Regression Constants

Freq.	a	b
2	116.55	17.52
5		
10	172.47	20.02
25		
50		
100	255.06	21.38

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999.

Appendix 6B-7 Rainfall Intensity Inches Per Hour Frederick Co.

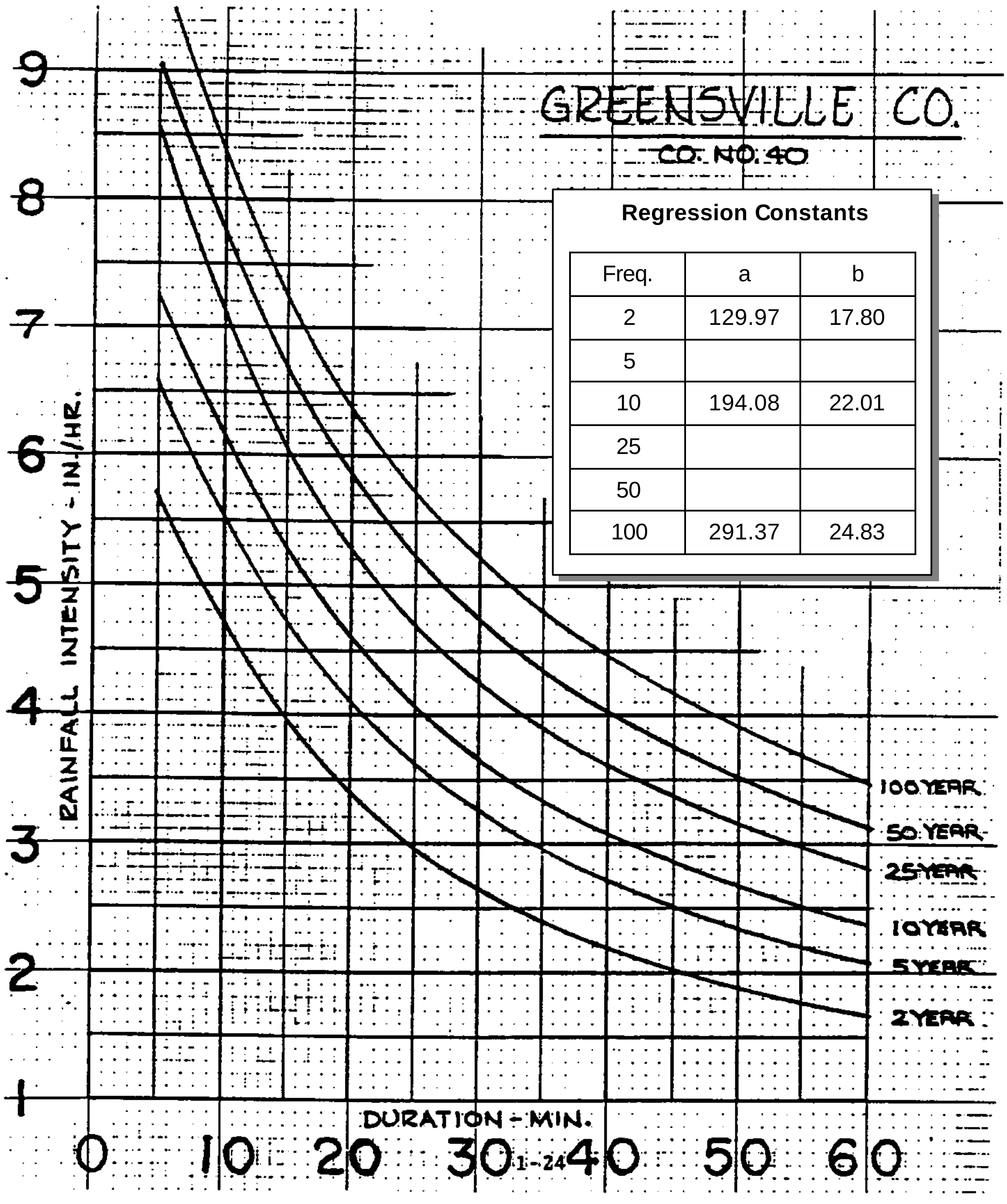


Regression Constants

Freq.	a	b
2	93.79	13.15
5		
10	141.02	15.77
25		
50		
100	211.40	17.42

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6B-8 Rainfall Intensity Inches Per Hour Greensville Co.

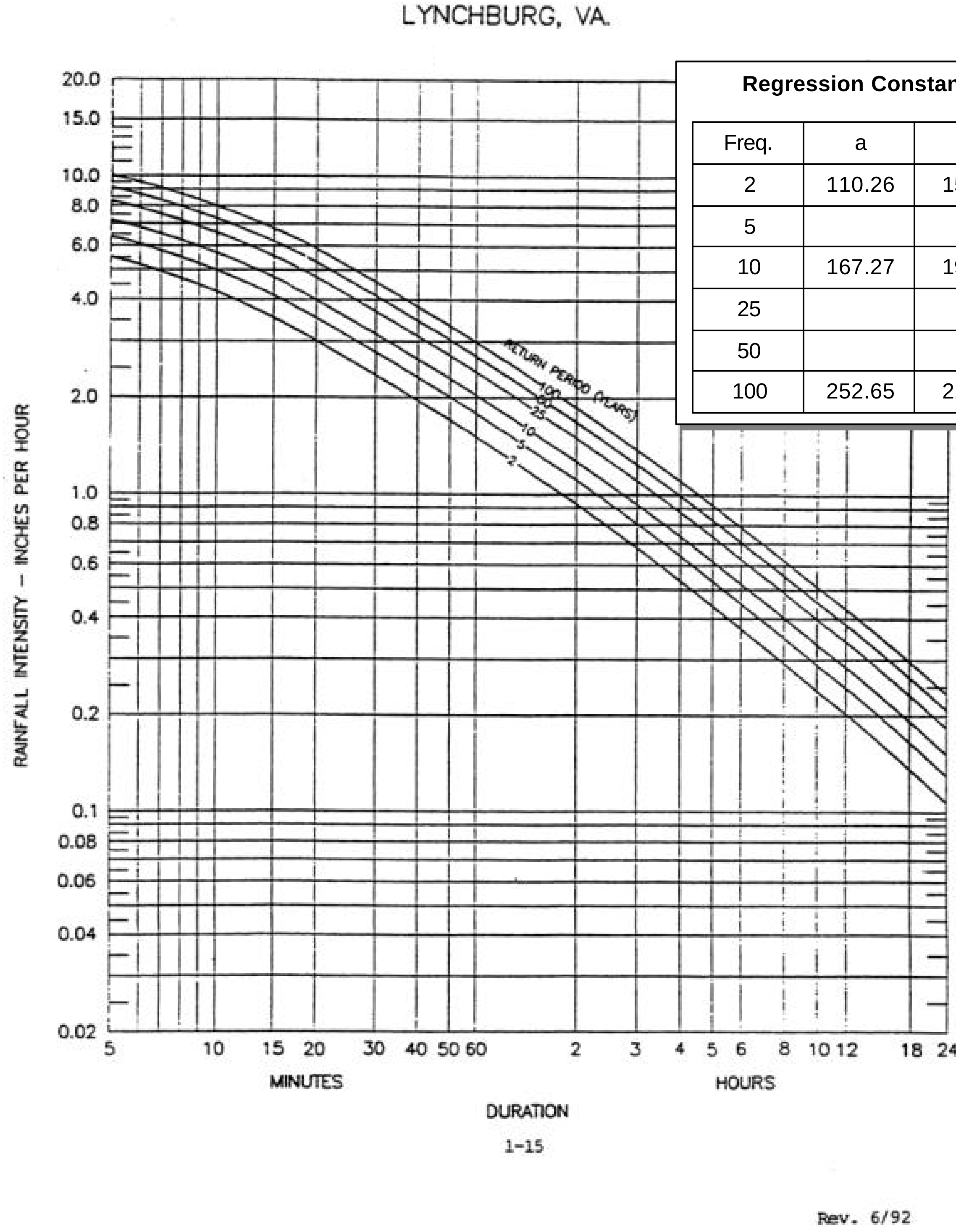


Regression Constants

Freq.	a	b
2	129.97	17.80
5		
10	194.08	22.01
25		
50		
100	291.37	24.83

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6B-9 Rainfall Intensity Inches Per Hour Lynchburg

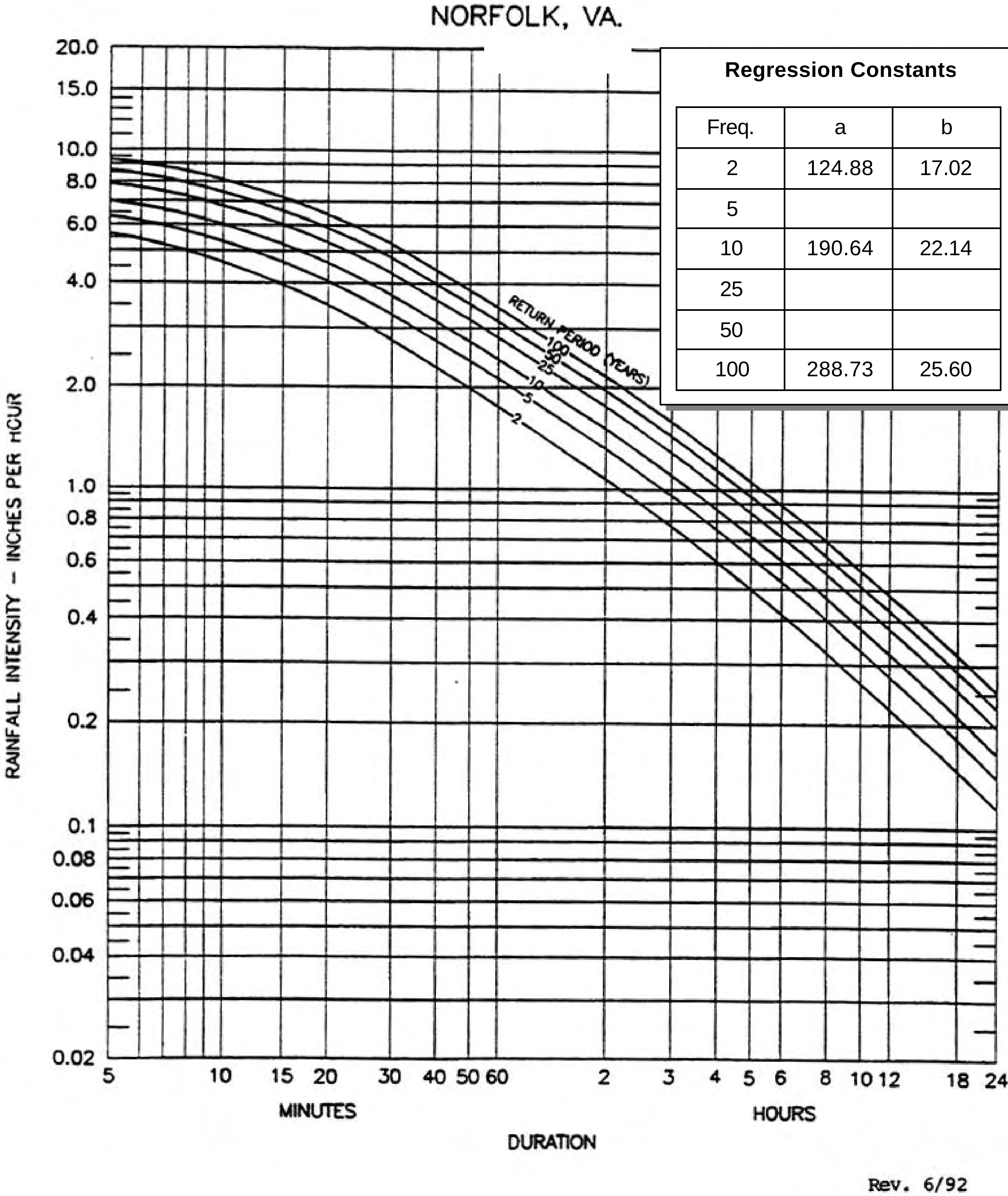


Regression Constants

Freq.	a	b
2	110.26	15.76
5		
10	167.27	19.18
25		
50		
100	252.65	21.56

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6B-10 Rainfall Intensity Inches Per Hour Norfolk

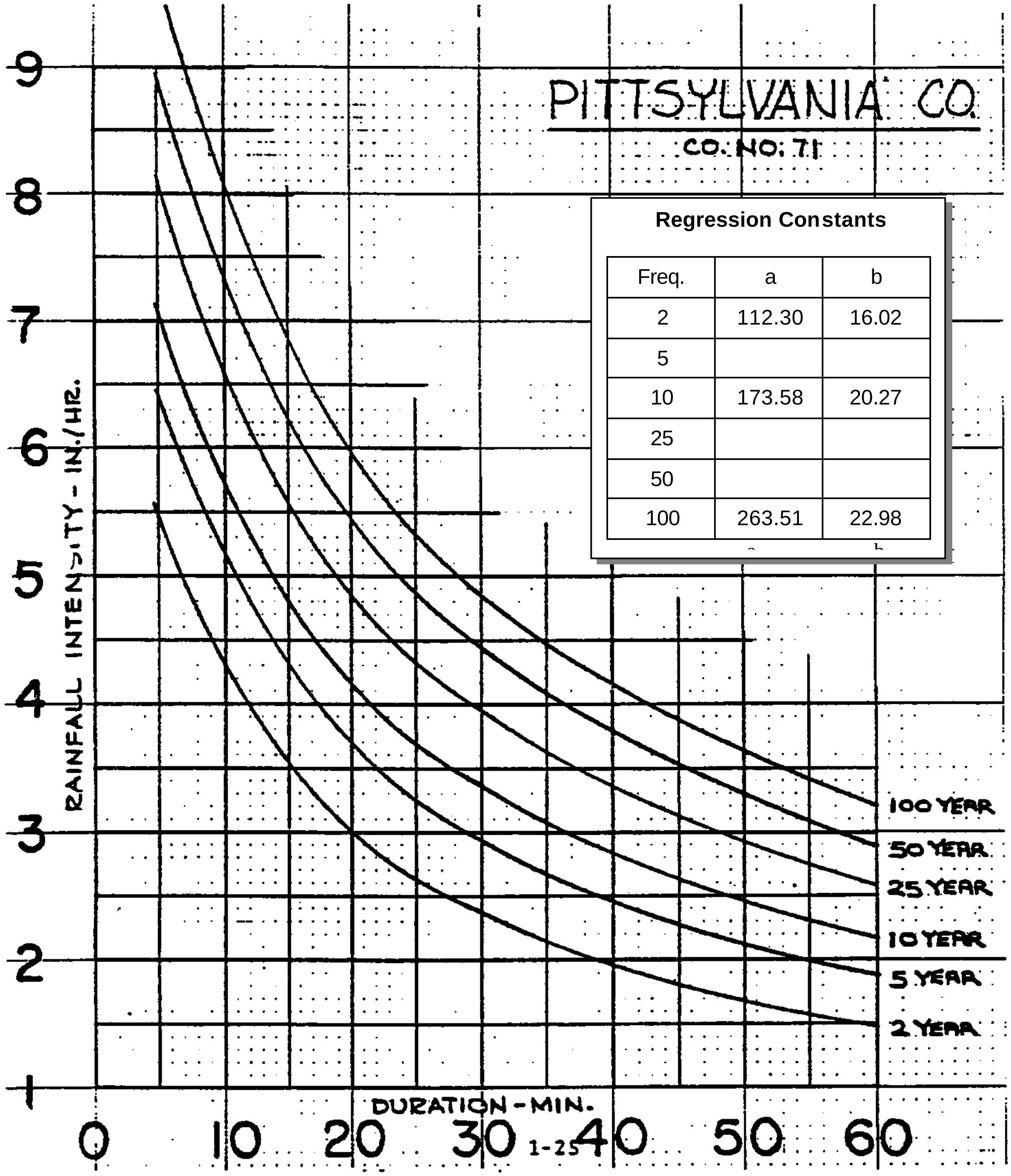


Regression Constants

Freq.	a	b
2	124.88	17.02
5		
10	190.64	22.14
25		
50		
100	288.73	25.60

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6B-11 Rainfall Intensity Inches Per Hour Pittsylvania Co.

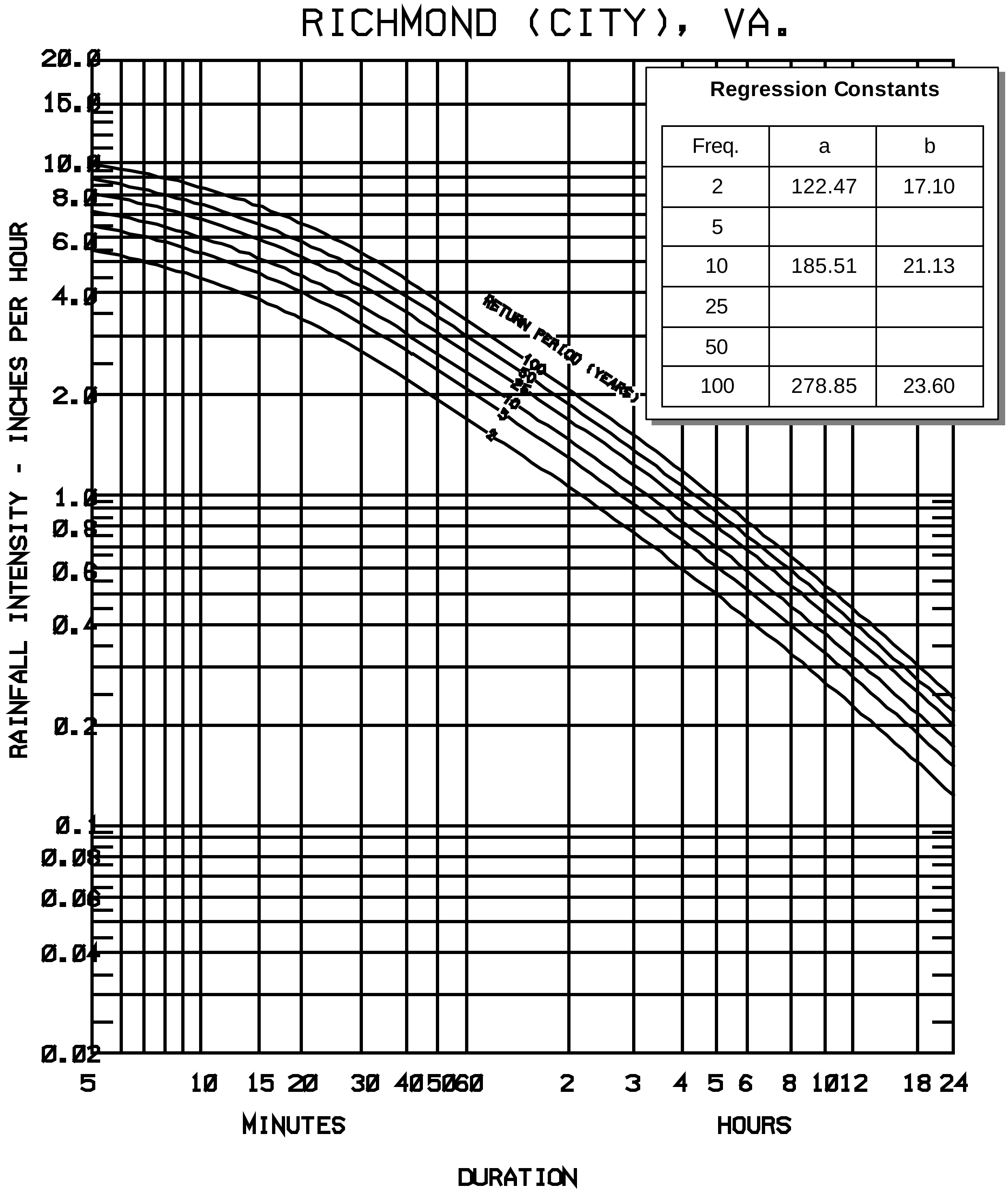


Regression Constants

Freq.	a	b
2	112.30	16.02
5		
10	173.58	20.27
25		
50		
100	263.51	22.98

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6B-12 Rainfall Intensity Inches Per Hour Richmond (City)

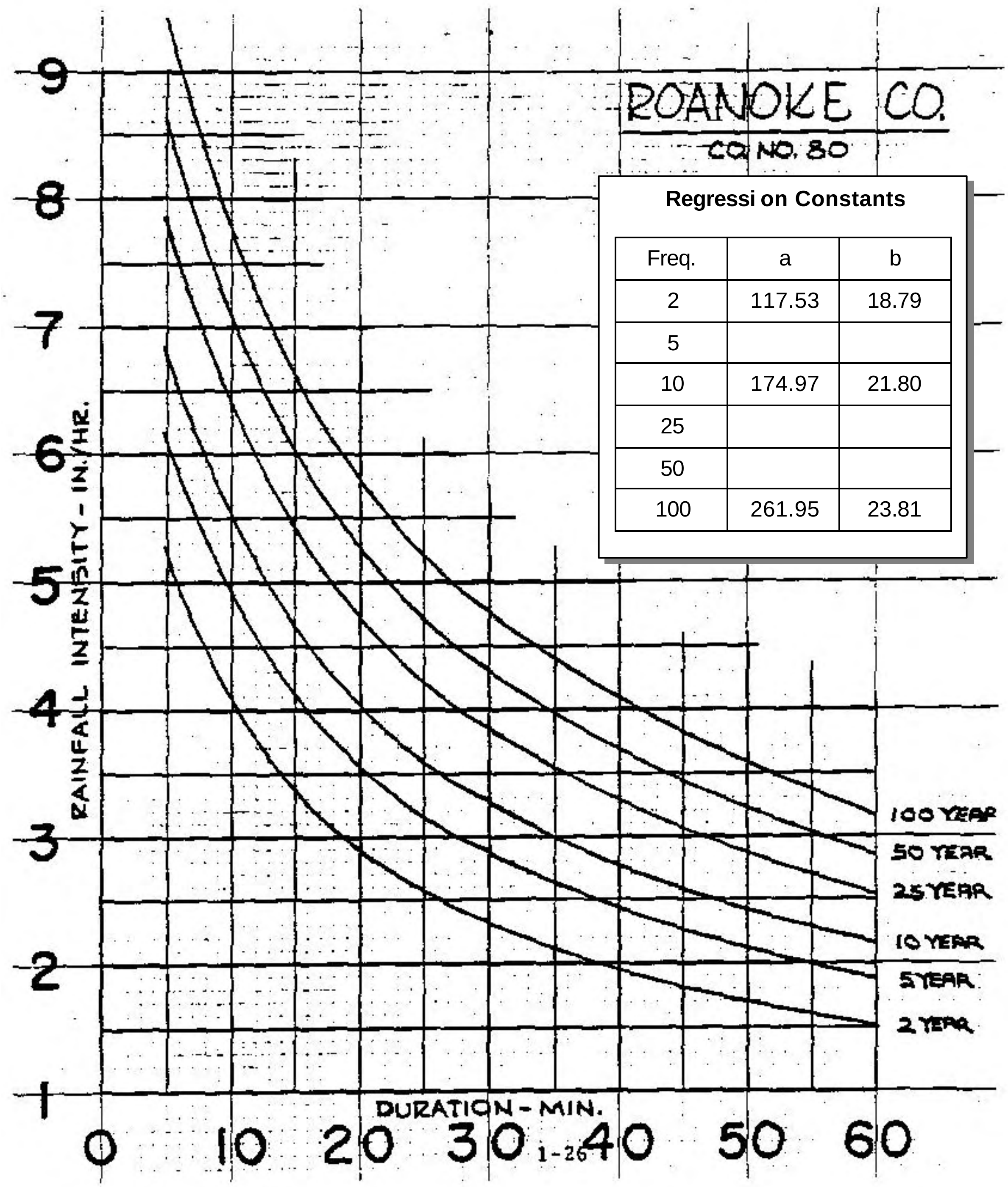


Regression Constants

Freq.	a	b
2	122.47	17.10
5		
10	185.51	21.13
25		
50		
100	278.85	23.60

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6B-13 Rainfall Intensity Inches Per Hour Roanoke Co.

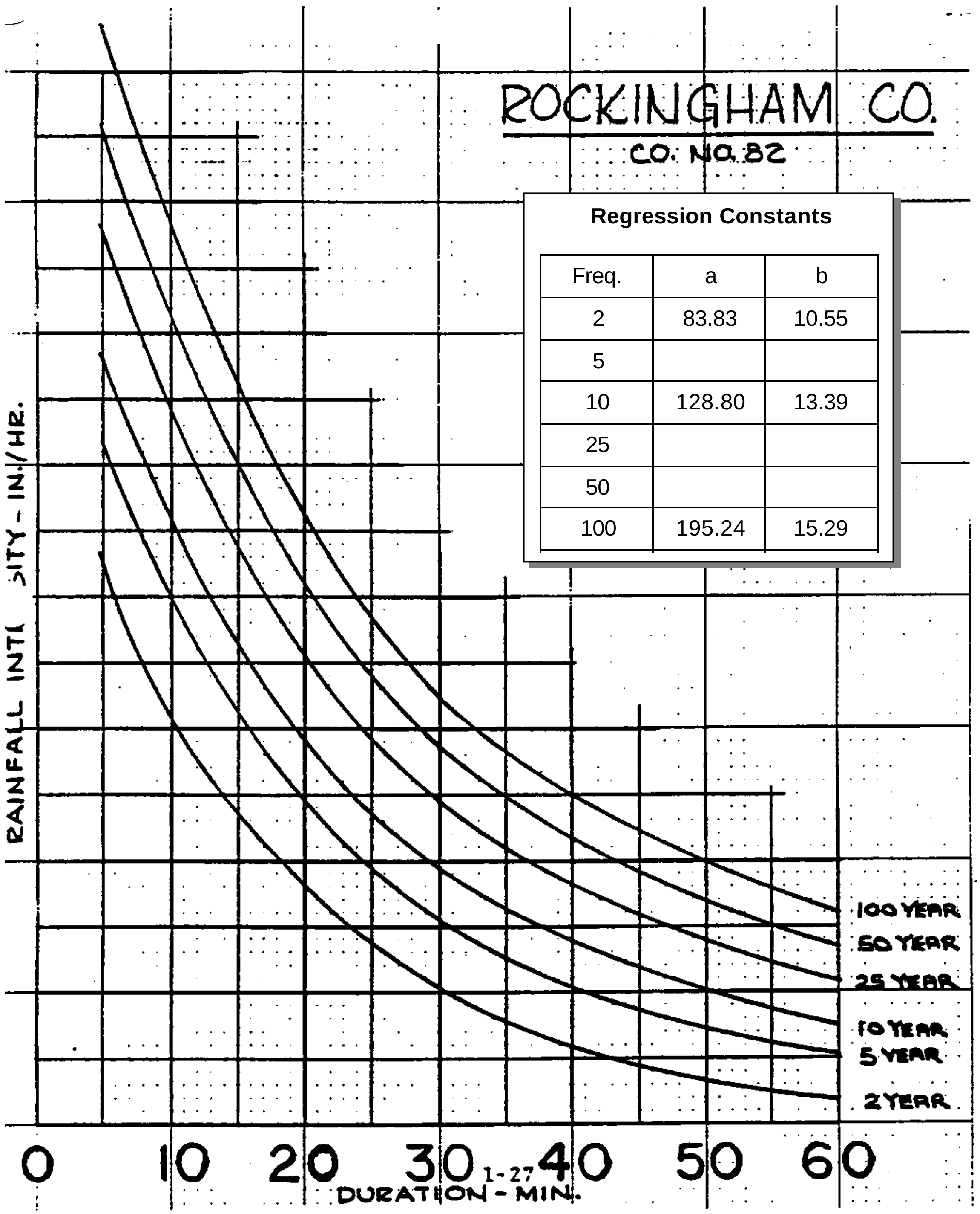


Regression Constants

Freq.	a	b
2	117.53	18.79
5		
10	174.97	21.80
25		
50		
100	261.95	23.81

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6B-14 Rainfall Intensity Inches Per Hour Rockingham Co.

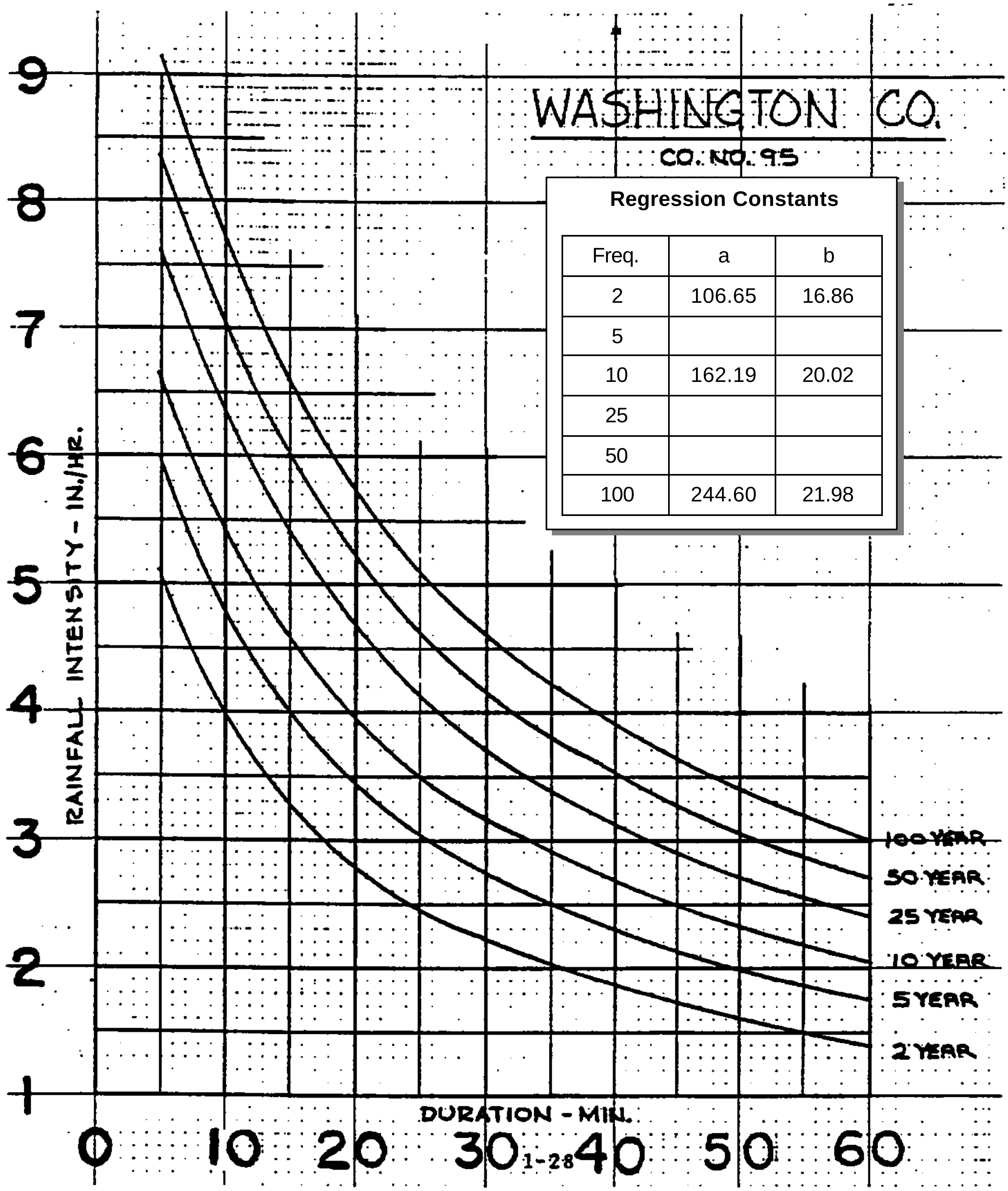


Regression Constants

Freq.	a	b
2	83.83	10.55
5		
10	128.80	13.39
25		
50		
100	195.24	15.29

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6B-15 Rainfall Intensity Inches Per Hour Washington Co.

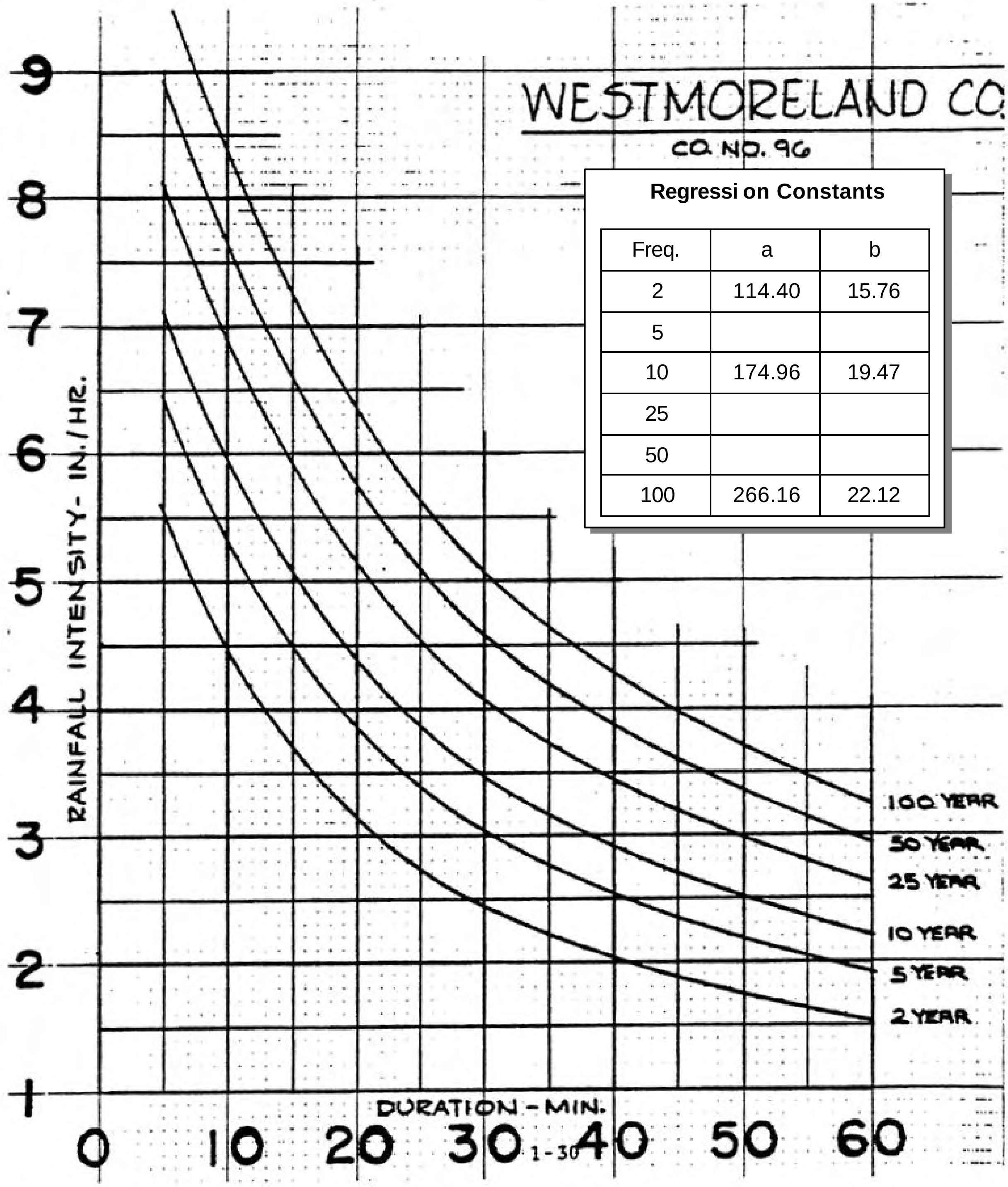


Regression Constants

Freq.	a	b
2	106.65	16.86
5		
10	162.19	20.02
25		
50		
100	244.60	21.98

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6B-16 **Rainfall Intensity Inches Per Hour**
Westmoreland Co.

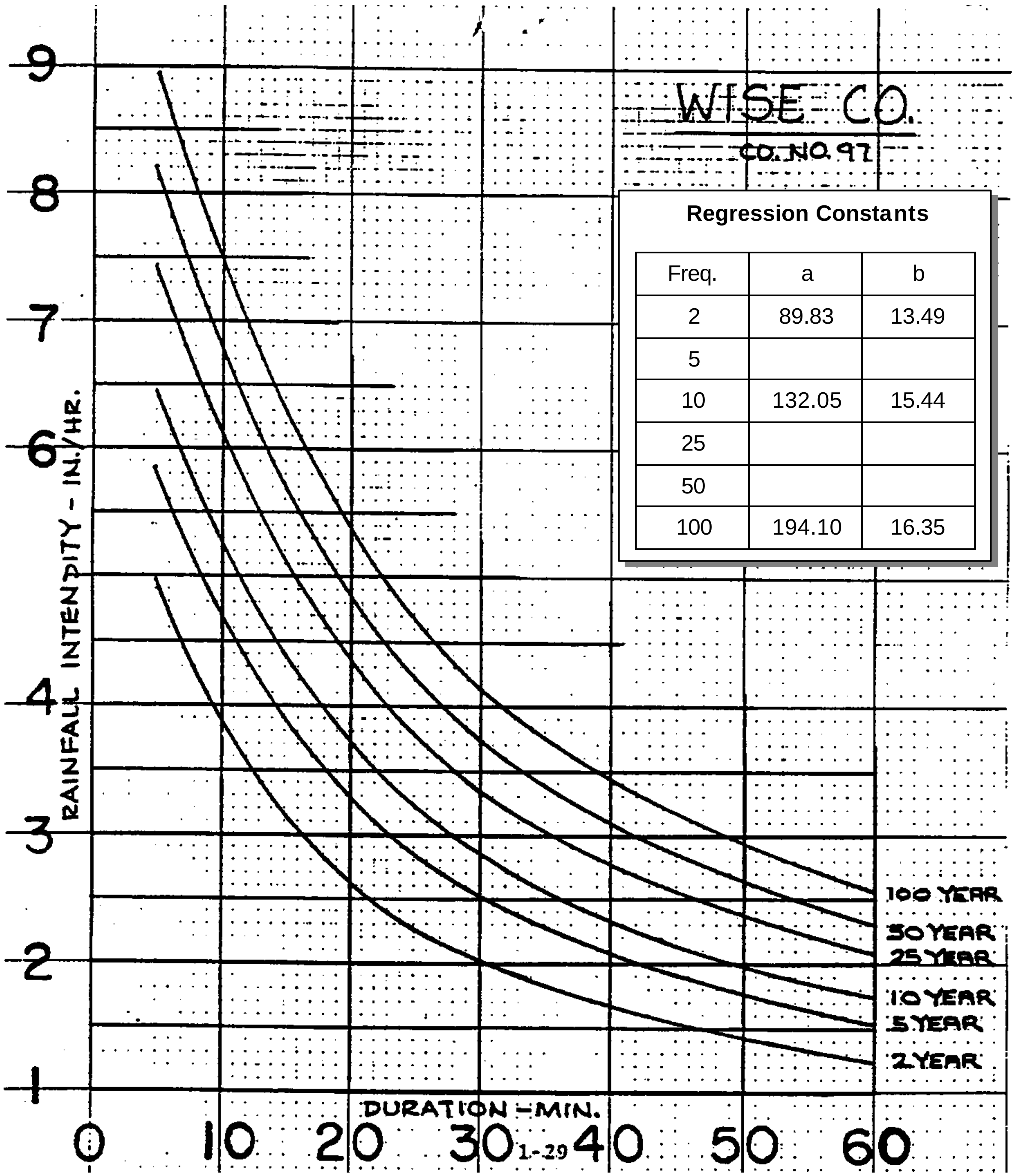


Regressi on Constants

Freq.	a	b
2	114.40	15.76
5		
10	174.96	19.47
25		
50		
100	266.16	22.12

Source:
Regression Constants from Virginia Stormwater Management Handbooi, 1st Ed., Vol. II, 1999

Appendix 6B-17 Rainfall Intensity Inches Per Hour
Wise Co.

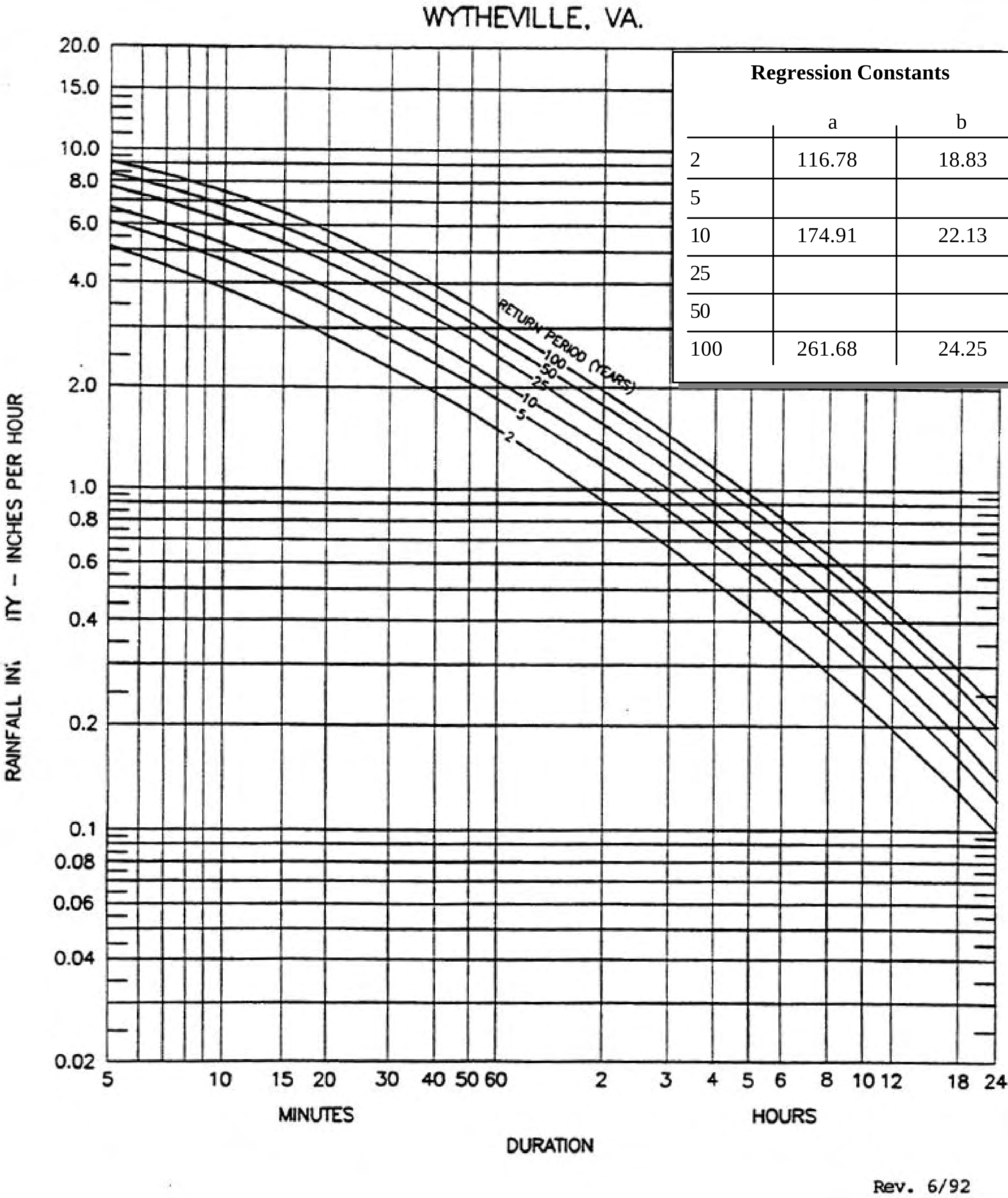


Regression Constants

Freq.	a	b
2	89.83	13.49
5		
10	132.05	15.44
25		
50		
100	194.10	16.35

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6B-18 Rainfall Intensity Inches Per Hour Wytheville



Regression Constants

	a	b
2	116.78	18.83
5		
10	174.91	22.13
25		
50		
100	261.68	24.25

Source:
Regression Constants from Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 6C-1 IDF Curves with B, D, and E Factors

B, D and E Factors that Define Intensity-Duration-Frequency (IDF) Curves for Use with the Rational Method and the Modified Rational Method

The rainfall IDF curves are also 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 are available for individual counties and may be obtained through the VDOT Central Office Hydraulics Section.

A sample of the information available is shown in Appendix 6C-2.

Appendix 6C-1 IDF Curves with B, D, and E Factors

Rainfall Report – Chesterfield County

Rainfall Type: Custom Rainfall

Rainfall Filename: Chesterf.rnd

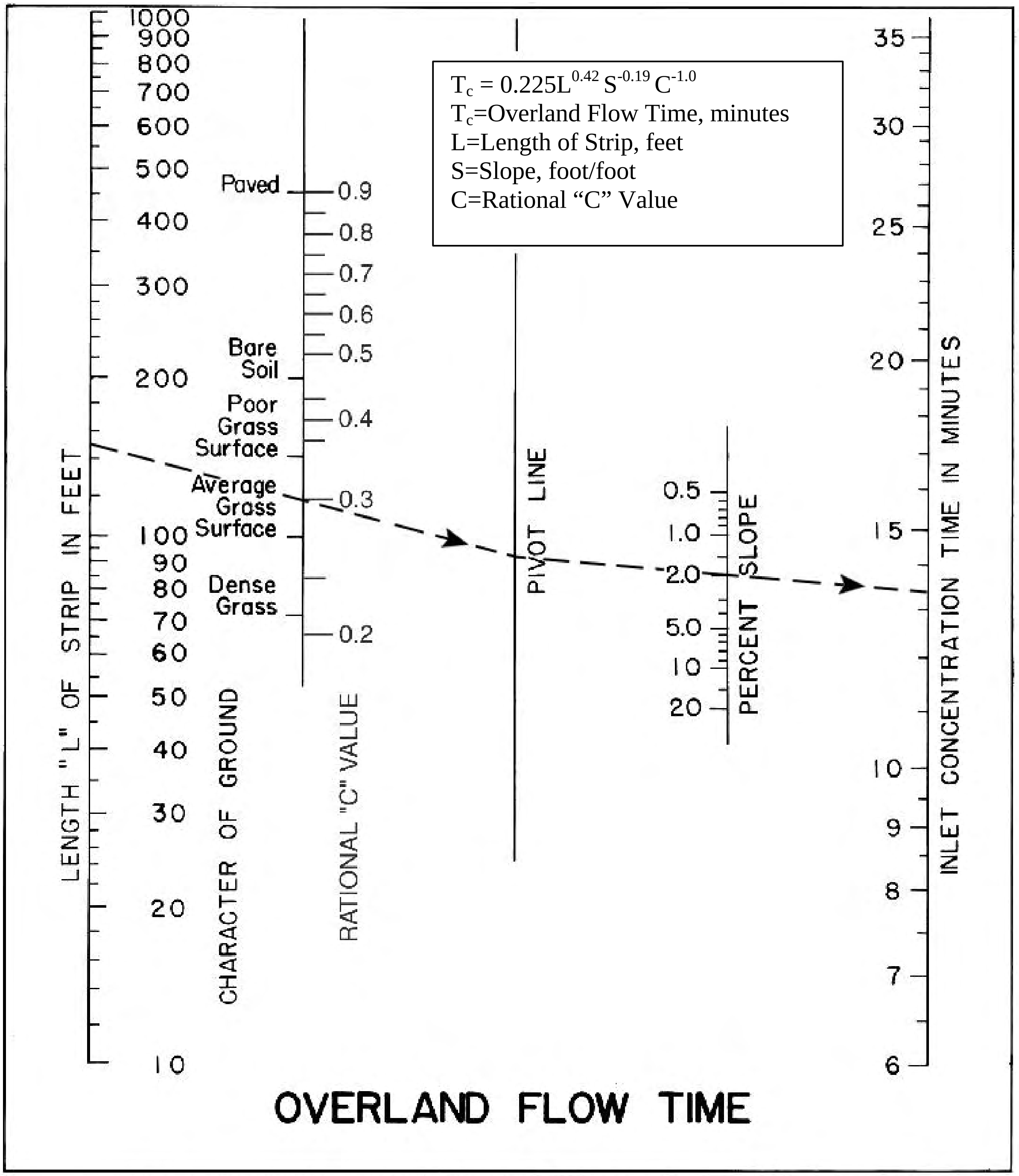
INTERMEDIATE INTENSITIES (in/hr)

	5 min	15 min	30 min	60 min	6 hr	24 hr
2 yr	5.71	3.80	2.56	1.62	0.40	0.13
5 yr	6.56	4.60	3.16	2.02	0.48	0.14
10 yr	7.22	5.16	3.58	2.31	0.54	0.16
25 yr	8.21	6.00	4.20	2.72	0.66	0.19
50 yr	8.99	6.68	4.68	3.03	0.67	0.18
100 yr	9.77	7.32	5.14	3.35	0.78	0.22

BDE VALUES $\text{Intensity} = \frac{B}{(t_c + D)^E}$

	B	D	E
2 yr	58.01	10.75	0.84
5 yr	92.96	14.75	0.89
10 yr	111.70	16.25	0.90
25 yr	146.51	18.25	0.91
50 yr	213.71	21.25	0.97
100 yr	221.97	21.25	0.95

Appendix 6D-1 Overland Flow Time - Seelye



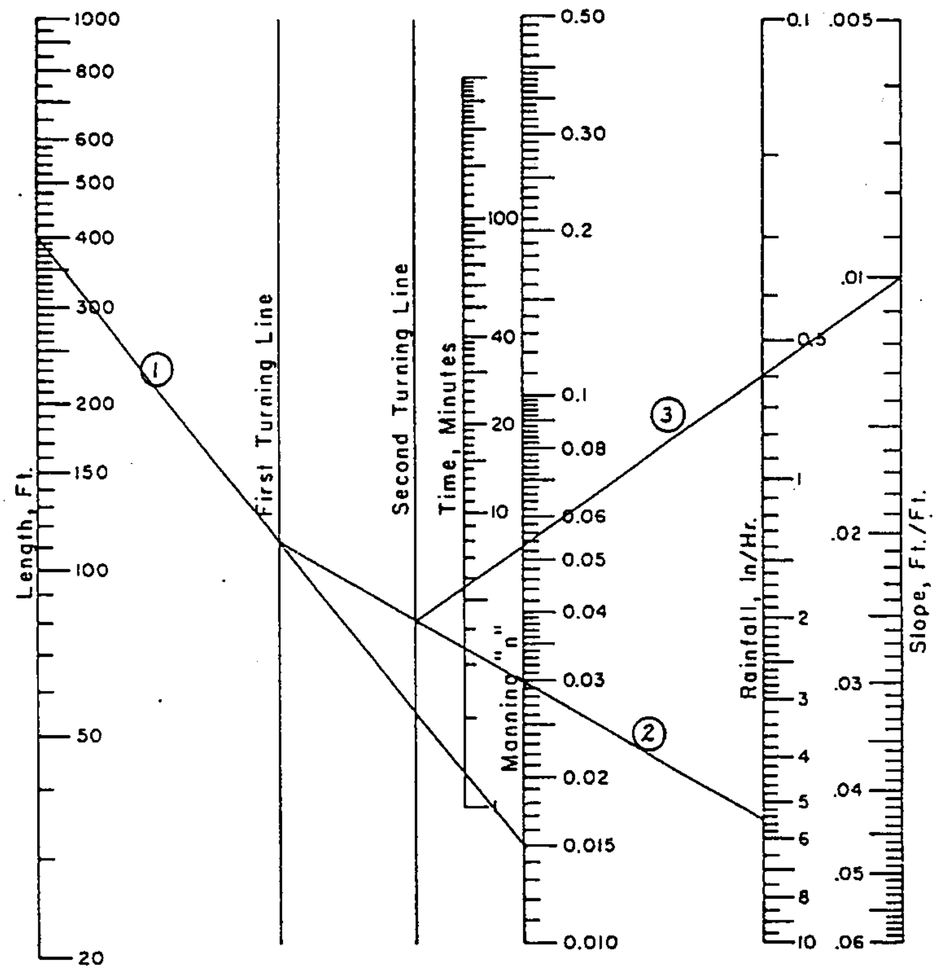
REPRINTED WITH PERMISSION FROM "DATA BOOK FOR CIVIL ENGINEERS" VOL. I - DESIGN
2ND EDITION (1951) BY E. E. SEELYE

Comments:
VDOT added a 'C-VALUE' scale 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 manual 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.

Appendix 6D-2 Kinematic Wave Formulation Overland Flow

Equation solved by nomograph:

$$t_c\,(min) = .93\,\frac{L^{0.6}n^{0.6}}{i^{0.4}S_0^{0.3}}$$



The initially assumed value of i and the nomograph value of t must be checked against the applicable intensity-duration-frequency curve by trial and error.

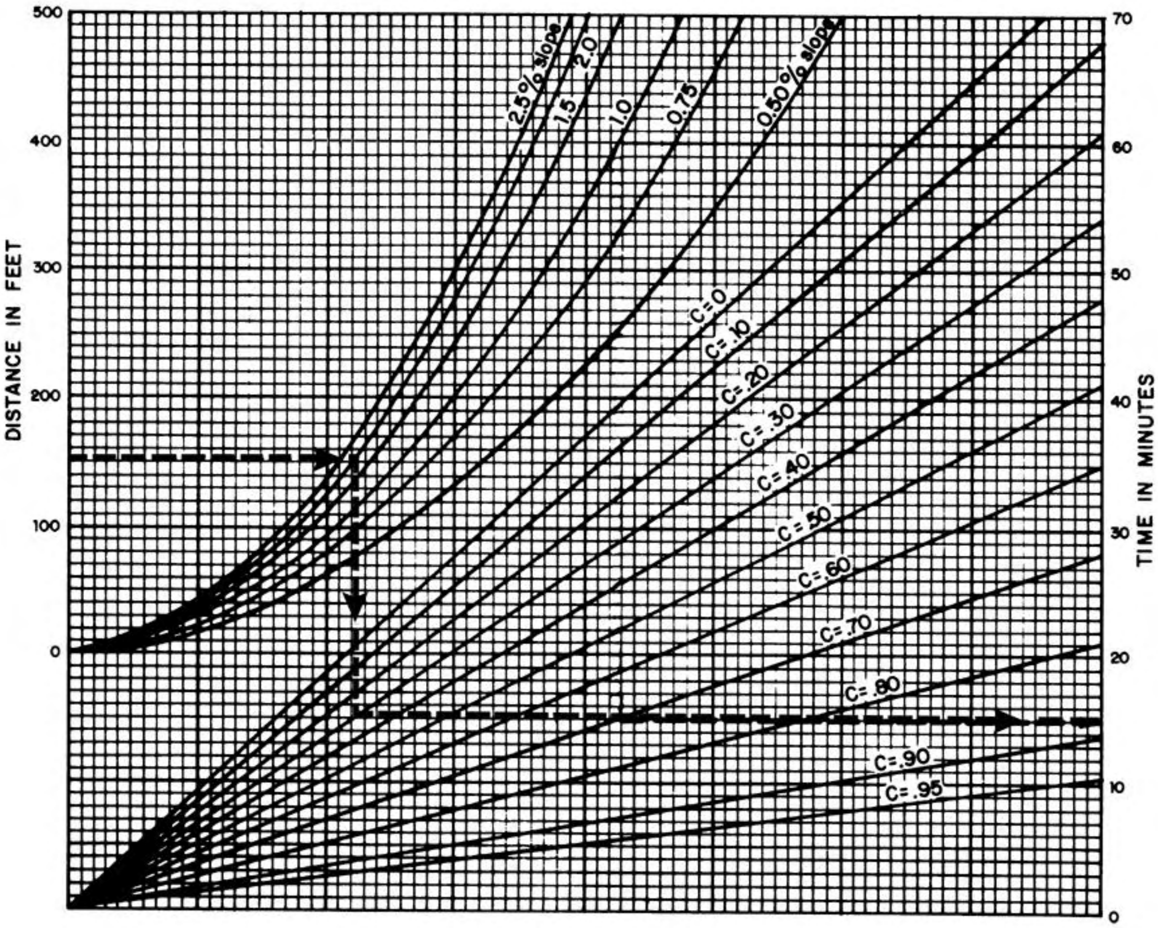
Example:
- L = 400 ft.
- n = 0.015
- i = 5.5 in./hr.
- S_0 = 0.01
- t = 5.5 min.

ONE INCH is 25.4mm
ONE FOOT is 0.3048m

Nomograph for determining time of concentration for overland flow, Kinematic Wave Formulation. *(After Ragan.)*

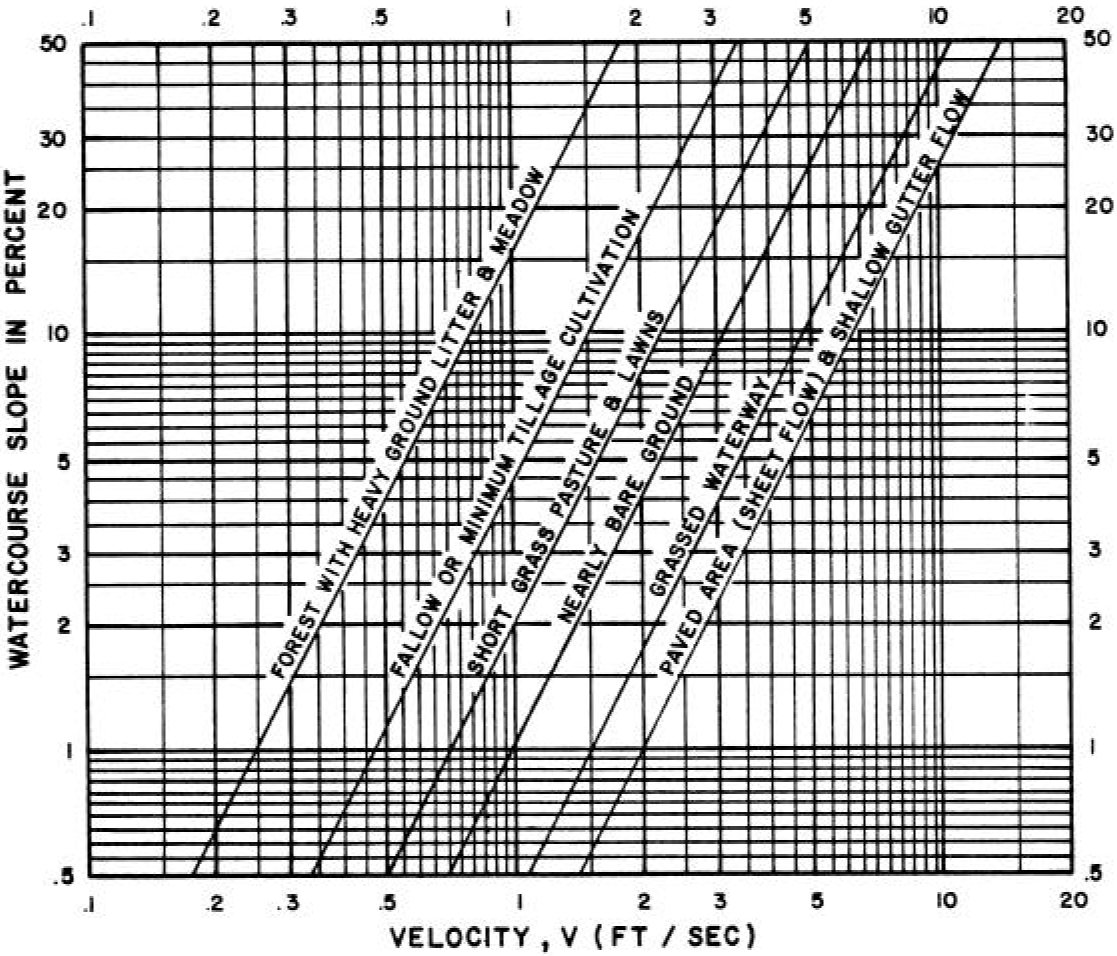
Comments:
VDOT has determined that the Kinematic Wave Method should only be used for:
a) Impervious Surfaces
b) n = 0.05 or less
c) Length = 300' Maximum

Appendix 6D-3 Overland Time of Flow



Source:
Airport Drainage, Federal Aviation Administration, 1965

Appendix 6D-4 Overland Flow Velocity

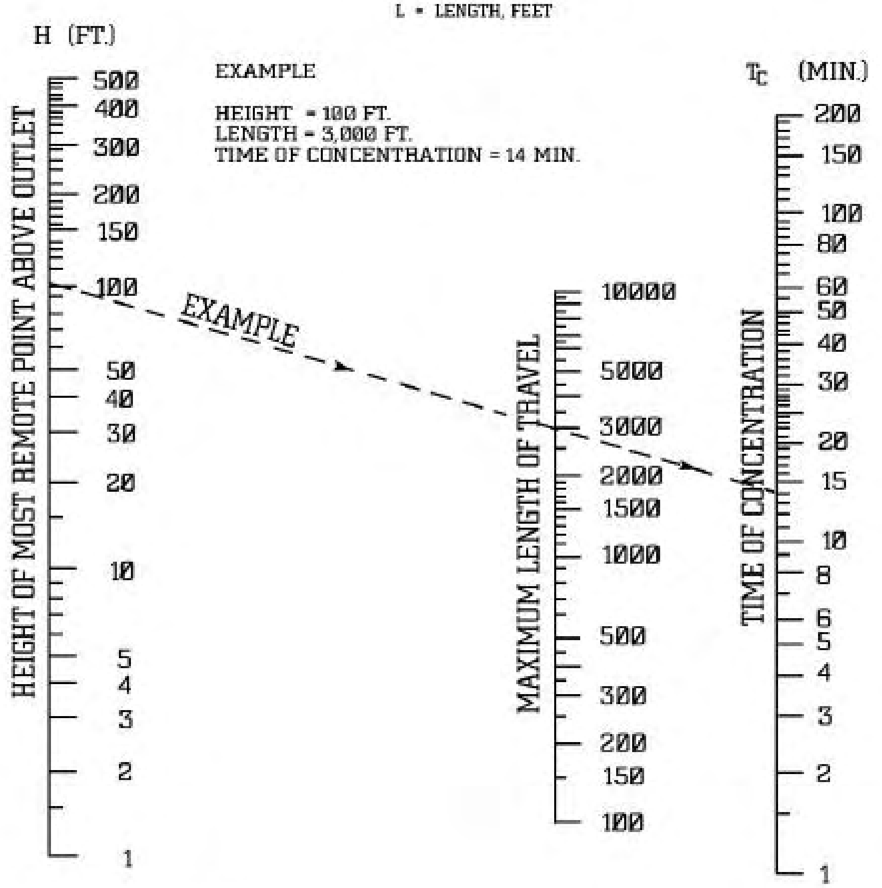


Source:
HEC No. 19, FHWA

Appendix 6D-5 Time of Concentration for Small Drainage Basins - Kirpich

$$T_C = 0.00948\ H^{-0.38}\ L^{1.13}$$

T_C = FLOW TIME, MINUTES
H = HEIGHT, FEET
L = LENGTH, FEET



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

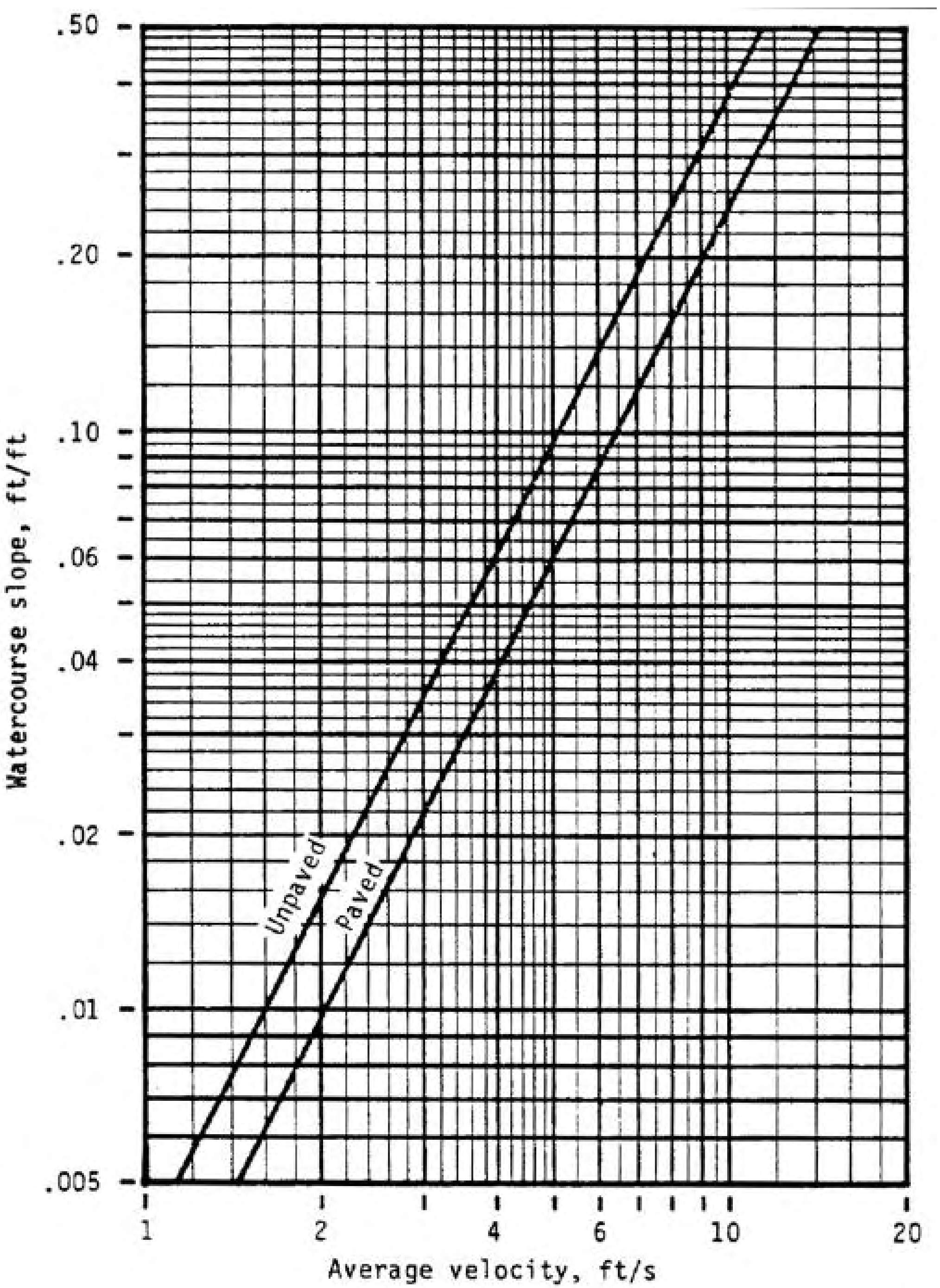
TIME OF CONCENTRATION OF SMALL DRAINAGE BASINS

* NOTE:
USE NOMOGRAPH FOR NATURAL BASINS WITH WELL-DEFINED CHANNELS AND FOR MOWED GRASS ROADSIDE CHANNELS.

Comments:
VDOT derived an equation from and added it to this nomograph. This was done without the author's permission in the interest of providing the user with an optional mathematical solution. The Department warrants neither the accuracy nor the validity of this equation and cautions the user that it be used at their own risk.

****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:
SCS, 210-VI-TR-55, Second Edition, June, 1986
VDOT has determined that this nomograph produces essentially the same flow time as the “Kirpich” Method.

Appendix 6E-1 Rational Method Runoff Coefficients

Recommended Coefficient of Runoff Values for Various Selected Land Uses

Description of Area	Runoff Coefficients
Business: Industrial and Commercial	0.80-0.90
Apartments and Townhomes	0.65-0.75
Schools	0.50-0.60
Residential - lots 10,000 sq. ft.	0.40-0.50
- lots 12,000 sq. ft.	0.40-0.45
- lots 17,000 sq. ft.	0.35-0.45
- lots ½ acre or more	0.30-0.40
Parks, Cemeteries and Unimproved Areas	0.20-0.35
Paved and Roof Areas	0.90
Cultivated Areas	0.50-0.70
Pasture	0.35-0.45
Lawns	0.25-0.35
Forest	0.20-0.30
Steep Grass (2:1)*	0.40-0.70
Shoulder and Ditch Areas *	0.35-0.50

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 5%).

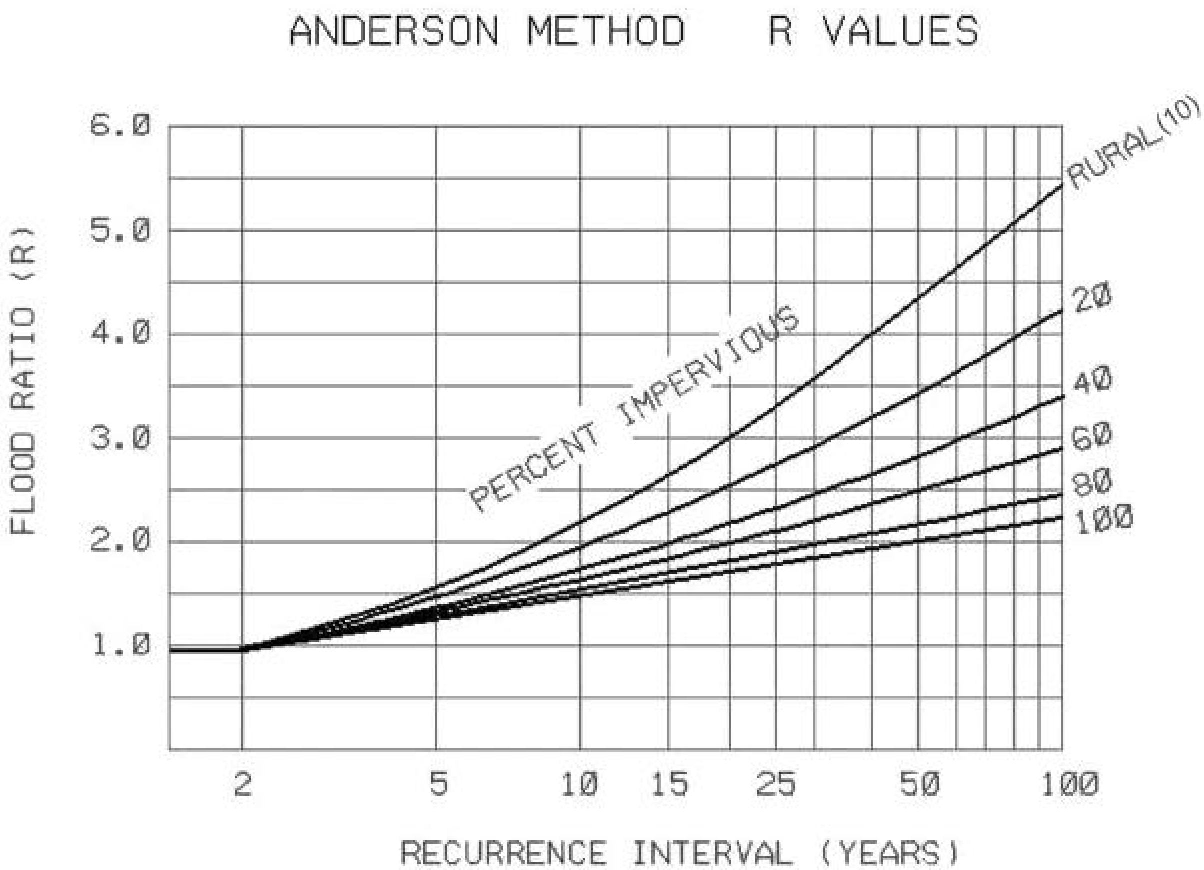
3. The highest range of runoff coefficients shall be used for steep areas (areas where the majority of the grades are greater than 5%), for cluster areas, and for development in clay soil areas.

*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.

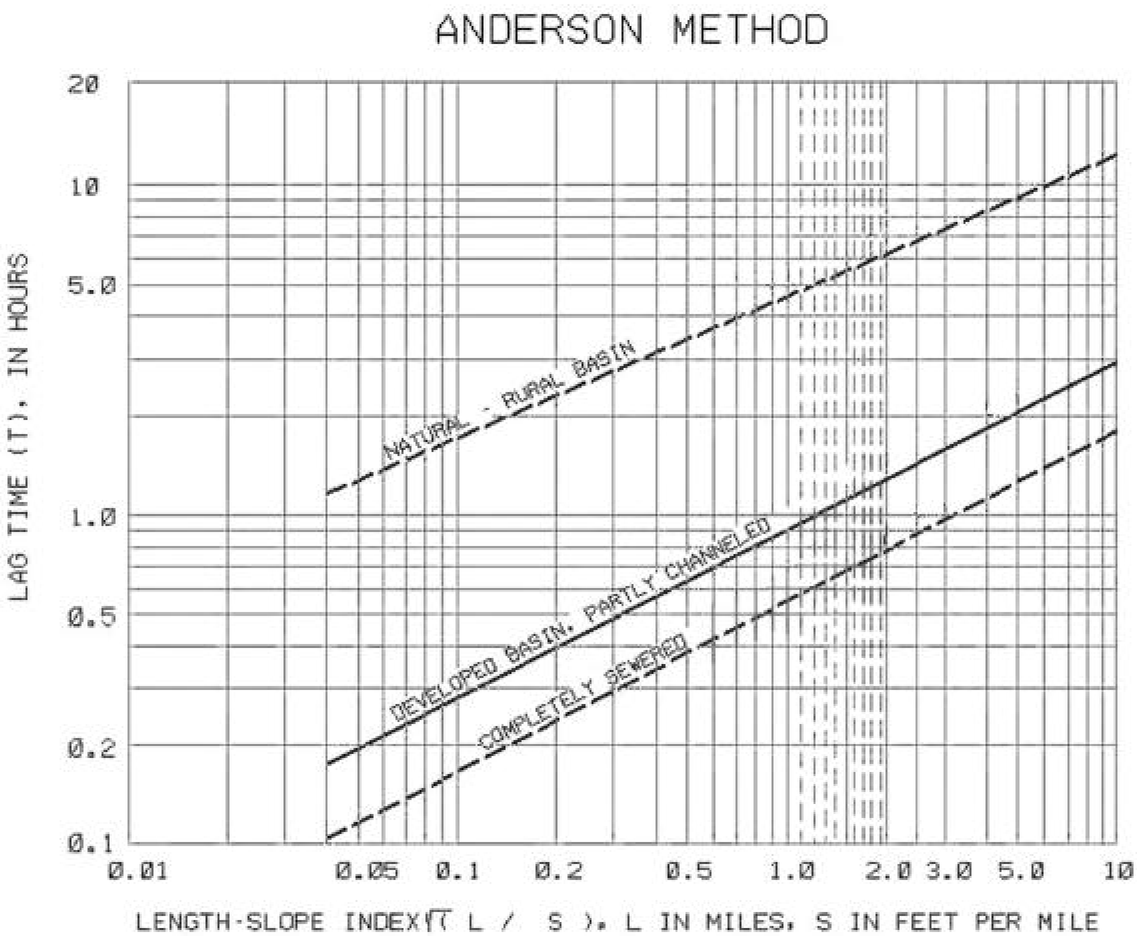
Source:
Runoff Coefficients and Inlet Times, Fairfax County Public Facilities Manual, Chart A6-19, 1988
VDOT Drainage Manual

Appendix 6F-1 Anderson Method: R Values



Source:

Appendix 6F-2 Lag Time vs. Length-Slope Index



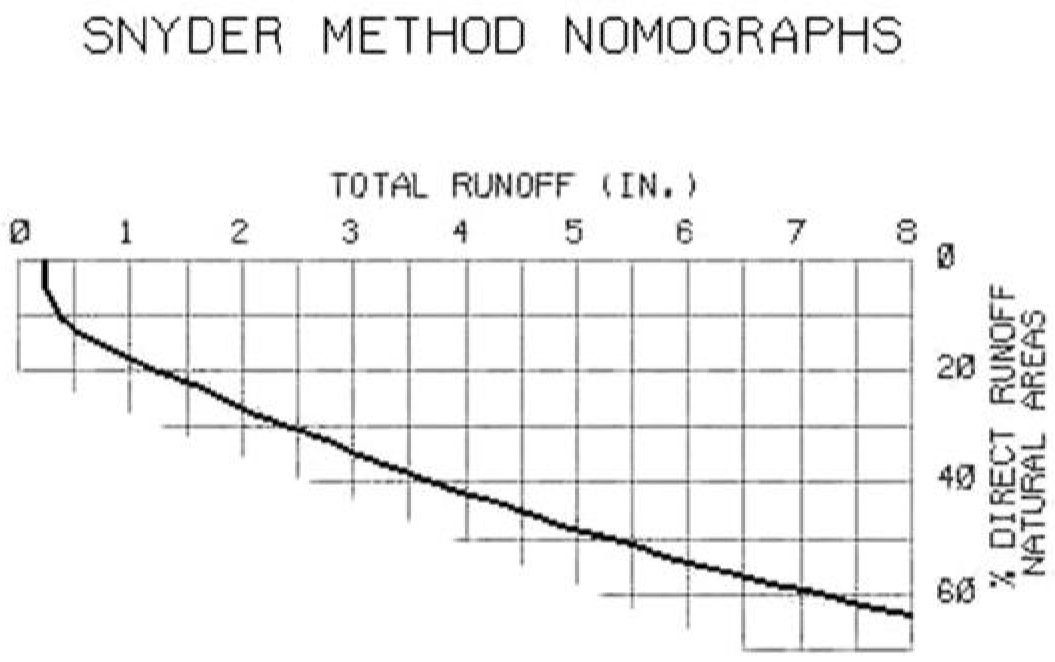
Natural Rural Basin $T=4.64\left(\frac{L}{\sqrt{S}}\right)^{0.42}$

Developed Basin Partly Channelized $T=0.90\left(\frac{L}{\sqrt{S}}\right)^{0.50}$

Completely Developed and Sewered Basin $T=0.56\left(\frac{L}{\sqrt{S}}\right)^{0.52}$

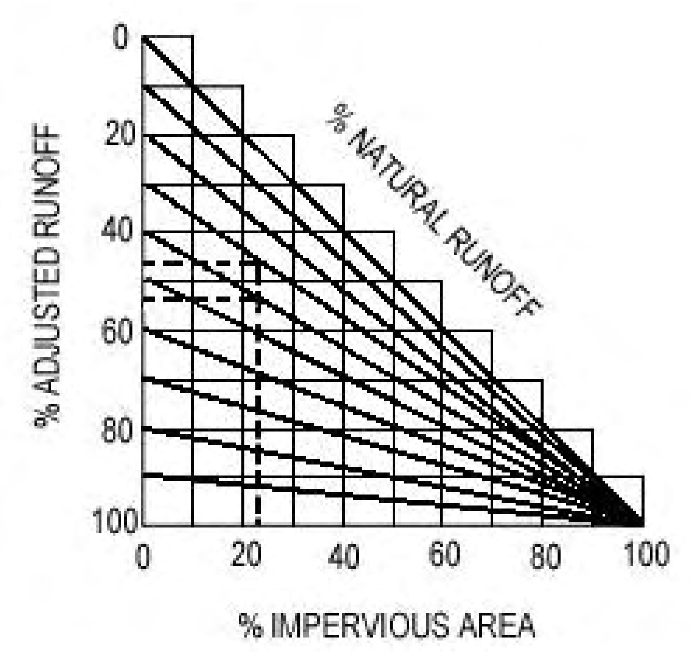
Source:

Appendix 6G-1 Total Runoff vs. % Direct Runoff



Source:

Appendix 6G-2 % Impervious Area vs. % Adjusted Runoff



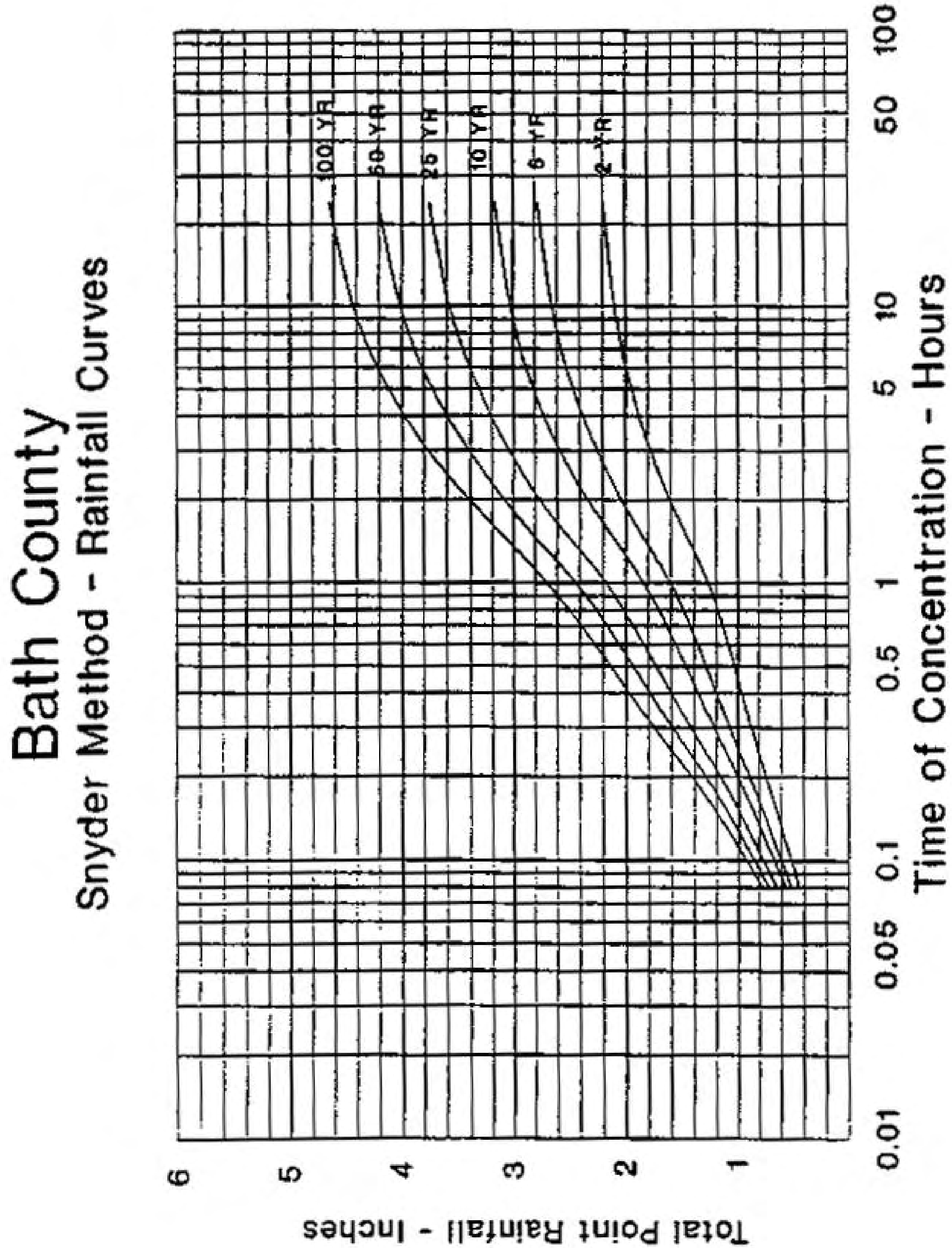
Source:

Appendix 6G-3 Alternate Values for Coefficient of Runoff in Urban Areas

Description of Area	Runoff Coefficients	Percent Impervious
Business, Commercial & Industrial	0.80-0.90	90%
Apartments & Townhouses	0.65-0.75	75%
Schools & Churches	0.50-0.60	50%
Residential:		
Single Family Units		
Lots 10,000 SF	0.40-0.50	35%
Lots, 12,000 SF	0.40-0.45	30%
Lots, 17,000 SF	0.35-0.45	25%
Lots ½ Acre or More	0.30-0.40	20%
Parks, Cemeteries & Unimproved Areas	0.25-0.35	15%
Pavement & Roofs	0.90	-
Lawns	0.25-0.35	varies

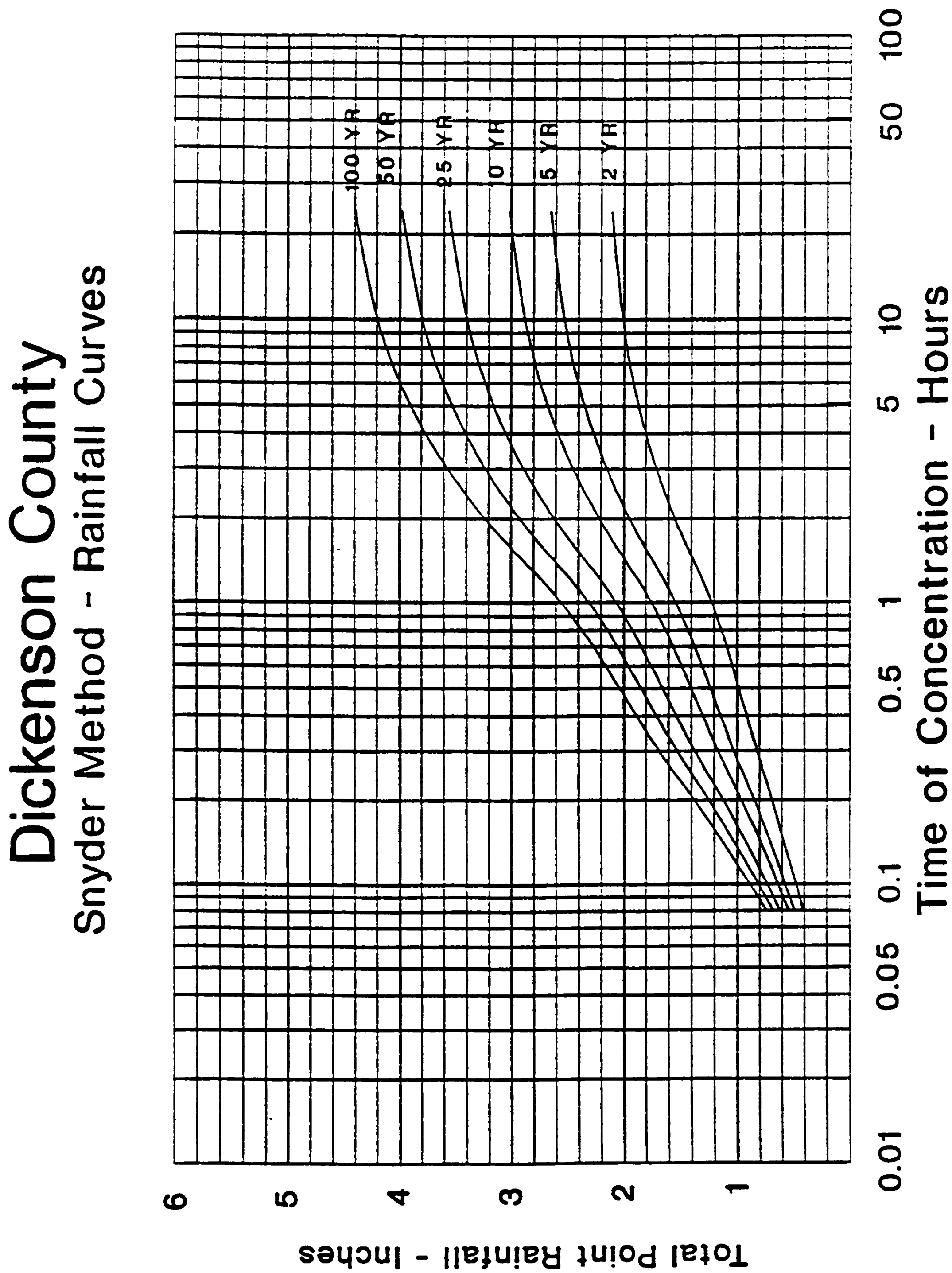
Source:

Appendix 6G-4 **Snyder Method: Bath County Rainfall Depth Curve**



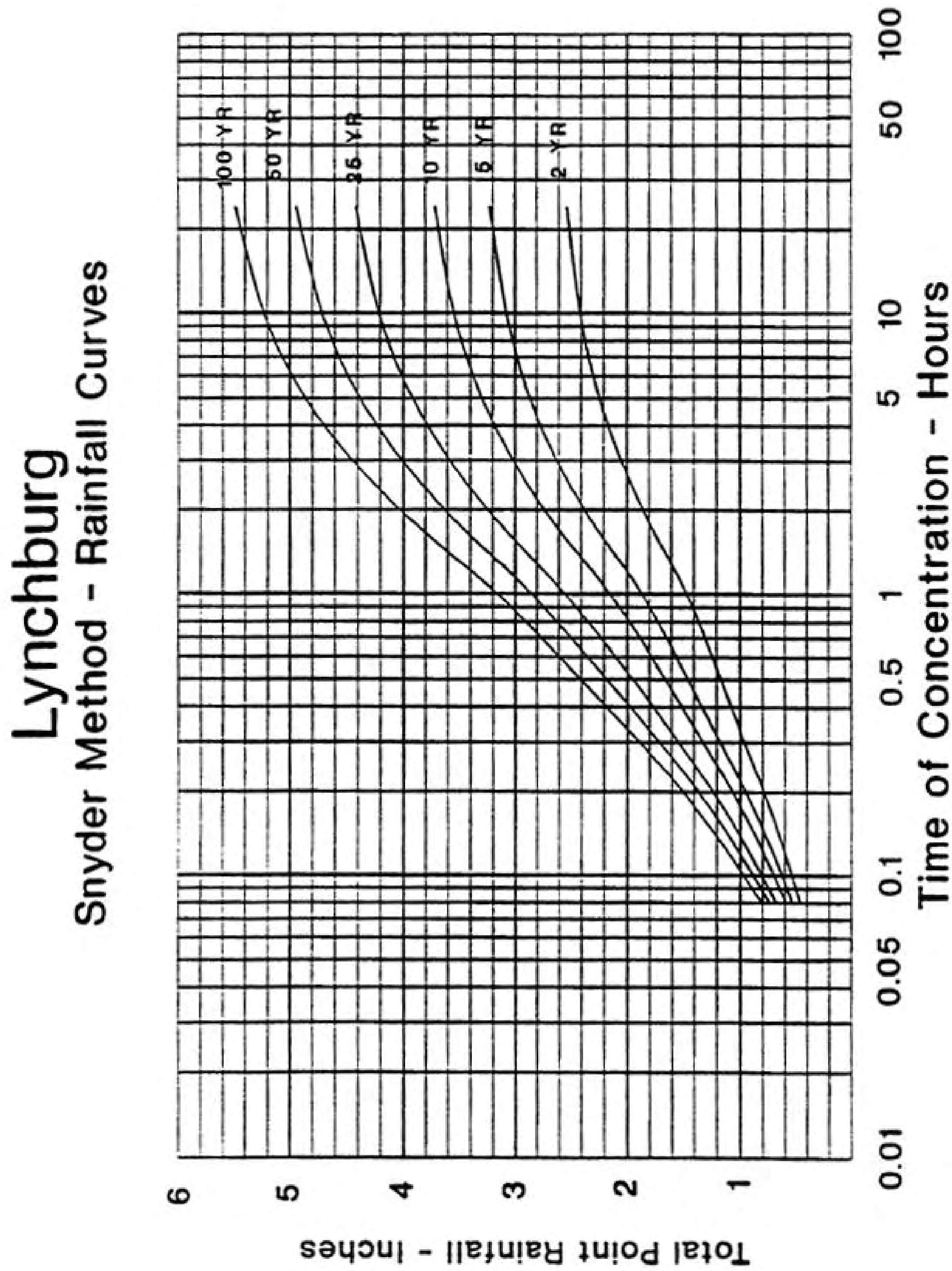
Source:

Appendix 6G-5 Snyder Method: Dickenson County Rainfall Depth Curve



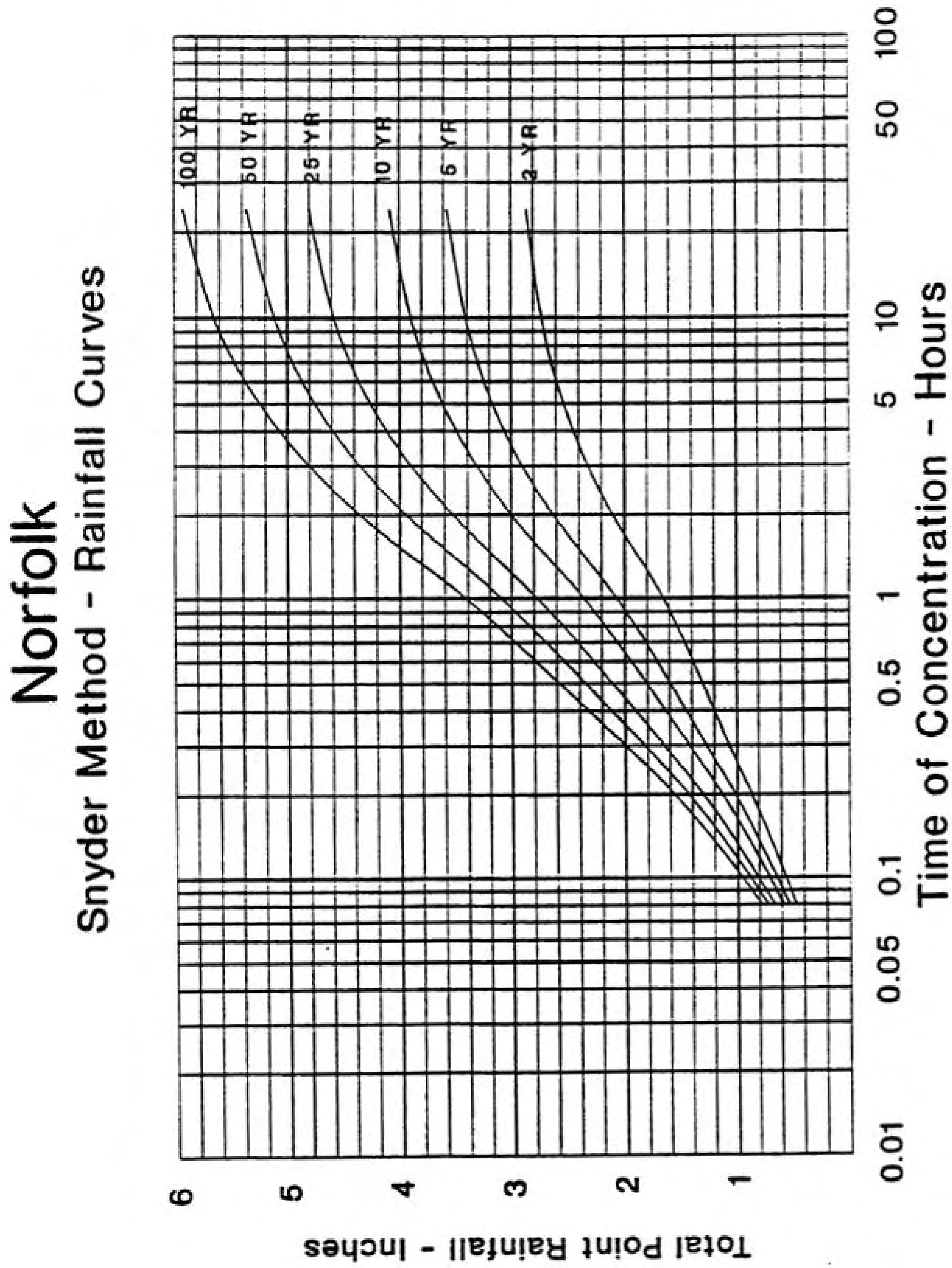
Source:

Appendix 6G-6 **Snyder Method: Lynchburg Rainfall Depth Curve**



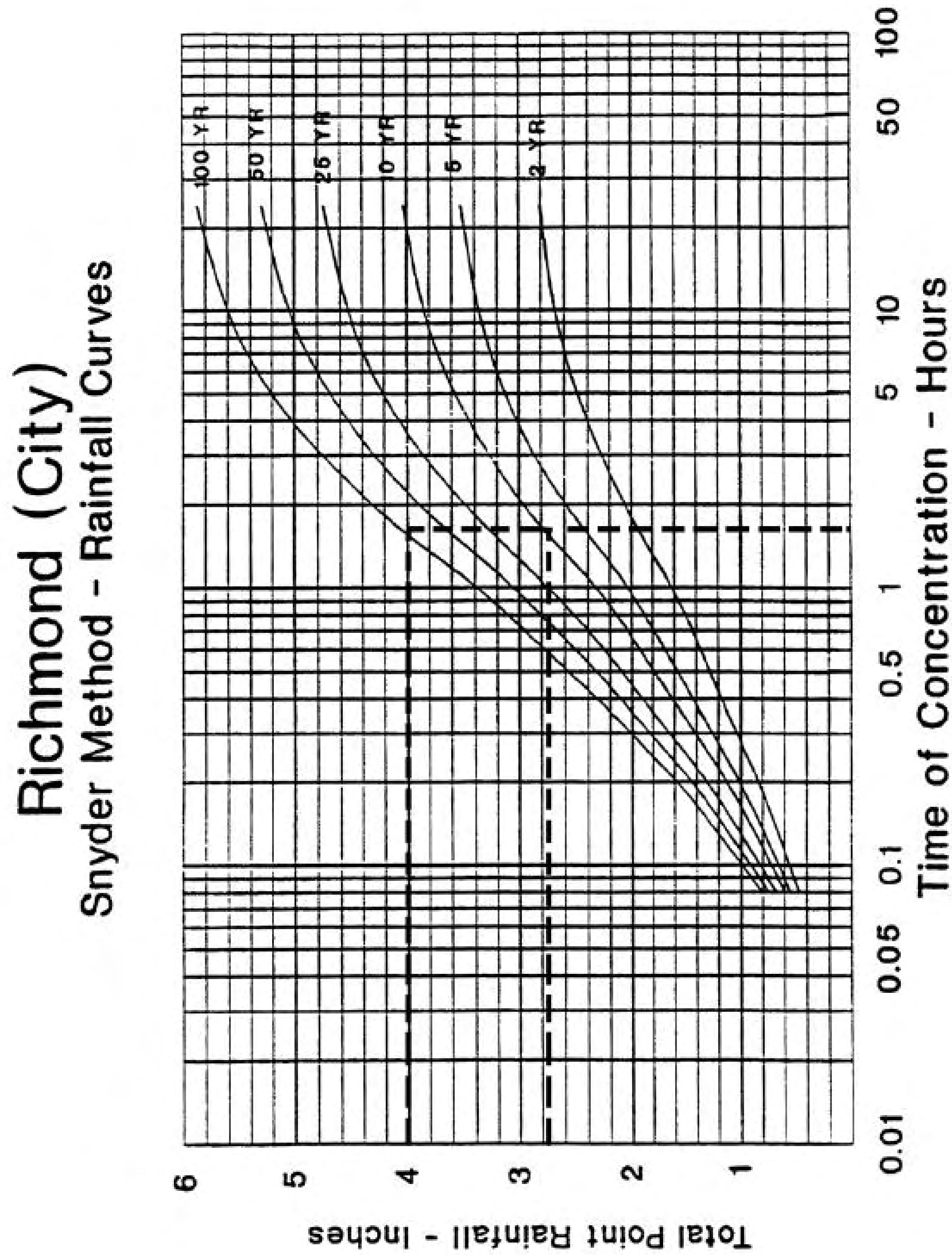
Source:

Appendix 6G-7 **Snyder Method: Norfolk Rainfall Depth Curve**



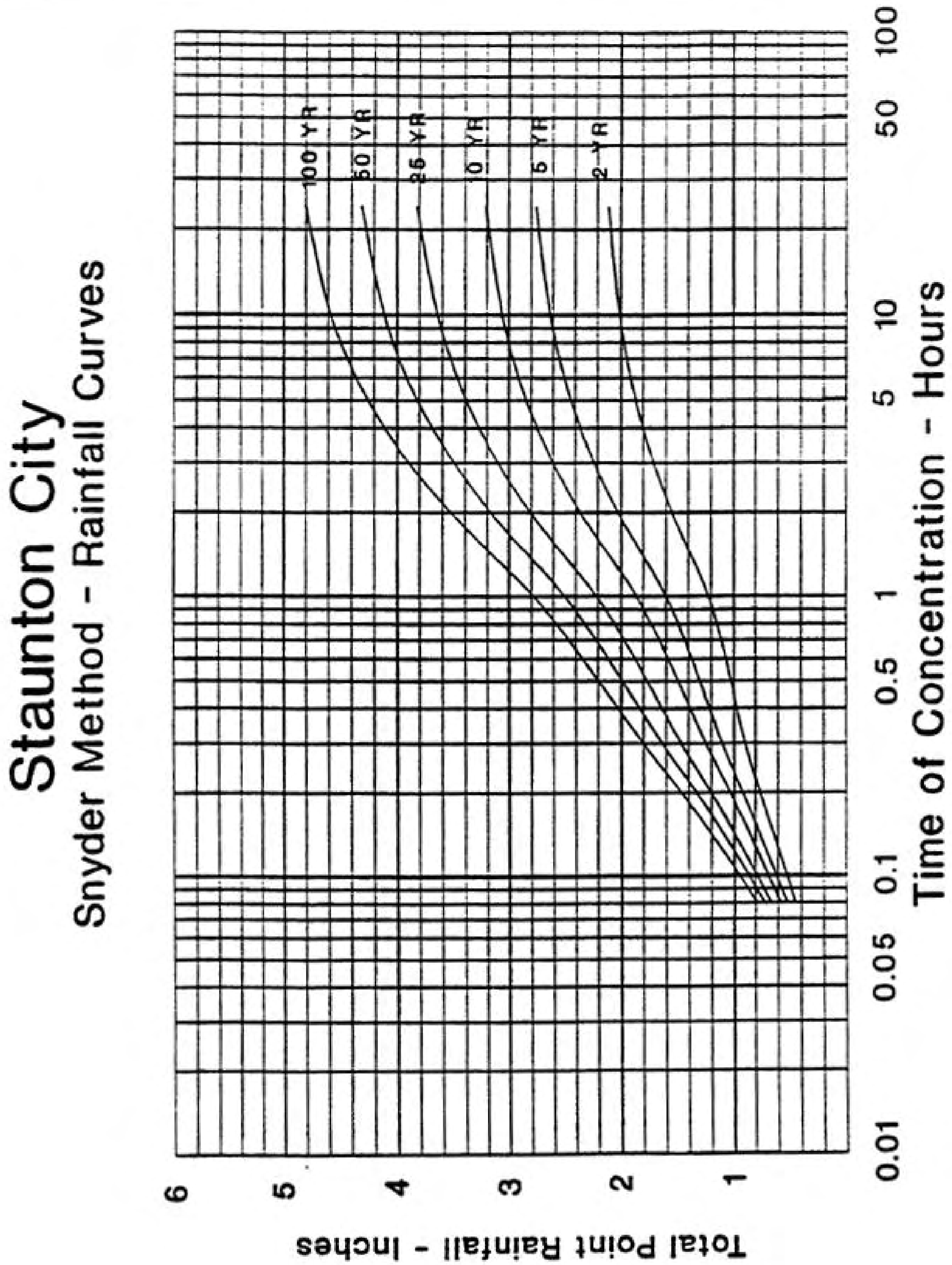
Source:

Appendix 6G-8 **Snyder Method: Richmond (City) Rainfall Depth Curve**



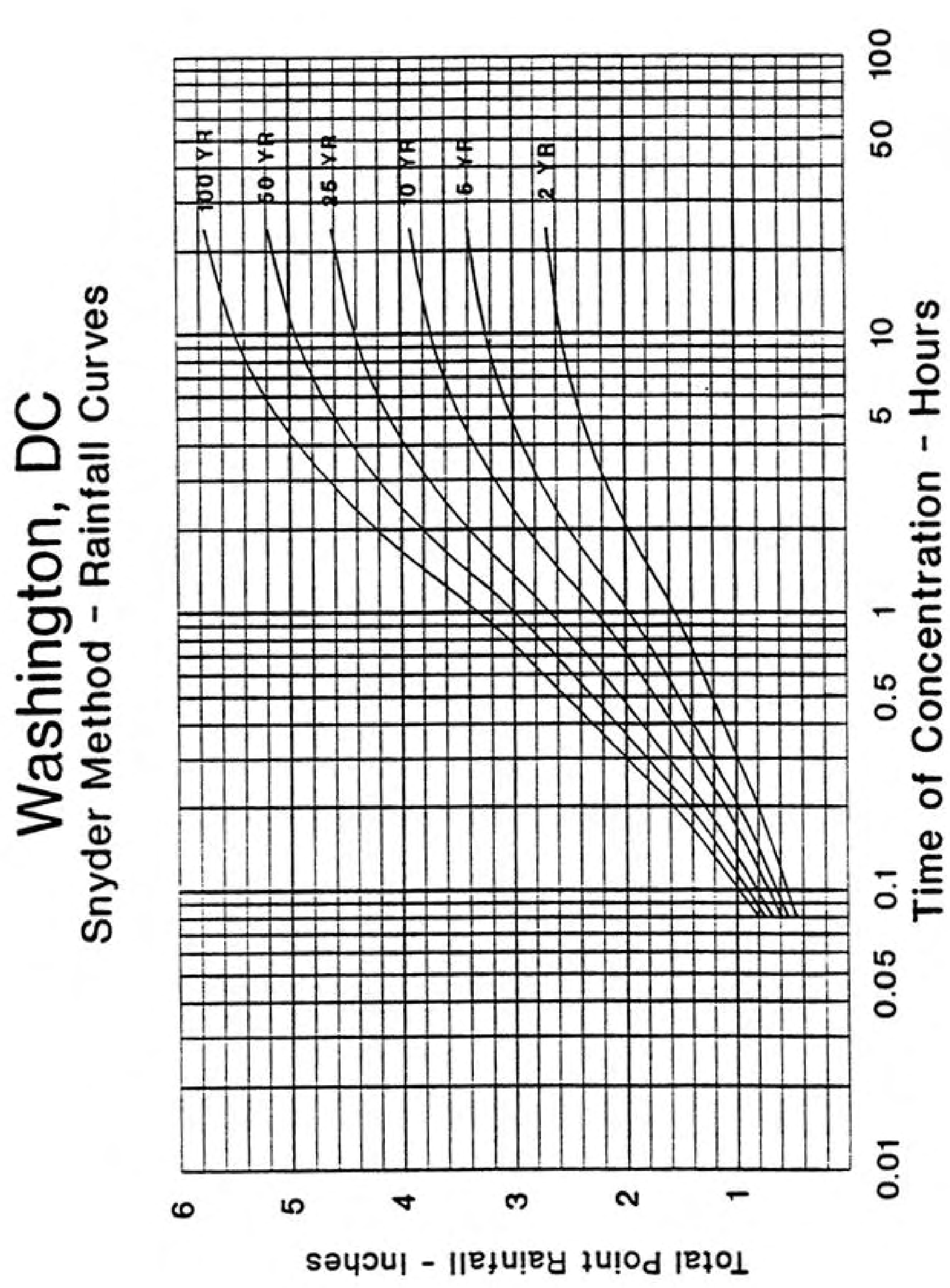
Source:

Appendix 6G-9 Snyder Method: Staunton (City) Rainfall Depth Curve



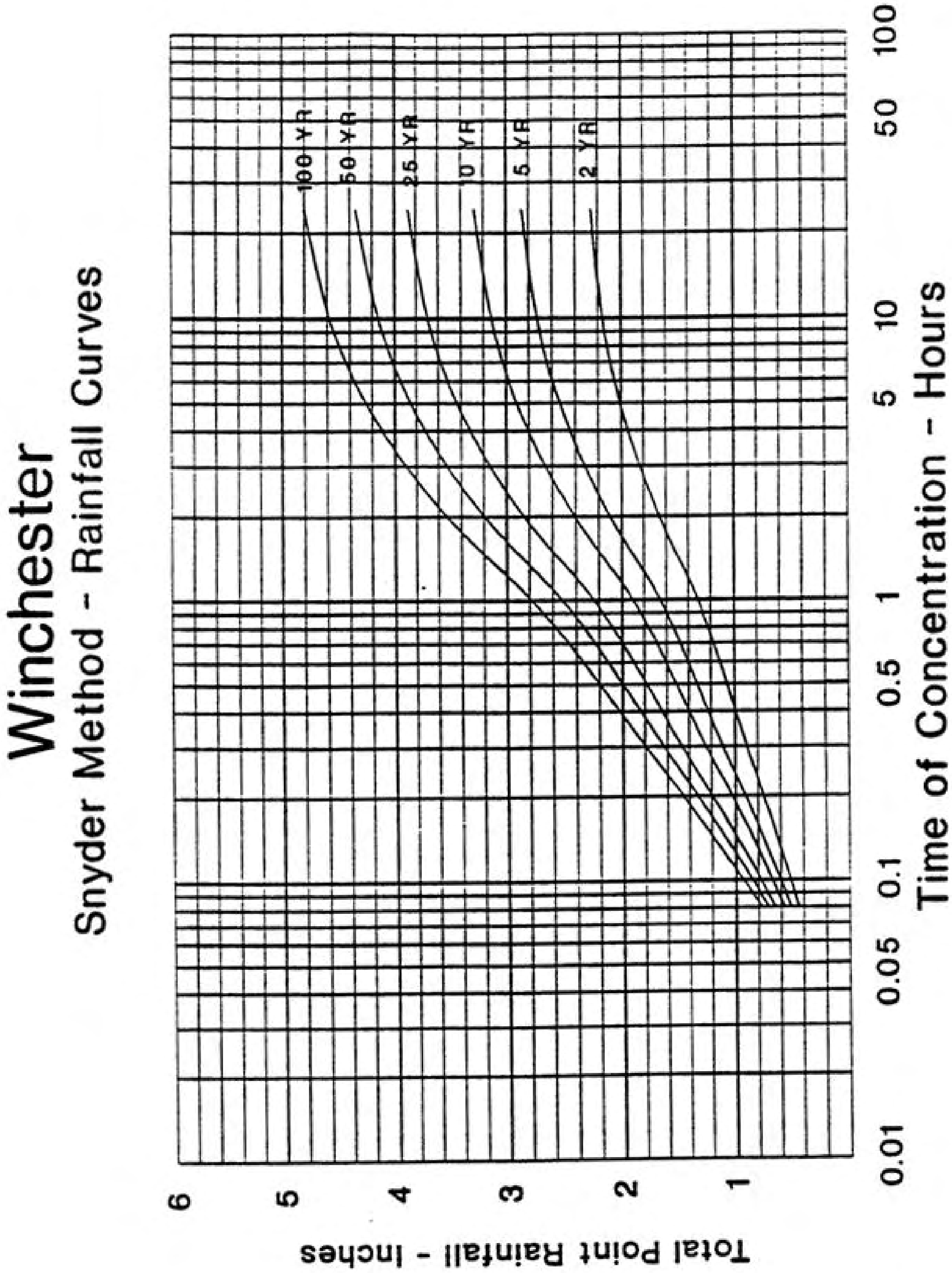
Source:

Appendix 6G-10 Snyder Method: Washington, D.C. Rainfall Depth Curve



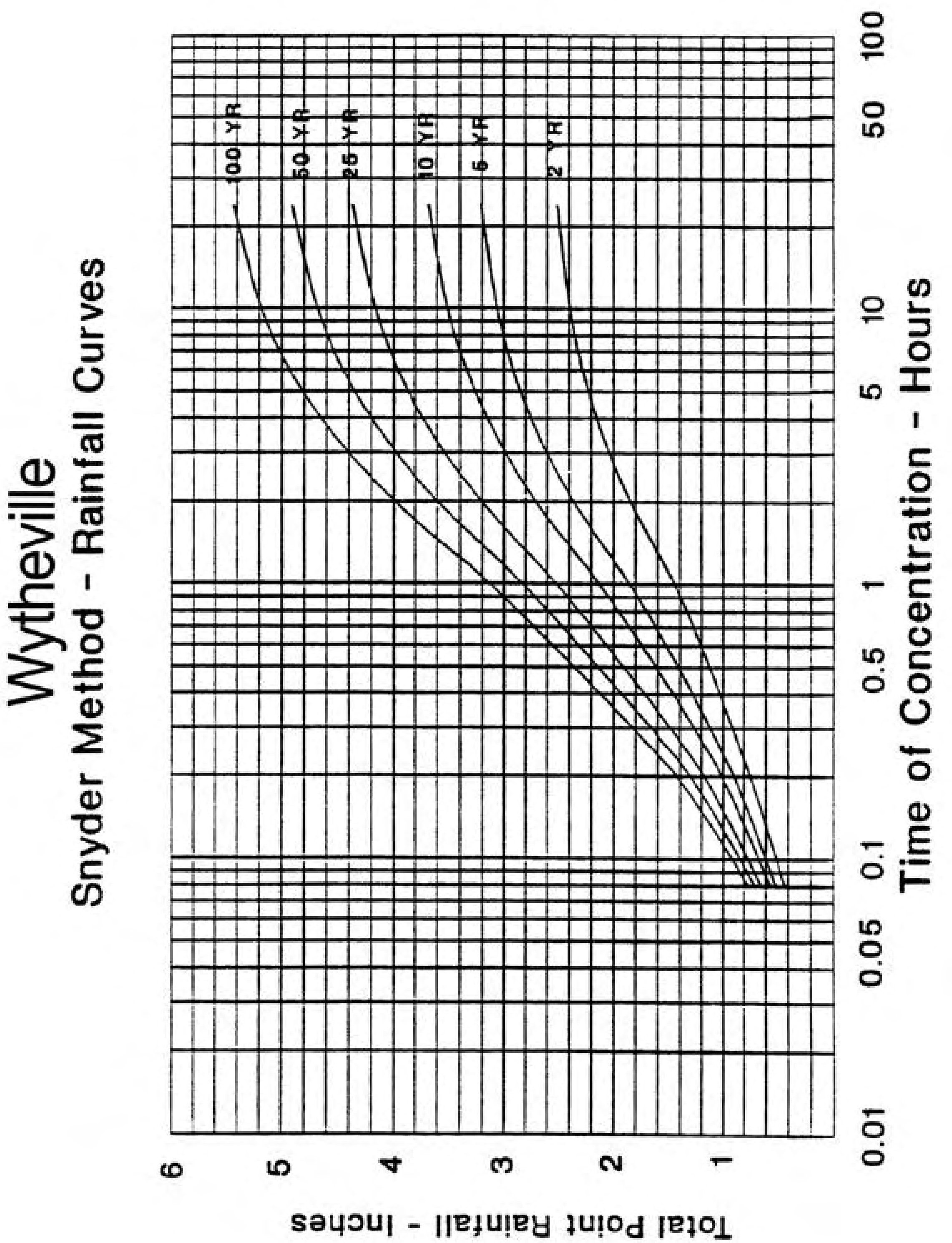
Source:

Appendix 6G-11 Snyder Method: Winchester County Rainfall Depth Curve



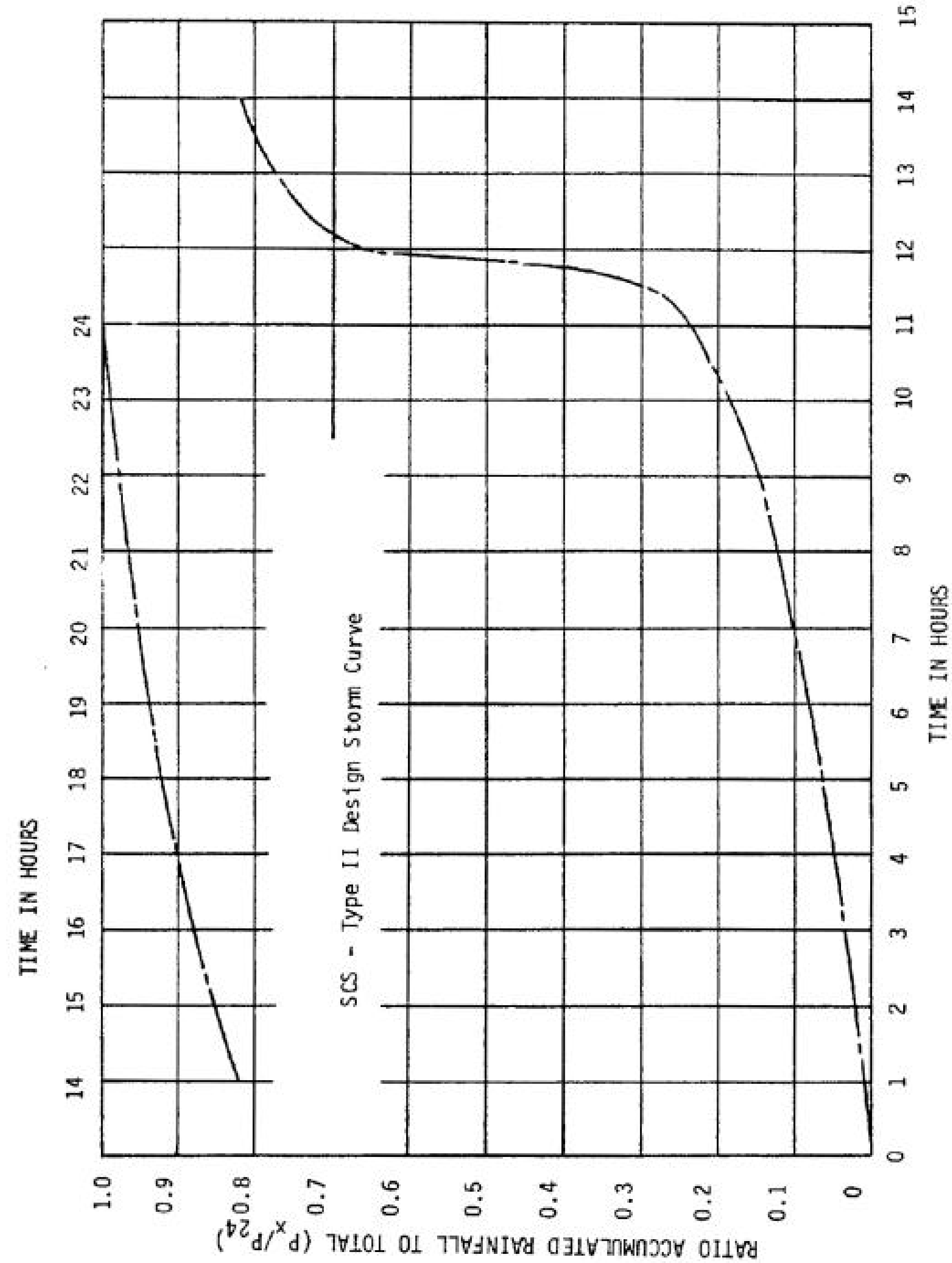
Source:

Appendix 6G-12 **Snyder Method: Wytheville Rainfall Depth Curve**



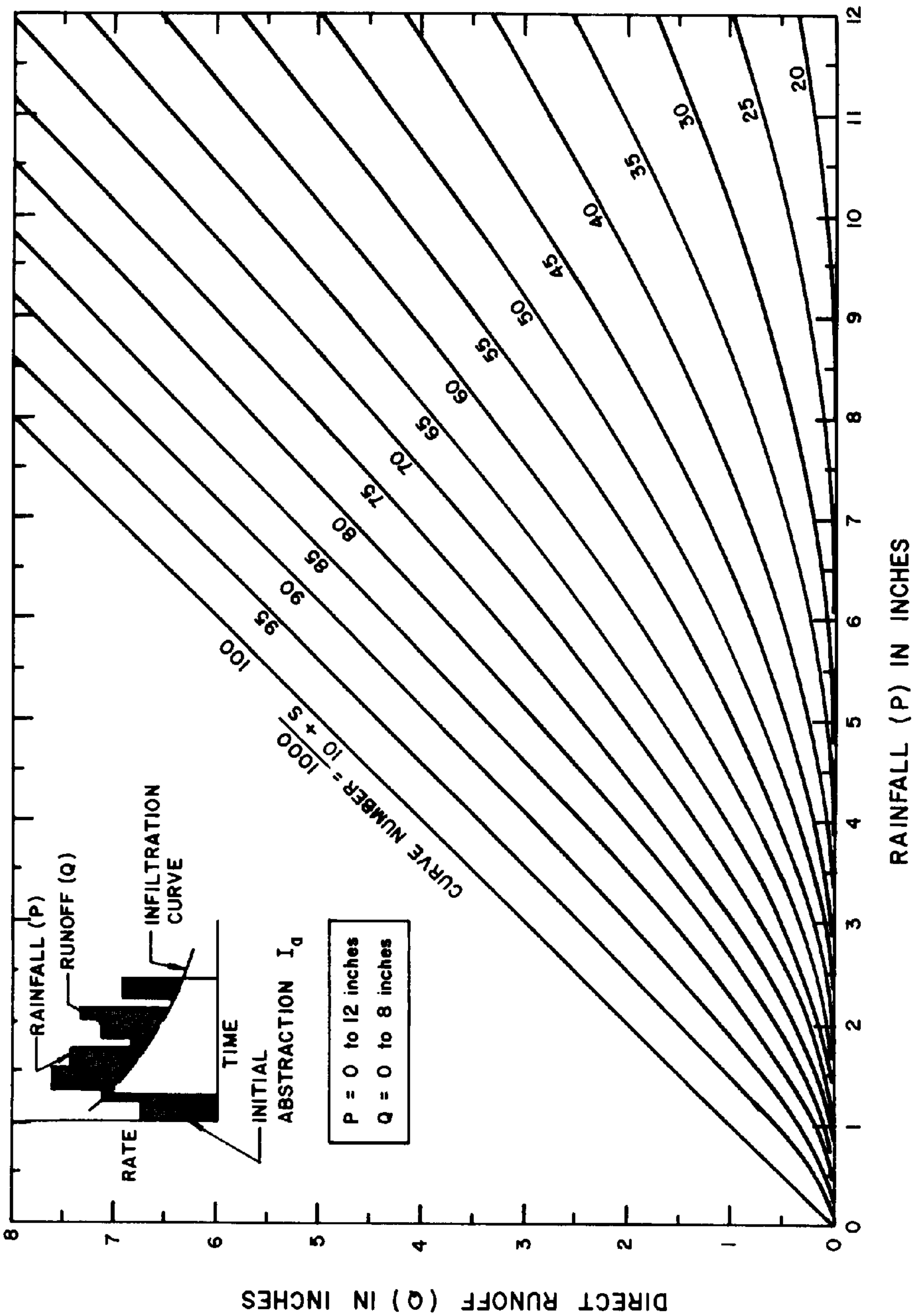
Source:

Appendix 6H-1 Type II Design Storm Curve



Source: SCS-TP-149

Appendix 6H-2 SCS Relation Between Direct Runoff, CN and Precipitation



Source: HEC No. 19, FHWA

Appendix 6H-3 Runoff Curve Numbers for Urban Areas[1]

Cover description	Curve numbers for hydrologic soil groups				
Cover type and hydrologic condition	Average percent impervious area[2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 25- to 50-mm sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
0.10 or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acre	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94
Idle lands (CNs are determined using cover types similar to those in Table 6 - 11).					

Comments:

[1] Average runoff condition, and $I_a = 0.2S$

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

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

Source: SCS-TP-149

Appendix 6H-4 Runoff Curve Numbers for Cultivated Agricultural Areas

Cover type	Treatment[2]	Hydrologic condition[3]	A	B	C	D
	Cover description		Curve numbers for hydrologic soil group			
Fallow	Bare soil	-	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row Crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C & T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
	Small grain SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
	Close-seeded SR	Poor	66	77	85	89
	or broadcast	Good	58	72	81	85
	Legumes or C	Poor	64	75	83	85
	Rotation	Good	55	69	78	83
	Meadow C&T	Poor	63	73	80	83
		Good	51	67	76	80

Comments:

[1] Average runoff condition, and $I_a = 0.2S$.
[2] Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
[3] Hydrologic condition is based on a combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or closed-seeded legumes in rotations, (d) percent of residue cover on the land surface (good > 20%) and (e) degree of roughness.
Poor: Factors impair infiltration and tend to increase runoff.
Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

Source: SCS-TP-149

Appendix 6H-5 Runoff Curve Numbers for Other Agricultural Areas

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range-continuous forage for grazing[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	[4]30	48	65	73
Woods—grass combination (orchard or tree farm)[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	[4]30	55	70	77
Farmsteads—buildings, lanes, driveways and surrounding lots	—	59	74	82	86

Comments:

[1] Average runoff condition, and $I_a = 0.2S$

[2] Poor: < 50% ground cover or heavily grazed with no mulch
Fair: 50 to 75% ground cover and not heavily grazed
Good: > 75% ground cover and lightly or only occasionally grazed

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

[4] Actual curve number is less than 30; use CN = 30 for runoff computations.

[5] CNs shown were computed for areas with 50% grass (pasture) cover. Other combinations of conditions may be computed from CNs for woods and pasture.

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

Source: SCS-TP-149

Appendix 6H-6 Conversion from Average Antecedent Moisture Condition to Dry and Wet Conditions

CN for Average Conditions	Corresponding CNs for Dry	Corresponding CNs for Wet
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
15	6	30
5	2	13

Source: USDA Soil Conservation Service TP-149 (SCS-TP-149), "A Method for Estimating Volume and Rate of Runoff in Small Watersheds," revised April 1973.

Appendix 6I-1 Joint Probability – Flood Frequency Analysis

a) Concept

One of the most frequently occurring hydrologic and hydraulic problems about which little literature is available is the problem of joint or coincidental occurrence of two or more events. If the events are caused by the same factors, then the events may be assumed to occur coincidentally. On the other hand, if the events are mutually independent and they leave probabilities of P_1 and P_2, then the probability of a coincidental occurrence is P_1 x P_2. In many cases, the events are somewhat related so that the probability of a joint occurrence is something other than P_1 x P_2.

The ideal solution to this problem would be a frequency analysis segregated with respect to the second variable. Unfortunately, unless long records are available, the number of primary events in any class interval of the secondary variables may be so limited that reliable analysis is not possible.

b) Antecedent and sequential conditions

- AMC II
- Type II Storm Distribution
- 1- to 10-day Storms

c) Outlet blockage

Tides – Fortunately the Norfolk District has developed a coincidental frequency analysis of the tide and precipitation. The results of this study are shown in Plates A-4 and A-5. A stage-frequency analysis of all tides is shown in Plate C-8. Again, fortunately, tide and precipitation are not totally independent and in fact, the severity of joint occurrences is less than if the variable s were total independent.

For design purposes the following combinations of tide and precipitation may be used.

Frequencies for Coincidental Occurrences
(Norfolk Harbor)

10-Year Design		100-Year Design	
Tide	Ppt.	Tide	Ppt.
6.3	0.0	8.4	0.00
2.5	1-year	5.4	1-year
2.0	5-year	4.2	5-year
1.5	10-year	1.5	100-year

The combinations of events at the extremities of the tables will usually be critical and often the intermediate combinations may be ignored.

d) Channel Flow

For the case of a tributary stream its relative independence may be qualitatively evaluated by a comparison of its drainage area with that of the mainstream. A short duration storm which causes peak discharge on a small basin may not be critical for a larger basin. Also, it may safely be assumed that if the same storm causes peak discharge on both basins, the peaks will be out of phase.

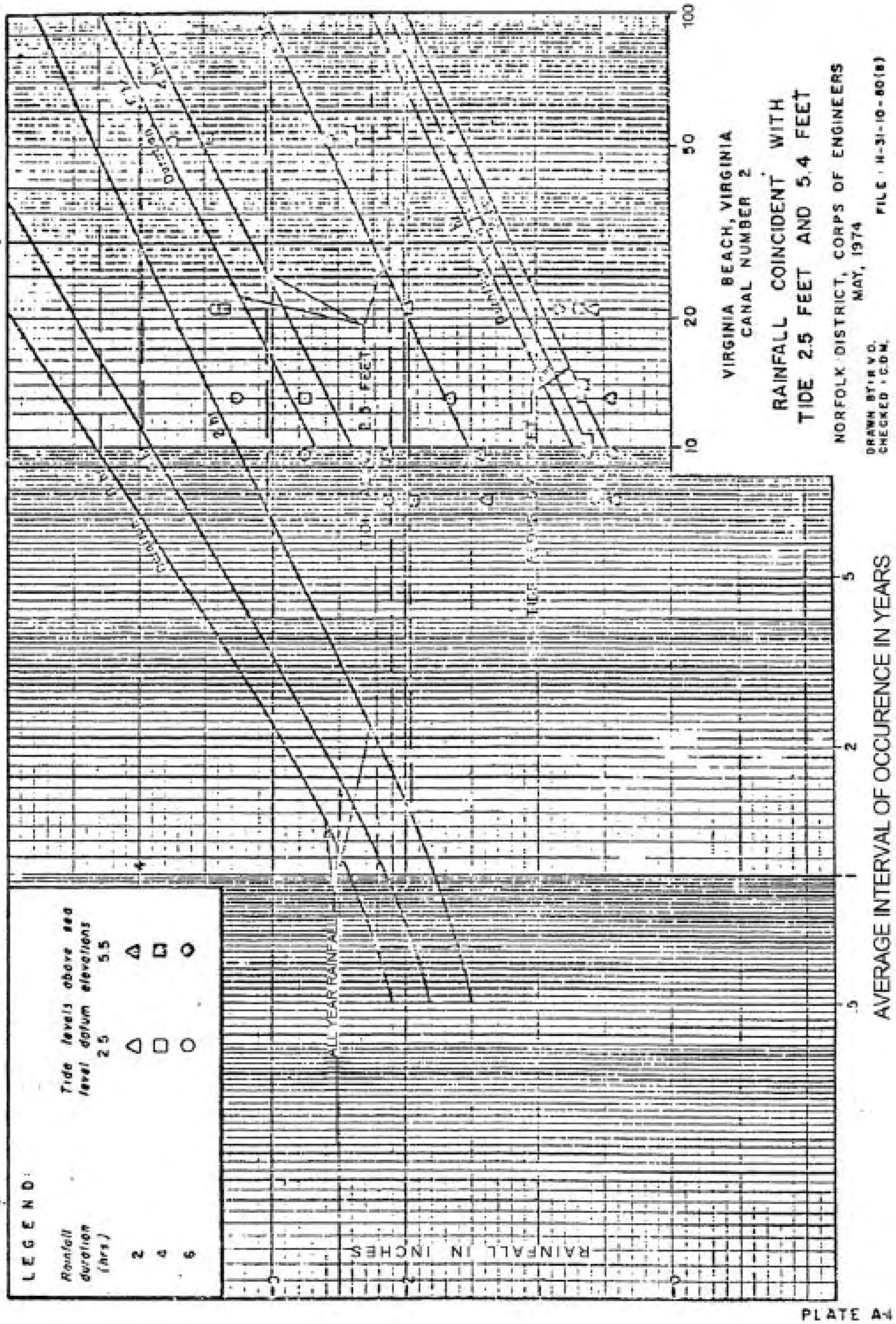
As an aid to the evaluation of joint probabilities, the following criteria are suggested:

Joint Probability Analysis

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

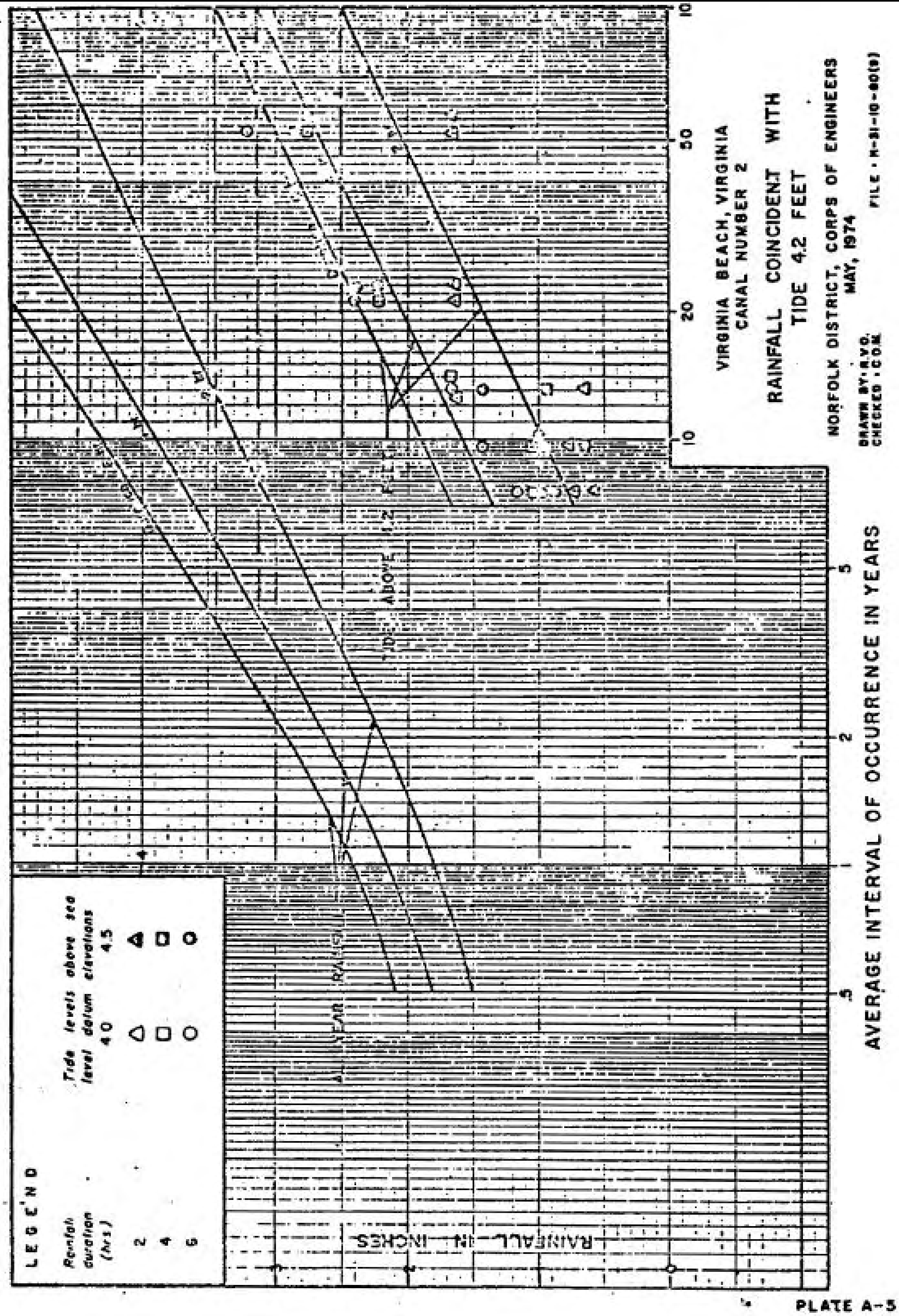
The design frequencies suggested in the table represent the extreme combinations of frequencies for each event. Experience has shown that these combinations will usually be critical. For example, the combinations of frequency for a 1,000 to 1 area ratio and a 100-year design frequency area 10- and 100-year frequencies. A 20- and 50-year frequency combination would have the same joint frequency, but the stage elevations, for instances, for these combinations will usually be less than the stages for the given combinations. Maximum stages should be reached when the 100-year storm on the tributary is coincident with the 10-year storm on the mainstream (greatest channel flow with moderate backwater effects) or when the 10-year storm on the tributary is coincidental with the 100-year storm on the mainstream (greatest backwater effect with moderate channel flow). Both cases must be analyzed.

Appendix 6I-2 **Rainfall Coincident with Tidal EL. 2.5 FT and 5.4 FT**



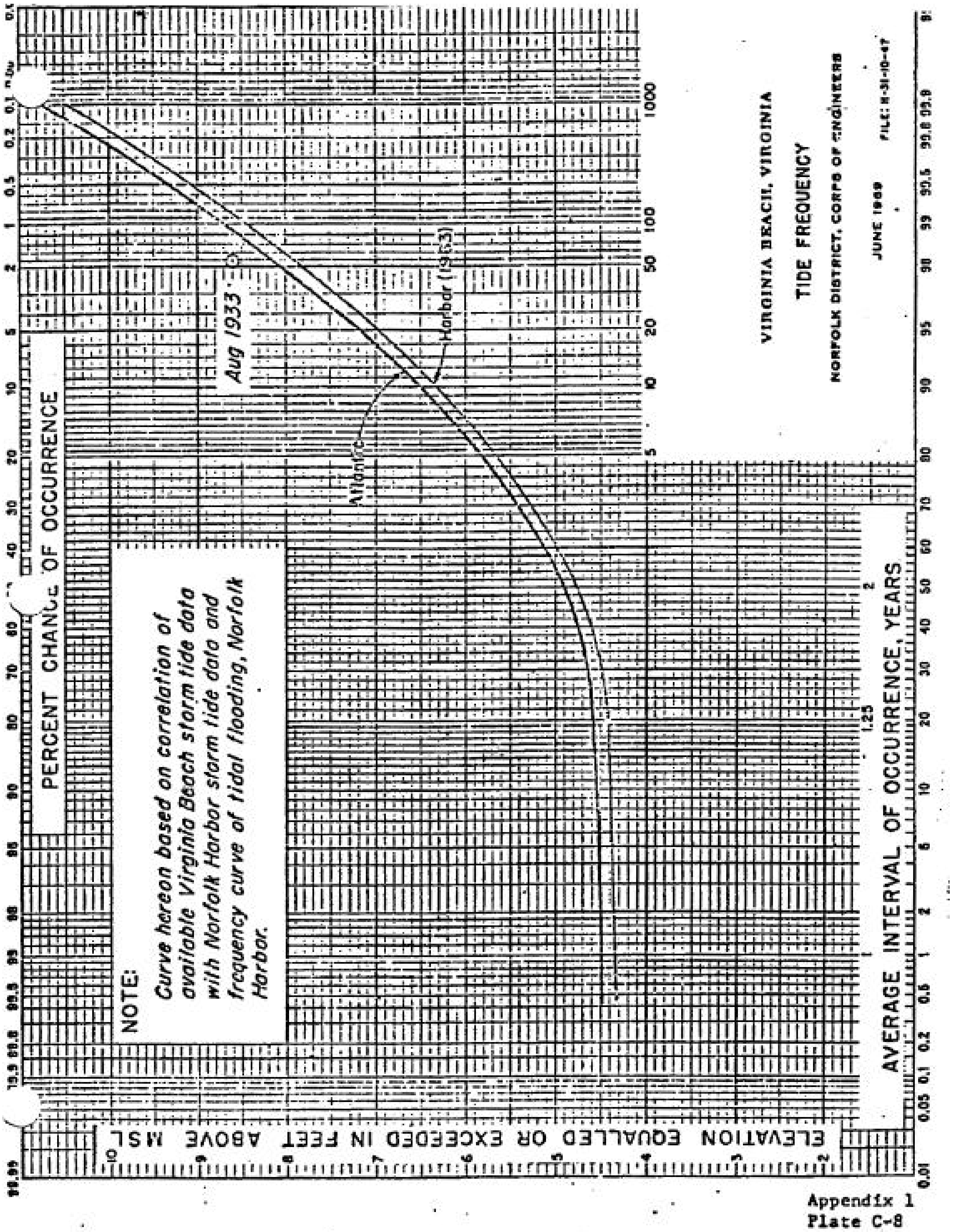
Source: U. S. Army Corps of Engineers

Appendix 6I-3 Rainfall Coincident with Tidal EL. 4.2 FT



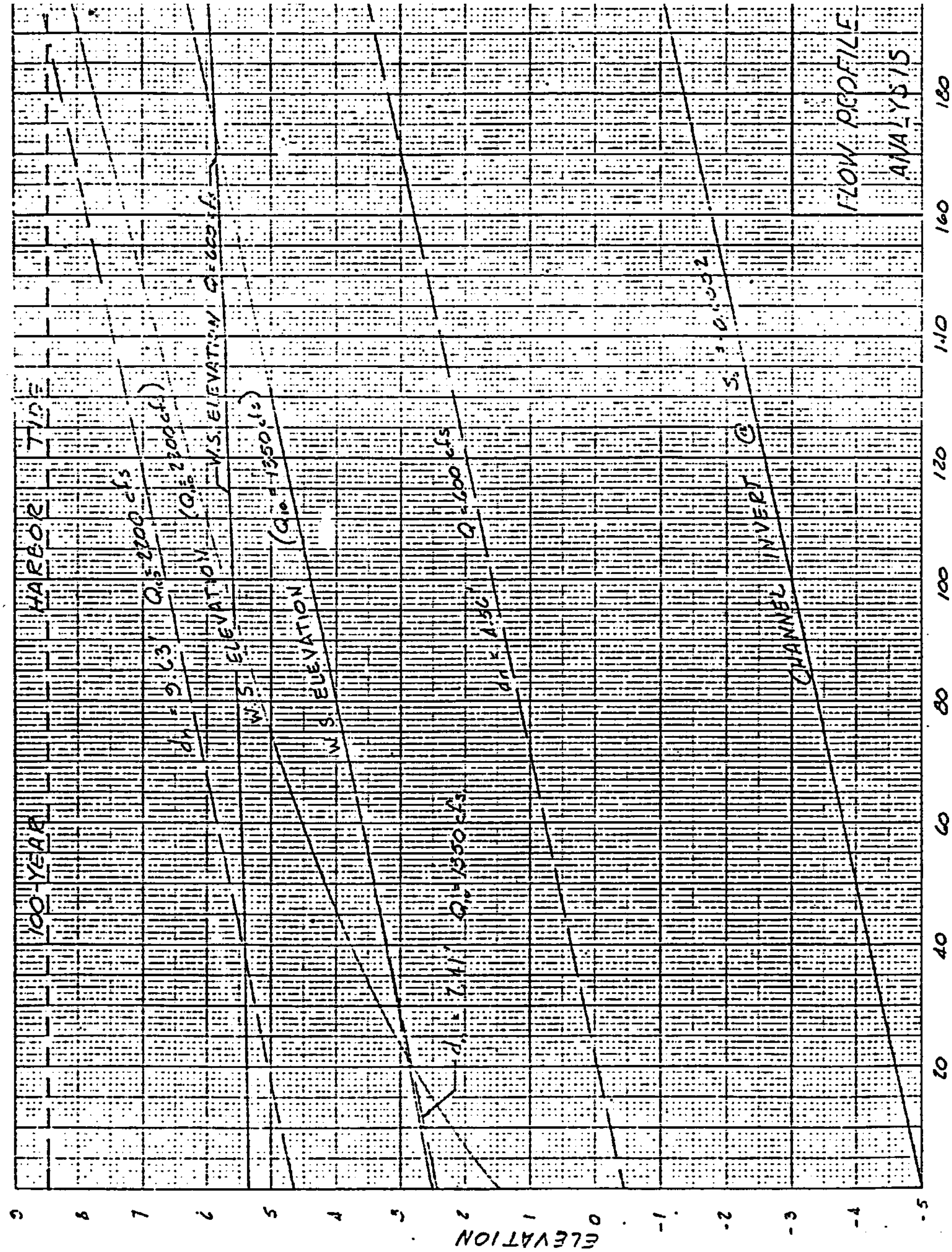
Source: U. S. Army Corps of Engineers

Appendix 6I-4 Tide Frequency, Virginia Beach



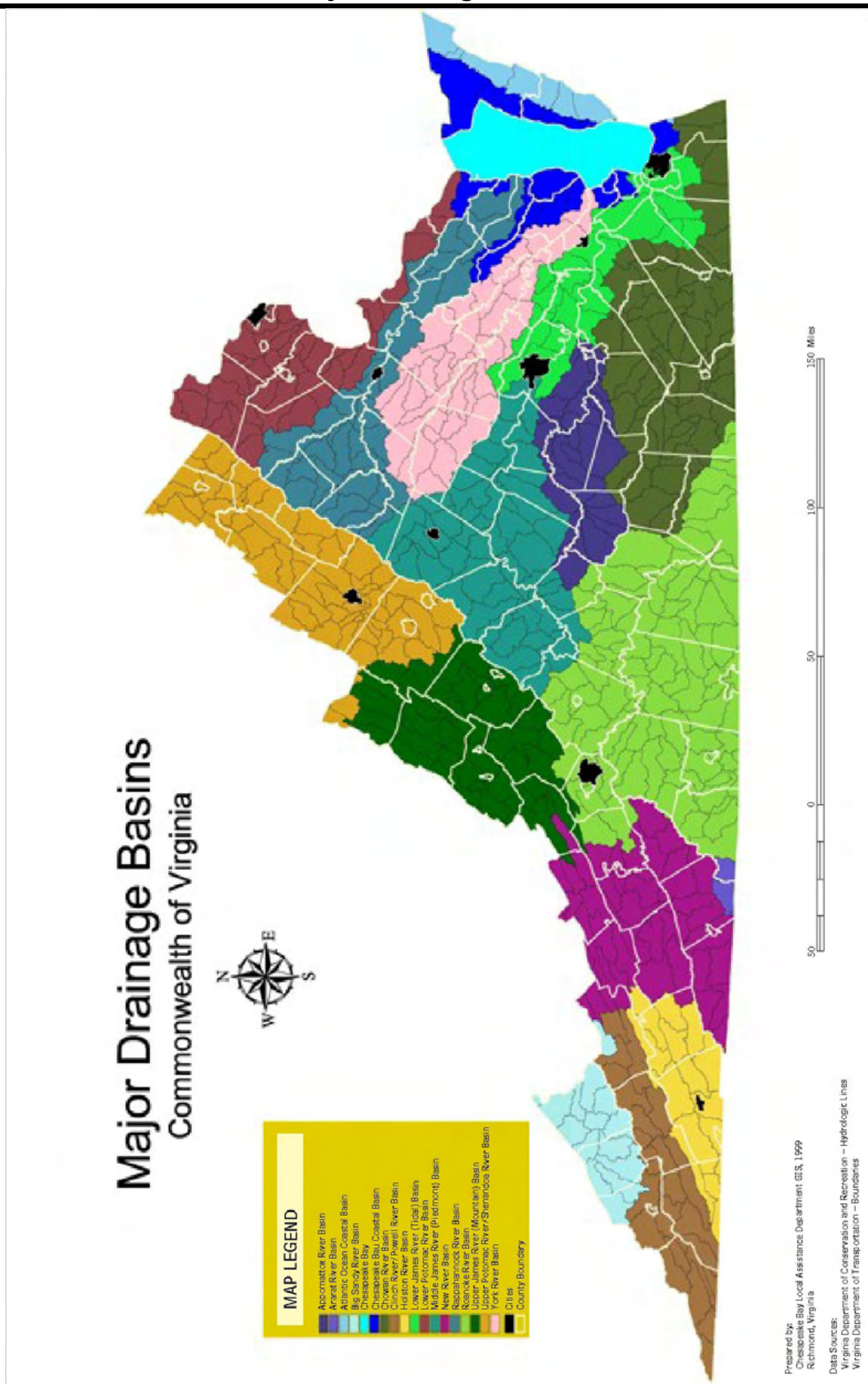
Source: U. S. Army Corps of Engineers

Appendix 6I-5 Flow Profile Analysis



Source: U. S. Army Corps of Engineers

Appendix 6J-1 Major Drainage Basins



Source: CBLAD

Chapter 7 — Ditches and Channels

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Chapter 7 - Ditches and Channels

7.1 Introduction and Objective

The objective of ditches and open channels is to convey stormwater runoff from, through, 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.

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.dcr.state.va.us/>) 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
- 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

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.

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 cubic feet per second (cfs) or less in minor channels. Minor channels usually parallel the highway embankment and are within the highway right-of-way. 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 off-site areas, and convey that flow to an adequate discharge point on- or off-site. 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 a well-defined, natural or man-made off-site receiving channel, pipe, or storm sewer system. If there is no well-defined off-site 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-year 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-year storm without overtopping its banks, and bed or bank erosion shall not occur due to a 2-year storm, (3) Pipes and storm sewer systems shall contain the 10-year storm.

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-year 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 off-site 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-year storm to increase when outfall is into a natural channel, or will not cause the pre-development rate from a 10-year 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.

All on-site channels must be adequate.

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.

7.3.2 Minor Channels (Roadside Ditches)

Minor channels are normally V-shaped and sometimes trapezoidal in cross section and lined with grass or a special protective lining. They are usually designed to convey the 10-year discharge, and to resist erosion from the 2-year discharge. Higher design discharges may be necessary when the channel intercepts off-site 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.

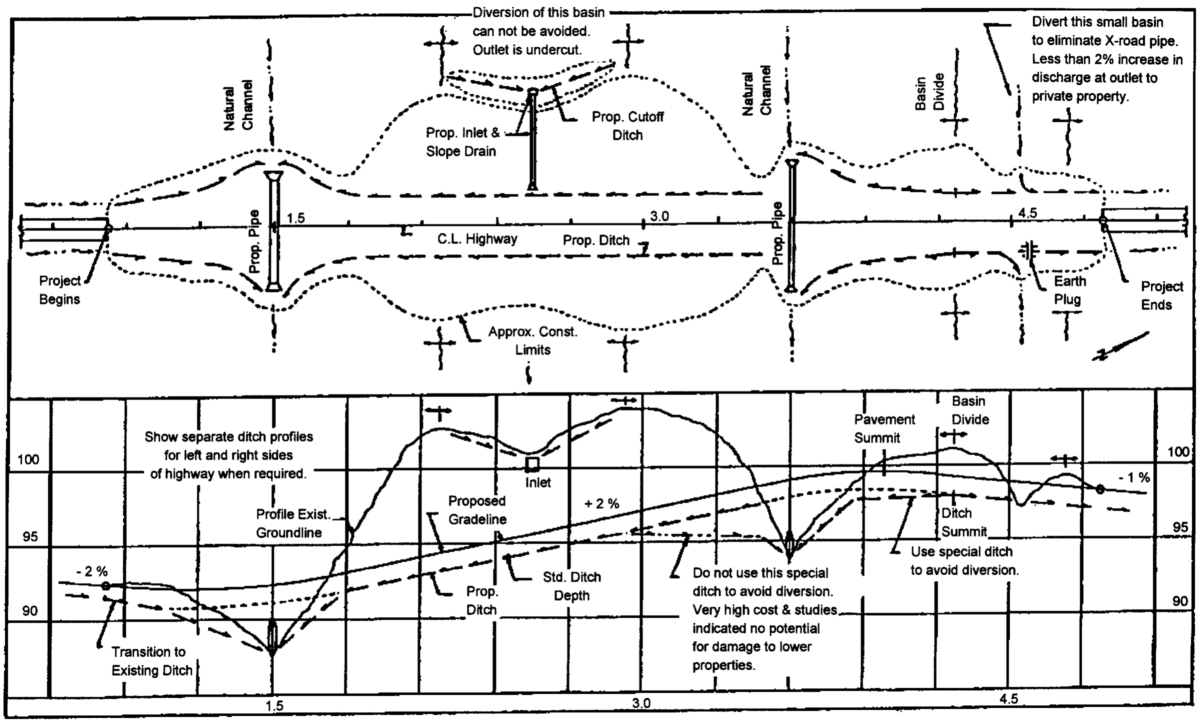


Figure 7-1. Sample Roadside Ditch Plan and Profile

7.3.3 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-year storm but may be greater based upon risk. Erosive protection is based on the 2-year 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.

7.3.4 Natural Channels

The hydraulic effects of floodplain encroachments should be evaluated over a full range of frequency-based peak discharges from the 2-year through the 500-year recurrence intervals on any major highway facility, as deemed necessary by the Department.

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 natural channel design may be necessary.

7.4 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 usually be 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.

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, 1970; 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.

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" (<http://www.fhwa.dot.gov/bridge/hec20.pdf>) and "Highways in the River Environment" (<http://www.fhwa.dot.gov/bridge/hire1990.pdf>) provide additional and more detailed information on making such assessments.

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.

A. Where the 100-year 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-year 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

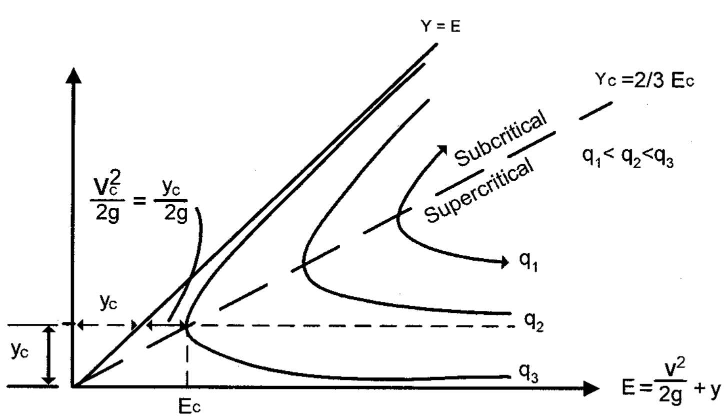
Specific energy (E) is defined as the energy head relative to the channel bottom. If the channel slope is less than 10 percent and the streamlines are nearly straight and parallel (so that the hydrostatic assumption holds), the specific energy (E) becomes the sum of the depth and velocity head:

$$E = d + \alpha \frac{V^2}{2g} \qquad (7.1)$$

Where:

- d = Depth of flow, ft
- α = Velocity distribution coefficient (see Equation 7.2)
- v = Mean velocity, fps
- g = 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.



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 nonuniform distribution of velocities in a channel section, the velocity head of an open channel is usually greater than the average velocity head computed as $(Q/A_t)^2/2g$. A weighted average value of the velocity head is obtained by multiplying the average velocity head, above, by a velocity distribution coefficient (α) defined as:

$$\alpha = \frac{\sum_{i=1}^{n} \frac{K_i^3}{A_i^2}}{\frac{K_t^3}{A_t^2}} \qquad (7.2)$$

Where:

K_i = Conveyance in subsection (see Equation 7.8)
K_t = Total conveyance in section (see Equation 7.8)
A_i = Cross-sectional area of subsection, ft^2
A_t = Total cross-sectional area of section, ft^2
N = 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:

$$F_r = \frac{V}{\sqrt{\frac{gH_D}{\alpha}}} \qquad (7.3)$$

Where:

α = Velocity distribution coefficient
V = Mean velocity = Q/A, fps
g = Acceleration due to gravity, 32.2 ft/s^2

H_D = Hydraulic depth = $\left(\frac{A}{T}\right)$, ft
(For rectangular channels the hydraulic depth is equal to the flow depth.)
A = Cross-sectional area of flow, ft^2
T = Channel top width at the water surface, ft
Q = Discharge, cfs

This expression for Froude number applies to channel flow at <u>any</u> 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
- $F_r \geq 1$, Supercritical Flow

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:

$$\frac{\alpha Q^2}{g} = \frac{A^3}{T} \tag{7.4}$$

Where:

α = Velocity distribution coefficient
Q = Discharge, cfs
g = Gravitational acceleration, 32.2 ft/s^2
A = Cross-sectional area of flow, ft^2
T = Channel top width at the water surface, ft

When flow is at critical depth, 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:

$$Q = \frac{A_1}{V_1} = \frac{A_2}{V_2} \tag{7.5}$$

Where:

Q = Discharge, cfs
A = Cross-sectional area of flow, ft^2
V = 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:

$$V = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \tag{7.6}$$

Where:

V = Velocity, fps
n = Manning's roughness coefficient
R = Hydraulic radius = A/P, ft
A = Flow area, ft^2
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:

$$Q = \frac{1.486}{n} AR^{\frac{2}{3}} S^{\frac{1}{2}} \qquad (7.7)$$

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.

7.4.4.2.10Conveyance

In channel analysis, it is often convenient to group the channel properties in a single term called the channel conveyance (K):

$$K = \frac{1.486}{n} AR^{\frac{2}{3}} \qquad (7.8)$$

Then Equation 7.7 can be written as:

$$Q = KS^{\frac{1}{2}} \qquad (7.9)$$

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 (α).

7.4.4.2.11Energy Equation

The energy equation (Bernoulli's Equation) expresses conservation of energy in open channel flow defined as energy per unit weight of fluid. This dimensions of length 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:

$$h_1 + \alpha_1 \frac{V_1^2}{2g} = h_2 + \alpha_2 \frac{V_2^2}{2g} + h_L \quad (7.10)$$

Where:

h_1 = Upstream water surface elevation, ft.
h_2 = Downstream water surface elevation, ft.
α = 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.

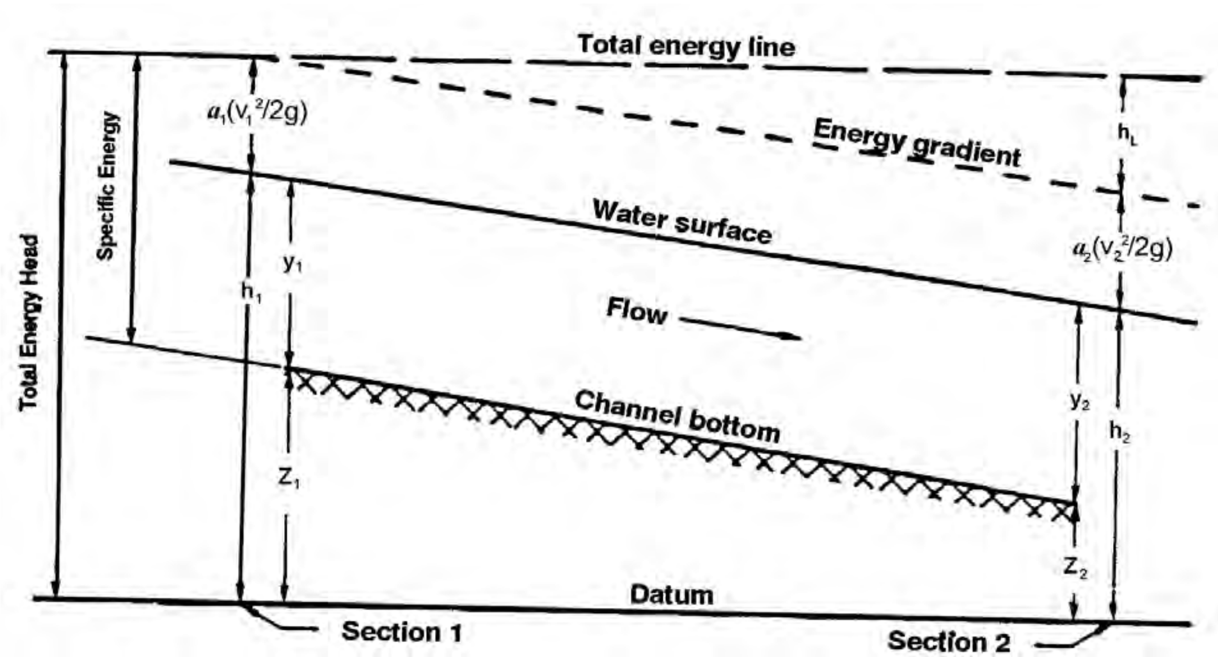


Figure 7-3. Terms in the Energy Equation

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 α, 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.

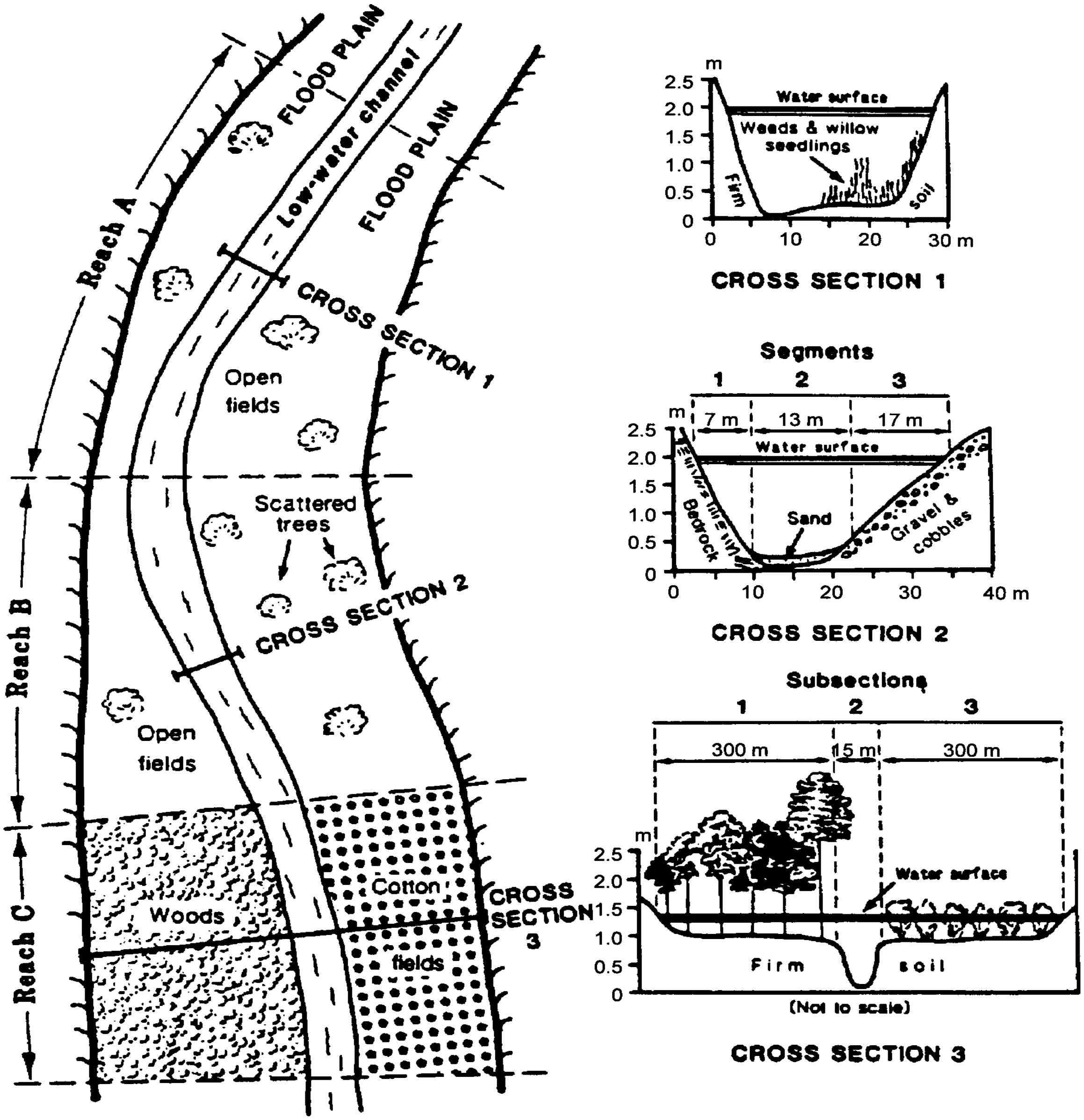


Figure 7-4. Hypothetical Cross Section Showing Reaches, Segments, and Subsections Used in Assigning n-Values

7.4.4.3.2 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.

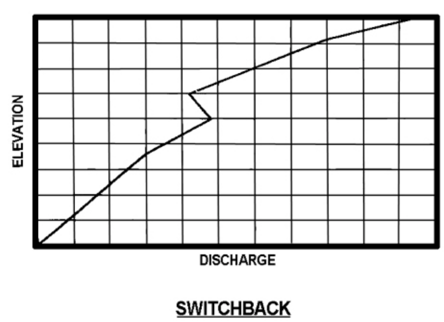


Figure 7-5. Example of Switchback Phenomenon

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.

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-year) 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 FHWA/USGS program WSPRO or the Corps of Engineers HEC-2 or HEC-RAS models 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.

$$d_1 + \alpha_1 \frac{V_1^2}{2g} + z_1 = d_2 + \alpha_2 \frac{V_2^2}{2g} + z_2 + h_L \qquad (7.11)$$

Where:

d = Depth of flow, ft
V = Mean velocity, fps
z = Elevation of flow line, ft
h_L = Total head loss, ft

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.

$$h_L = h_f + h_o \qquad (7.12)$$

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.

$$h_f = LS_A \qquad (7.13)$$

Where:

L = Length of channel segment, ft.
S_A = Average energy slope of channel segment, ft./ft.

$$S_A = \frac{S_1 + S_2}{2} \qquad (7.14)$$

Where:

S_1 = Energy slope at Section 1, ft./ft.
S_2 = 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.

$$S = \frac{Q^2 n^2}{2.25 R^{\frac{4}{3}} A^2} \qquad (7.15)$$

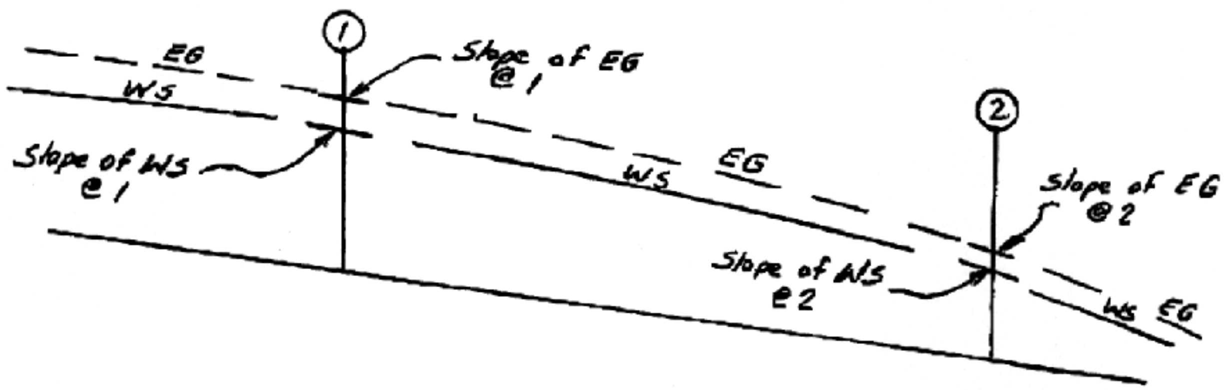


Figure 7-6. Energy Slope between Two Channel Sections

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.

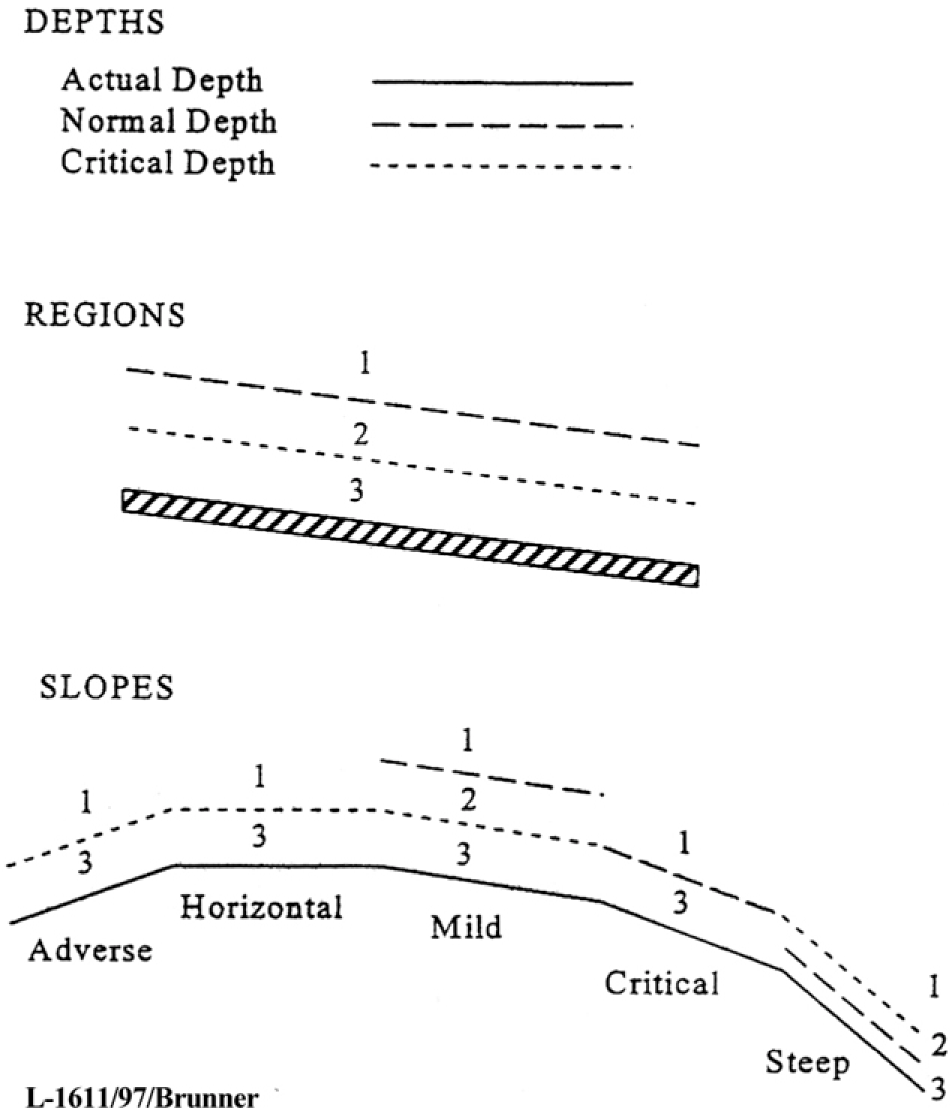


Figure 7-7. Notation for Classifying Water Surface Profiles

GRADUALLY VARIED FLOW

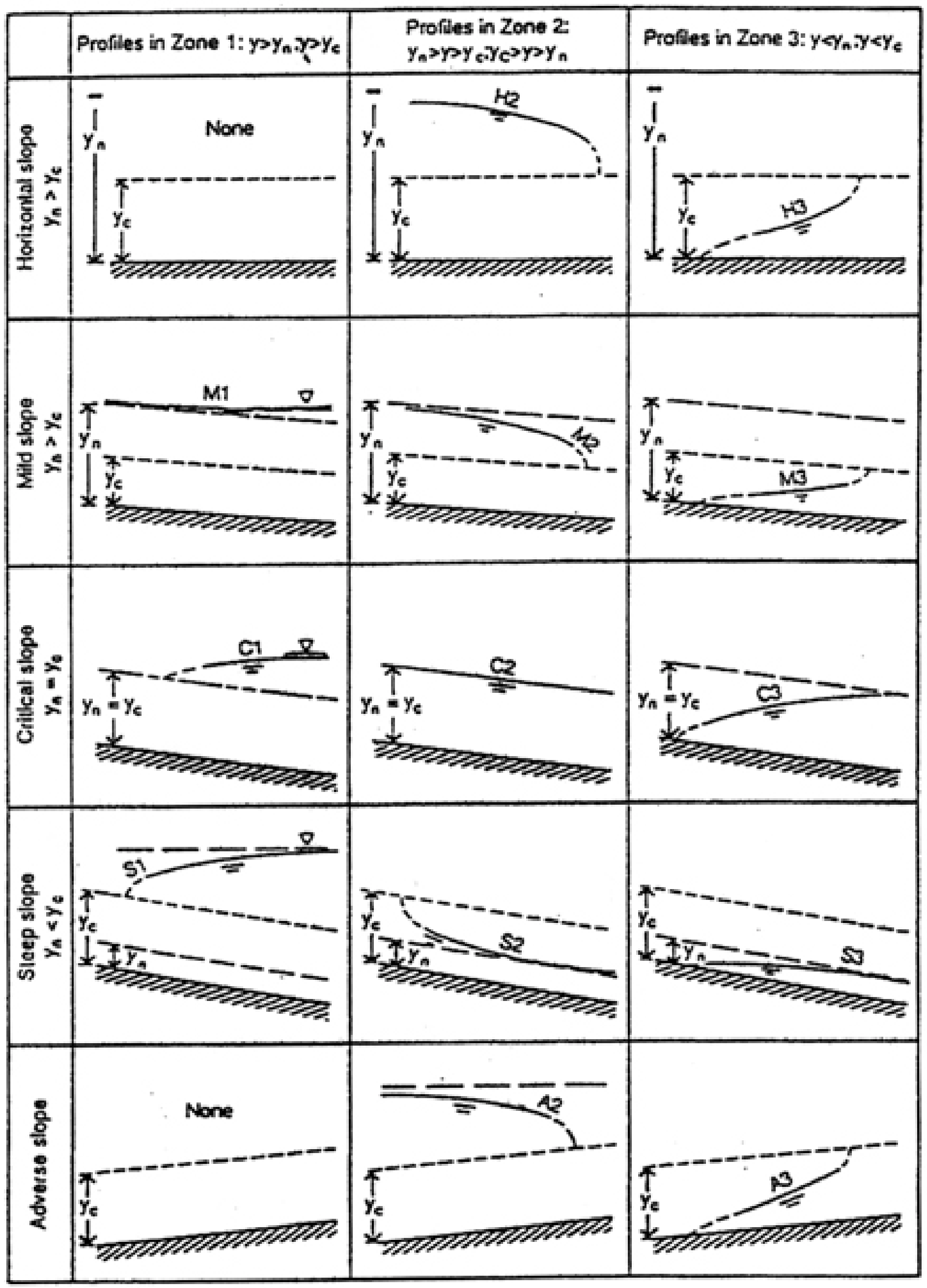


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Figure 7-8. Types of Backwater Profiles

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 Sediment Routing Alluvial River Simulation) Computer Model, developed by the National Cooperative Highway Research Program and the FHWA (Molinas, 1994). 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."

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 protective covering (VDOT Standard EC-2) or a permanent soil stabilization mat (VDOT Standard EC-3, Type A, or Type B). 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 used on steep grades due to high velocities 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 should provide any design details that are needed to protect the undercutting of the lining and preserve the integrity of the ditch.

Two methods are commonly applied to determine whether a channel is stable from an erosion standpoint: Permissible Velocity and Tractive Force. These methods are described 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 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 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 percent higher than 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 is widely used, the tractive force (permissible shear) method provides a more physically based and realistic model for particle detachment and erosion processes (FHWA's HEC-15 , “Design of Roadside Channels with Flexible Linings”). It is recommended that the permissible velocity method be used for roadside ditches and that the tractive force method be used for major channel analyses.

7.4.6.2 Tractive Force

The Tractive Force Method takes into account the physical factors of bed material, channel geometry, depth, and velocity of flow. The method is confined to non-cohesive materials for which the permissible tractive force is related to particle size and shape and the sediment load in the water. Particle diameters are based on the equivalent spherical volume and assume 75 percent 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 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.

$$K_1 = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} \qquad (7.16)$$

Where:

θ = Side slope angle (measured from the horizontal), deg.

ϕ = Natural angle of repose of material under consideration (measured from the horizontal), deg.

Note: The descriptions for θ and ϕ (above) are reversed in Appendix 7E-8.

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.

The average tractive force formula is:

$$\tau_o = 62.4RS_o \qquad (7.17)$$

Where:

τ_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.

$$\tau_{max} = 62.4dS_o \qquad (7.18)$$

Where:

d = Depth of flow, ft

A convenient form that can be used for manually performing this computation is found in Appendix 7B-3.

7.4.6.3 Geotextile Channel Linings

Geotextile materials designated as Standard EC-3 (Type A and B) Soil Stabilization Mat are used for protective linings in ditches. Standard EC-3 Soil Stabilization Mat is intended to be used as a protective ditch lining material to be applied when the design velocity exceeds the allowable velocity for Standard EC-2, Jute Mesh.

When the design velocity exceeds the allowable velocity for Standard EC-3, a paved (or riprap) lining is required.

The Standard EC-3 (Type C) Soil Stabilization Mat 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

Standard EC-3 Type A is to be employed where the design velocity in the ditch is within the range of 4 to 7 feet per second (fps).

Standard EC-3 Type B is to be employed where the design velocity is within the range of 7 to 10 fps.

A Manning's n-value of 0.05 should be used with Standard EC-3.

Typically, the use of Standard EC-3 Type A should begin at the point where the flow velocity exceeds 4 fps (velocity is assumed to be for flow in an EC-2 lined channel) and change to EC-3, Type B at the appropriate point, until the design velocity exceeds 10 fps or until the velocity decreases so that the use of a ditch lining can be discontinued.

Experience has shown that the installation of this material is particularly critical. The design drawings should stress that the material must be installed in strict accordance with the standard drawings and manufacturer's specifications.

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 percent of the total mass. The size of the 50 percent 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 percent stone is referred to by its W_{50}. The Department has six standard riprap classifications whose 50 percent 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 per cubic yard (148 lbs. per cu. ft.) for plan estimating purposes, unless otherwise specified by the District Administrator.

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 percentages of voids for all VDOT standard riprap classes is 25 percent 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

It is VDOT practice to place riprap over a geotextile bedding in situations where:

- The riprap is being placed on bridge spill slopes
- The riprap is being placed on fill or other loose material
- The impinging velocity is 10 fps or higher (with the Manning n-value having been based on the riprap)
- The Materials Division has recommended it
- It is required for environmental reasons

When it is necessary to use geotextile bedding under the riprap, 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 inches thick

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. A computer program developed by the FHWA is available from various sources such as the National Technical Information Service (NTIS) and McTrans, for sizing riprap and other flexible linings in accordance with the 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.

7.5 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.

7.5.1 Documentation Requirements

These items establish a minimum requirement for all channels except roadside ditches. Also, see Documentation Section 3.3.1. 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.

7.5.2 Roadside Ditches and Minor Channels

7.5.2.1 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.

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 0.2 percent

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), or

B. Use the method of allowable tractive force (HEC-15)

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

7.5.2.2 VDOT Method for Design of Roadside Ditch Linings

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 several years.

Before the computational analysis can be performed, a soils report from the Materials Division 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 <u>each</u> <u>ditch</u> 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.

7.5.2.2.1 Design Procedure for Roadside Ditch Linings Using VDOT Method

A computer software program "RDDITCH" is available upon request from VDOT to assist in performing these computations, or manual 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.

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.

7.5.2.2.2 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.

7.5.2.3 Roadside Ditch Lining Sample Problem

Given:

LOCATION: Lynchburg Area

ALLOWABLE VELOCITY: 2.0 fps (Bare Earth)

DITCH SLOPE:

STATION	SLOPE
1-2	0.5%
2-3	2.0%
3-4	3.0%
4-5	4.0%
5-6	4.0%

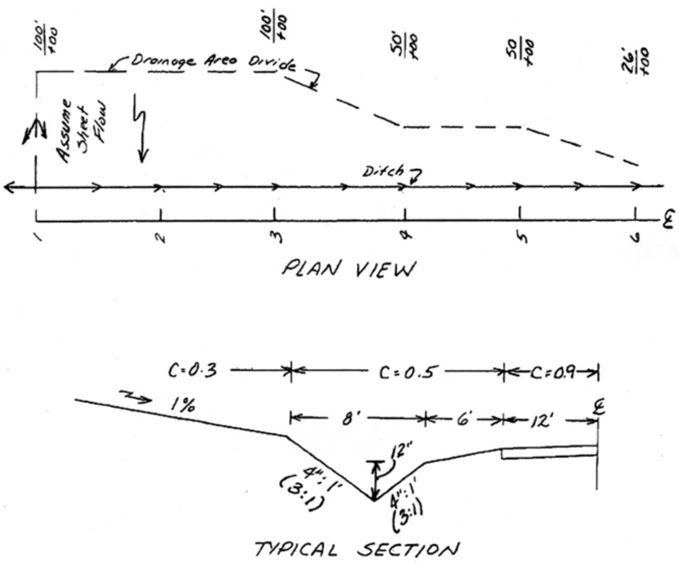
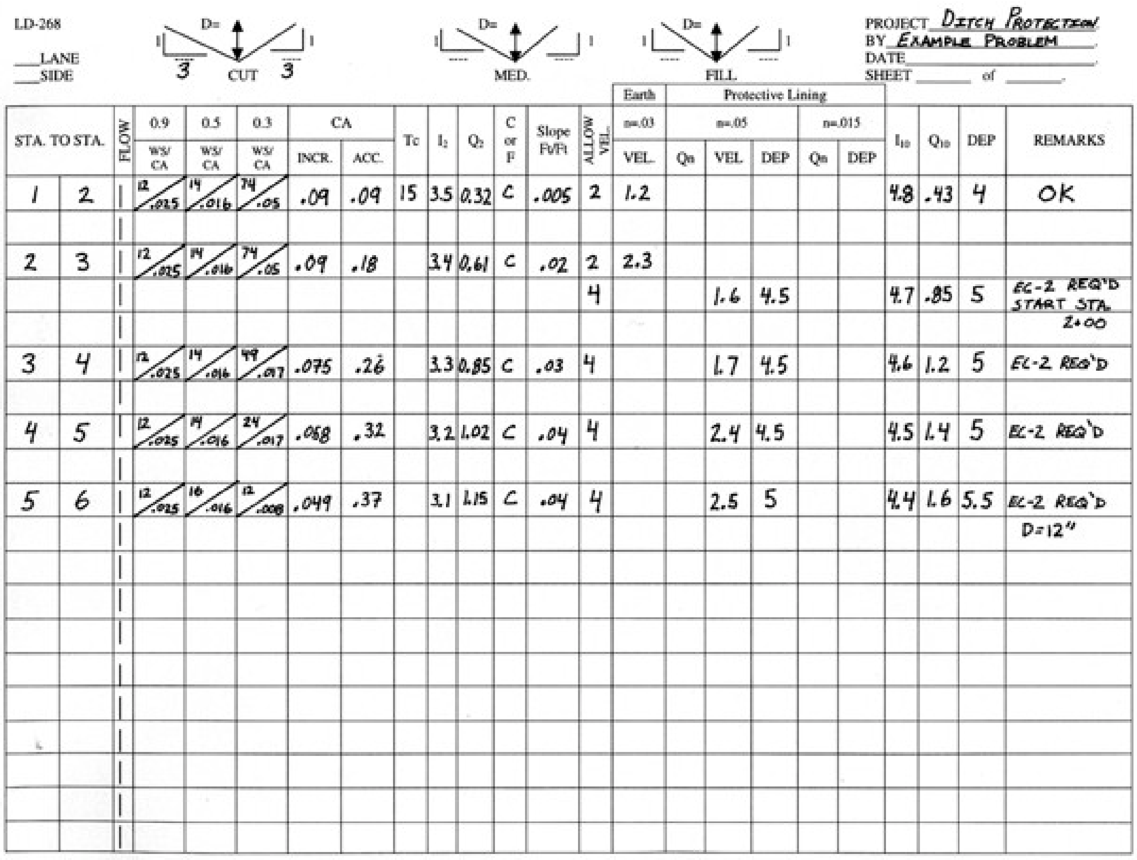


Figure 7-9. Roadside Ditch Protection Sample Problem, Plan and Section



LD-268

___LANE
___SIDE

D= 3 CUT 3 | D= MED. | D= FILL

PROJECT Ditch Protection
BY Example Problem
DATE
SHEET ___ of ___

STA.	TO STA.	FLOW	0.9 WS/CA	0.5 WS/CA	0.3 WS/CA	CA INCR.	CA ACC.	Tc	I_2	Q_2	C or F	Slope Ft/Ft	ALLOW VEL.	Earth n=.03 VEL.	Protective Lining n=.05 Qn	n=.05 VEL	n=.05 DEP	n=.015 Qn	n=.015 DEP	I_{10}	Q_{10}	DEP	REMARKS
1	2	|	12/.025	14/.016	74/.05	.09	.09	15	3.5	0.32	C	.005	2	1.2						4.8	.43	4	OK
2	3	|	12/.025	14/.016	74/.05	.09	.18		3.4	0.61	C	.02	2	2.3									
		|											4			1.6	4.5			4.7	.85	5	EC-2 REQ'D START STA. 2+00
3	4	|	12/.025	14/.016	49/.017	.075	.26		3.3	0.85	C	.03	4			1.7	4.5			4.6	1.2	5	EC-2 REQ'D
4	5	|	12/.025	14/.016	24/.017	.068	.32		3.2	1.02	C	.04	4			2.4	4.5			4.5	1.4	5	EC-2 REQ'D
5	6	|	12/.025	16/.016	12/.008	.049	.37		3.1	1.15	C	.04	4			2.5	5			4.4	1.6	5.5	EC-2 REQ'D D=12"

Figure 7-10. Worksheet (LD-268) for Calculation of Roadside Ditch Protection , Sample Problem

7.5.3 Major Channels

The following procedures are used to analyze flow in major channels.

7.5.3.1 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.

7.5.3.2 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 - d_n) - Water Surface Profile computations can be started at a point in the channel where uniform flow can be assumed and, therefore, normal depth (d_n) 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 (d_c)

B. Known water surface elevation

C. Slope/Area method (d_n)

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

$$h_1 = d_1 + \frac{V_1^2}{2g} + z_1$$

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 (d_2) at the unknown cross section.

C. Compute area (A_2), wetted perimeter (P_2), hydraulic radius (R_2) and velocity (V_2) for the unknown cross section based on the assumed depth (d_2).

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

$$S_1 = \frac{Q^2 n^2}{2.25 R_1^{\frac{4}{3}} A_1^2}$$

F. Compute energy slope (S_2) at the unknown cross section

$$S_2 = \frac{Q^2 n^2}{2.25 R_2^{\frac{4}{3}} A_2^2}$$

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 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.

7.5.3.2.1 Water Surface Profile Sample Problem

GIVEN: Q = 100 cfs
n = 0.15
Cross section shown in Figure 7-11

REQUIRED: Compute Water Surface Profile (WSP) between sections 1 and 3 with a tolerance of 0.2 feet.

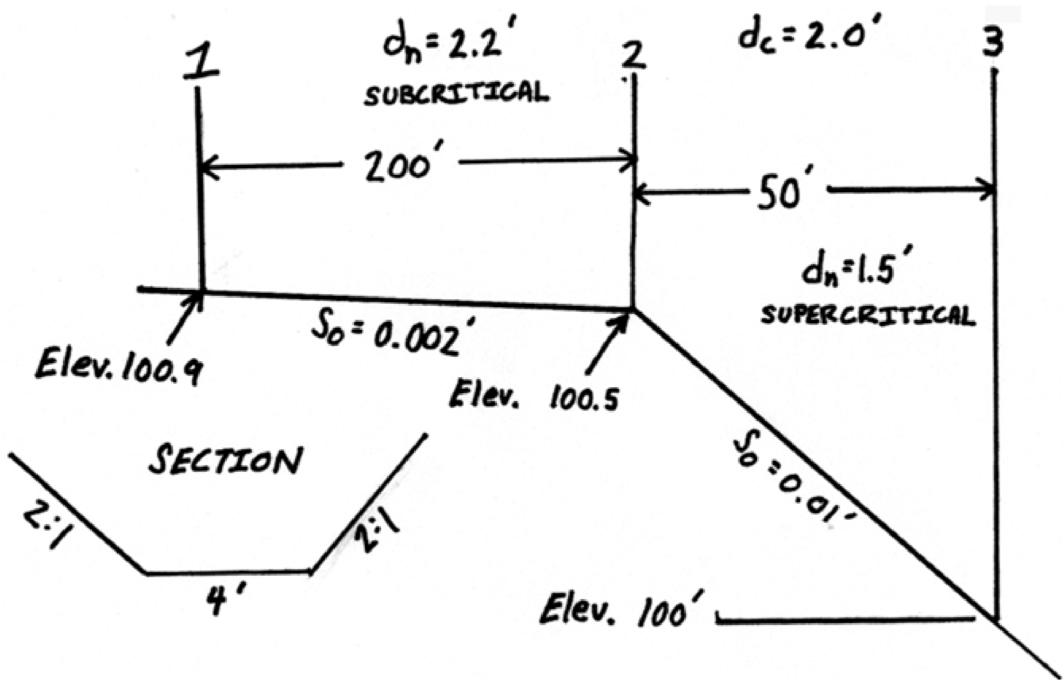


Figure 7-11. Water Surface Profile Sample Problem

(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)

Worksheet For Non–Uniform Flow In Open Channels

	$d_1 + v_1^2/2g + Z_1$	depth	X–Section Area at Station 2	Wetted Perimeter	$A_2 \div P_2$	$Q_2 \div A_2$	$v^2 \div 2(32.2)$	Flow Line Elevation	Distance Between Stations	$\frac{Q^2 \cdot n^2}{2.25 \cdot R^{4/3} \cdot A^2}$	$\frac{Q^2 \cdot n^2}{2.25 \cdot R^{4/3} \cdot A^2}$	$\frac{S_1 + S_2}{2}$	$S_A \cdot L$	$d_2 + v_2^2/2g + Z_2$	Tolerance = ± 0.2'
STA.	H_1	d_2	A_2	P_2	R_2	V_2	$v^2/2g$	Z_2	L	S_1	S_2	S_A	h_1	H_2	$H_{UP} = H_{DOWN} + h_1$
2	103.11	2/d_c	A_1/16	P_1/12.94	R_1/1.24	V_1/6.25	$v_1^2/2g$ 0.69	Z_1/100.5	—	.00293					ASSUME WSP STARTS @ SEC. 2 WITH d_c 2' ∴
							.61								WSE 100.5 + 2 102.5'
3 →	103.11	1.5/d_n	10.5	10.71	0.98	9.52	1.41	100	50	.00293	.00932	.00612	0.31	102.91	103.11 ≈ 102.91 + 0.31 = 103.22 OK
															∴ WSE = 100 + 1.5 = 101.5'
← 1	103.11	2.2/d_n	18.48	13.84	1.34	5.41	0.45	100.9	200	.00293	.00198	.00246	0.49	103.55	103.55 ≈ 103.11 + 0.49 = 103.6 OK
															∴ WSE = 100.9 + 2.2 = 103.1'

Figure 7-12. Worksheet for Calculation of Non-Uniform Flow in Open Channels, Sample Problem

7.5.4 Natural Channels

7.5.4.1 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.

7.5.4.2 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.

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

7.5.5 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.

7.5.5.1 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). A computer program entitled “RIPRAP” performs all necessary design calculations in accordance with HEC-11. It is available on request.

The riprap design procedure presented in HEC-11 is based on the equation:

$$D_{50} = C\left[\frac{0.001V_a^3}{K_1^{1.5}\sqrt{d_{avg}}}\right] \tag{7.19}$$

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

In HEC-11, K_1 is by Equation 7.22:

$$K_1 = \sqrt{1-\frac{\sin^2\theta}{\sin^2\phi}} \tag{7.20}$$

Where:

θ = Side slope angle (measured from the horizontal), deg.

ϕ = Natural angle of repose of material under consideration (measured from the horizontal), deg.

$$C = C_{sg}\, C_{SF} \tag{7.21}$$

Where:

C_{sg} = Adjustment factor for specific gravity of stone
C_{SF} = Adjustment factor for stability

$$C_{sg} = \frac{2.12}{\left(S_g - 1\right)^{1.5}}$$

Where:

S_g = Specific gravity of the rock riprap (Assume 2.65 for all VDOT standard riprap sizes.)

$$C_{SF} = \left(\frac{SF}{1.2}\right)^{1.5} \tag{7.22}$$

Where:

SF = Stability Factor, See Appendix 7D-5, (Usually 1.2)

Notes:

1. Nomographs for the solutions of ϕ, 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.

7.5.5.1.1 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 ft. 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: $$C_{sg} = \frac{2.12}{\left(S_g - 1\right)^{1.5}} = \frac{2.12}{\left(2.65 - 1\right)^{1.5}} = 1$$

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: $$D_{50} = C\left[\frac{0.001V_a^3}{K_1^{1.5}\sqrt{d_{avg}}}\right] = 1\left[\frac{0.001(5.4)^3}{(0.74)^{1.5}\sqrt{2}}\right] = 0.17 \text{ ft.}$$

Step 7: Closest standard VDOT riprap size = Class AI (D_{50} = 0.8 ft).

7.5.5.2 Riprap Lining Design Method for Minor Channels (HEC-15)

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 (τ_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 7-1. Manning's n-Values for Depth Ranges

Riprap Size	0-0.5 ft	0.5-2.0 ft	>2.0 ft
Class AI	0.04	0.069	0.035
Class I	--	0.078	0.040

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 (τ_o):

$$\tau_o = \gamma d S_o \tag{7.23}$$

Where:

γ = Unit weight of water (62.4 lbs/ft^3)
d = Depth of flow, ft.
S = Average ditch flowline slope, ft./ft.

If $\tau_o > \tau_p$, 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°.

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 ($D_{50\ side}$)

$$D_{50\ side} = \frac{K_1}{K_2} D_{50\ bottom}$$

From a practical standpoint, whatever riprap size is indicated for the ditch side slope should be used on the bottom as well.

7.5.5.2.1 Channel Stability Sample Problem - Tractive Force Calculation

Check the stability of channel's native material using tractive force calculation.

Given: A natural channel with a bed and banks of native materials composed of cobbles and pebbles. Mean diameter is 1.25 inches for the D_{75} size stone. The channel bottom width (B) is 10 ft. 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 ft.

Determine whether the channel is stable for the indicated condition.

Solution:

Step 1: Determine the permissible shear stress (τ_p)

From Appendix 7E-2, for a D_{75} particle diameter of 1.25 inches, read a permissible tractive force (τ_p) on the channel bottom of 0.5 lbs/ft^2.

Step 2: For a side slope ratio of 2:1, the sine of the slope angle ($\theta = 26.6^o$) is 0.5

Step 3: From Appendix 7E-1, for a particle diameter of 1.25 inches, read an angle of repose (ϕ) of 40^o

The sine of $40^o = 0.643$

Step 4: $K_1 = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} = \sqrt{1 - \frac{0.447^2}{0.643^2}} = 0.72$

Step 5: Permissible tractive force on the side slopes (τ_s)

$$\tau_s = K_1\tau_p = 0.72(0.05) = 0.36 \text{ lbs/ft.}^2$$

Step 6: Compute the width to depth ratio

$\frac{B}{d} = \frac{10}{2} = 5$, which is less than 10, therefore use $\tau_o = 62.4RS_o$

Step 7: Flow cross-sectional area

$$\begin{aligned} A &= Bd + zd^2 \\ &= 10(2) + 2(2)^2 \\ &= 28 \text{ ft.}^2 \end{aligned}$$

Step 8: Wetted perimeter

$$\begin{aligned} P &= B + 2d\sqrt{1+z^2} \\ &= 10 + 2(2)\sqrt{1+2^2} \\ &= 18.94 \text{ ft.} \end{aligned}$$

Step 9: Hydraulic radius

$$R = \frac{A}{P} = \frac{28}{18.94} = 1.48 \text{ ft.}$$

Step 10: Compute the average tractive force

$$\begin{aligned} \tau_o &= 62.4RS_o \\ &= 62.4(1.48)(0.008) \\ &= 0.74 \text{ lbs/ft.}^2 \end{aligned}$$

Step 11: Compare the average tractive force of the channel to the allowable tractive force of the channel sides and bottom

τ_o of 0.74 > $\tau_{p\ (bottom)}$ of 0.5 lbs/ft^2 or $\tau_{s\ (side\ slope)}$ of 0.36 lbs/ft^2

Native material is unstable. Channel protection is required.

7.5.5.2.2 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 ft, 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 ft

τ_p = 3.2 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 ft/ft, B = 2 ft, z = 2, Qn = 20 (0.069) = 1.38,

Read d/B = 0.78. d = 0.78 (2) = 1.56 ft. 0.5 < 1.56 <2.0,

Therefore, the assumed n-value of 0.069 is acceptable.

Step 4: Determine the maximum shear stress

$$\tau_{max} = \gamma d S_o = 62.4(1.56)(0.005) = 0.49\ \text{lb}/\text{ft.}^2$$

Step 5: Evaluate the maximum shear stress with the permissible shear stress

$\tau_{max} < \tau_p$,

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

$\phi = 42^o$

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^o, read K_2 = 0.73.

Step 9: Determine the D_{50} for the side slopes

$$D_{50\ side} = \frac{K_1}{K_2} D_{50\ bottom} = \frac{0.90}{0.73}(0.8) = 0.98\ \text{ft.}$$

Therefore, it would probably be best to use Standard Class I riprap (D_{50} – 1.1 ft) for both the channel bottom and side slopes in lieu of the originally proposed Class AI (D_{50} = 0.8 ft).

7.6 References

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Appendix 7A-1 Definitions and Abbreviations

Abbreviations:

BRI-STARS	Bridge Stream Tube Model for Sediment Routing Alluvial River Simulation
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
NTIS	National Technical Information Service
USCOE/USACE	United States Corps of Engineers
VDOT	Virginia Department of Transportation

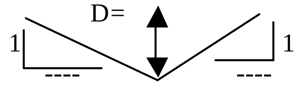
Appendix 7A-2 Symbols

SYMBOL	DEFINITION	UNITS
A	Cross-sectional area of flow	ft^2
α	Velocity distribution coefficient	-
C	Stone size correction factor	-
C_{sg}	Adjustment to the specific gravity of stone	-
C_{SF}	Adjustment to the stability factor	-
d	Depth of flow	ft
D_{50s}	Required D_{50} for side slopes	ft
d_c	Critical depth	ft
d_n	Normal depth	ft
d_{avg}	Average flow depth in the main channel	ft
E	Specific energy	ft
F_r	Froude Number	-
g	Acceleration due to gravity	ft/s^2
h_L	Total head loss due to local minor and friction losses	ft
h	Stage or head	ft
h_f	Friction loss	ft
H_D	Average hydraulic depth	ft
h_o	Summation of minor losses	ft
K	Channel conveyance	-
L	Discharge-weighted or conveyance reach length	ft
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft
Q	Discharge	cfs
R	Hydraulic radius	ft
S_o	Channel slope	ft/ft
S	Slope of the energy grade line	ft/ft
S_A	Average slope of the energy grade line	ft/ft
S_g	Specific gravity of rock riprap	-
SF	Stability factor applied	
T	Top width at the water surface	ft
τ_o	Average tractive force	lbs/ft^2
τ_{max}	Maximum tractive force	lbs/ft^2
τ_p	Permissible shear stress	lbs/ft^2
τ_s	Side slope shear stress	lbs/ft^2
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity	ft/s
V_a	Average velocity in main channel	ft/s
z	Elevation head	ft
S_f	Mean slope of the energy grade line	ft/ft
γ	Unit weight of water	lb/ft^3
θ	Side slope angle	deg.
ϕ	Angle of repose of material	deg.

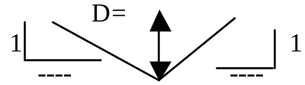
Appendix 7B-1 LD-268 Roadside and Median Ditch Design Form

LD-268

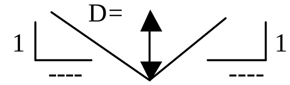
____LANE
____SIDE



CUT



MED.

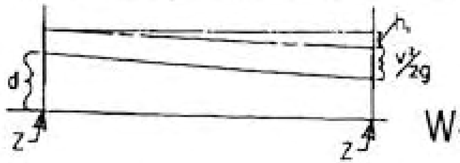


FILL

PROJECT ________________________.
BY______________________________.
DATE____________________________.
SHEET ________ of ________.

STA. TO STA.		FLOW	0.9	0.5	0.3	CA		Tc	I_2	Q_2	C or F	Slope Ft/Ft	ALLOW VEL.	Earth	Protective Lining					I_{10}	Q_{10}	DEP	REMARKS
														n=.03	n=.05			n=.015					
			WS/ CA	WS/ CA	WS/ CA	INCR.	ACC.							VEL.	Qn	VEL	DEP	Qn	DEP				

Appendix 7B-2 Water Surface Profile Calculation Form



Worksheet For Non–Uniform Flow In Open Channels

	$d_1 + v_1^2/2g + Z_1$	depth	X–Section Area at Station 2	Wetted Perimeter	$A_2 \div P_2$	$Q_2 \div A_2$	$v_2^2 \div 2(32.2)$	Flow Line Elevation	Distance Between Stations	$\frac{Q^2 \cdot n^2}{2.25 \cdot R_1^{4/3} \cdot A_1^2}$	$\frac{Q^2 \cdot n^2}{2.25 \cdot R_2^{4/3} \cdot A_2^2}$	$\frac{S_1 + S_2}{2}$	$S_A \cdot L$	$d_2 + v_2^2/2g + Z_2$	Tolerance = ± ______
STA.	H_1	d_2	A_2	P_2	R_2	V_2	$v_2^2/2g$	Z_2	L	S_1	S_2	S_A	h_f	H_2	$H_{UP} = H_{DOWN} + h_f$

Source:

Appendix 7B-3 CHANNEL STABILITY WORK SHEET

CHANNEL DATA

Q = _________(cfs) P = _________(ft.) Native Material

S_o = _________(ft/ft) R = _________(ft.) D_{50} = _________

d_n = _________(ft.) V_n = _________(fps) D_{75} = _________

A = _________(ft^2) Side Slope = _____ :1 n = _________

STABILITY OF NATIVE MATERIAL

$\tau_o = 62.4 \bullet R \bullet S_o$ = 62.4 • ________ • ________ = ________

τ_p Bed = ________ (Appendix 7E-2 or 3)

For D_{50} = ________ ϕ = _______ ^{o} (Appendix 7E-1)

For D_{75} = ________ ϕ = _______ ^{o} (Appendix 7E-9)

Side Slope = ___ :1 θ = _______ ^{o}

$K_1 = [1 - (\sin^2 \theta / \sin^2 \phi)]^{0.5}$

K_1 = [1 - ($\sin^2$ _______ ^{o} / $\sin^2$ _______ ^{o})]$^{0.5}$ = ________

τ_s Side Slope (SS) = τ_p Bed • K = ________ • ________ = ________

τ_p Bed (______) (<) (=) (>) τ_o (______)

∴ Native Material on Bed is (stable) (unstable)

τ_s SS (______) (<) (=) (>) τ_o (______)

∴ 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 = ________

S_o = _________(ft/ft) R = _________(ft.)

d_n = _________(ft.) V_n = _________(fps)

A = _________($ft.^2$) Side Slope = _____ :1

DETERMINE RIPRAP SIZE

$\phi = 42^o$ Side Slope = _____ :1 θ = ______ ^{o}

$K_1 = [1 - (\sin^2 \theta / \sin^2 \phi)]^{0.5}$

$K_1 = [1 - (\sin^2$ _______ $^o / \sin^2 42^o)]^{0.5}$ = ________

For Specific Gravity = 2.65 and Stability Factor = 1.2

$D_{50} = 0.001 \bullet V_a^3 / (d_{avg}^{0.5} \bullet K_1^{1.5})$

$D_{50} = 0.001 \bullet$ ________ ^{3} / (________ $^{0.5}$ $\bullet$ ________ $^{1.5}$)

D_{50} Computed = _____________

Note: All VDOT standard riprap (Class AI through Type II) is assumed to have a ϕ of approximately 42^o and a Specific Gravity of 2.65. Therefore, the Computed D_{50} should be adjusted by the Stability Correction Factor (C_{SF}) (if any) to derive a Final D_{50}. The VDOT standard class of riprap with the next higher D_{50} should be specified.

Correction Factor For Stability Factor (SF) other than 1.2 (Default = 1.0)

$C_{SF} = (SF / 1.2)^{1.5}$ = (________ / 1.2)$^{1.5}$ = ________

Final $D_{50} = C_{SF} \bullet$ Computed D_{50} = ________ $\bullet$ ________ = ________

RIPRAP RECOMMENDATION: VDOT (Class) (Type) ____________

Thickness (T) = _________" (2 $\bullet$ D_{50} MSD minimum)

Source: VDOT

Appendix 7B-5 RIPRAP DESIGN WORK SHEET FOR OTHER THAN VDOT STANDARD RIPRAP SIZES

CHANNEL DATA

Q = _________(cfs) P = _________(ft.) n = ________

S_o = _________(ft/ft) R = _________(ft.)

d_n = _________(ft.) V_n = _________(fps)

A = _________(ft^2) Side Slope = _____ :1

ASSUMED RIPRAP SIZE - D_{50} = ________

VERIFY ASSUMED RIPRAP SIZE

ϕ = _______^{o} (Appendix 7E-1)
Side Slope = _____ : 1 θ = ______^{o}
$K_1 = [1 - (\sin^2 \theta / \sin^2 \phi)]^{0.5}$
K_1 = [1 - ($\sin^2$ _______^{o} / $\sin^2$ _______^{o})]$^{0.5}$ = ________

For Specific Gravity = 2.65 and Stability Factor = 1.2

$D_{50} = 0.001 \bullet V_a^{3} / (d_{avg}^{\ 0.5} \bullet K_1^{\ 1.5})$

D_{50} = 0.001 • ________^{3} / (________$^{0.5}$ • ________$^{1.5}$) = _____________

D_{50} Computed (______) (<) (=) (>) D_{50} Assumed (______)

Assumed D_{50} is (correct) (incorrect)

Note: The above process of assuming a D_{50} size, determining the natural angle of repose (ϕ) and computing a D_{50} size should be repeated until the Assumed D_{50} size equals the Computed D_{50} size. Once the D_{50} 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 D_{50}.

Correction Factor For Riprap Specific Gravity (S_s) other than 2.65 (Default = 1.0)

$C_{sg} = 2.12 / (S_s - 1)^{1.5}$ = 2.12 / (________ - 1)$^{1.5}$ = _________

Correction Factor For Stability Factor (SF) other than 1.2 (Default = 1.0)

$C_{SF} = (SF / 1.2)^{1.5}$ = (________ / 1.2)$^{1.5}$ = ________

Final Correction Factor = C = $C_{sg} \bullet C_{SF}$ = ________ • ________ = ________

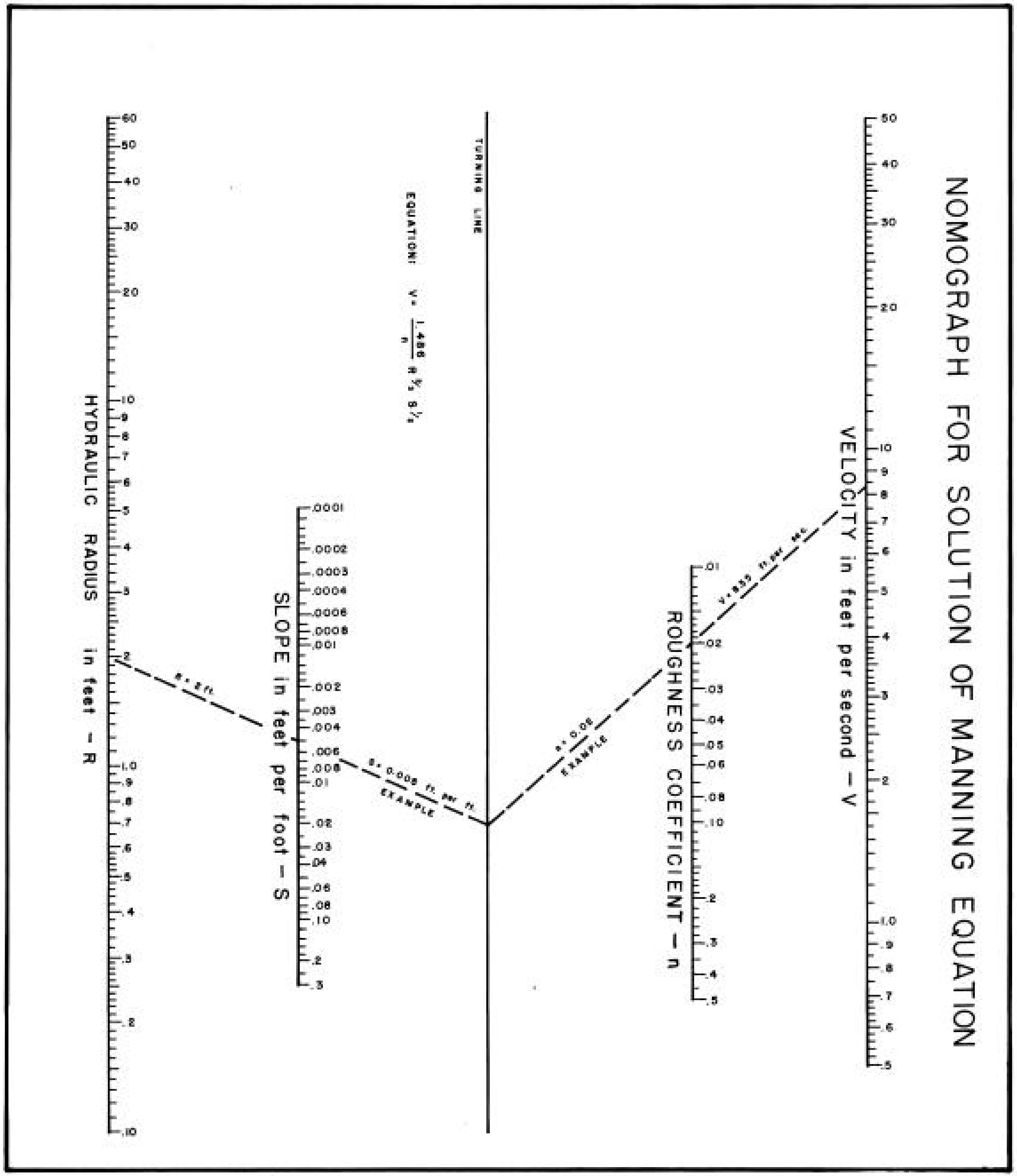
Final D_{50} = C • Computed D_{50} = ________ • ________ = ________

RIPRAP RECOMMENDATION: _________________

Thickness (T) = _________” (2 • D_{50} MSD minimum)

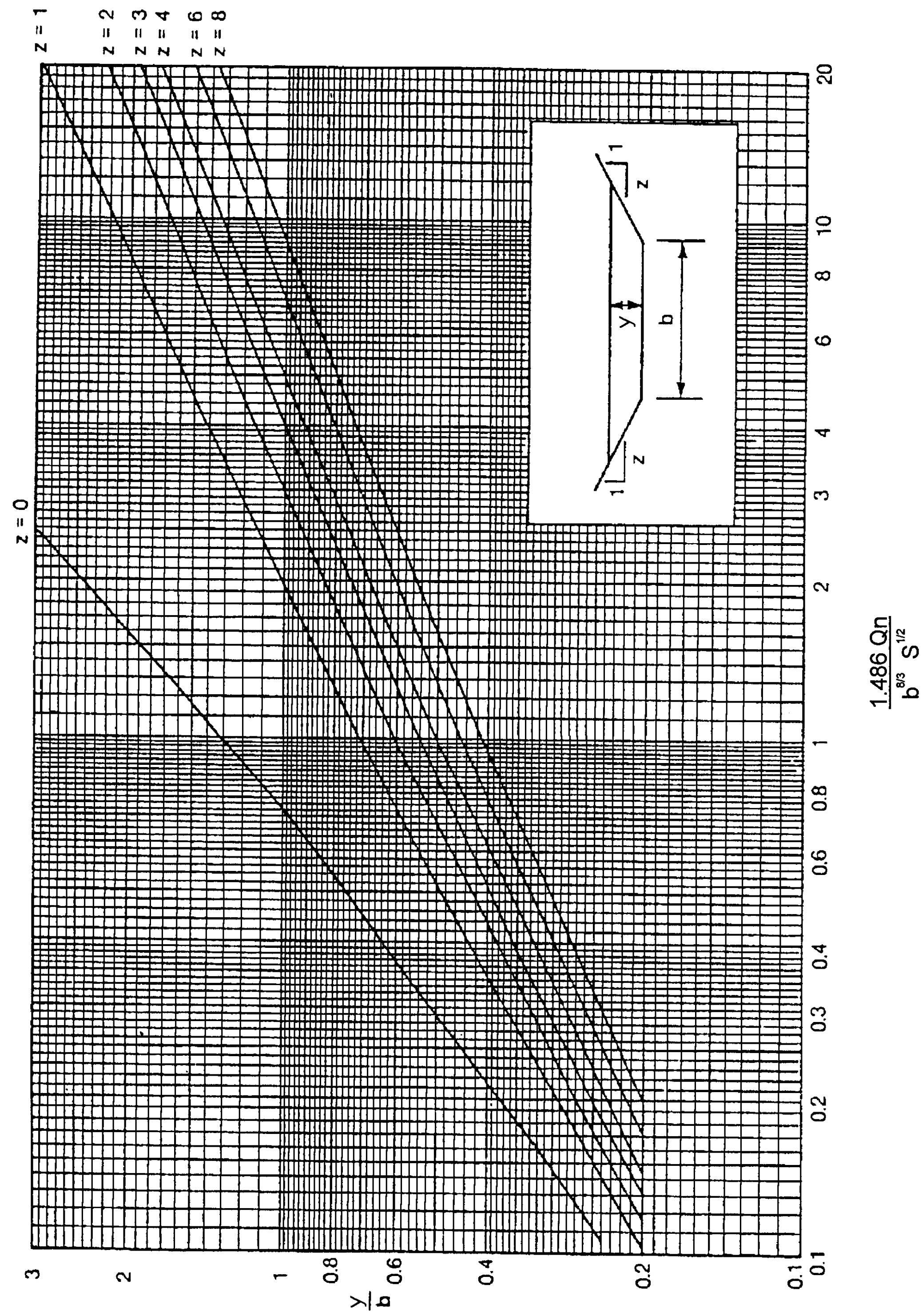
Source: VDOT

Appendix 7C-1 Nomograph for Solution of Manning's Equation



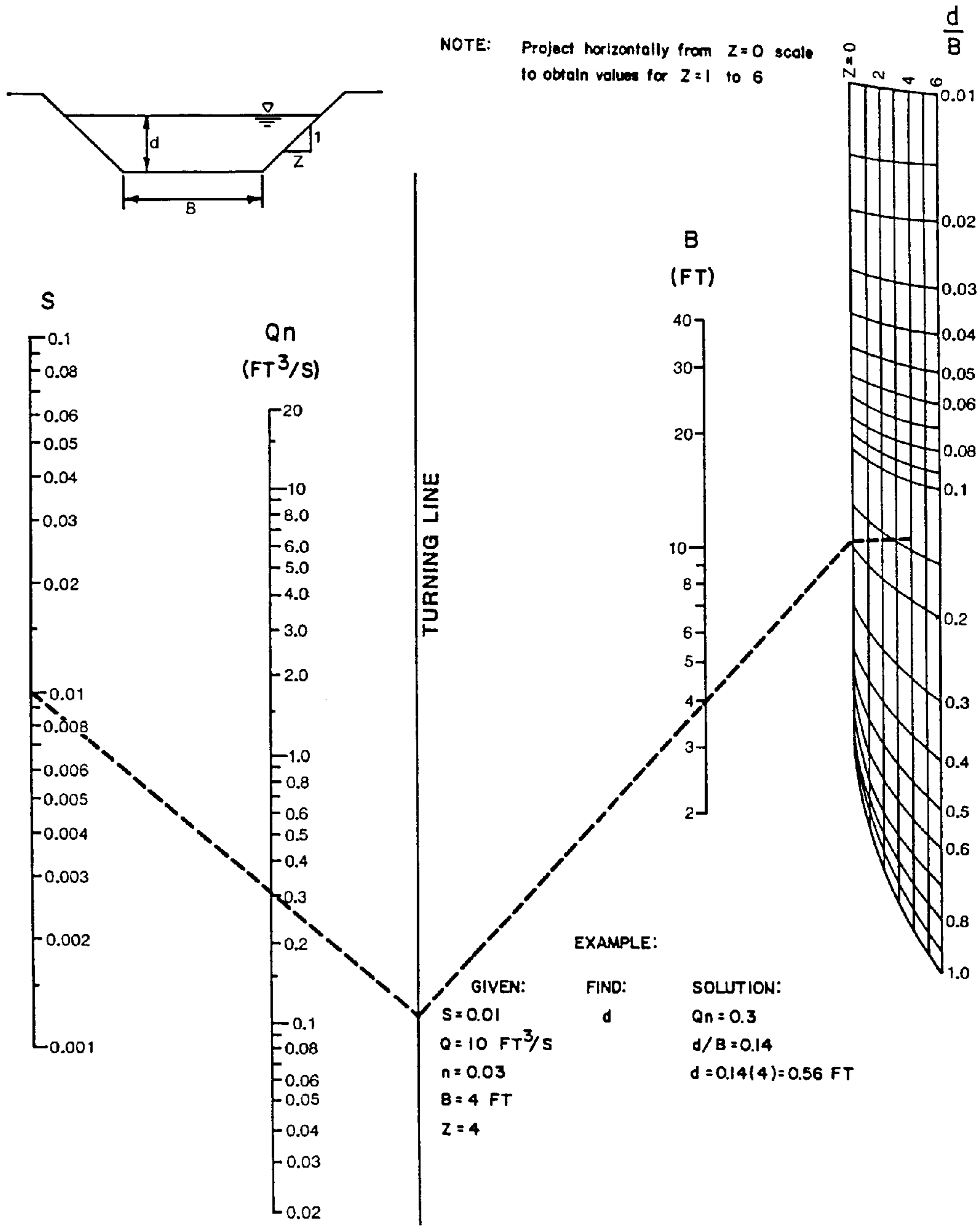
Source: VDOT

Appendix 7C-2 Trapezoidal Channel Capacity Chart



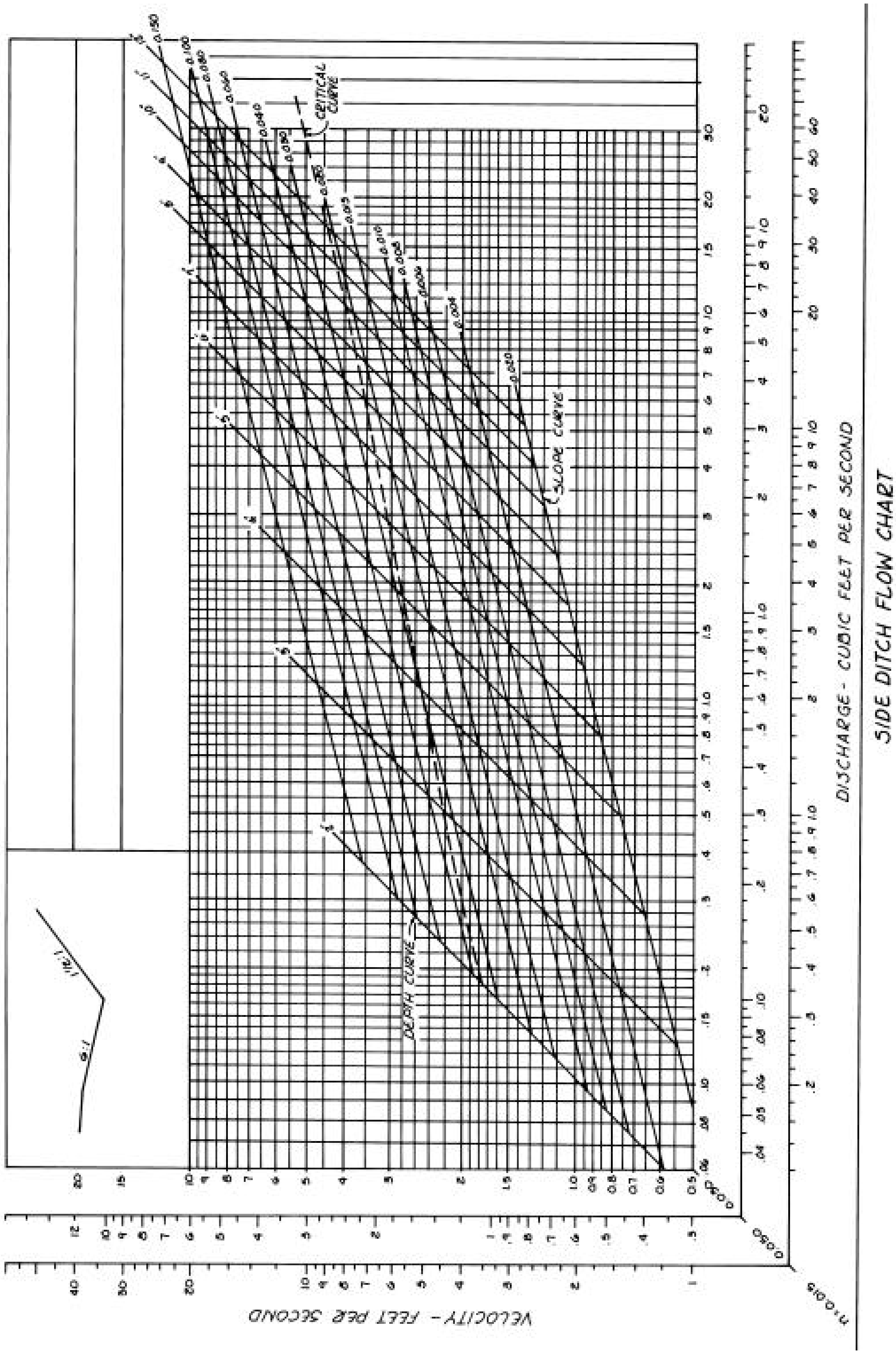
Source:

Appendix 7C-3 Nomograph for Solution of Normal Depth



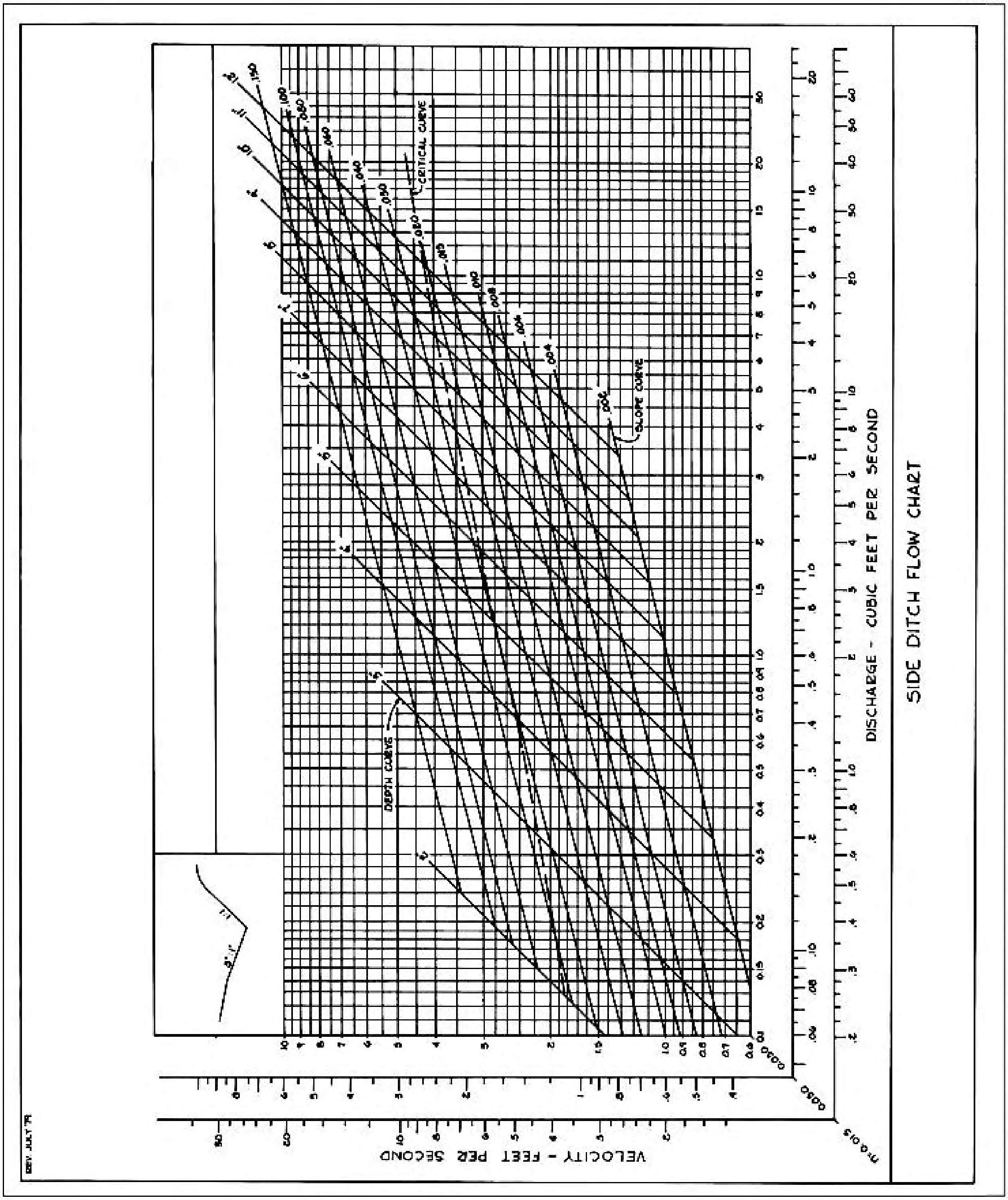
Source: HEC-15

Appendix 7C-4 Side Ditch Flow Chart (Side Slopes = 6:1, 1.5:1)



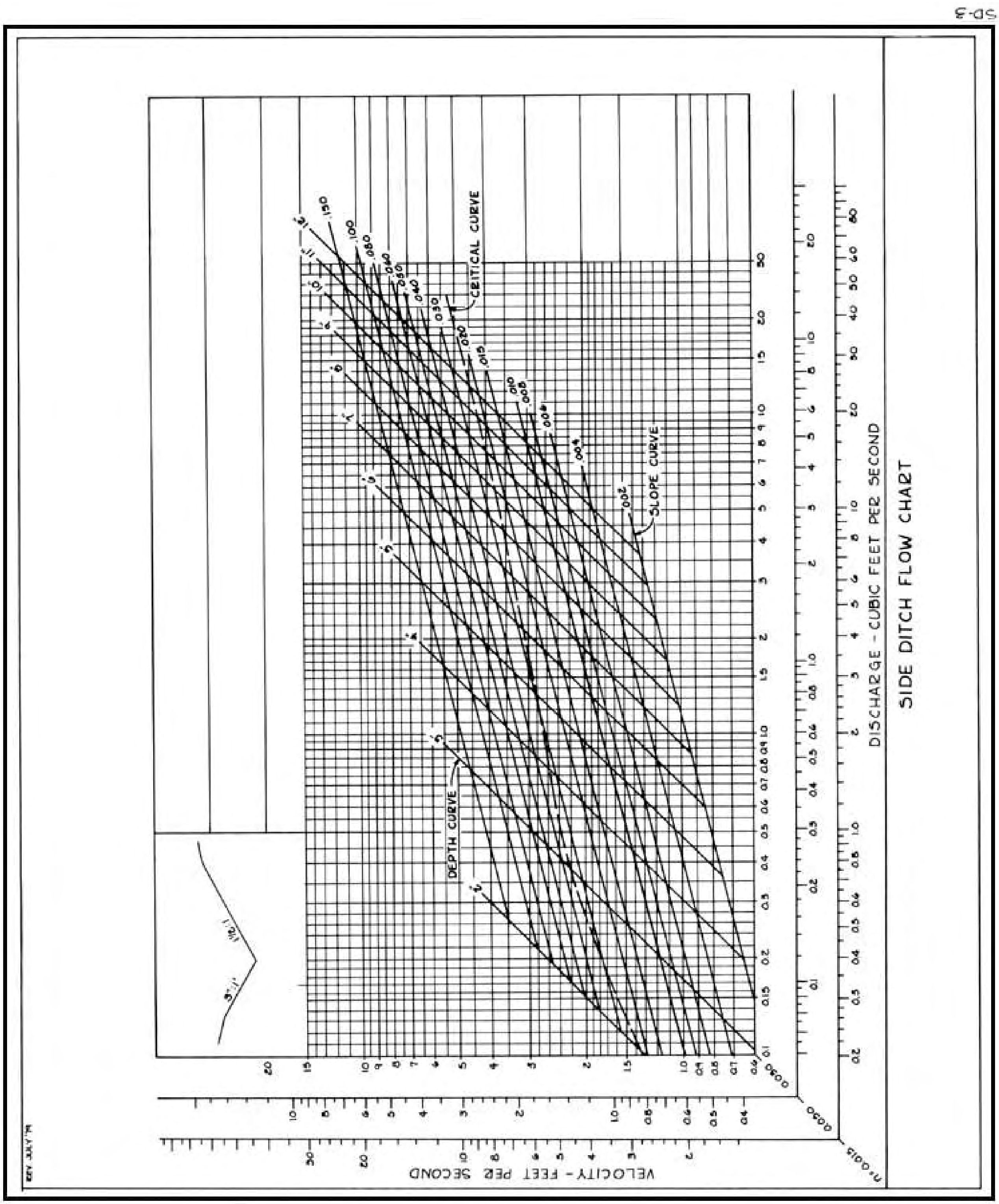
Source: VDOT

Appendix 7C-5 Side Ditch Flow Chart (Side Slopes = 4:1, 1:1)



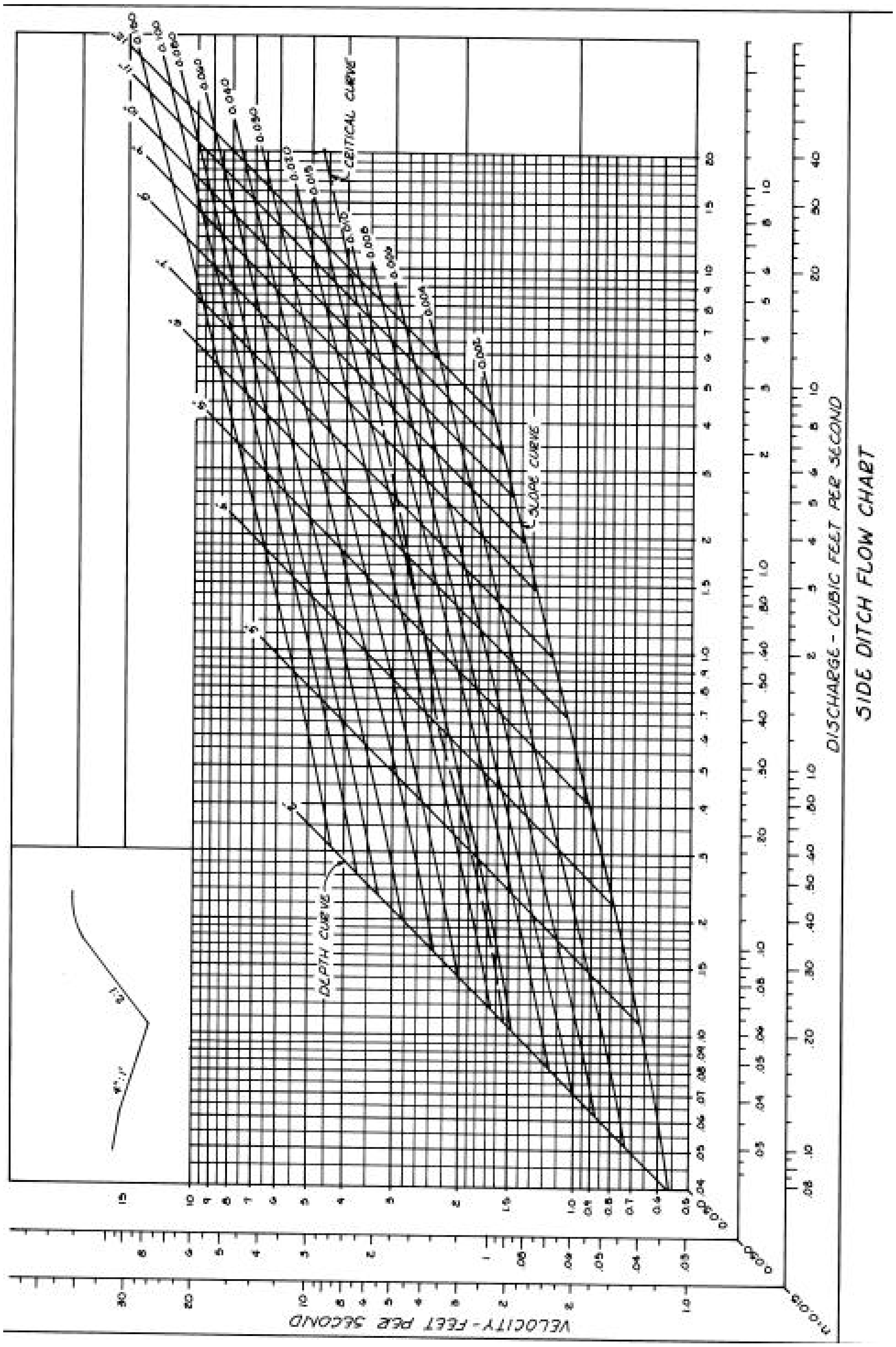
Source: VDOT

Appendix 7C-6 **Side Ditch Flow Chart (Side Slopes = 4:1, 1.5:1)**



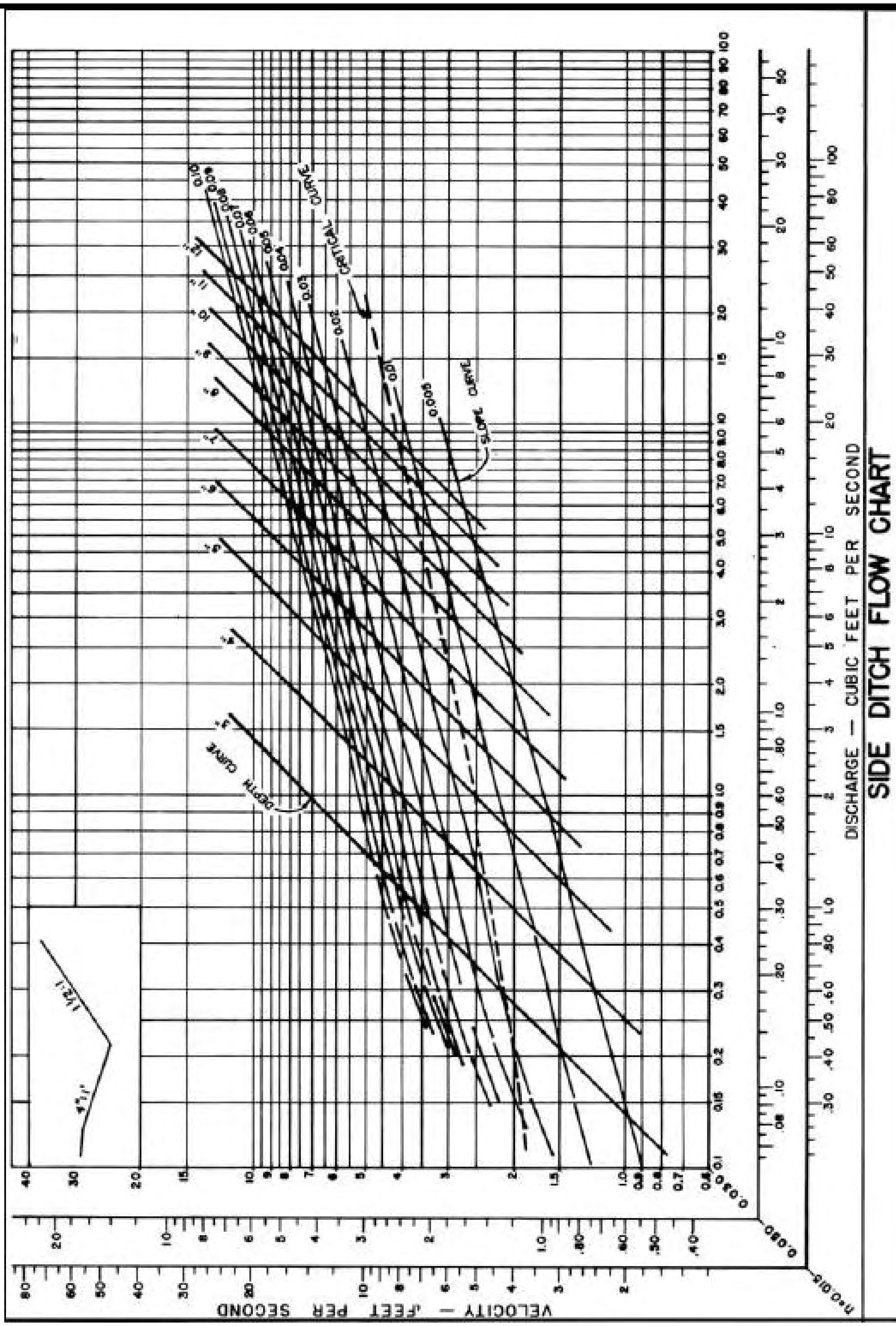
Source: VDOT

Appendix 7C-7 Side Ditch Flow Chart (Side Slopes = 3:1, 2:1)



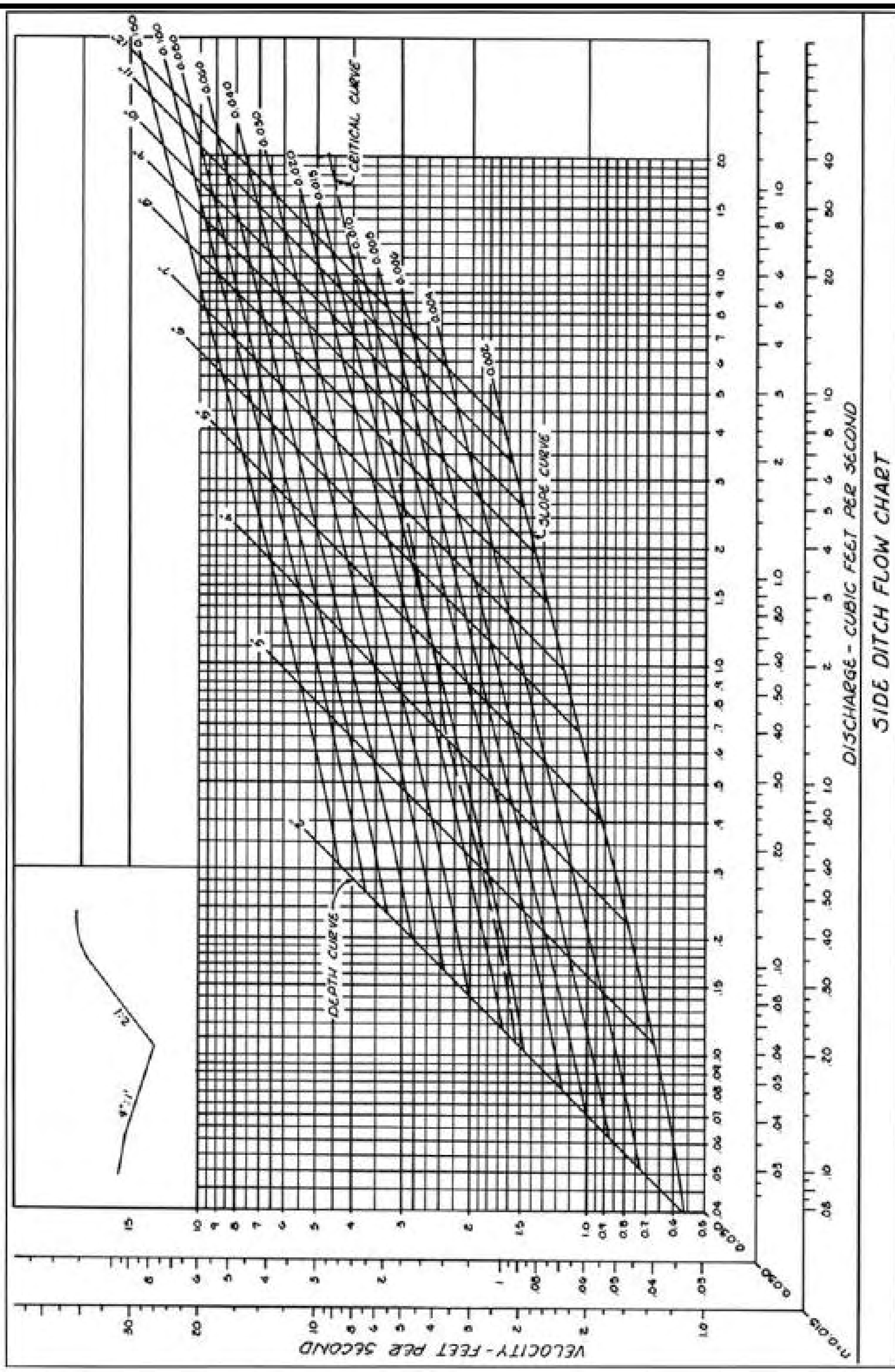
Source:

Appendix 7C-8 **Side Ditch Flow Chart**
(Side Slopes = 3:1, 1.5:1)



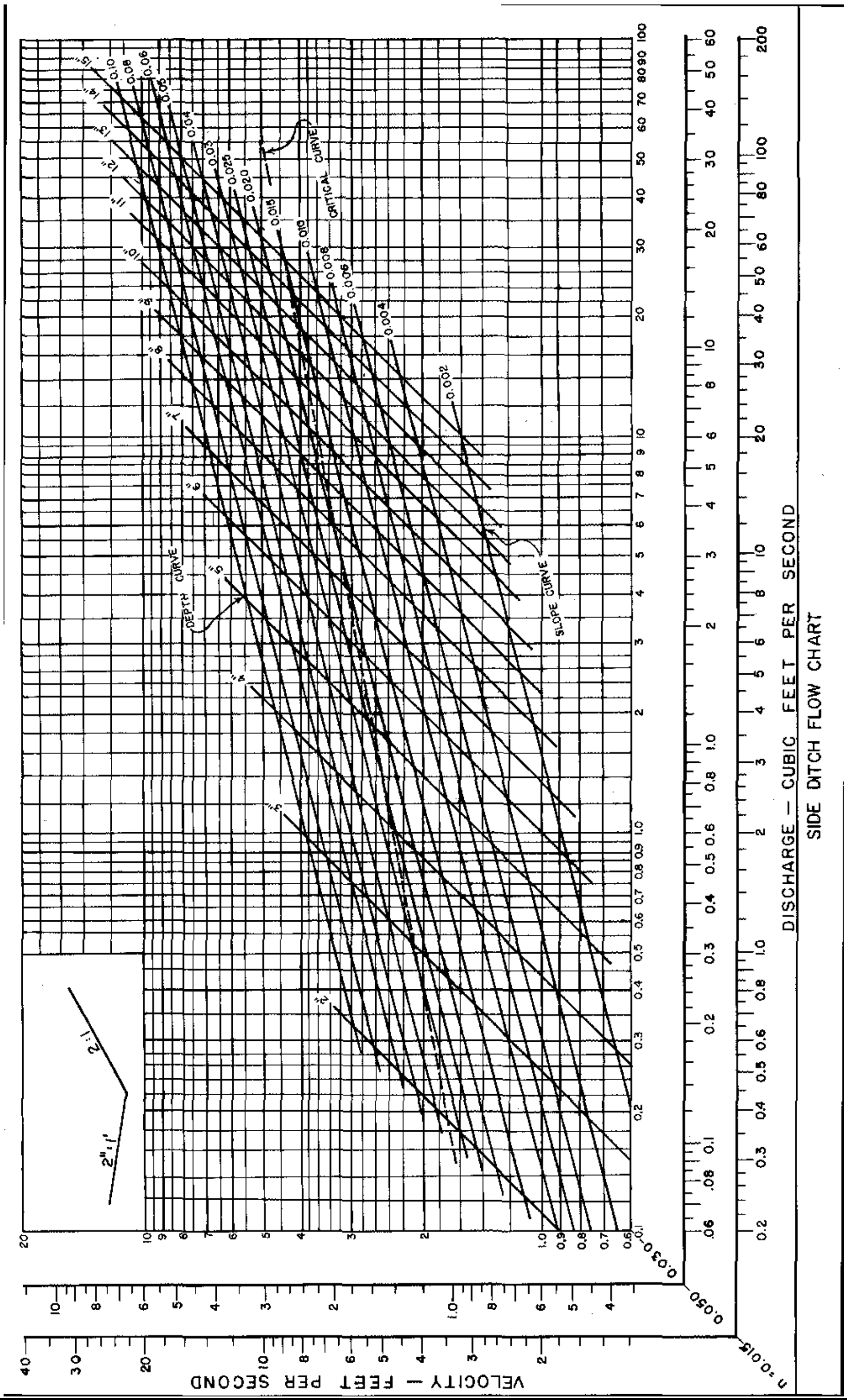
Source: VDOT

Appendix 7C-9 **Side Ditch Flow Chart (Side Slopes = 3:1, 2:1)**



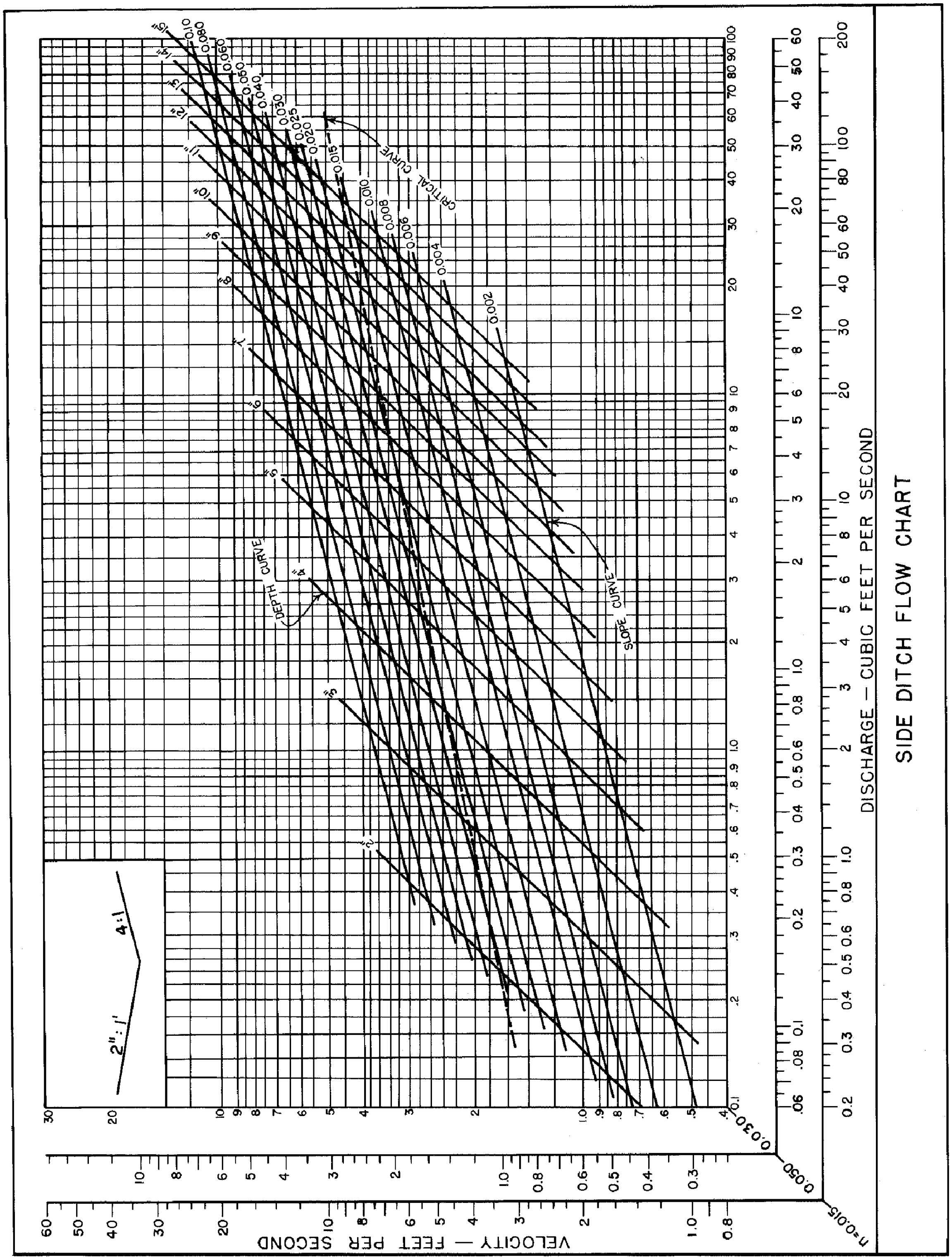
Source: VDOT

Appendix 7C-10 **Side Ditch Flow chart**
(Side Slopes = 6:1, 2:1)



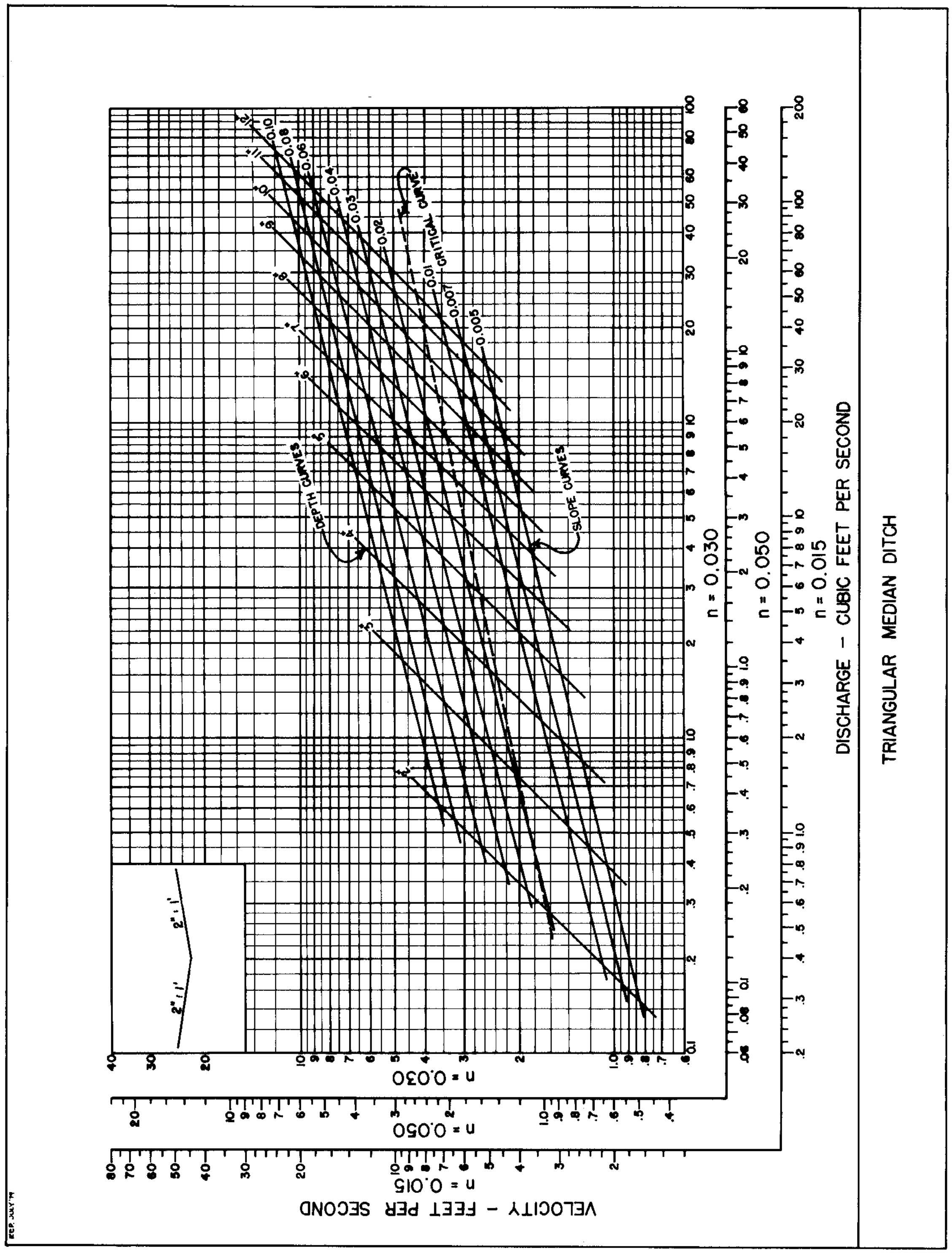
Source: VDOT

Appendix 7C-11 **Side Ditch Flow Chart (Side Slopes = 6:1, 4:1)**



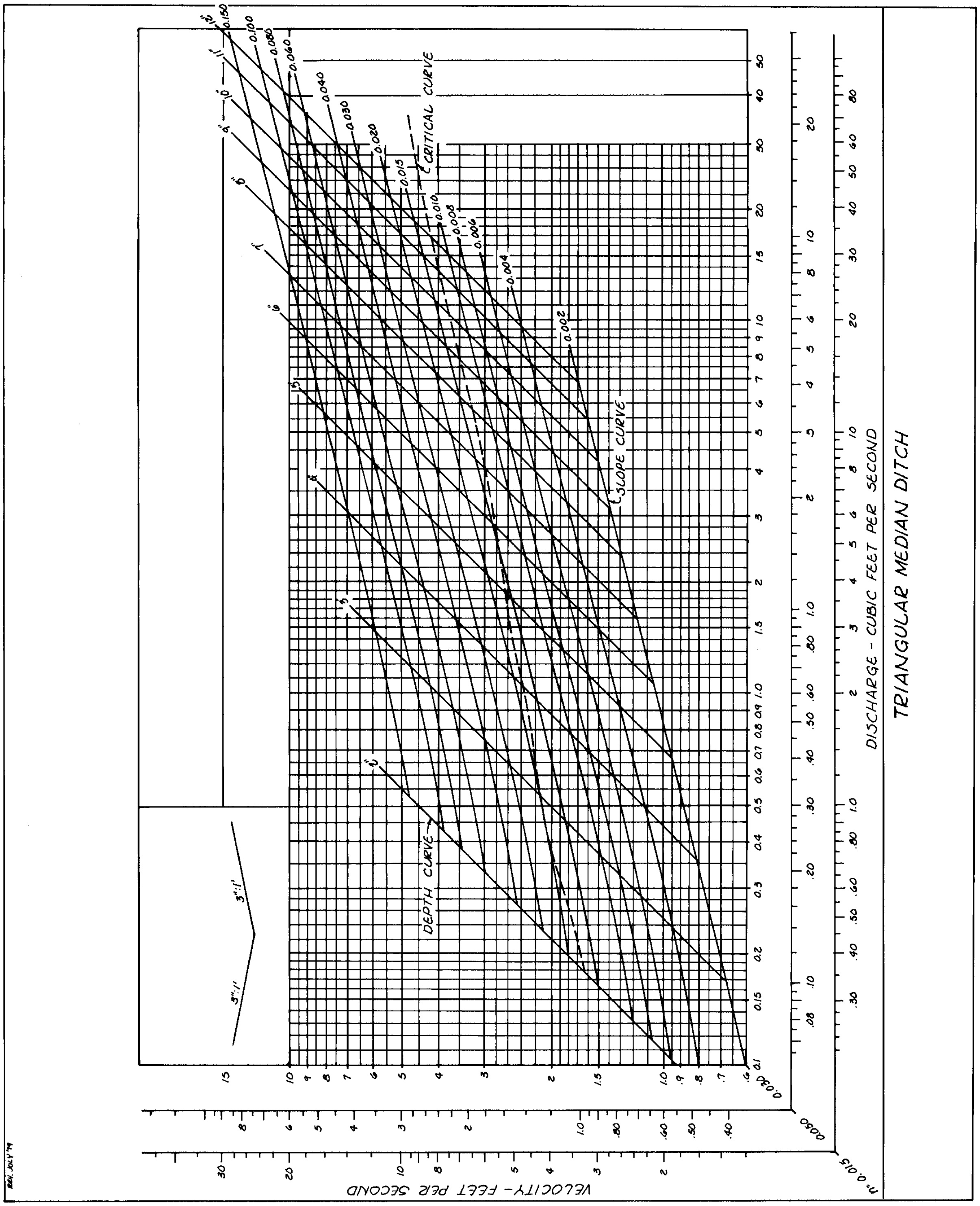
Source: VDOT

Appendix 7C-12 Triangular Median Ditch Flow Chart (Side Slopes = 6:1, 6:1)



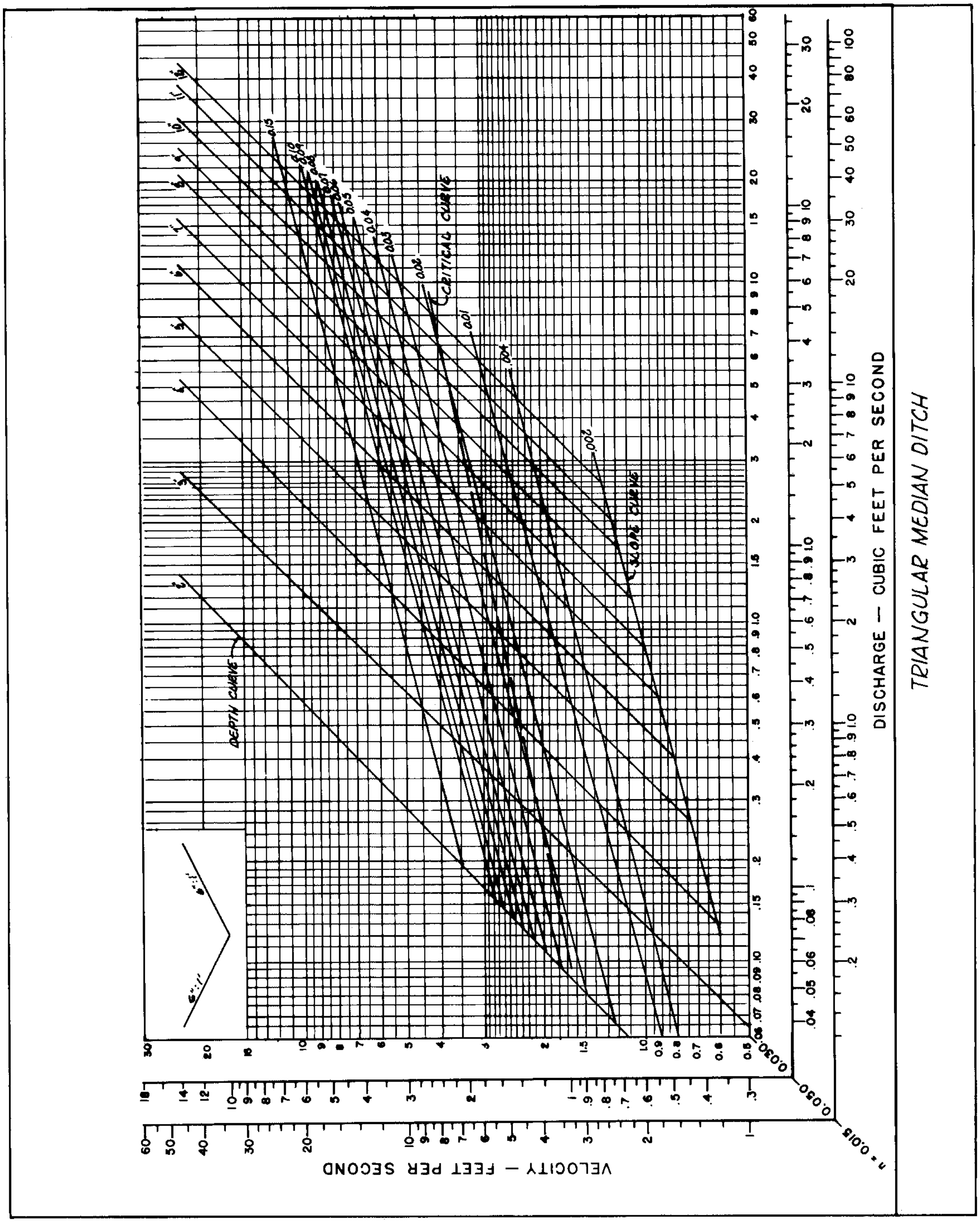
Source: VDOT

Appendix 7C-13 Triangular Median Ditch Flow Chart (Side Slopes = 4:1, 4:1)



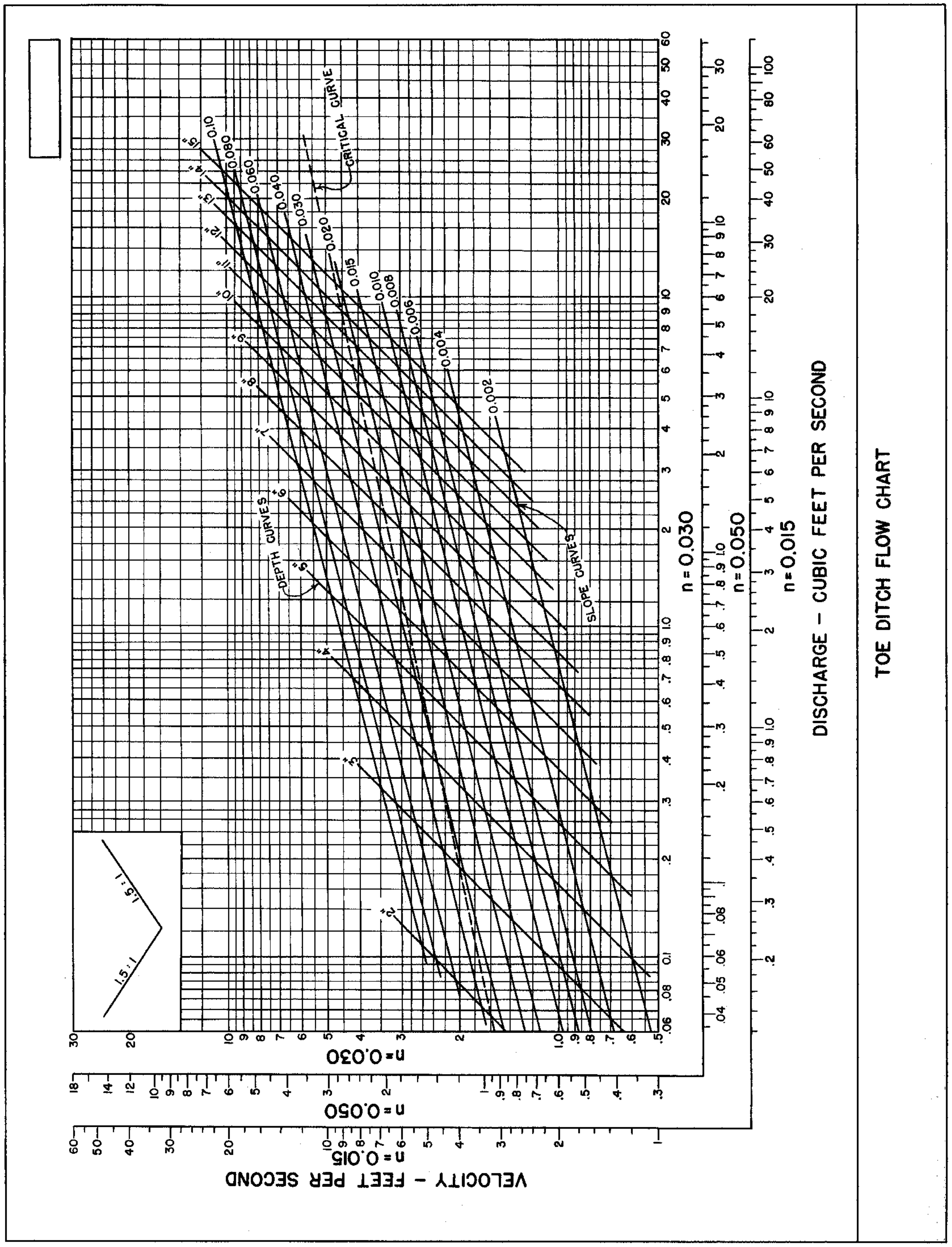
Source: VDOT

Appendix 7C-14 Triangular Median Ditch Flow Chart (Side Slopes = 2:1, 2:1)



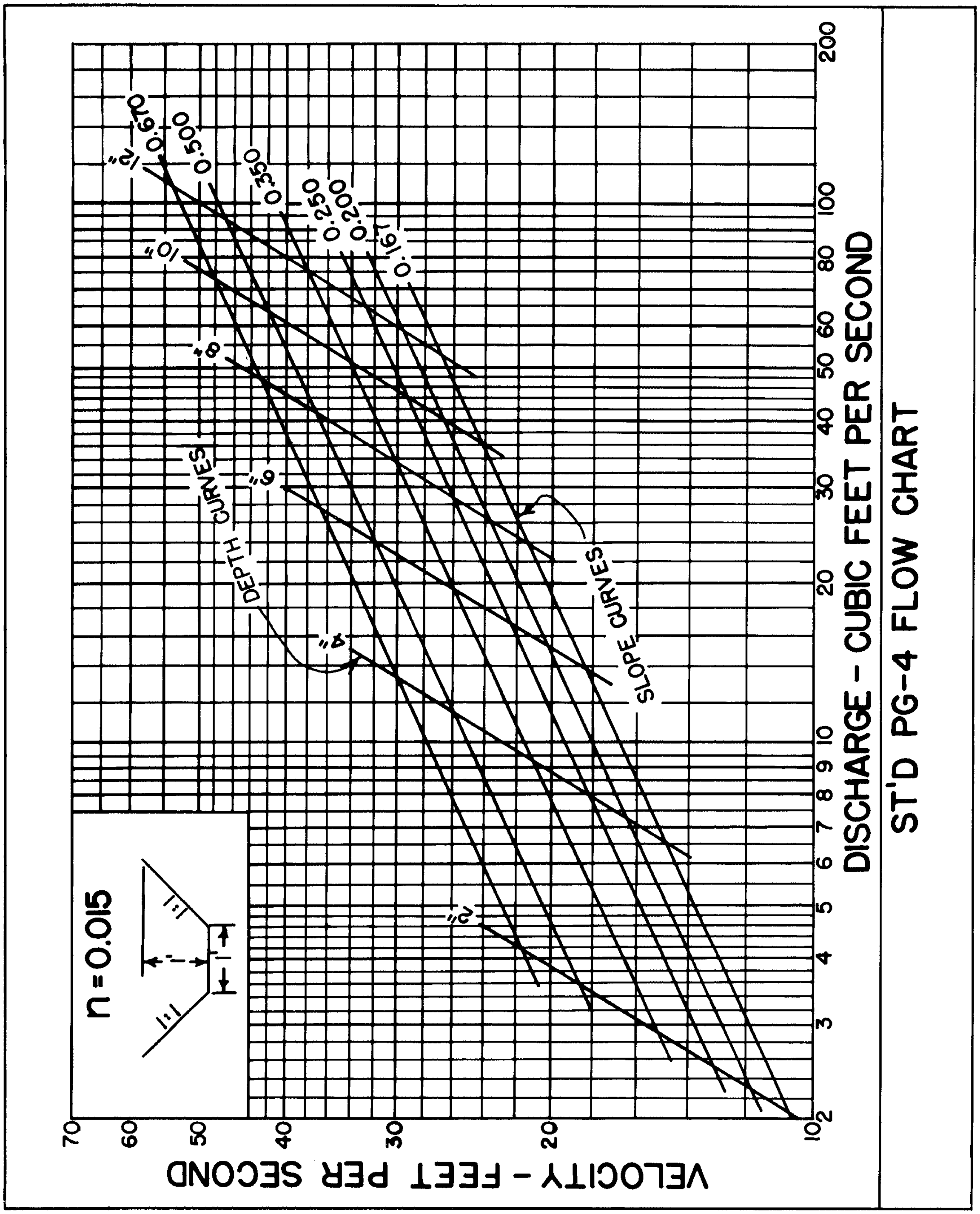
Source: VDOT

Appendix 7C-15 **Toe Ditch Flow Chart**
(Side Slopes = 1.5:1, 1.5:1)



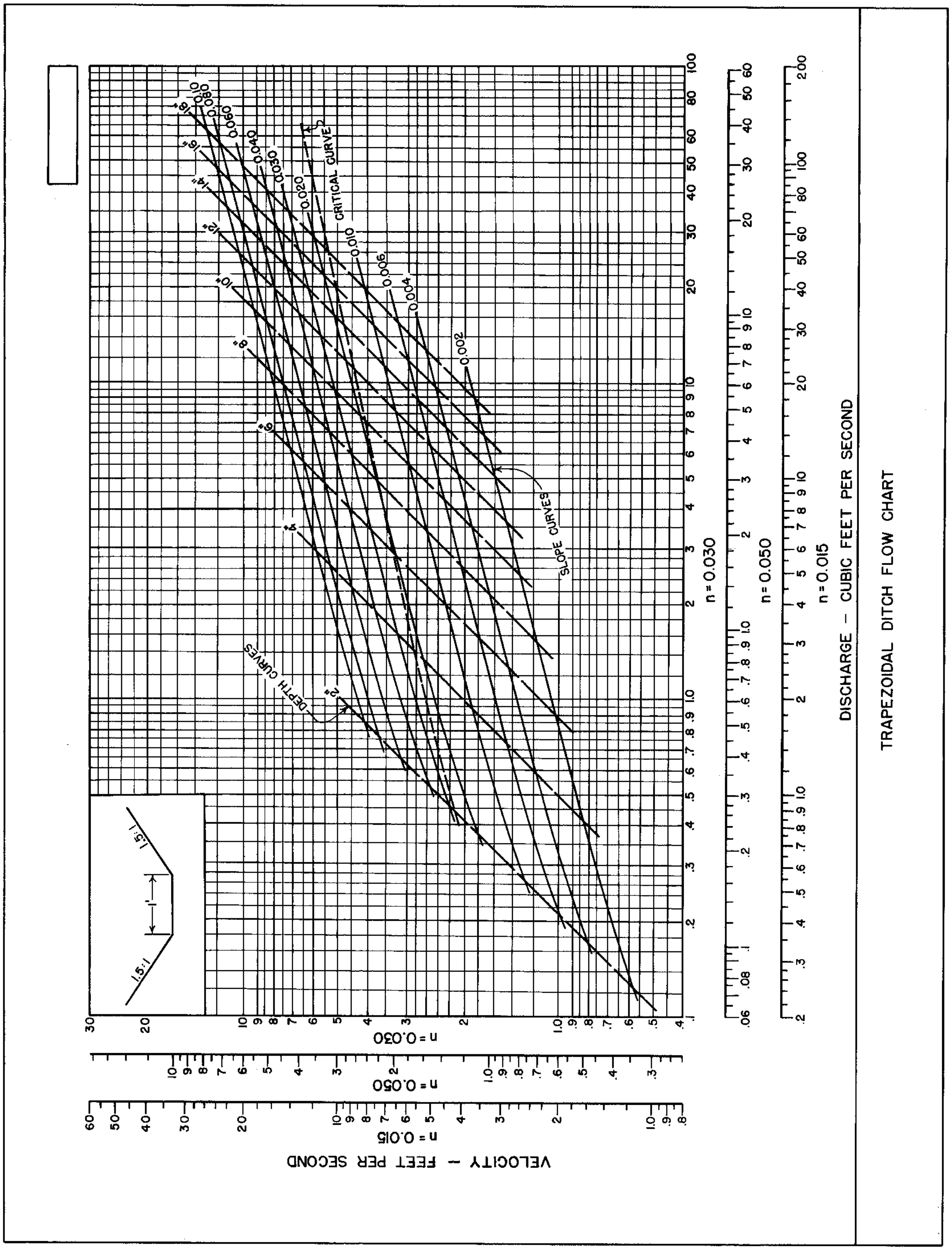
Source: VDOT

Appendix 7C-16 Standard PG-4 Flow Chart



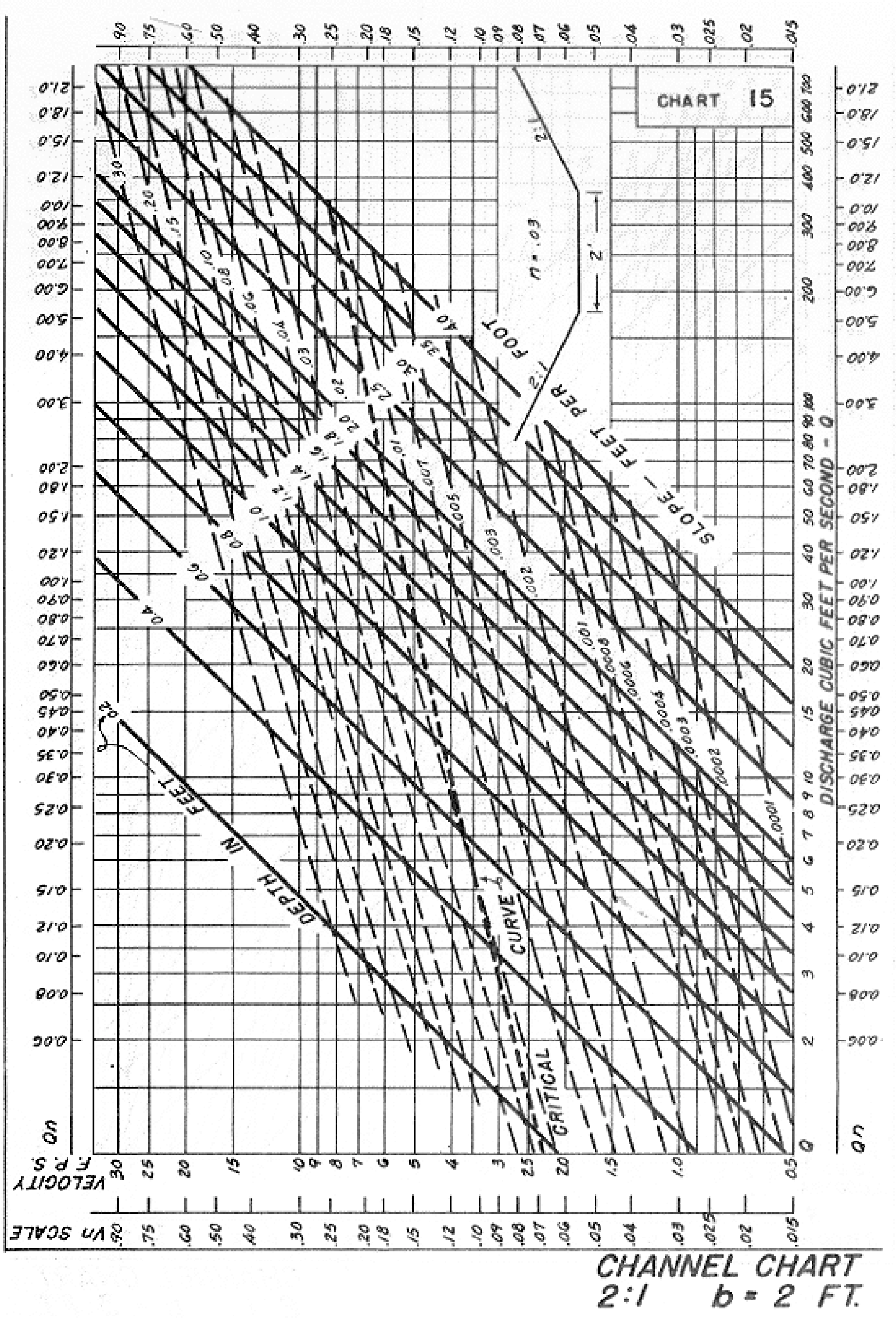
Source: VDOT

Appendix 7C-17 Trapezoidal Ditch Flow Chart (B=1', Side Slopes = 1.5:1)



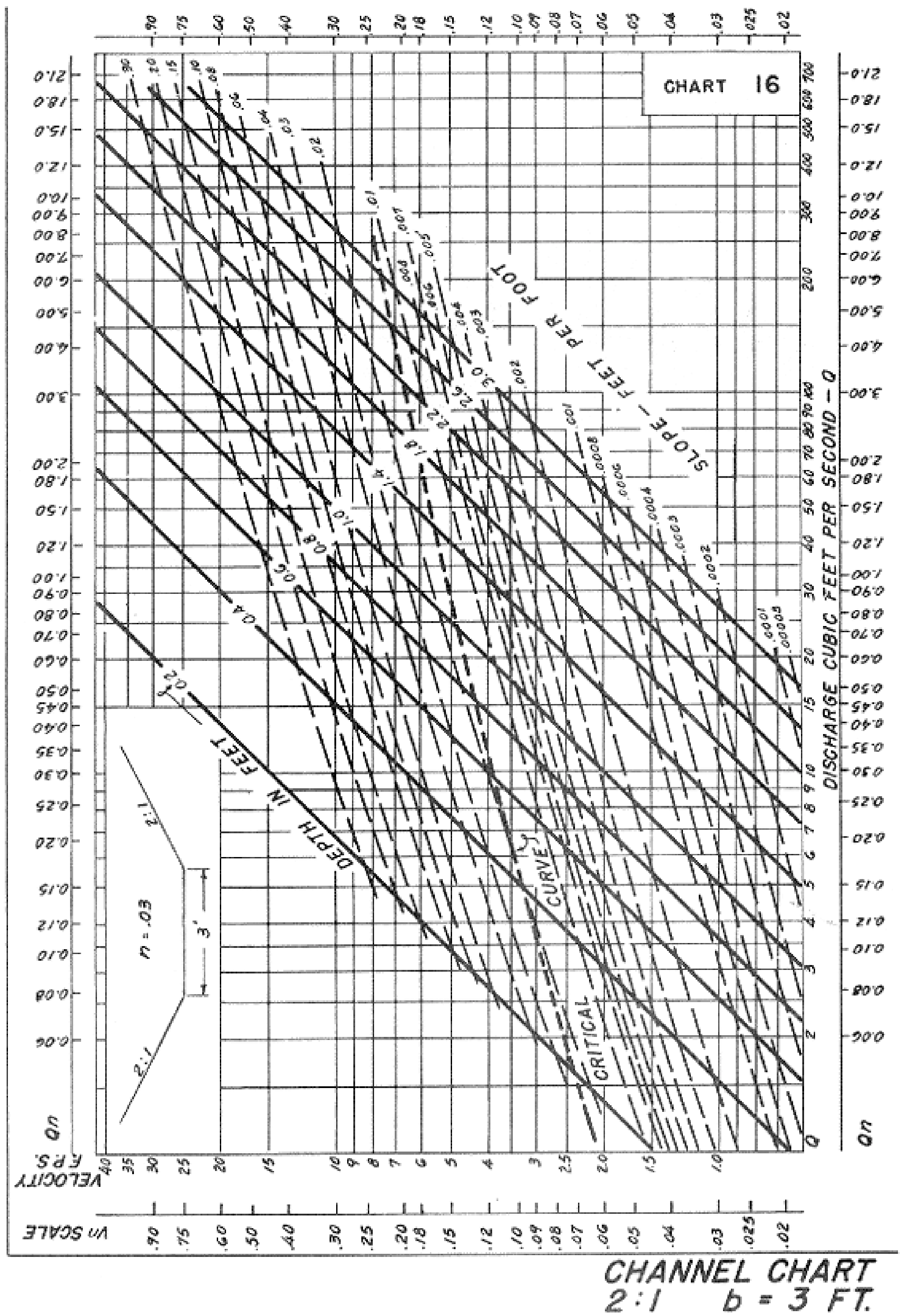
Source: HDS-3

Appendix 7C-18 Trapezoidal Channel Flow Chart (B=2', Side Slopes = 2:1)



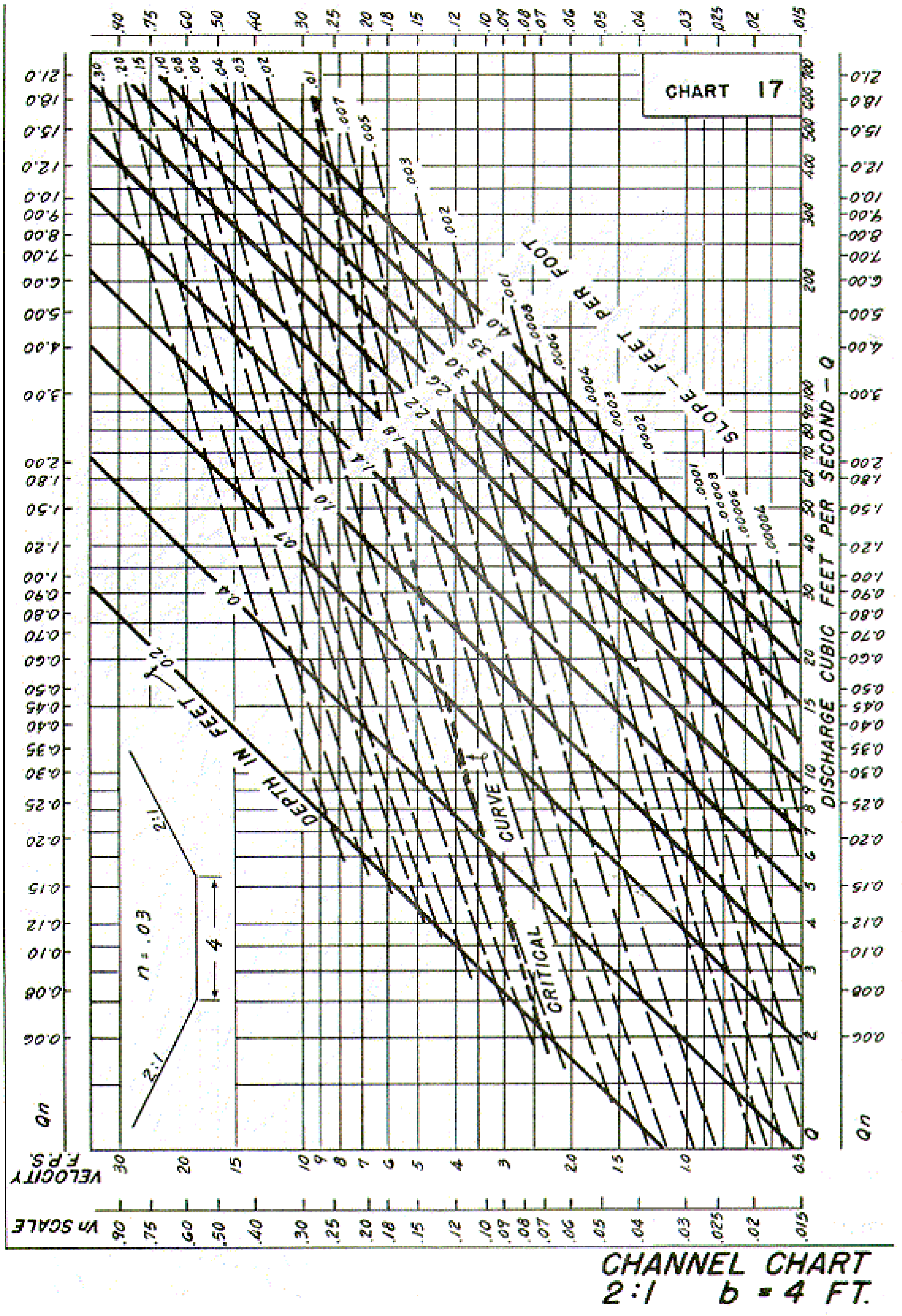
Source: HDS-3

Appendix 7C-19 Trapezoidal Channel Flow Chart (B=3', Side Slopes = 2:1)



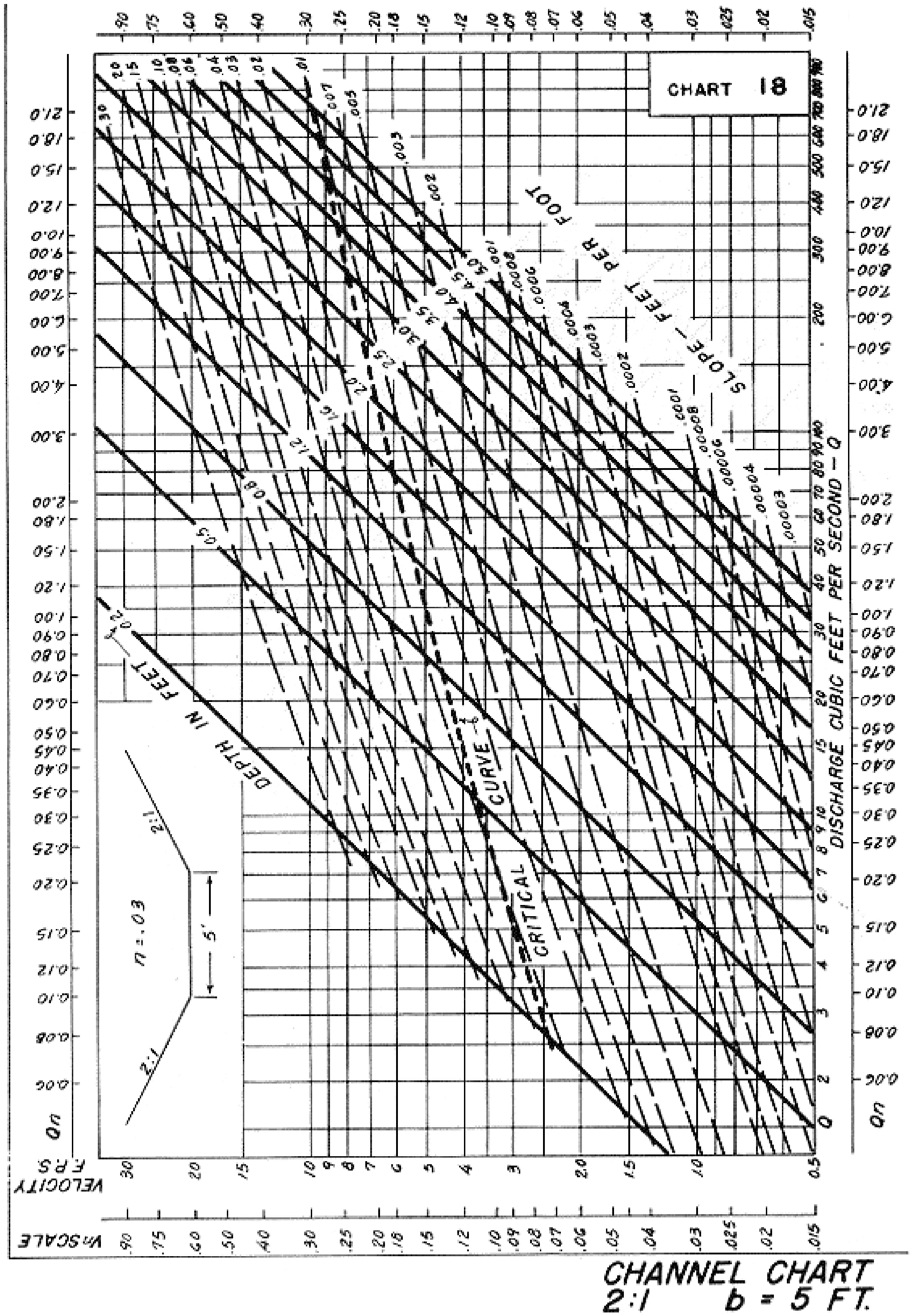
Source: HDS-3

Appendix 7C-20 Trapezoidal Channel Flow Chart (B=4', Side Slopes = 2:1)



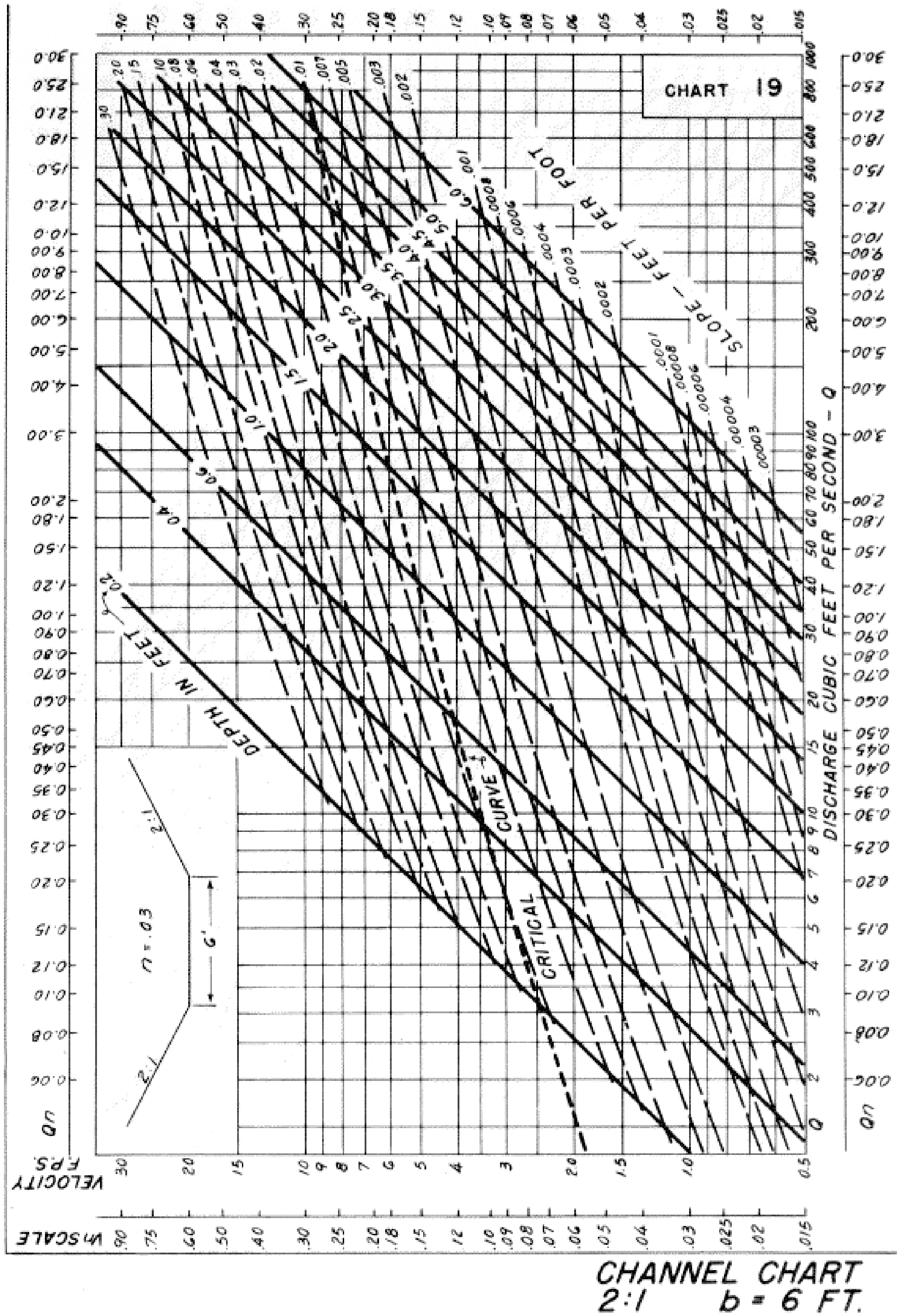
Source: HDS-3

Appendix 7C-21 Trapezoidal Channel Flow Chart (B = 5', Side Slopes = 2:1)



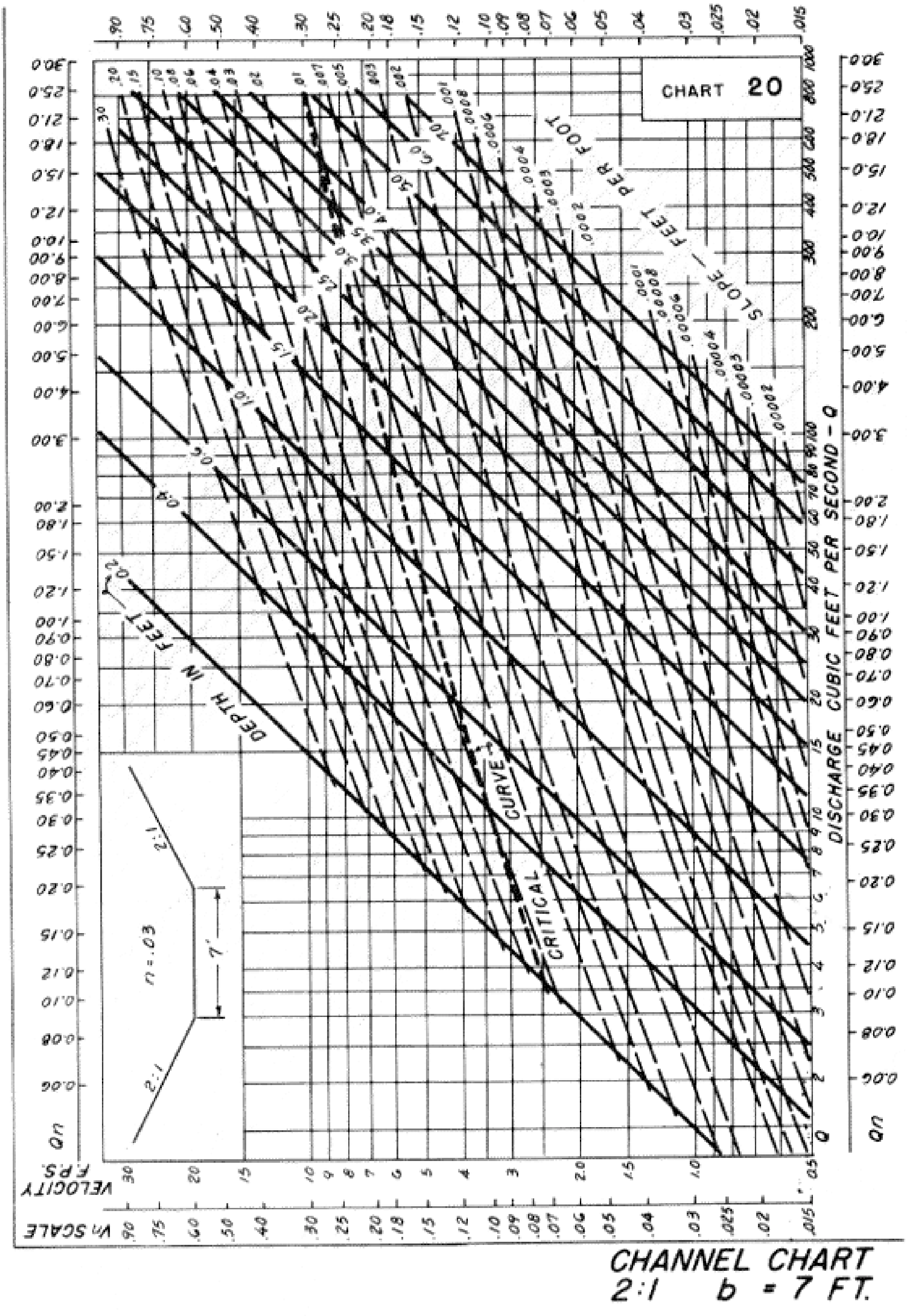
Source: HDS-3

Appendix 7C-22 Trapezoidal Channel Flow Chart (B = 6', Side Slopes 2:1)



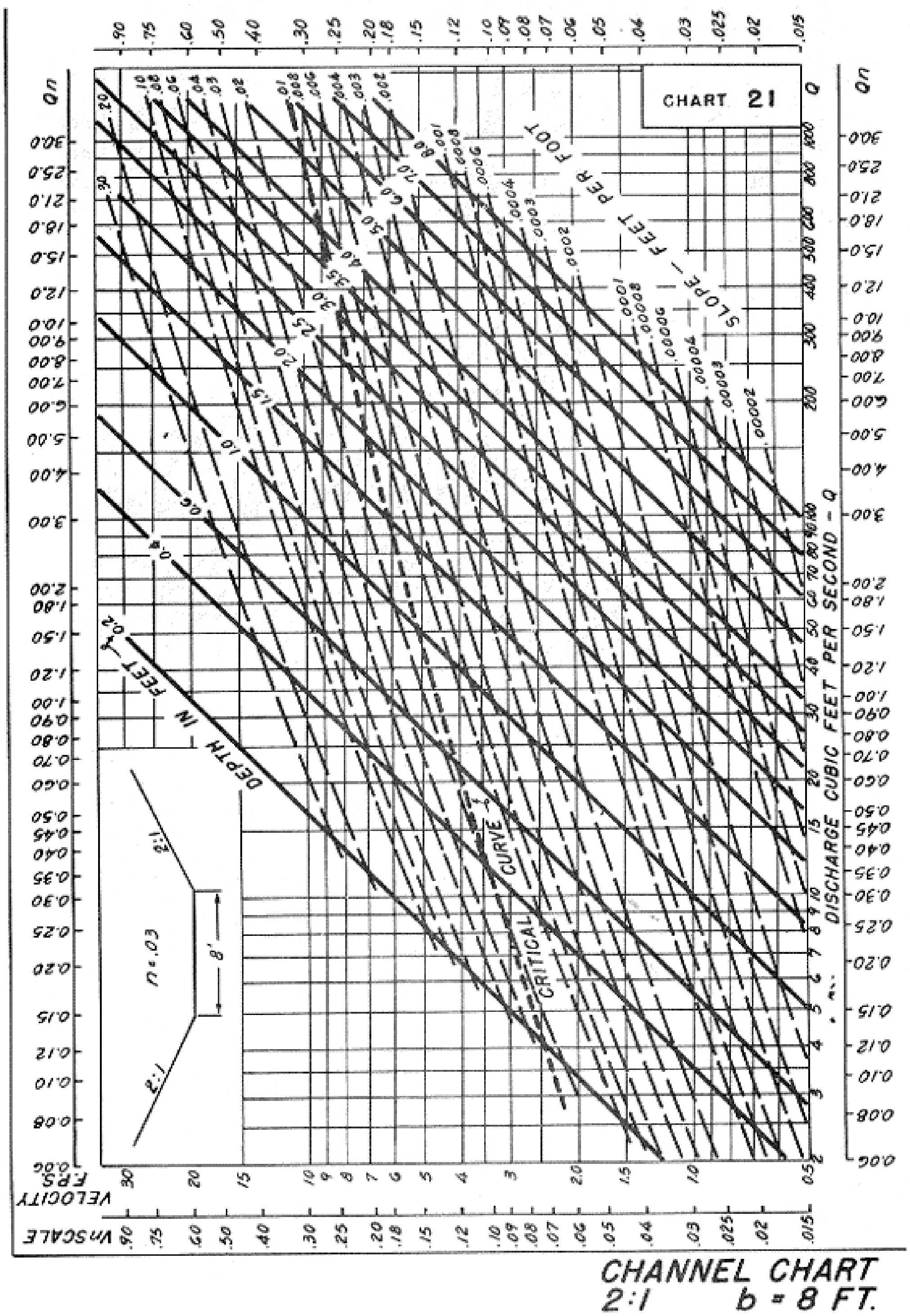
Source: HDS-3

Appendix 7C-23 Trapezoidal Channel Flow Chart (B = 7’, Side Slopes = 2:1)



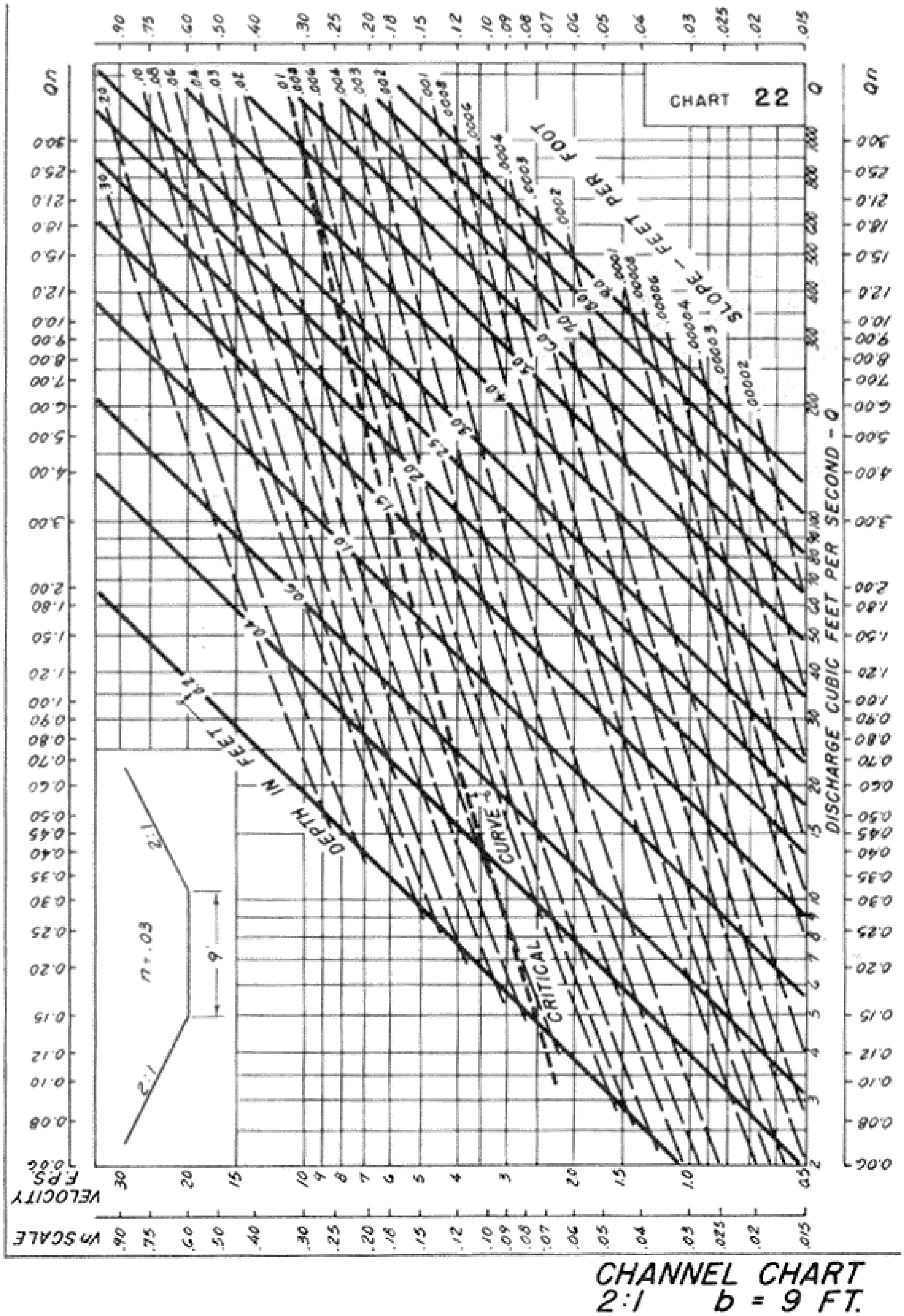
Source: HDS-3

Appendix 7C-24 Trapezoidal Channel Flow Chart (B = 8', Side Slopes = 2:1)



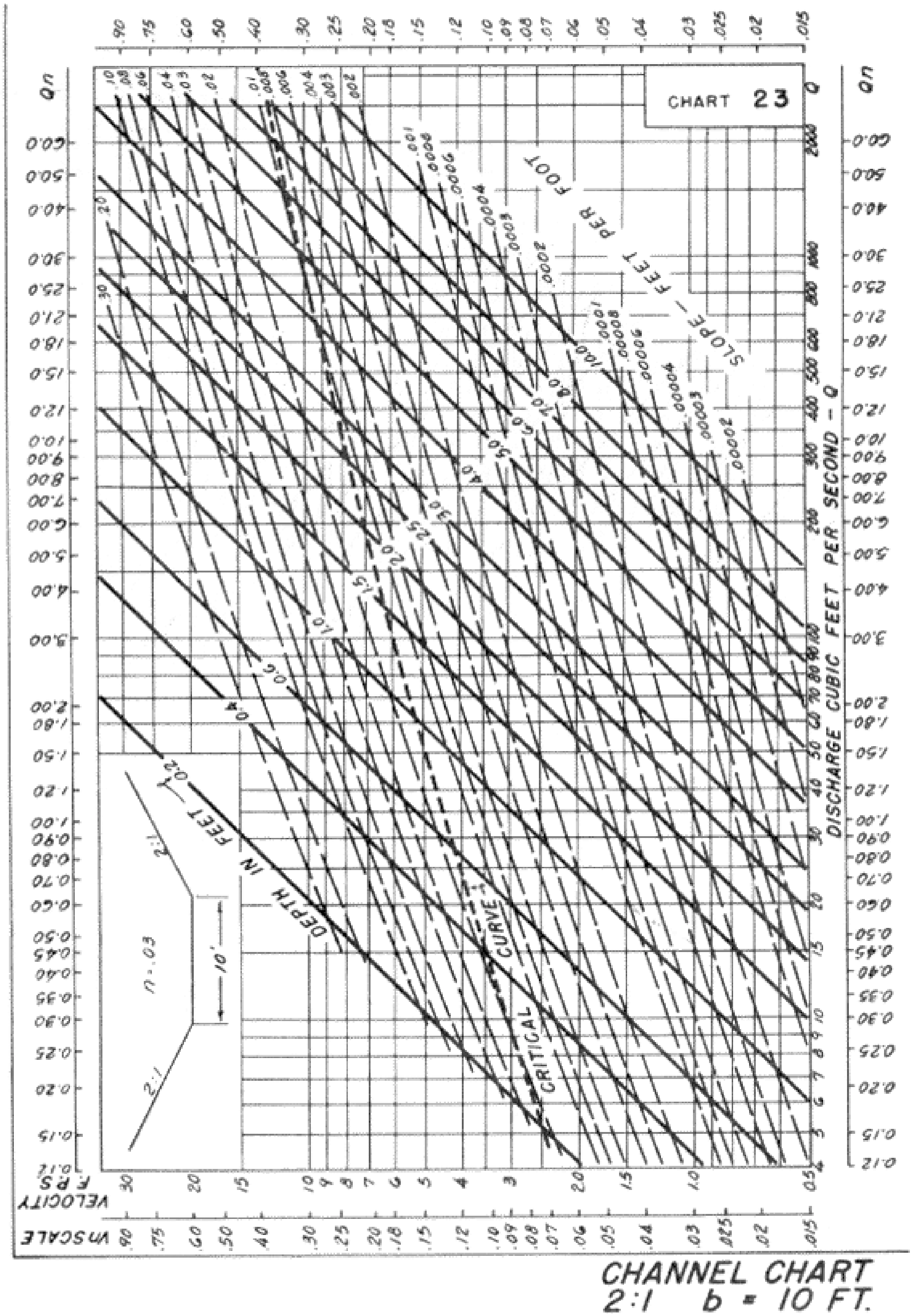
Source: HDS-3

Appendix 7C-25 Trapezoidal Channel Flow Chart (B = 9', Side Slopes = 2:1)



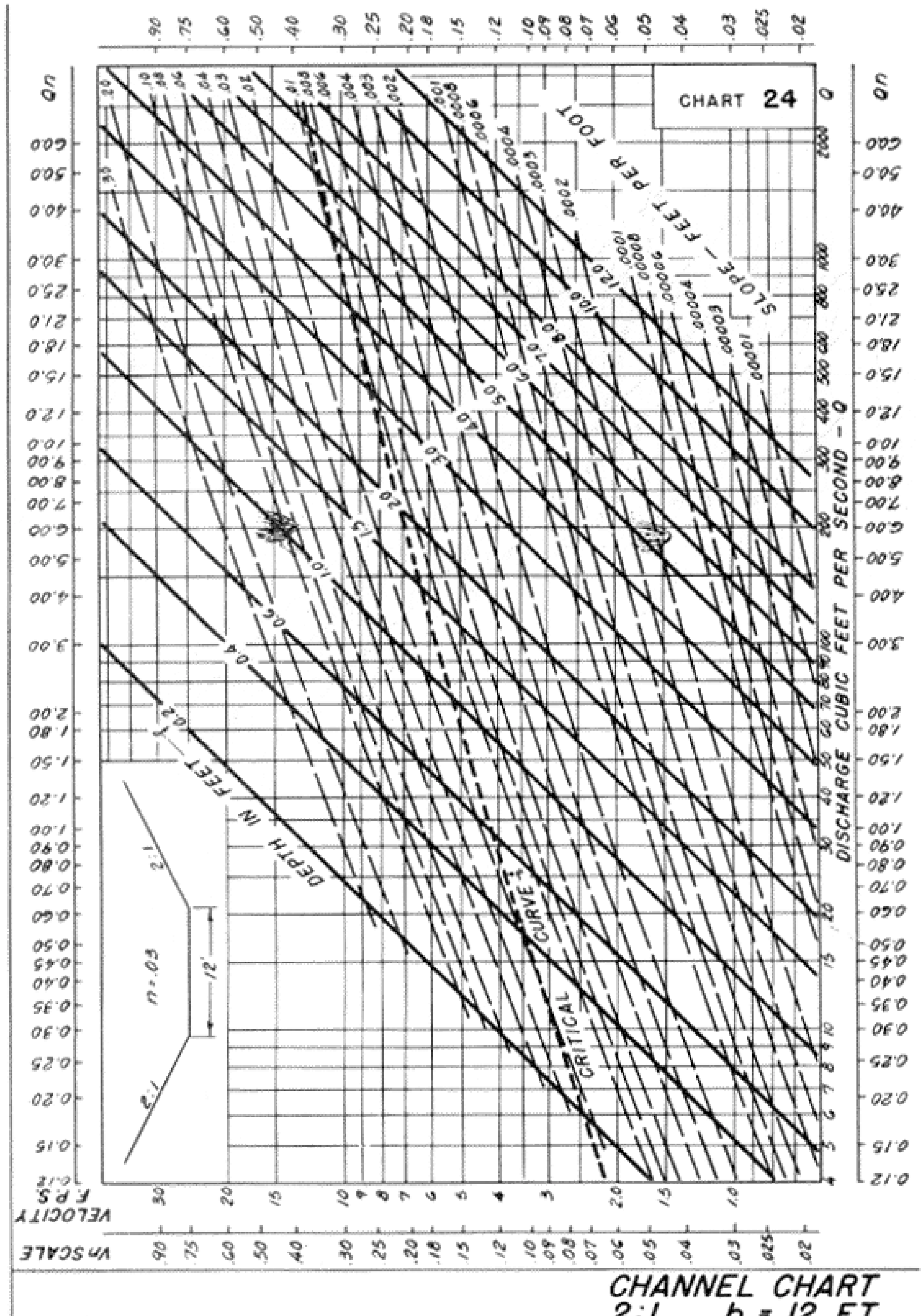
Source: HDS-3

Appendix 7C-26 Trapezoidal Channel Flow Chart (B = 10’, Side Slopes = 2:1)



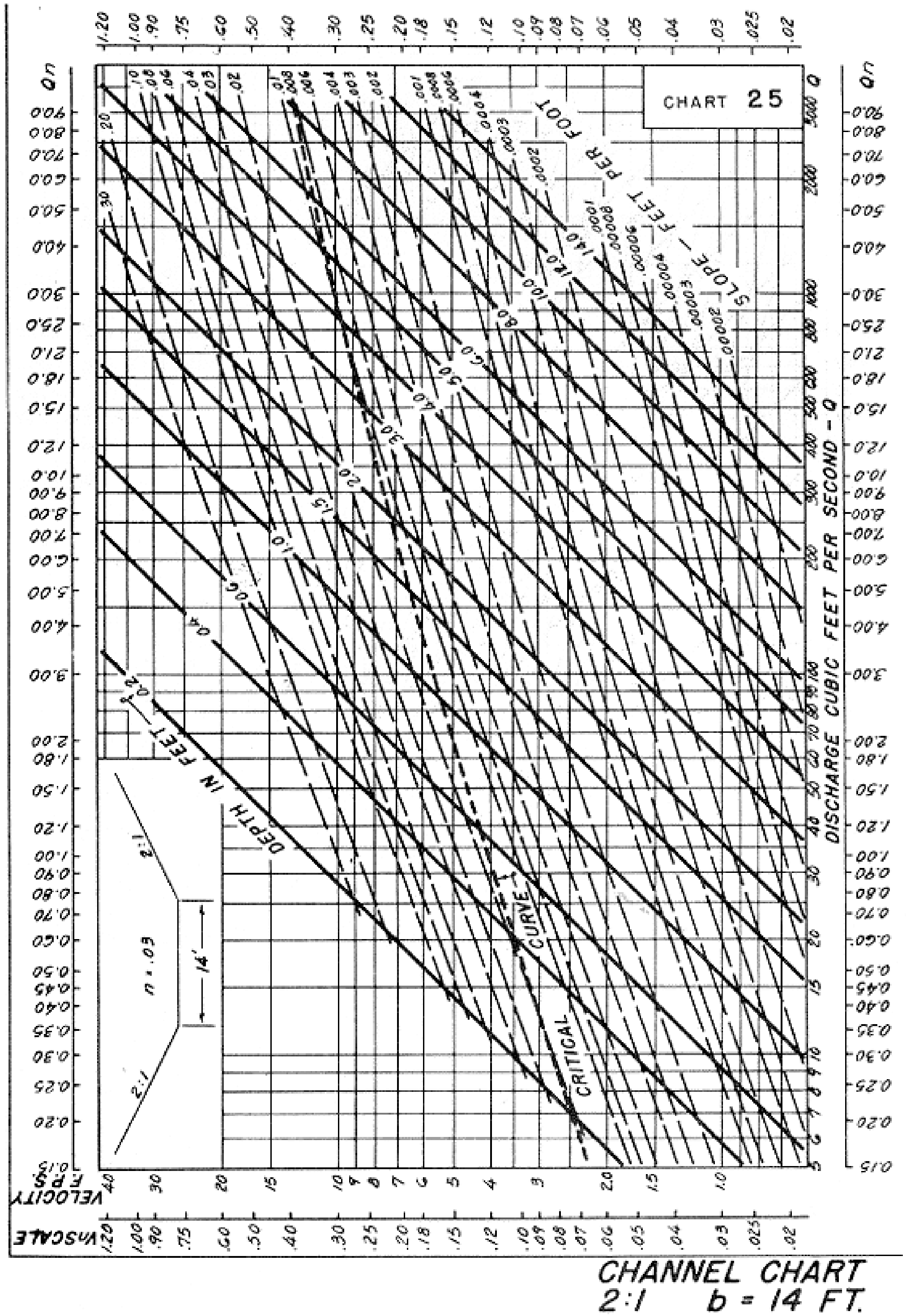
Source: HDS-3

Appendix 7C-27 Trapezoidal Channel Flow Chart (B = 12', Side Slopes = 2:1)



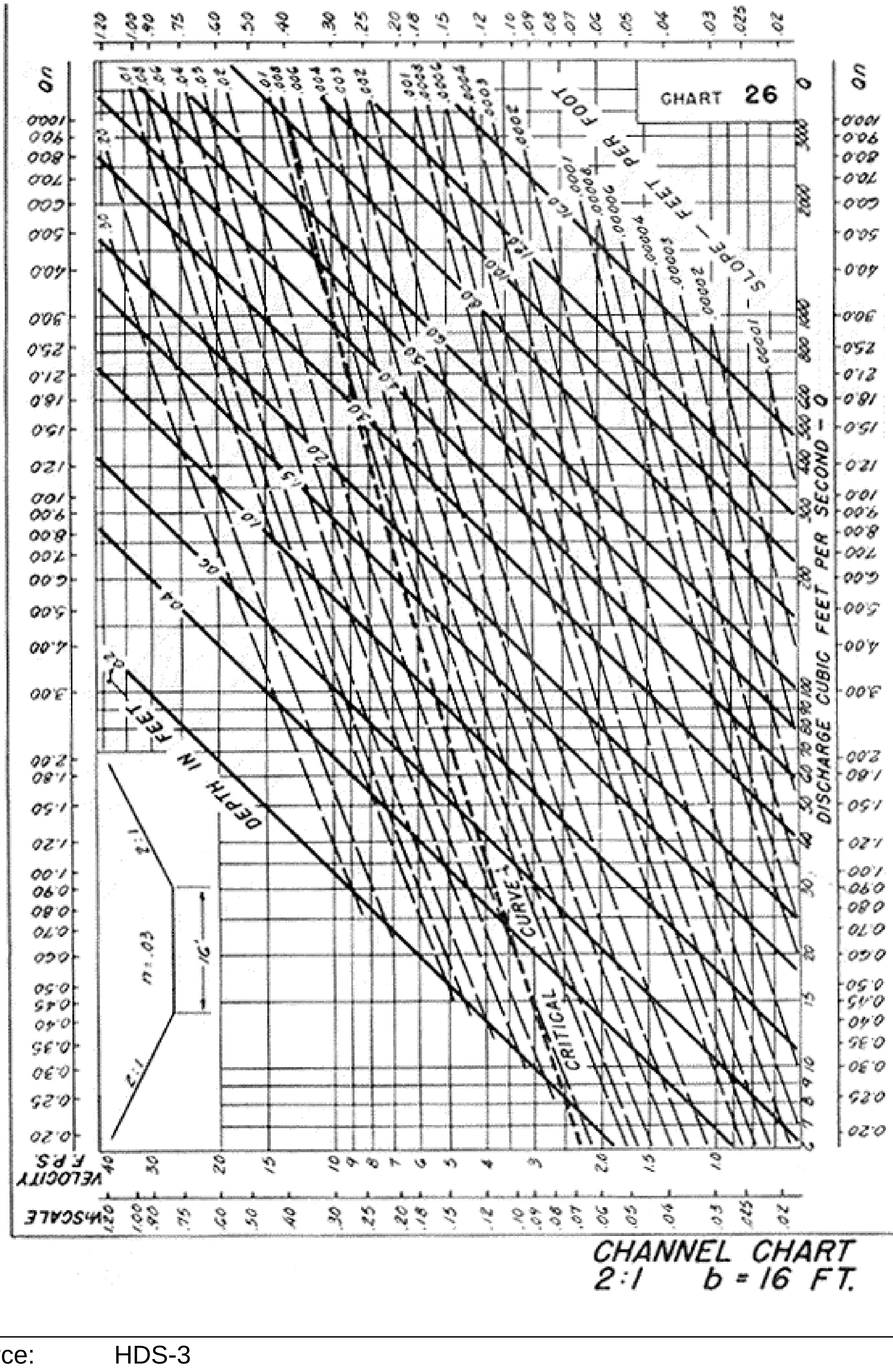
Source: HDS-3

Appendix 7C-28 Trapezoidal Channel Flow Chart (B = 14', Side Slopes = 2:1)



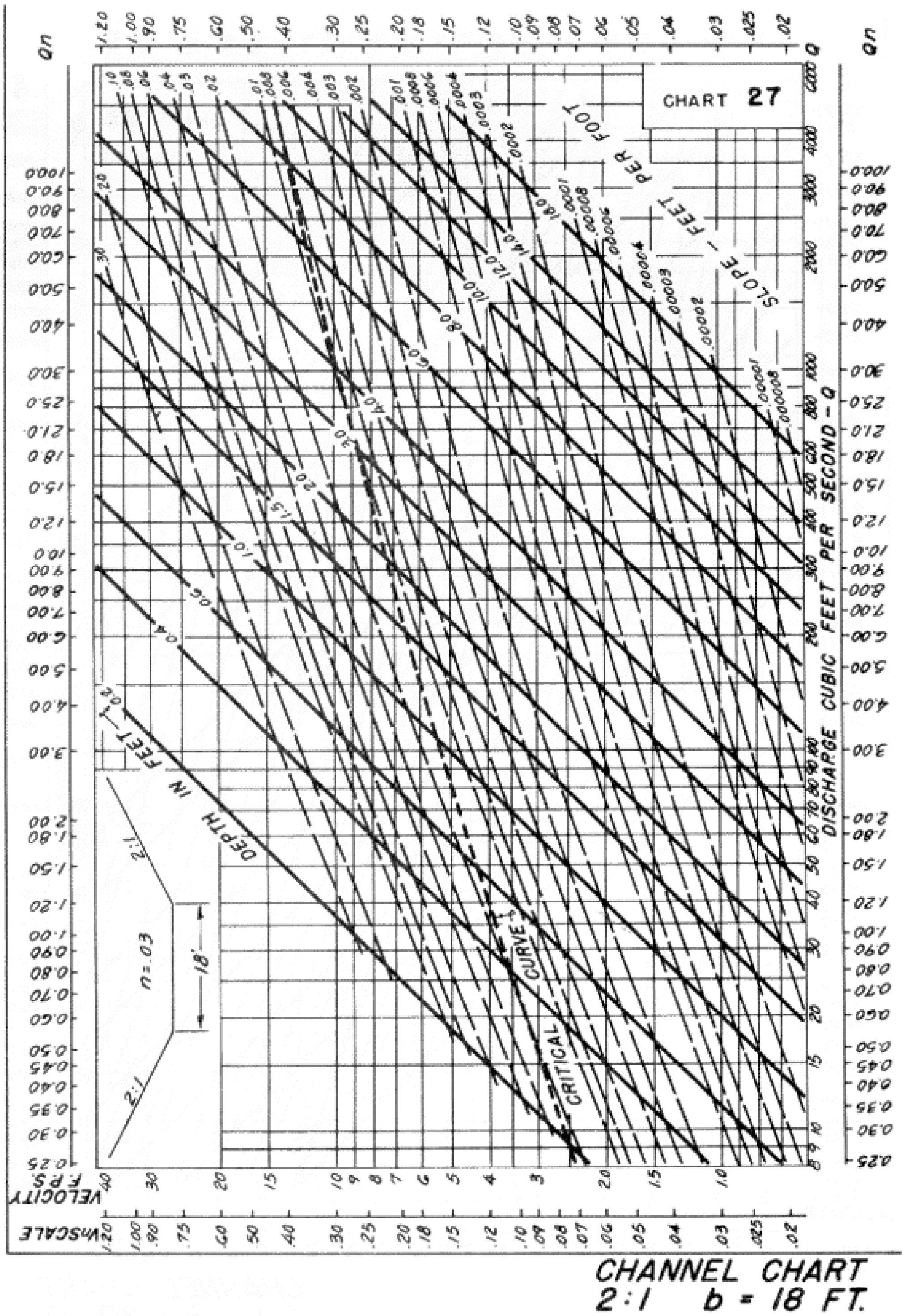
Source: HDS-3

Appendix 7C-29 Trapezoidal Channel Flow Chart (B = 16', Side Slopes = 2:1)



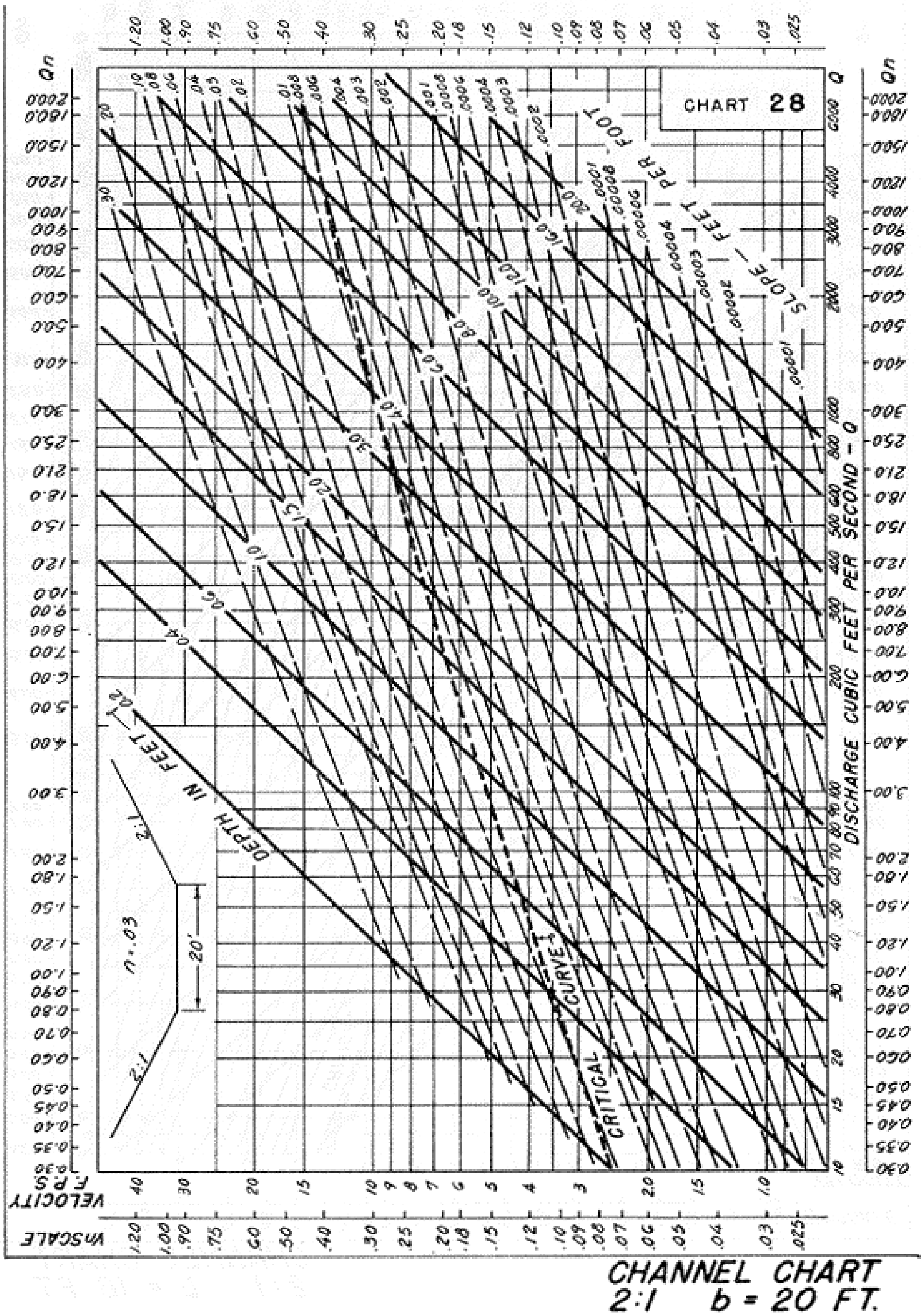
Source: HDS-3

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)

Type of Channel and Description	Minimum	Normal	Maximum
LINED CHANNELS (Selected linings)			
a. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Gunite, good section	0.016	0.019	0.023
b. Asphalt			
1. Smooth	0.013	0.013	-
2. Rough	0.016	0.016	-
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 deep pools	0.025	0.030	0.033
2. Same as above, but more stones/weeds	0.030	0.035	0.040
3. Clean, winding, some pools/shoals	0.033	0.040	0.045
4. Same as above, but some weeds/stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070

Appendix 7D-1 Values of Roughness Coefficient n (Uniform Flow)

Type of Channel and Description	Minimum	Normal	Maximum
2. Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.700	0.100	0.160
d. Trees			
1. Dense Willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
3. Major Streams (top width at flood stage > 100 ft)			
The n-value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	-	0.060
b. Irregular and rough section	0.035	-	0.100

Source: Chow, V.T.

Appendix 7D-2 Recommended Maximum Water Velocities and Manning's n as a Function of Soil Type and Flow Depth

ASSHTO Classification	ASSHTO Soil Description	Fortier and Scobey Soil Description	Maximum Water Velocity (ft/s)	Manning's n -Flow Depth 0.5-2.0 ft
	BROKEN ROCK and COBBLES	Cobbles and Shingles	5.5	0.030
A-1-a	Stone fragments or **GRAVEL,** with or without well-graded [1] binder[2]	Coarse gravel, non-colloidal	4.5	0.025
same	same	Fine gravel	3.5	0.020
A-1-b	Coarse **SAND**, with or without well-graded [1] binder[2]	Graded loam to cobbles when non-colloidal	4.0	0.030
A-2 (A-2-4. A-2-5. A-2-6, A-2-7)	Mixture of **GRAVEL** and **SAND**, with silty or clay fines[3], or nonplastic silt fines	Graded silts to cobbles when colloidal	4.5	0.030
same	same	Sandy loam , non-colloidal	2.0	0.020
A-3	Fine **SAND,** without silty clay fines; e.g. beach sand or stream -deposited fine sand	Fine Sand, non-colloidal	1.5	0.020
same	same	Silt loam, non-colloidal	2.3	0.020
A-4	Non- to moderately plastic[4] **SILT;** mixtures of silt, sand, and/or gravel, with a minimum silt content of 36%	Alluvial silts, non-colloidal	2.3	0.020
A-5	Moderately to highly plastic [4] **SILT**. Soil; mixtures of silt, sand, and/or gravel, with a minimum fines [3] content of 36%	Ordinary firm loam	2.5	0.020
A-6	Plas tic[4] **CLAY** soil; mixtures of clay, sand, and/or gravel, with a minimum fines [3] content of 36%	Alluvial silts, colloidal	3.5	0.025
A-7	Moderately to highly plastic, **CLAY**; mixtures of clay, sand, and/or gravel, with a minimum clay content of 36%	Stiff clay, very colloidal	4.0	0.025

1) Well-graded-containing a broad range of particle sizes with no intermediate sizes missing.
1) Binder - soil particles consisting of fine sand, silt, and clay.
2) Fines - particle sizes finer than 0.074 mm (e.g., silt and clay particles).
3) 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.

Appendix 7D-3 Standard VDOT Riprap Classifications, Weights, and Blanket Thickness

Classification	D_{50} (ft)	W_{50} (lbs)	T (in)
Class AI	0.8	50	20
Class I	1.1	100	26
Class II	1.6	300	38
Class III	2.2	1000	53
Type I	2.8	2000	60
Type II	4.5	8000	97

Appendix 7D-4 Approximate Rock Dimensions and Equivalent Weights for Riprap

WEIGHT	MEAN SPERICAL DIAMETER	RECTANGULAR SHAPE	
		LENGTH	HT./WIDTH
25 lbs.	0.7’	1.1’	0.4’
50 lbs.	0.8’	1.4’	0.5’
75 lbs.	1.0’	1.6’	0.5’
100 lbs.	1.1’	1.75’	0.6’
150 lbs.	1.3’	2.0’	0.67’
300 lbs.	1.6’	2.6’	0.9’
500 lbs.	1.9’	3.0’	1.0’
1000 lbs.	2.2’	3.7’	1.25’
1500 lbs	2.6’	4.7’	1.5’
2000 lbs.	2.75’	5.4’	1.8’
2 tons	3.6’	6.0’	2.0’
3 tons	4.0’	6.9’	2.3’
4 tons	4.5’	7.6’	2.5’
10 tons	6.1’	10.0’	3.3’

Appendix 7D-5 Selection of Stability Factors

CONDITION	STABILITY FACTOR RANGE
Uniform flow; straight or mildly curving reach (curve radius/ channel width >30); impact from wave action and floating debris is minimal; little or no uncertainity in design parameters.	1.0 - 1.2
Gradually varying flow; moderate bend curvature (30 > curve radius/channel width > 10); impact from waves or floating debris is moderate.	1.3 - 1.6
Approaching rapidly varying flow; sharp bend curvature (30 > curve radius/channel width >10); significant impact potential from floating debris and/or ice; significant wind and/or bore generated wves (1-2 ft); high flow turbulence; mixing flow at bridge abutments; significant uncertainty in design parameters.	1.6 - 2.0
Channel bends when ratio of curve radius to channel width (R/W) > 30.	1.2
Channel bends when 30 > R/W > 10.	1.3 - 1.6
Channel bends when R/W < 10.	1.7

Appendix 7D-6 Permissible Velocities for Erodible Linings

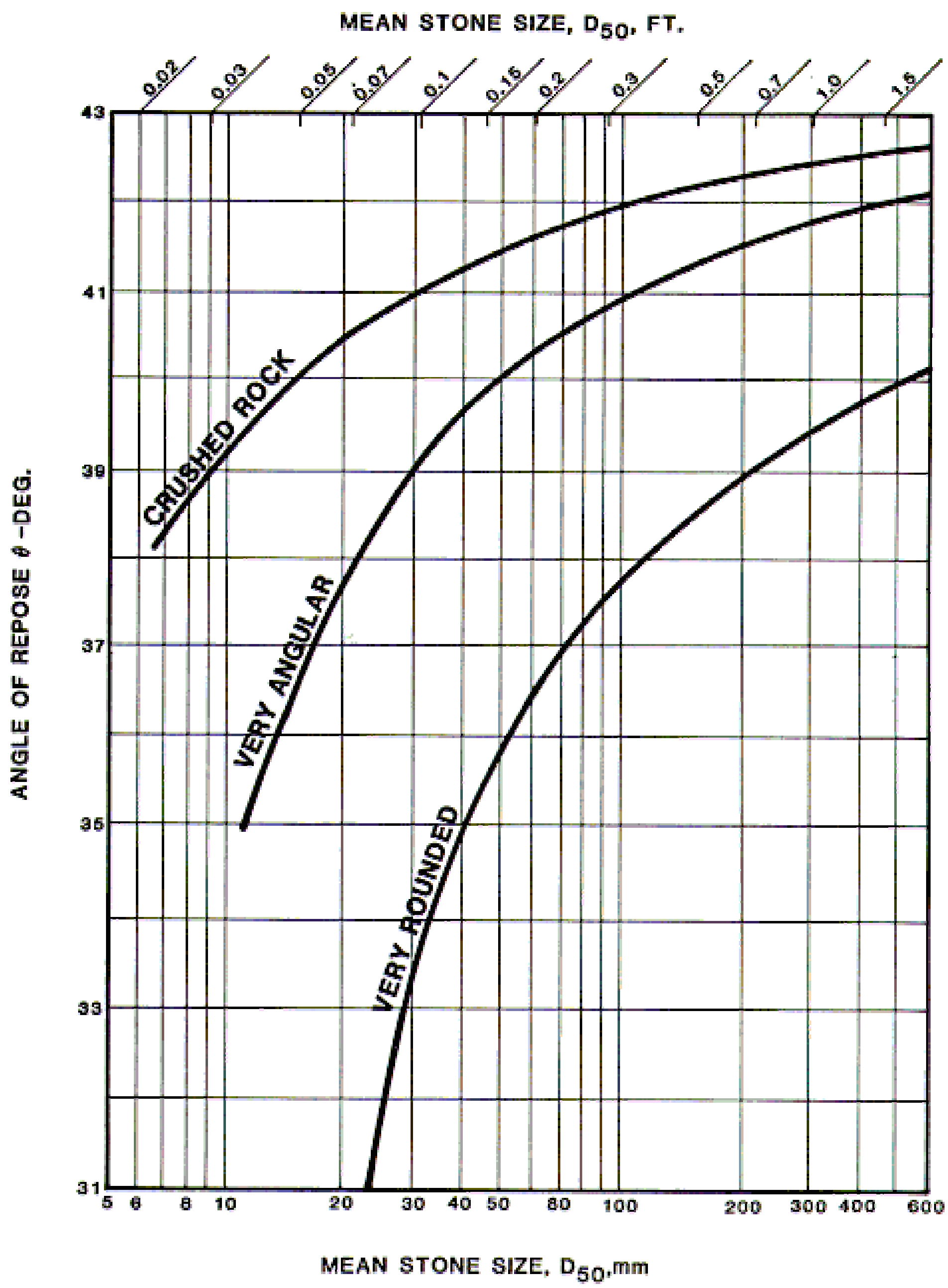
Permissible velocities for channels with erodible linings, based on uniform flow in continuously wet, aged channels[1]:

Soil type or lining (earth; no vegetation)	Maximum permissible velocities for		
	Clear water	Water carrying fine silts	Water carrying sand and gravel
	F.p.s.	*F.p.s.*	*F.p.s.*
Fine sand (noncolloidal)	1.5	2.5	1.5
Sandy loam (noncolloidal)	1.7	2.5	2.0
Silt loam (noncolloidal)	2.0	3.0	2.0
Ordinary firm loam	2.5	3.5	2.2
Volcanic ash	2.5	3.5	2.7
Fine gravel	2.5	5.0	3.7
Stiff clay (very colloidal)	3.7	5.0	3.0
Graded, loam to cobbles (noncolloidal)	3.7	5.0	5.0
Graded, silt to cobbles (colloidal)	4.0	5.5	5.0
Alluvial silts (noncolloidal)	2.0	3.5	2.0
Alluvial silts (colloidal)	3.7	5.0	3.0
Coarse gravel (noncolloidal)	4.0	6.0	6.5
Cobbles and shingles	5.0	5.5	6.5
Shales and hard pans	6.0	6.0	5.0

Source:

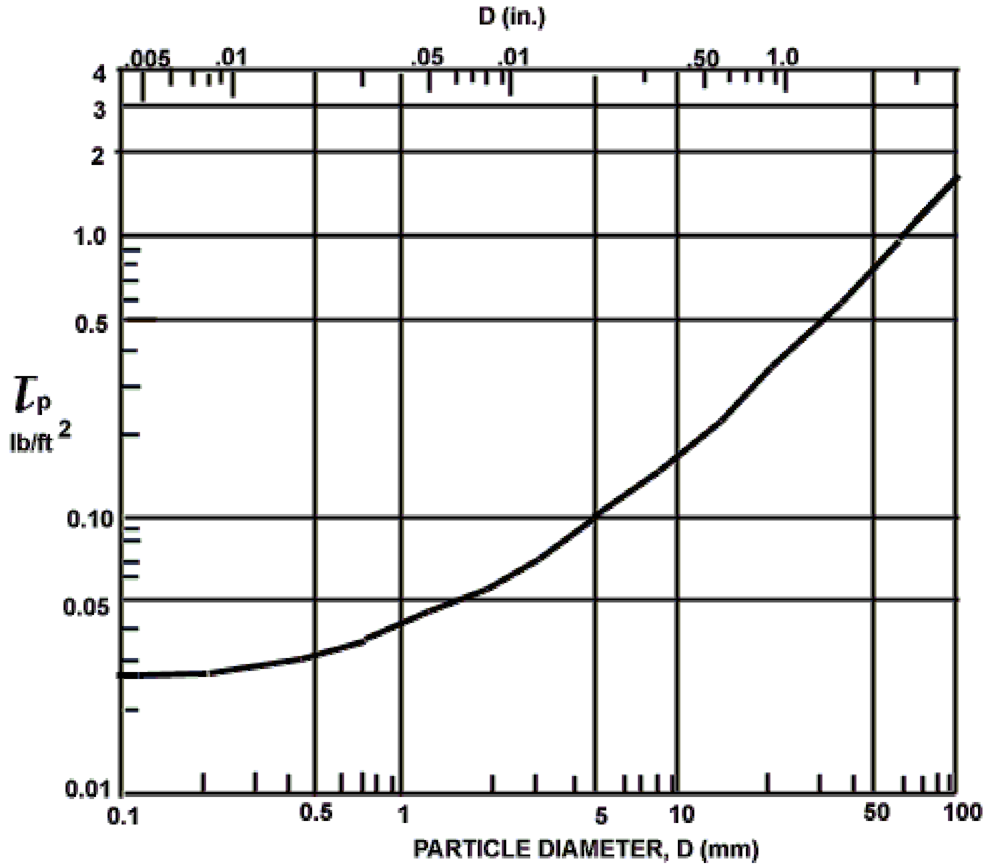
[1]As recommended by Special Committee on Irrigation Research, American Society of Civil Engineers, 1926.

Appendix 7E-1 Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone



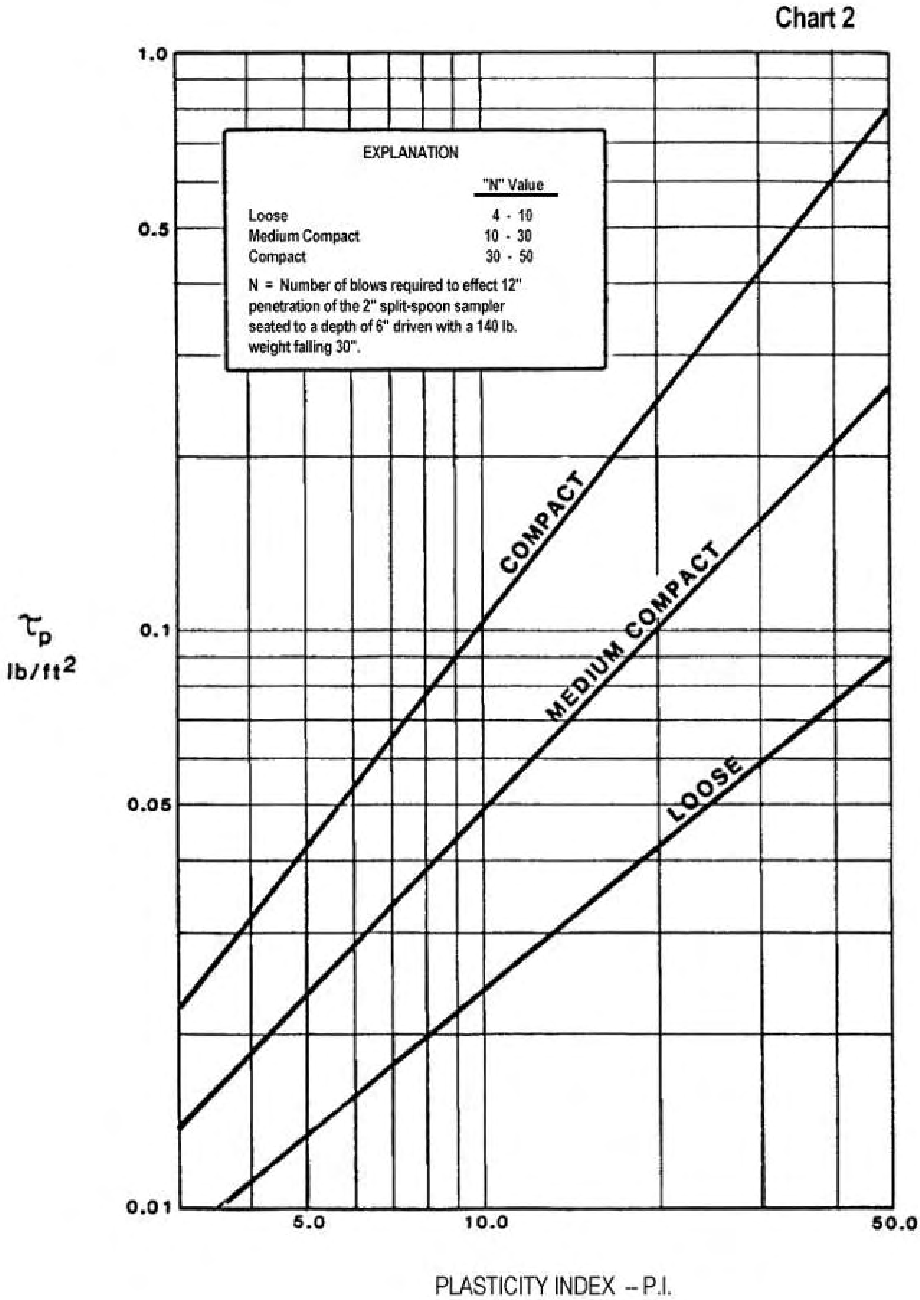
Source: HEC-15

Appendix 7E-2 Permissible Shear Stress for Non-Cohesive Soils



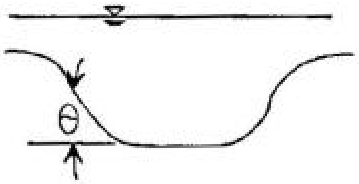
Source: HEC-15

Appendix 7E-3 Permissible Shear Stress for Cohesive Soils



Source: HEC-15

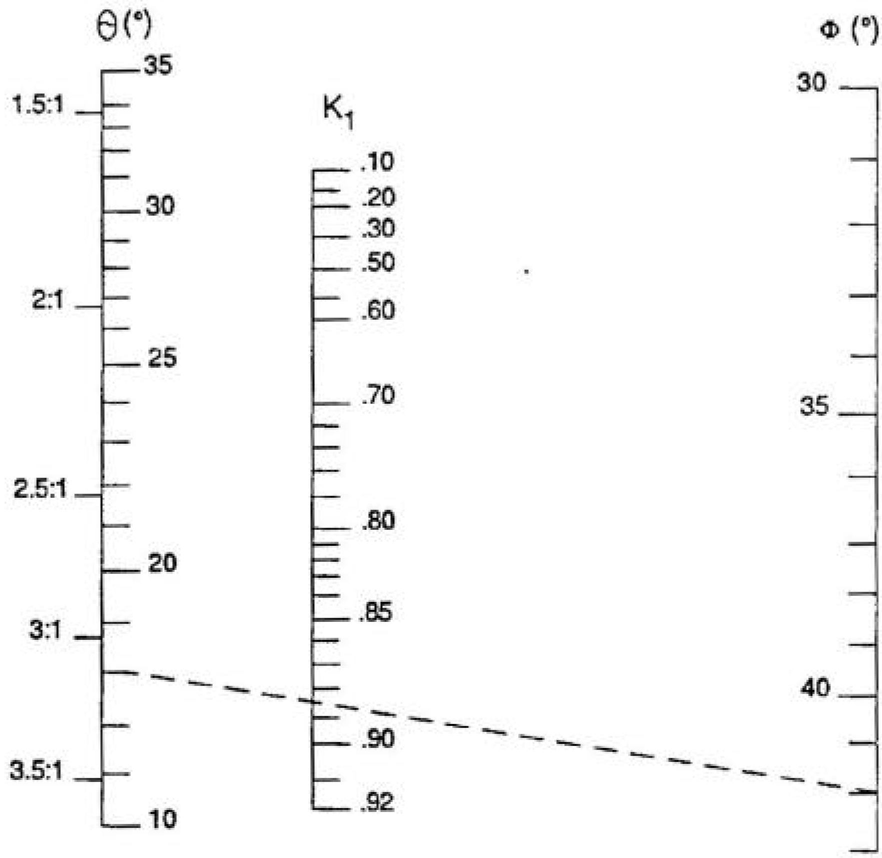
Appendix 7E-4 Bank Angle Correction Factor (K_1) Nomograph



$$K_1 = \left[1 - \frac{\sin^2\theta}{\sin^2\Phi}\right]^{0.5}$$

θ = Bank angle with horizontal

Φ = Material angle of repose (See chart 4)



Example

Given:
θ = 18°
Very Angular
D_{50} = 1.5 ft.

Find:
K_1

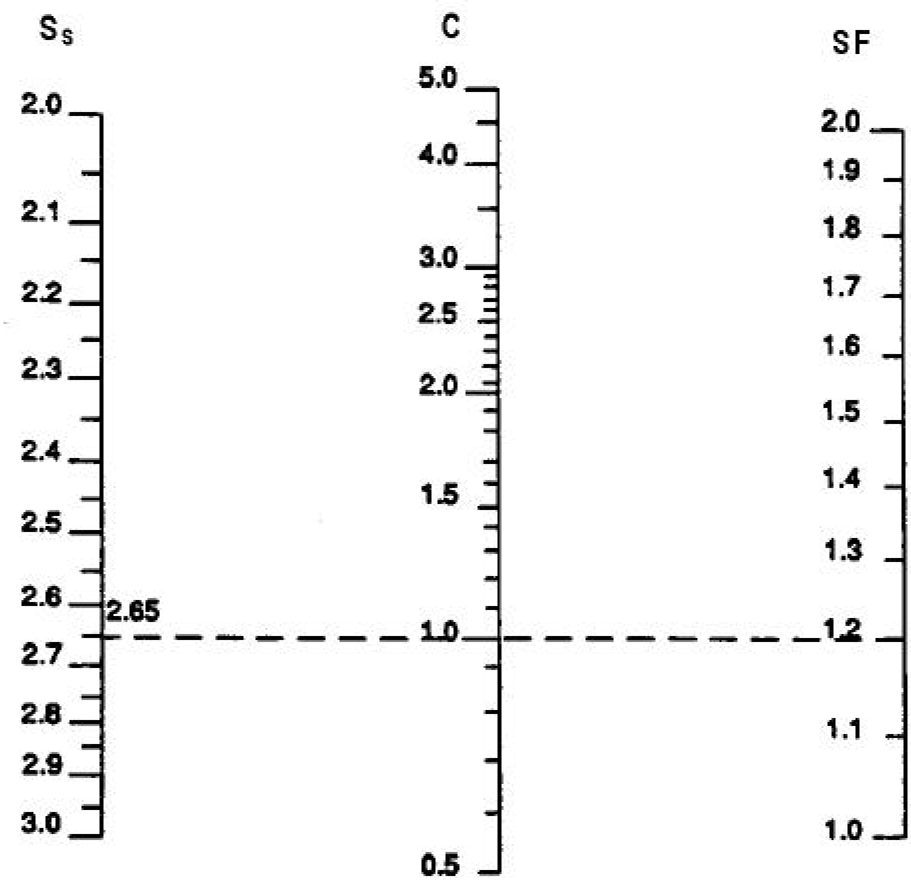
Solution:
Φ = 42°
K_1 = 0.885

Source: HEC-11

Appendix 7E-5 Correction Factor for Riprap Size

$C = 1.61\ SF^{1.5}/(S_S - 1)^{1.5}$

C = D_{50} CORRECTION FACTOR
SF = STABILITY FACTOR
S_S = SPECIFIC GRAVITY OF ROCK



Example:

Given:
S_S = 2.65
SF = 1.2

Solution:
C = 1.0

Source: HEC-11
Comment: S_s=S_g (text)

Appendix 7E-6 Riprap Size Relationship

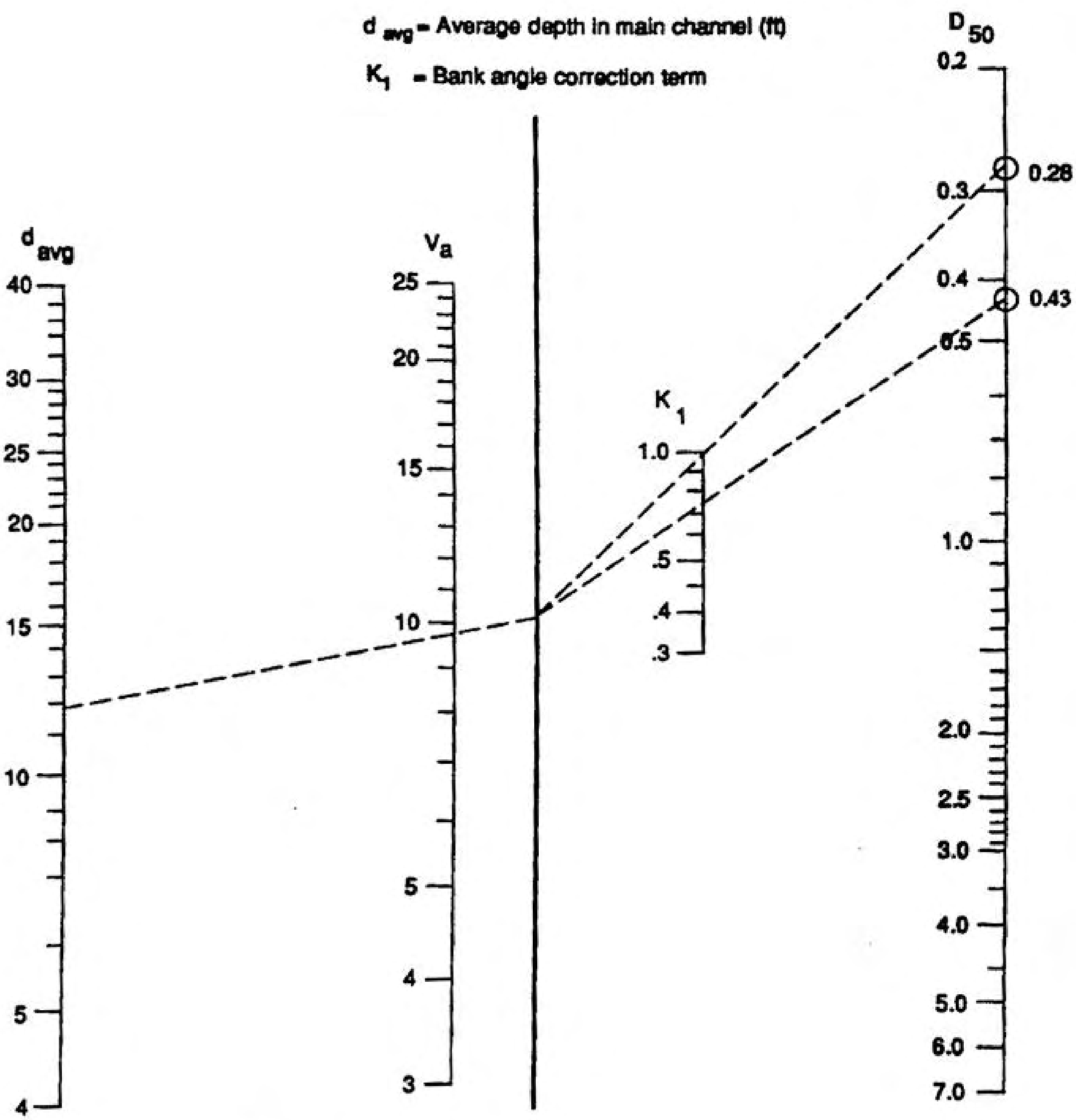
$$D_{50} = 0.001\, V_a^3 / (d_{avg}^{1/2} K_1^{3/2})$$

D_{50} = Median Riprap Size (ft.)

V_a = Average velocity in main channel (ft/sec)

d_{avg} = Average depth in main channel (ft)

K_1 = Bank angle correction term



Example

Given:
$V_a = 9.7$
$d_{avg} = 11.8$ ft.
$K_1 = 0.73$

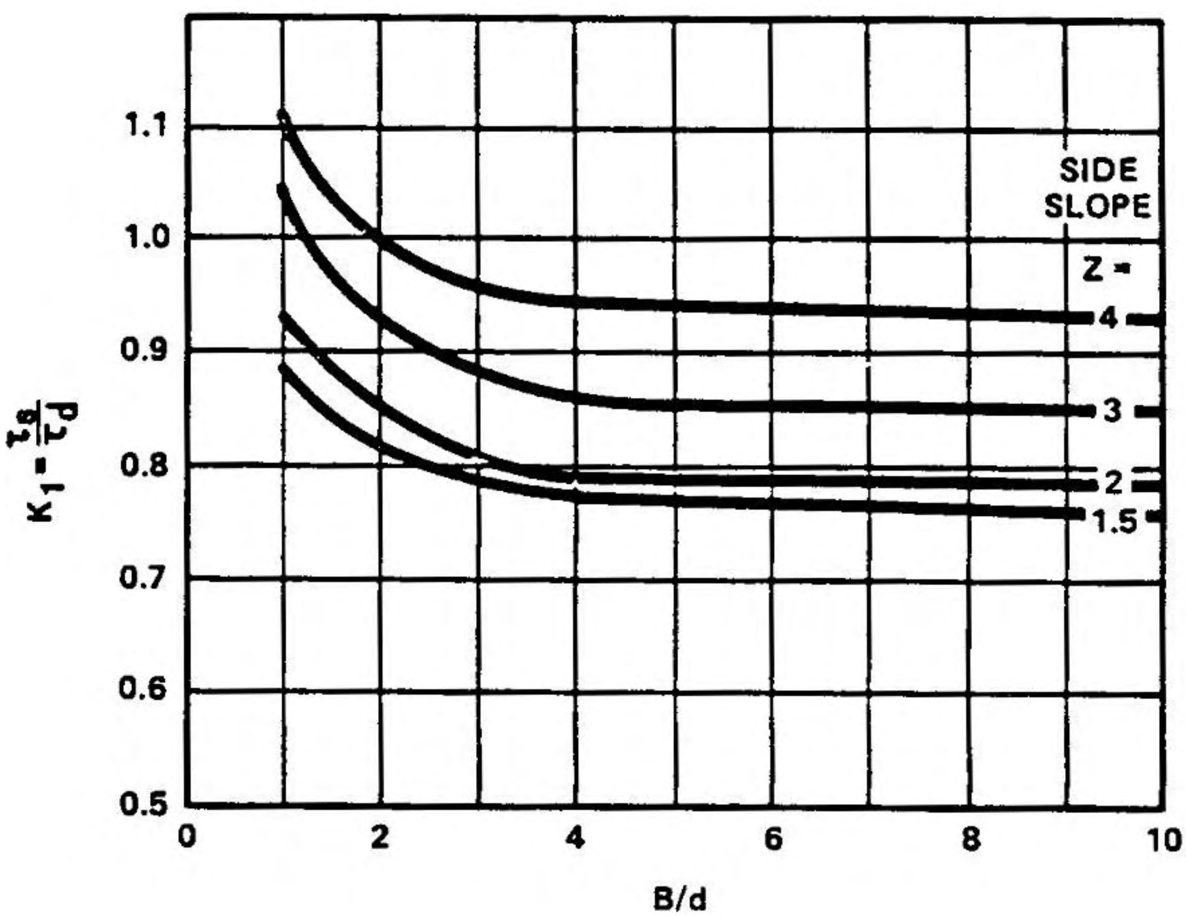
Find:
D_{50}

Solution:
$D_{50} = 0.43$

Source: HEC-11

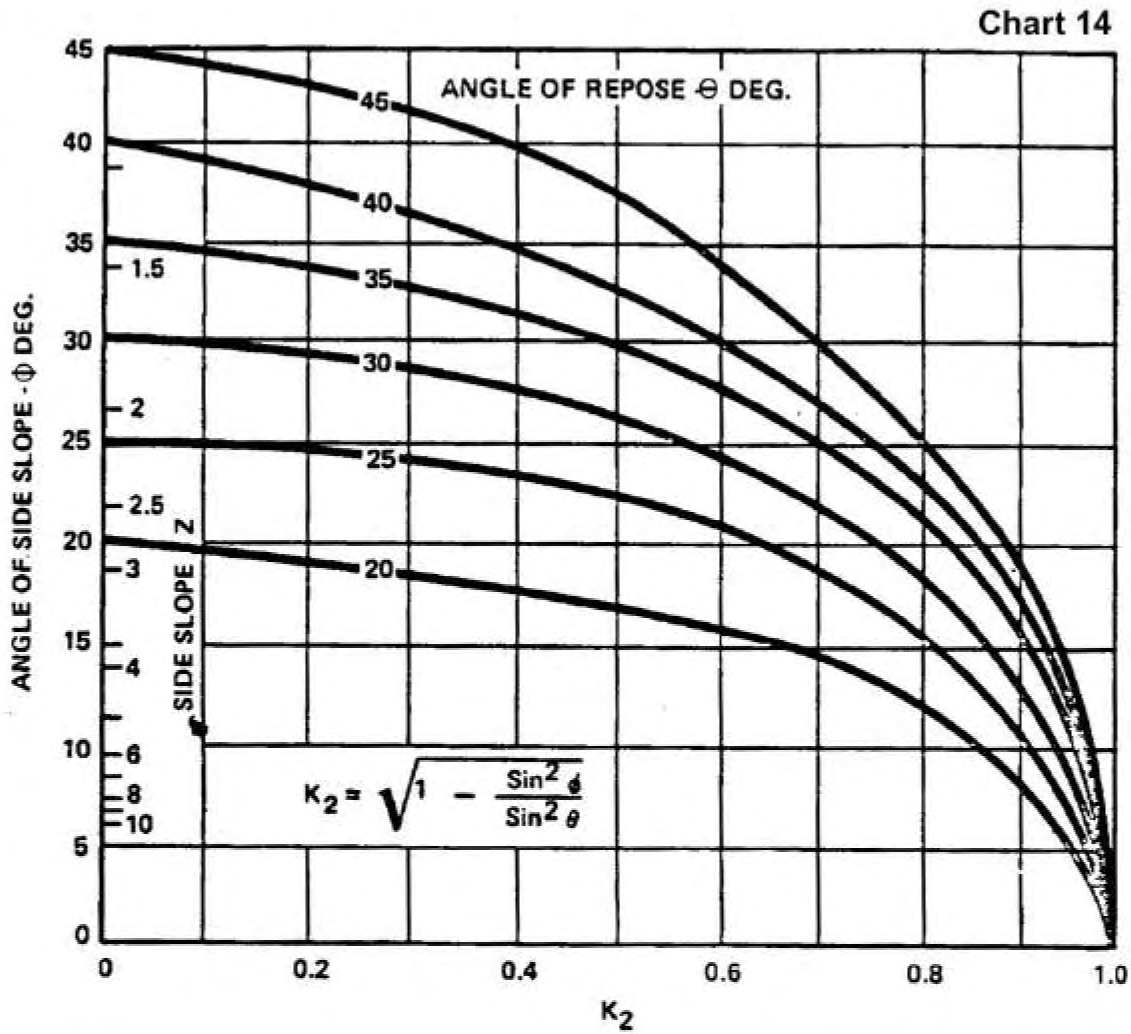
Appendix 7E-7 Channel Side Shear Stress to Bottom Shear Stress Ratio

Chart 13



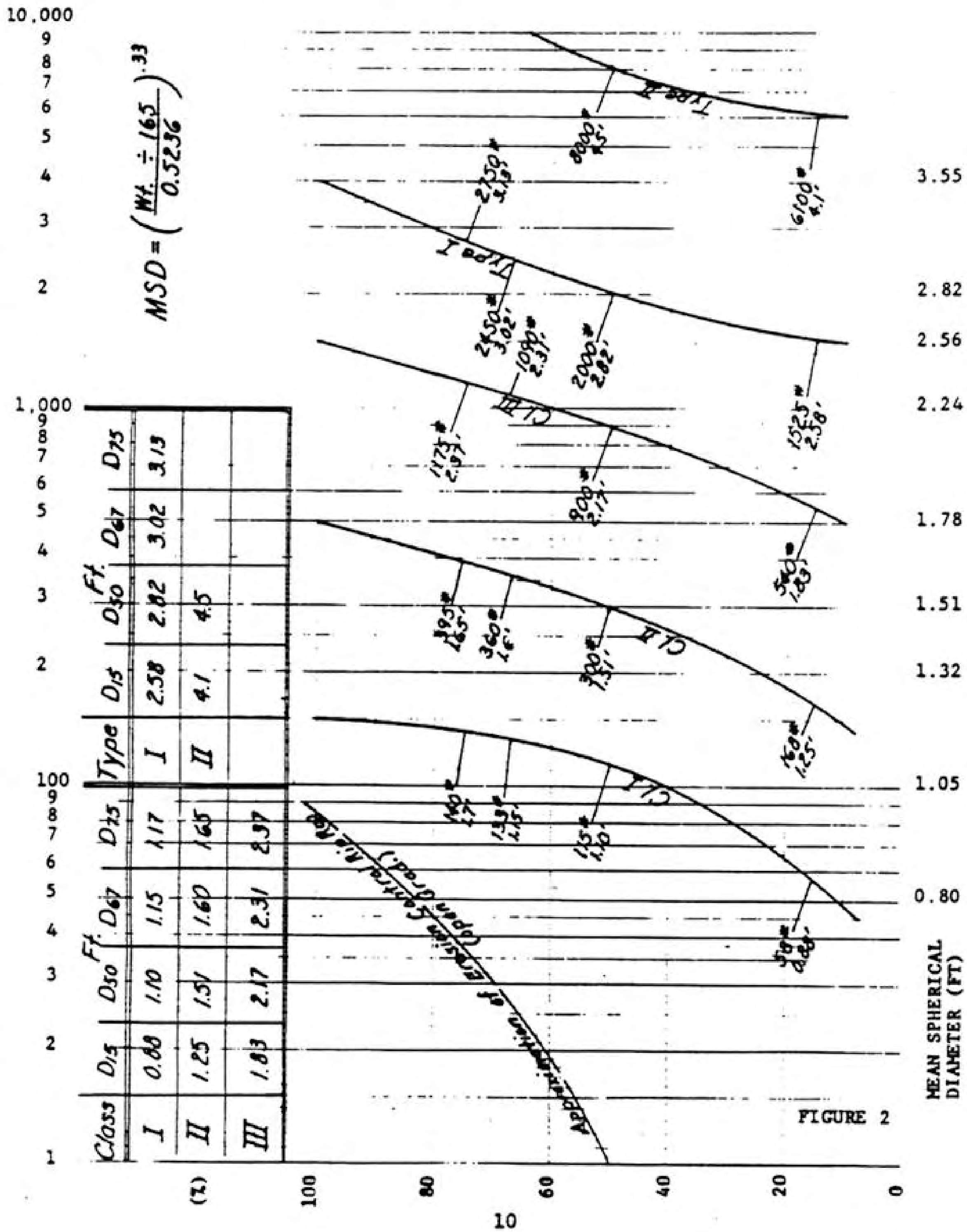
Source: HEC-15

Appendix 7E-8 Tractive Force Ratio (K_2)



Source: HEC-15
Comment: The symbols of Φ and θ 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

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Chapter 8 - Culverts

8.1 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.

For additional details on culvert design, refer also to Chapter 15, Drainage Design Instructions (DDM 2).

The FHWA web sites for specific publications may be used for additional information on the following topics:

HDS-5, Hydraulic Design Of Highway Culverts
<http://isddc.dot.gov/OLPFiles/FHWA/009342.pdf>

HEC-20, Stream Stability At Highway Structures,
<http://www.fhwa.dot.gov/bridge/hec20.pdf>

Highways In The River Environment (HIRE),
<http://www.fhwa.dot.gov/bridge/hire1990.pdf>

8.1.1 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. 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.

8.2 Design Policy

8.2.1 Federal and VDOT Policy

The following 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, highwater 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

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

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)

8.3.2 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 Flood Frequency

Culverts should be designed to accommodate the following minimum flood frequencies where the primary concern is the maintenance of traffic flow and the convenience of the highway user:

Roadway	***Flood Frequency (Annual Risk)***
Interstate	50-year (2%)
Primary & Arterial	25-year (4%)
Secondary	5 to 10 (20 to 10%)

The above requirements are minimum, and deviation requires approval from VDOT. Culverts should be designed to pass floods greater than those noted above where warranted by potential damage to adjacent property, loss of human life, injury, or heavy financial loss.

Future development of contributing watersheds and floodplains that have been zoned or delineated 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.

In compliance with the National Flood Insurance Program (NFIP) it is necessary to consider the 100-year frequency flood at all locations where construction will encroach on a floodplain. The 100-year floodplains and the 100-year water surface profiles are delineated and established for the Federal Emergency Management Agency (FEMA), the administrator of NFIP. The NFIP prohibits any construction in floodplains, when combined with all other existing and anticipated uses, which would increase the water surface elevation of the 100-year flood more than one foot at any given point.[1] It should be noted that it is VDOT's policy not to allow any increase in the level of the 100-year flood. This does not necessarily require that the culvert be sized to pass the 100-

[1] *This value of 1.0 foot is to be considered maximum and local conditions may require a lesser value for the particular site'. Also see The Governor of Virginia's Policy Memorandum 3-78 (1)*

year flood, provided the capacity of the culvert plus flow bypassing the culvert is sufficient to accommodate the 100-year flood without raising the water surface elevation excessively. An example of this may be where the culvert is designed to keep the roadway free from inundation during the 10-year design storm and the road grade is held down such that the culvert capacity plus flow over the roadway is sufficient to pass the 100-year flood without raising the upstream water surface elevation. In instances where a NFIP floodplain is involved, the same procedure used to establish the floodplain and water surface elevations (usually a step-backwater computer model such as HEC-RAS, HEC-2, WSPRO, etc.) should be used to design the proposed culvert.

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 inches 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

8.3.2.3 Review Headwater

The review headwater is the flood depth that:

- Does not increase the existing 100-year flood elevation in the NFIP mapped floodplains, or in the vicinity of insurable buildings
- Has a level of inundation that is tolerable to upstream property and roadway for the frequency of the check storm

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 and for major drainage installations with a 100-year discharge equal to or greater than 500 cfs, calculate the tailwater depth using backwater

methods (such as HEC-2, HEC-RAS, or WSPRO) or other step methods as appropriate. (Step backwater methods yield the most accurate tailwaters)

- When using 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 [$TW = (d_c + D)/2$] 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

8.3.2.6 Maximum Outlet Velocity

The outlet velocity of a culvert is the velocity computed at the downstream end of the culvert. It is usually higher than the maximum natural stream velocity.

Variations in the shape and size of a culvert barrel seldom have a significant effect on the outlet velocity. The slope and roughness of the culvert barrel are the principal factors affecting the outlet velocity.

If the outfall channel consists of material that will be subject to erosion, the following criteria should apply (refer to Table 8-1):

- The outlet velocities should be computed for the design discharge and the 25-year discharge. Whichever velocity is less should be used to ensure that the channel is stable.
- Erosion Control Stone, Class I or II should be in accordance with Standard EC-l
- Special design energy dissipators should consist of heavy riprap or other structures designed to provide protection for the specific site conditions (see Appendix 8E-1). The FHWA's HEC-14 publication provides analysis and design information on energy dissipators.

Table 8-1. Required Culvert End Treatment Based on Outlet Velocity

Outlet Velocity[1] (ft/s)	Culvert End Treatment (Type)	
	End Sections	Endwall or Headwall With Curtain Walls
0-6	None	None
6-10	EC - 1 Class I	None
10-14	EC - 1 Class I	EC - 1 Class I
14-19	EC - 1 Class II	EC - 1 Class II
19+	Special Design	Special Design

[1]Based on Q_{25} or design Q, whichever is less.

8.3.2.7 Minimum Velocity

The minimum velocity in a culvert barrel should be adequate to prevent siltation at low flow rates. 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 storage. If unusual circumstances require the consideration of upstream storage, the following concerns should be addressed:

- Limit the total area of flooding
- Limit the average time that bankfull stage is exceeded for the design flood to 48 hours in rural areas or 6 hours in urban areas
- Ensure that the storage area will remain available for the life of the culvert through the purchase of additional right-of-way or easements
- Consider environmental impacts
- Consider sediment deposition
- Consider the impacts of the delayed peak coinciding with downstream peak discharges

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 can occur and is sometimes encouraged, for higher discharges. Roadway overtopping may occur:

- When evaluating existing culvert installations for current design flows
- When performing check flood calculations
- When the design flood has less than a 100-year return period, and the roadway is held down so that the total flow through the culvert and over the roadway is sufficient to pass the 100-year storm without raising the 100-year water surface elevation upstream of the culvert
- Due to a flood event which exceeds the design storm and check storm

If roadway overtopping is indicated, 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

Storage routing may be useful in evaluating the impacts of roadway overtopping.

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 inches due to maintenance considerations
- For other systems, 15 inches will generally be the minimum culvert diameter, except that hydraulically adequate 12-inch diameter culverts may be used if the culvert length is less than 50 feet 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. 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, <u>all</u> barrels are laid 6 inches 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 degrees as measured from a line perpendicular to the roadway centerline without the approval of VDOT.

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 inches in 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. 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 inches in 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 inches in 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

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 ft. 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-feet. 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.

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.

8.3.5 Allowable Pipe Materials

Refer to I&IM LD- (D) 121

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 or erosion from undermining the culvert outlet.

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 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.

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 live stream, or where the protection of fish habitat warrants, as determined by the Environmental Division, the invert of a proposed culvert will be set 6 inches 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, all barrels or cells are to be lowered 6 inches below stream grade and a low flow diversion should be used to maintain low flow in the appropriate barrel(s) as shown in Appendix 8G-1. 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.

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 assume a constant discharge or to route a hydrograph
- 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).

8.4.2.1.1 Constant Discharge

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

8.4.2.1.2 Hydrograph and Storage Routing

Hydrograph and storage routing method:

- Is used when unusual circumstances exist
- Considers the storage capacity behind a highway embankment which attenuates a flood hydrograph and reduces the peak discharge
- May reduce the required culvert size, given adequate storage

- Is checked by routing the design hydrographs through the culvert to determine the outflow hydrograph and upstream water surface elevation
- Procedures are in Chapter 11, Stormwater Management, and in HDS-5, Section V

8.4.2.2 Computational Methods

Computational methods include hand methods (LD-269) and computer solutions. Hand 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 Hand Methods

Hand methods using design equations and nomographs through design form LD-269 (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

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.

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.

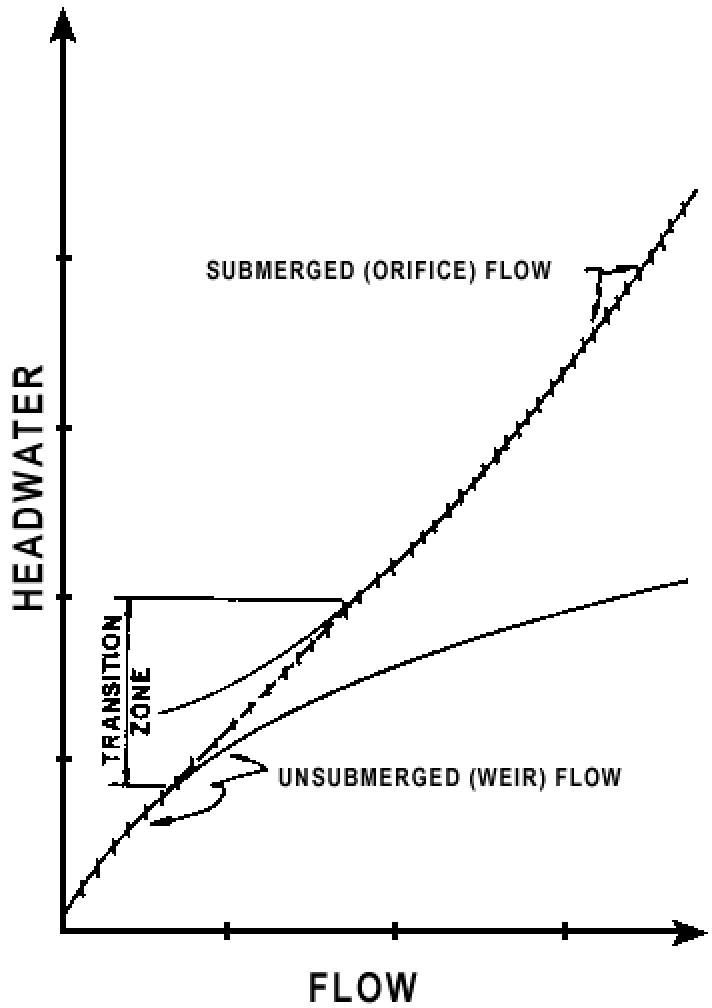


Figure 8-1. Performance Curves - Unsubmerged, Transition, and Submerged

Four types of inlet control flow profiles within culverts are shown in Figure 8-2.

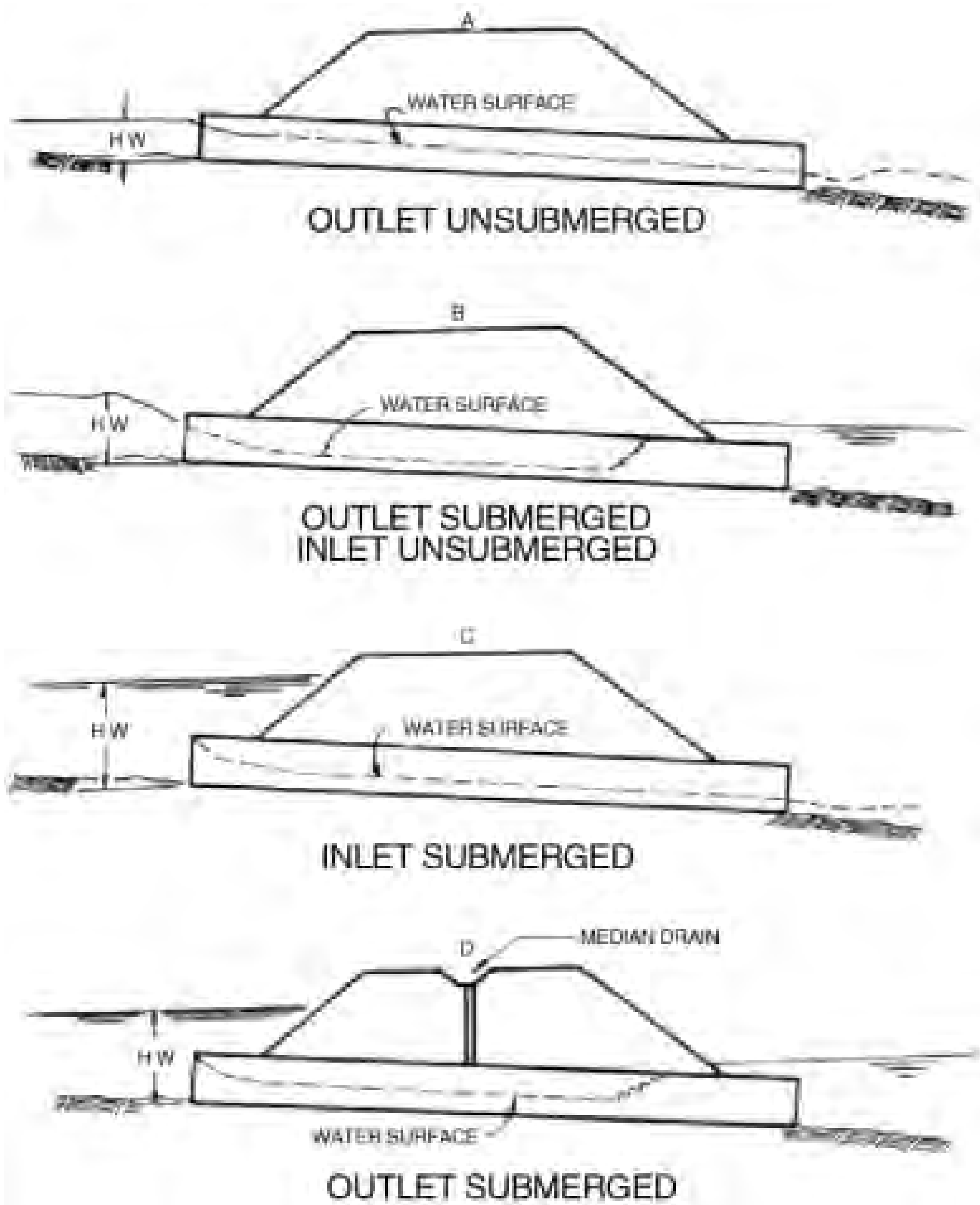


Figure 8-2. Types of Inlet Control Flow

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. 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.

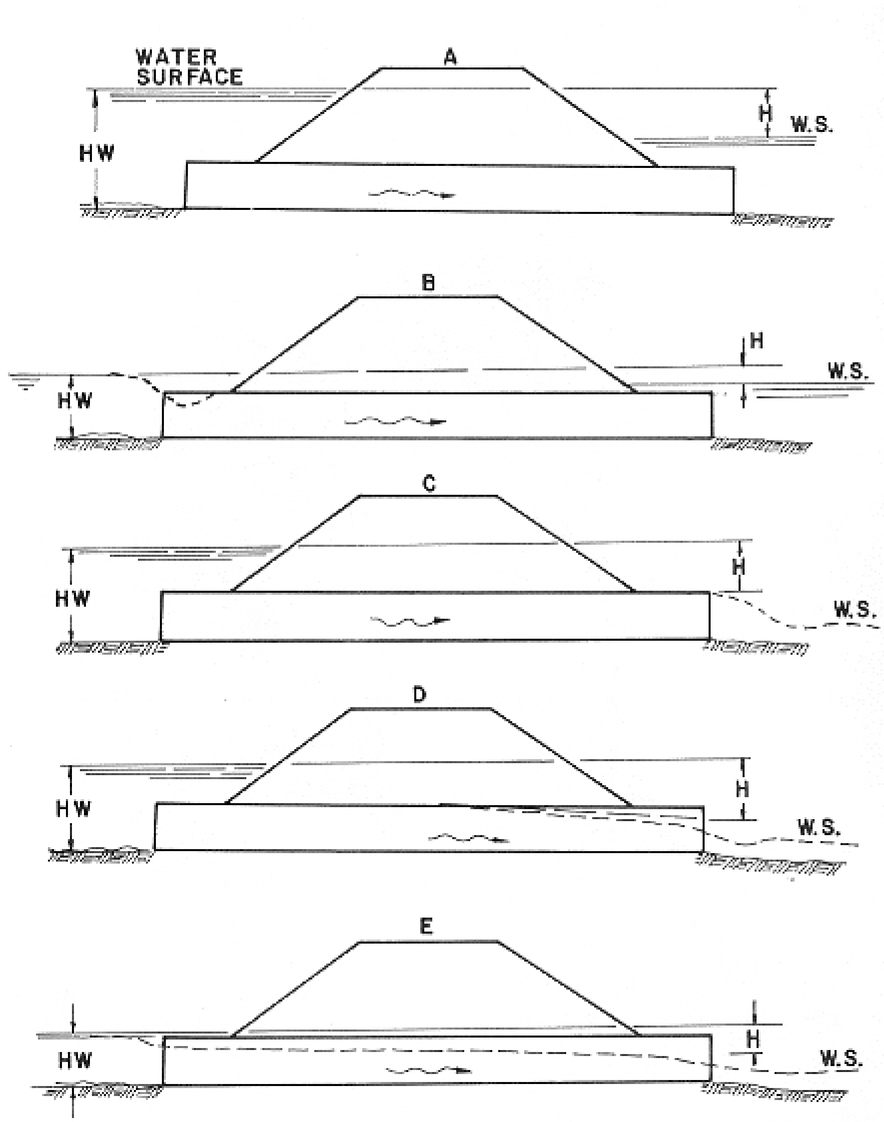


Figure 8-3. Types of Outlet Control Flow

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.

$$H_L = H_e+H_f+H_o+H_b+H_j+H_g+H_v \qquad (8.1)$$

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_e = 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.

$$V = \frac{Q}{A} \tag{8.2}$$

Where:

V = Average full barrel velocity, fps
Q = Flow rate, cfs
A = Cross sectional area of flow with the barrel full, sq. ft.

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.).

$$H_v = \frac{V^2}{2g} \tag{8.3}$$

Where:

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.

$$H_e = K_e\left(\frac{V^2}{2g}\right) \tag{8.4a}$$

Where:

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.

$$H_f = \frac{29n^2L}{R^{1.33}}\left(\frac{V^2}{2g}\right) \quad (8.4b)$$

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 = $\left(\frac{A}{P}\right)$, ft
A = Cross section area of pipe, sq. ft.
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.

$$H_o = 1.0\left[\frac{V^2}{2g} - \frac{V_d^2}{2g}\right] \quad (8.4c)$$

Where:

V_d = Channel velocity downstream of the culvert, fps (if downstream velocity is neglected, use Equation 8.4d).

$$H_o = H_v\left(\frac{V^2}{2g}\right) \quad (8.4d)$$

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.

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.

$$H = H_E + H_o + H_f$$

$$H = \left[1 + K_e + \frac{29n^2L}{R^{1.33}}\right]\left(\frac{V^2}{2g}\right) \quad (8.5)$$

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:

$$HW_o + \frac{V_u^2}{2g} = TW + \frac{V_d^2}{2g} + H_L \qquad (8.6)$$

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)

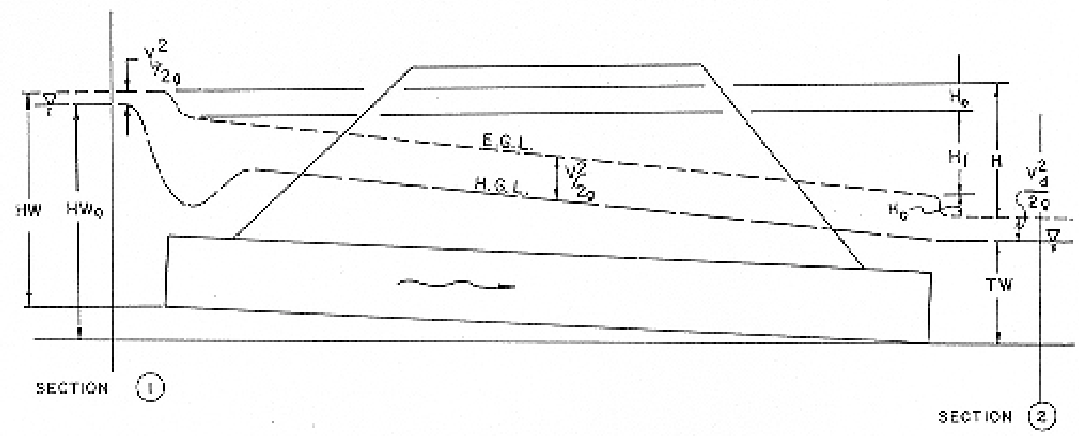


Figure 8-4. Full Flow Energy and Hydraulic Grade Lines

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.

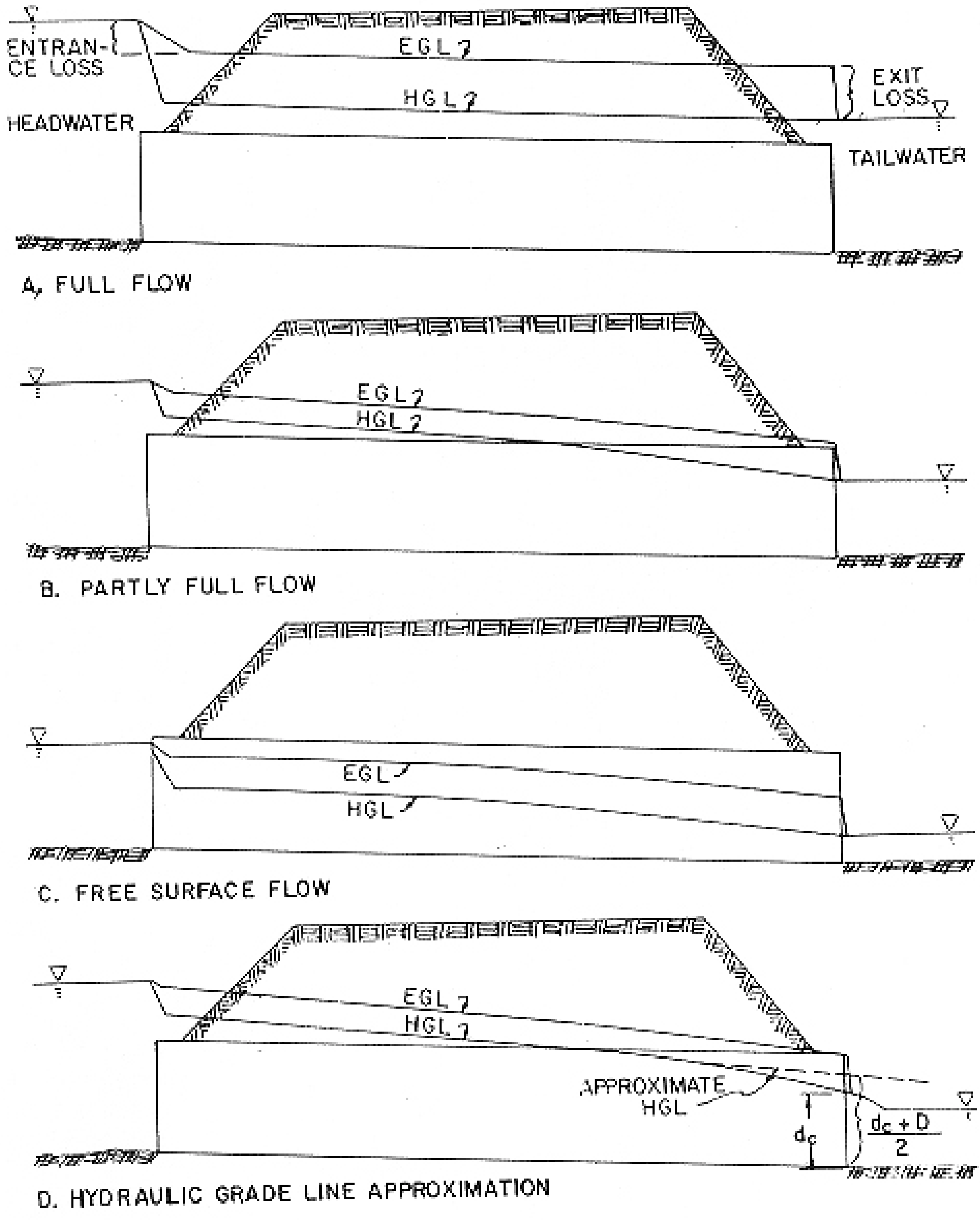


Figure 8-5. Outlet Control Energy and Hydraulic Grade Lines

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:

$$HW = TW + H - S_o L \qquad (8.7)$$

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):

$$HW = h_o + H - S_o L \qquad (8.8)$$

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.

8.4.3.5 Outlet Velocity

Culvert outlet velocities should be calculated to determine the need for erosion protection at the culvert exit (See Table 8-1). 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.

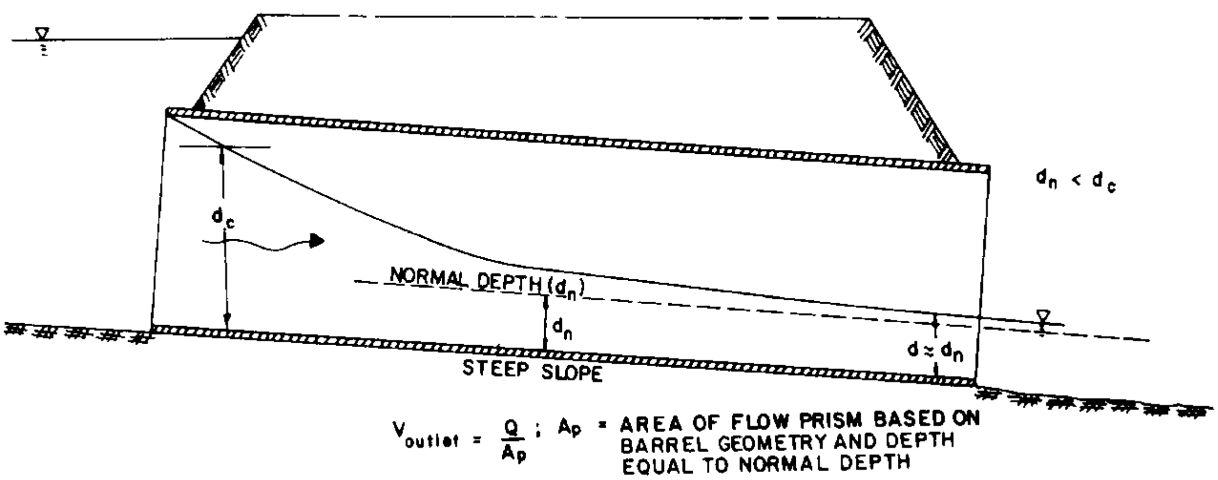


Figure 8-6. Outlet Velocity - Inlet Control

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

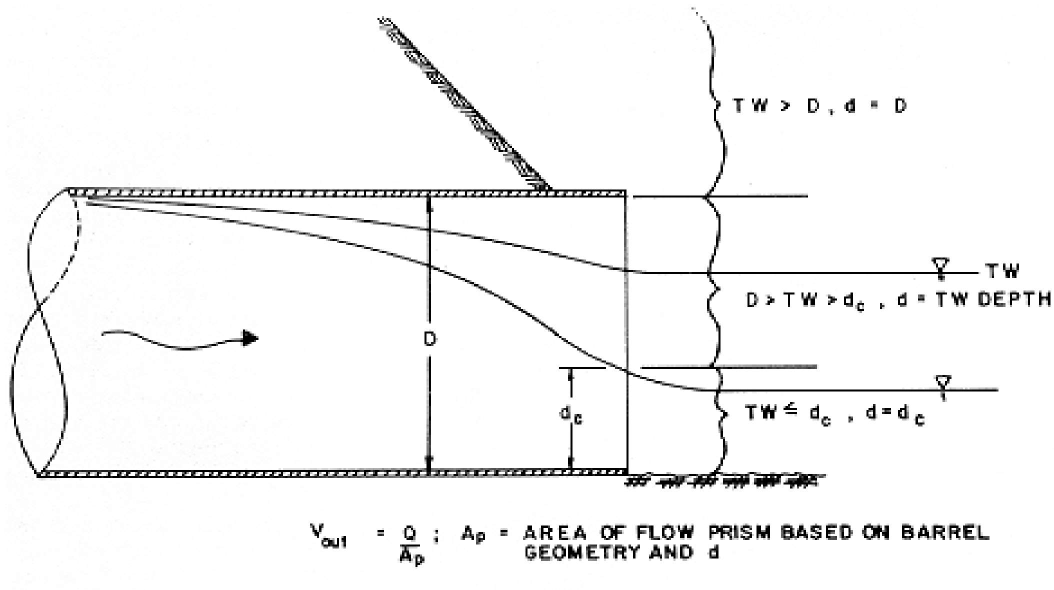


Figure 8-7. Outlet Velocity - Outlet Control

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.

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.

$$Q_r = C_d L HW_r^{1.5} \qquad (8.9)$$

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
HW_r = 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:

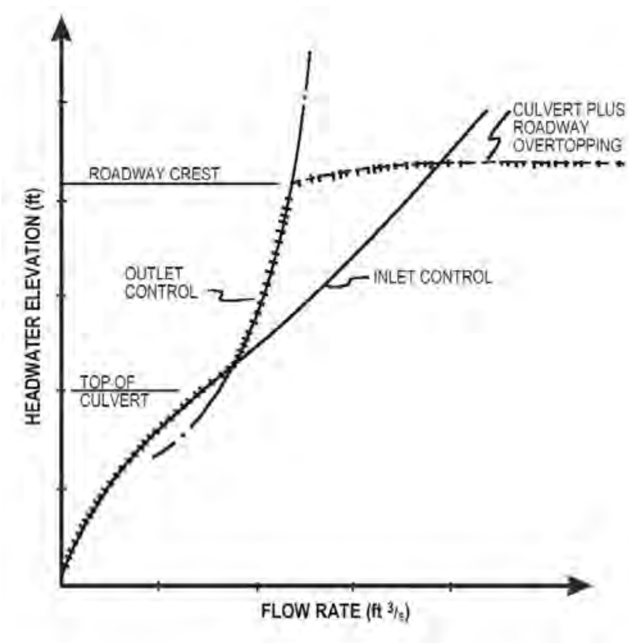


Figure 8-8. Overall Culvert Performance Curve

- 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. <u>Both inlet and outlet control headwaters should be calculated.</u>

  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.

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 percent (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.

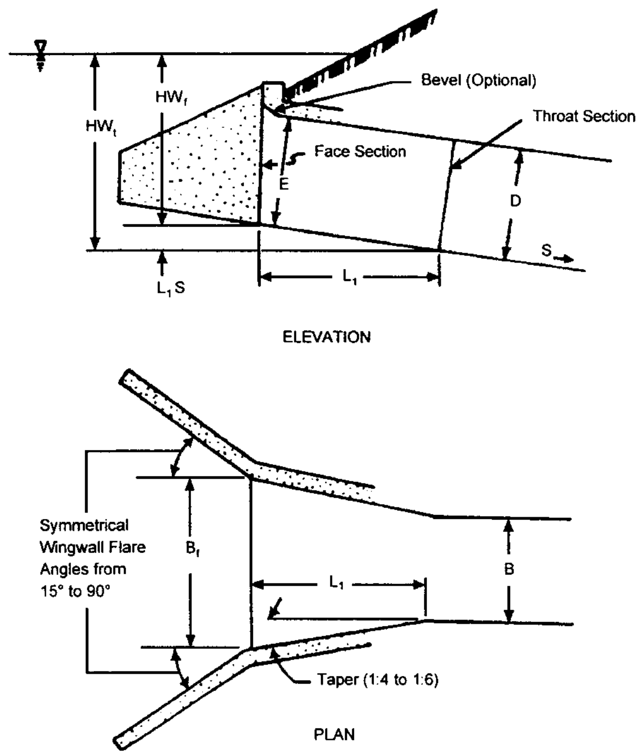


Figure 8-9. Side-Tapered Inlet

The side-tapered inlet throat should be designed to be the primary control section for the design range of flows and headwaters.

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.

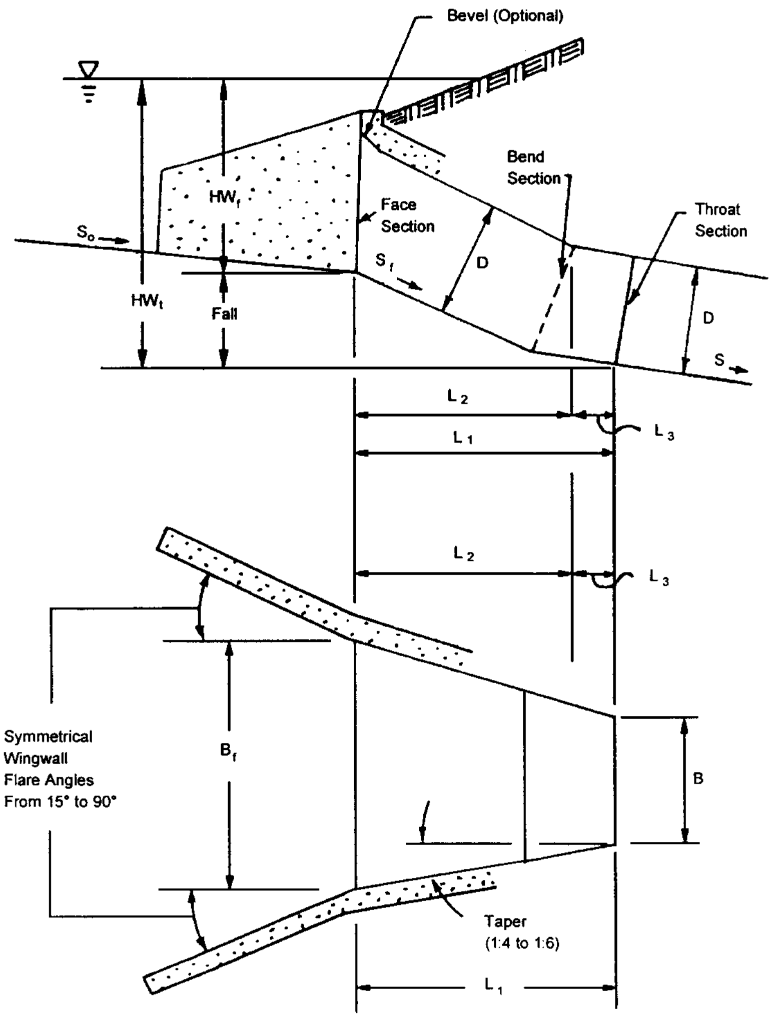


Figure 8-10. Slope-Tapered Inlet

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. 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 of 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.

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.

8.5 Design Procedures and Examples

8.5.1 Documentation Requirements

These items establish a minimum documentation requirement for culvert design. The following items used in the design or analysis should be included in the documentation file:

- Allowable headwater elevation and basis for its selection
- Cross section(s) used in the design highwater determinations
- Roughness coefficient assignments (n-values)
- Observed highwater, dates and discharges
- Stage discharge curve at outlet for undisturbed, existing and proposed conditions to include the depth and velocity measurements or estimates and locations for the design, and check floods
- Documentation showing the calculated backwater elevations, outlet velocities, scour for the design storm, check storm and any available historical floods
- Type of culvert entrance condition
- Culvert outlet appurtenances and energy dissipation calculations and designs
- Copies of all computer analyses, such as HY8 output, and VDOT Form LD-269 (Appendix 8B-1) if the culvert was designed manually
- Roadway geometry (plan and profile)
- Potential flood hazard to adjacent properties

All hydrologic and hydraulic design computations and all maps, studies, reports, comments and interviews pertinent to the culvert design must be concisely and legibly recorded in a form that can be easily filed and microfilmed for ready recall. This data must remain easily accessible for reference when filing for permits and when future changes or designs are made.

In most cases, the design flood, the overtopping flood, and the 100-year flood should be shown in the culvert design documentation. The use of the VDOT culvert form (LD-269) in Appendix 8B-1 provides a convenient form on which to record the pertinent data.

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 (HW_i)

- Use the appropriate inlet control nomographs in Appendix 8C

Step 8 Determine outlet control headwater depth at inlet (HW_{oi}):

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 (d_c) using the appropriate chart in Appendix 8C

   - Locate flow rate and read d_c
   - d_c cannot exceed D
   - If d_c>0.9D, consult Handbook of Hydraulics (King and Brater) for a more accurate d_c, if needed, since curves are truncated where they converge

c. Calculate $\left(\frac{d_c+D}{2}\right)$

d. Determine h_o

   h_o = the larger of TW or $\left(\frac{d_c+D}{2}\right)$

e. Determine K_e

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 (HW_{oi})

- Use Equation 8.8, if V_u and V_d 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 V_u and V_d.
- If HW_{oi} is less than 1.2D and control is outlet control:
  - ➢ The barrel may flow partly full
  - ➢ The approximate method of using the greater of tailwater or $\left(\frac{d_c + D}{2}\right)$ 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 (HW_c)

a. Compare HW_i and HW_{oi}, and use the higher

b. Compare HW to allowable HW criteria (cover, $\left(\frac{HW}{D}\right)$, 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 $\left(\frac{HW}{D}\right)$ should be at least 1.0 and not exceed 1.5
- The allowable overtopping flood frequency should not be exceeded

Step 14 Select and analyze check storm 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, in accordance with Table 8-1. 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

8.5.2.1 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

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
- 100-year frequency for check storm
- Allowable headwater depth for design flood = 8.5 ft.
- 100-year floodplain depth = 10.0 ft.

Step 2. Determine Hydrology

USGS Regression equations yield:

Q_{50} = 400 cfs
Q_{100} = 500 cfs

Step 3. Account for tailwater

Slope = 0.05 ft./ft.
Length = 100 ft.

The predetermined depths and velocities for the downstream channel are:

Q (cfs)	TW (ft)	V (ft/s)
400	2.8	18
500	3.1	19

Step 4. Summarize data on design form

(See Figure 8-11)

Step 5. Select trial design structure

Shape: Box
Size: 7.0 ft (B) by 6.0 ft (D)
Material: Concrete
Entrance: Wingwalls with 30^{o}-75^{o} flare

Step 6. Select design discharge

$Q_d = Q_{50} = 400$ cfs

Step 7. Determine inlet control headwater depth (HW_i)

Use inlet control nomograph - Appendix 8C-8

a. D = 6.0 ft

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 ft. for Q_{50} = 400 cfs

b. d_c = 4.6 ft. 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 ft.

e. K_e = 0.4 from Appendix 8D-2 (for 30^o-75^o wingwalls)

f. Determine (H) - use Appendix 8C-15

- K_e scale = 0.4
- Culvert length, L = 100 ft.
- n = 0.012 (same as on Appendix 8C-15)
- Area = 42 sq. ft.
- H = 2.3 ft.

g.
$$\begin{aligned} HW_{oi} &= h_o + H - LS_o \\ &= 2.3 + 5.3 - 100(0.05) \\ &= 2.6 \text{ ft.} \end{aligned}$$

Step 9. Determine controlling headwater (HW_c)

HW_i = 7.80 ft.

HW_{oi} = 2.5 ft.

HW_c = The greater of HW_i or HW_{oi}

$HW_c = HW_i$ = 7.80 ft.

The culvert is in inlet control

Step 10. Compute discharge over the roadway (Q_r)

Not applicable

Step 11. Compute total discharge (Q_r)

Q_t = 400 cfs

Step 12. Determine outlet velocity (V_o)

- Use Appendix 8C-83
- Enter 400 cfs on the horizontal, “Discharge” scale
- Read vertically to the “Slope” curve of 0.05
- Read horizontally to the “Velocity" scale and find a value of 27 fps

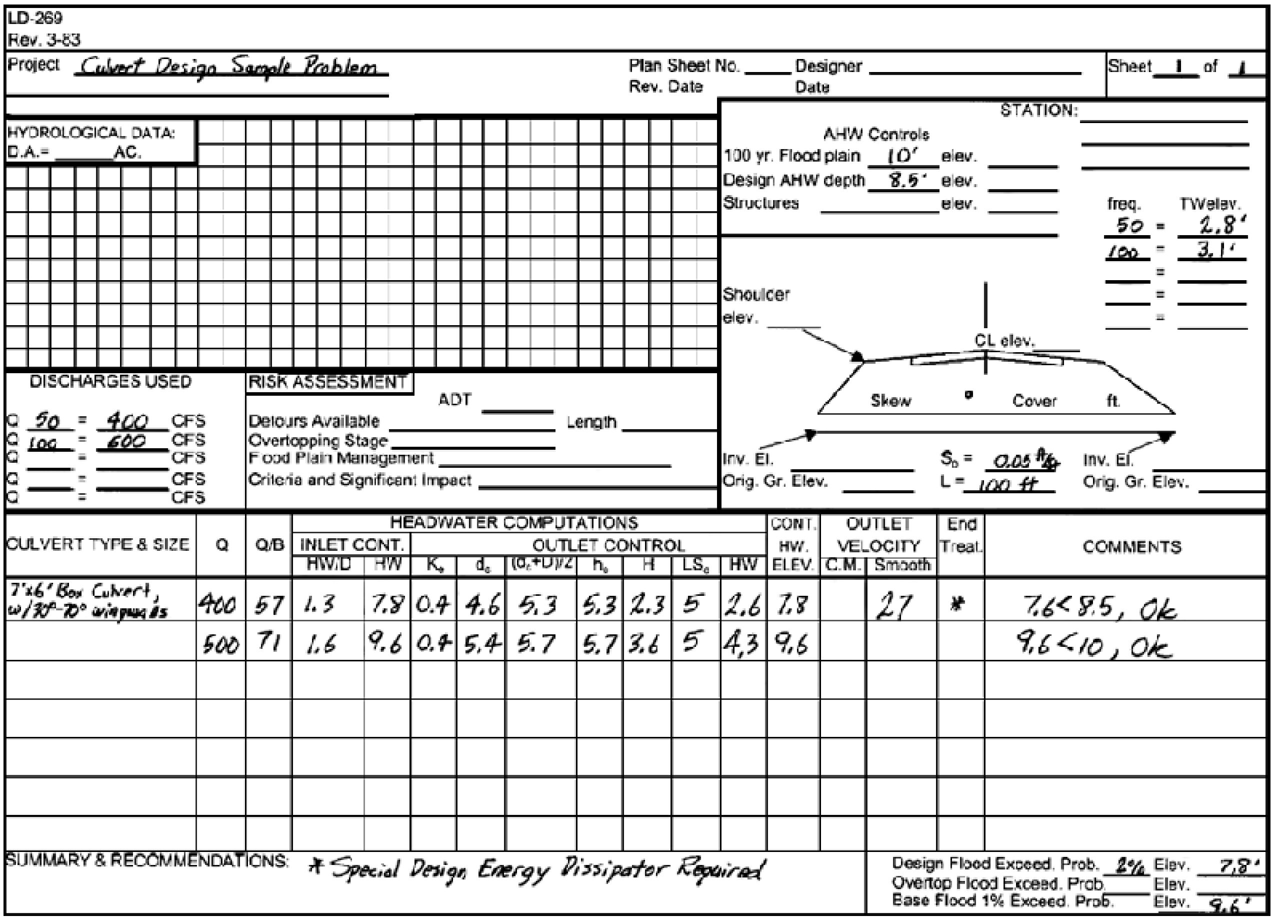
Step 13. Repeat steps 5-10 for check flood (100-yr.):

- Compare design with constraints and assumptions. If any of the following are exceeded, repeat steps 5 through 12:

- 100-year floodplain depth = 10.0 ft.>9.6 ft.
- Overtopping flood frequency > 50-yr.

Step 14. Design Considerations (None)

Step 15. Complete any additional necessary documentation



LD-269
Rev. 3-83

Project: Culvert Design Sample Problem — Plan Sheet No. ____ Designer ____ Rev. Date ____ Date ____ Sheet 1 of 1

HYDROLOGICAL DATA: D.A.= ____ AC.

STATION: ____

AHW Controls
100 yr. Flood plain 10' elev. ____
Design AHW depth 8.5' elev. ____
Structures ____ elev. ____

freq.	TWelev.
50	2.8'
100	3.1'

Shoulder elev. ____ CL elev. ____ Skew ° Cover ft.

Inv. El. ____ S_o = 0.05 ft/ft Inv. El. ____
Orig. Gr. Elev. ____ L = 100 ft Orig. Gr. Elev. ____

DISCHARGES USED
Q 50 = 400 CFS
Q 100 = 600 CFS
Q ____ = ____ CFS
Q ____ = ____ CFS
Q ____ = ____ CFS

RISK ASSESSMENT
ADT ____
Detours Available ____ Length ____
Overtopping Stage ____
Flood Plain Management ____
Criteria and Significant Impact ____

CULVERT TYPE & SIZE	Q	Q/B	INLET CONT. HW/D	INLET CONT. HW	OUTLET CONTROL K_e	d_c	$(d_c+D)/2$	h_o	H	LS_o	HW	CONT. HW. ELEV.	OUTLET VELOCITY C.M.	OUTLET VELOCITY Smooth	End Treat.	COMMENTS
7'x6' Box Culvert, w/30°-70° wingwalls	400	57	1.3	7.8	0.4	4.6	5.3	5.3	2.3	5	2.6	7.8		27	*	7.6<8.5, Ok
	500	71	1.6	9.6	0.4	5.4	5.7	5.7	3.6	5	4.3	9.6				9.6<10, Ok

SUMMARY & RECOMMENDATIONS: * Special Design Energy Dissipator Required

Design Flood Exceed. Prob. 2% Elev. 7.8'
Overtop Flood Exceed. Prob. ____ Elev. ____
Base Flood 1% Exceed. Prob. Elev. 9.6'

Figure 8-11. Completed Culvert Design Form, Sample Problem

8.5.3 Buoyancy Protection Procedure

Note: A software program is available from VDOT that performs this procedure.

8.5.3.1 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. per 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. per cubic foot

Weight of Fill (Saturated) - 37.6 lbs. per cubic foot

Weight of Water - 62.4 lbs. per cubic foot

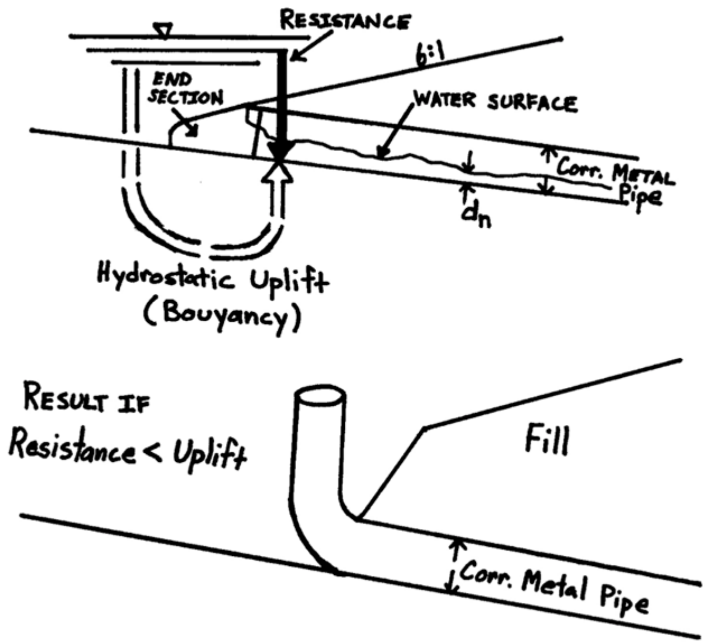


Figure 8-12. Hydrostatic Uplift Forces and Effects on Pipe

If Resistance < Hydrostatic Uplift:

Increase weight on end of pipe by adding concrete endwall or concrete anchor blocks.

8.5.3.2 Buoyancy Protection Sample Problem

Given:

48-inch Corrugated Metal Pipe, 12 gage
Fully Coated with Paved Invert
Std. ES-2 End Section
Q = 96 cfs

Computed Values:

HW/D= 1.25
HW= 5 ft.
d_n= 2.5 ft.
d_c= 2.8 ft.

Assumed Values:

Weight of Fill (Dry) = 100 lbs. per cu. ft.
Weight of Fill (Saturated) = 37.6 lbs. per cu. ft.
Weight of Water = 62.4 lbs. per cu. ft.
Weight of Pipe = 84 lbs. per L.F.
(Handling weight of corrugated steel pipe available in Appendix 8F-1 and 8F-2)

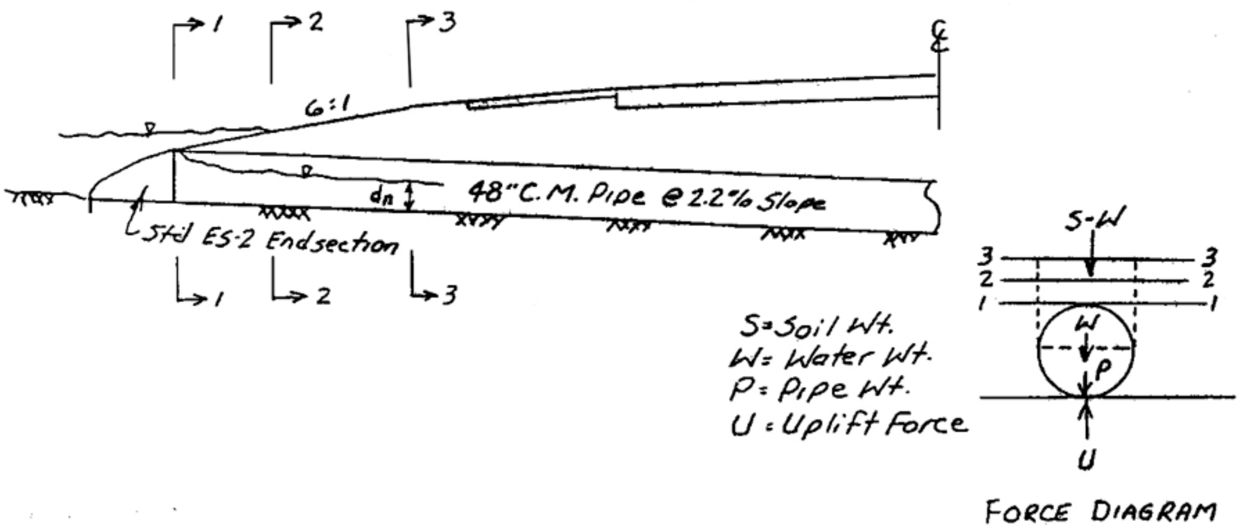


Figure 8-13. Buoyant Forces Acting on Pipe

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./cu. ft.)

Buoyant force = $L(A)(\gamma)$

Where:

L = Unit length of pipe, ft
A = Cross sectional area of pipe, sq. ft.
γ = Unit weight of water = 62.4 lbs/ft^3

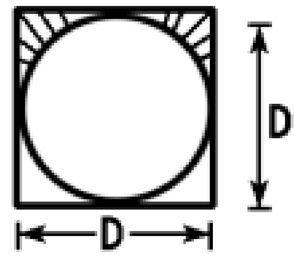


Figure 8-14. Weight of Fill, Section 1

$$= 1\left(\frac{\pi D^2}{4}\right)(62.4)$$

$$= 1\left(\frac{\pi(4)^2}{4}\right)(62.4)$$

$$= 784 \text{ lbs/ft.}$$

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./cu. ft. (saturated fill)

$$\text{Area} = \left[\frac{D^2 - \frac{\pi D^2}{4}}{2}\right] = \left[\frac{4^2 - \frac{\pi(4^2)}{4}}{2}\right] = 1.7 \text{ sq.ft.}$$

Weight of Fill = 1.7 (1)(37.6)

Weight of Fill = 64 lbs/ft

Compute weight of water (inside pipe)

Weight of Water = Area of flow x 1 ft. x 62.4 lbs./cu. ft.

Assume depth of water, $d = d_n = 2.5$ ft.

$$\frac{d}{D} = \frac{2.5}{4} = 0.625$$

$$\frac{\text{Area}}{D^2} = 0.516 \text{ (From Appendix 8F-5)}$$

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 ft 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)*

  Weight of Fill = Area ft^2 x 1 ft. x 37.6 lbs./ft^3 (saturated fill)

$$\text{Area} = \left[\frac{D^2 - \frac{\pi D^2}{4}}{2}\right] + 1(D) = \left[\frac{4^2 - \frac{\pi 4^2}{4}}{2}\right] + 1(4) = 5.7\ \text{sq.ft.}$$

  Weight of Fill = 5.7 ft.^{2} x 1 ft. x 37.6 lbs./ft^3

  Weight of Fill = 214 lbs/ft.

- *Compute weight of water (Inside Pipe)*

  Assume depth of water, $d = d_n = 2.5$ ft.

  Weight of Water = 515 lbs./ft.
  (Same as Section 1)

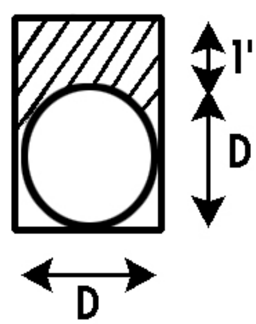


Figure 8-15. Weight of Fill Material, Section 2

- *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.

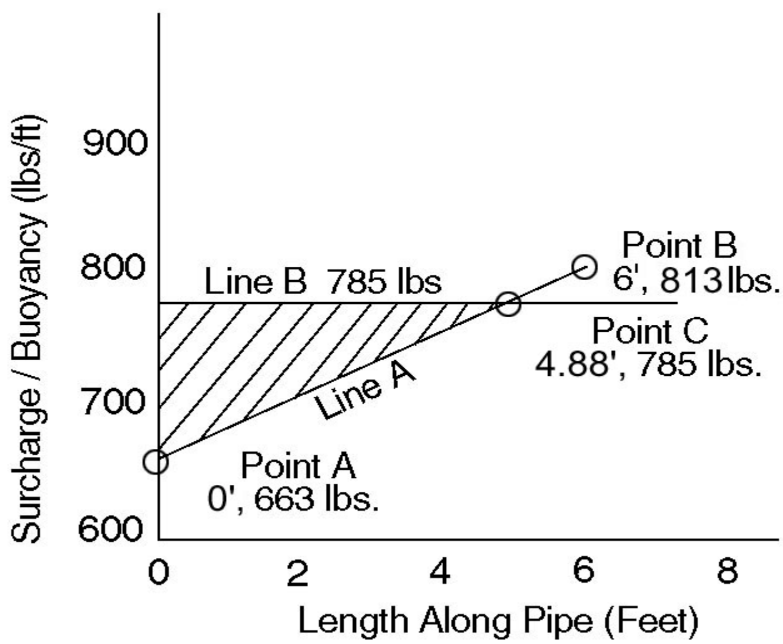


Figure 8-16. Surcharge/Buoyancy along Length of Pipe

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.

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 ft.

   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.

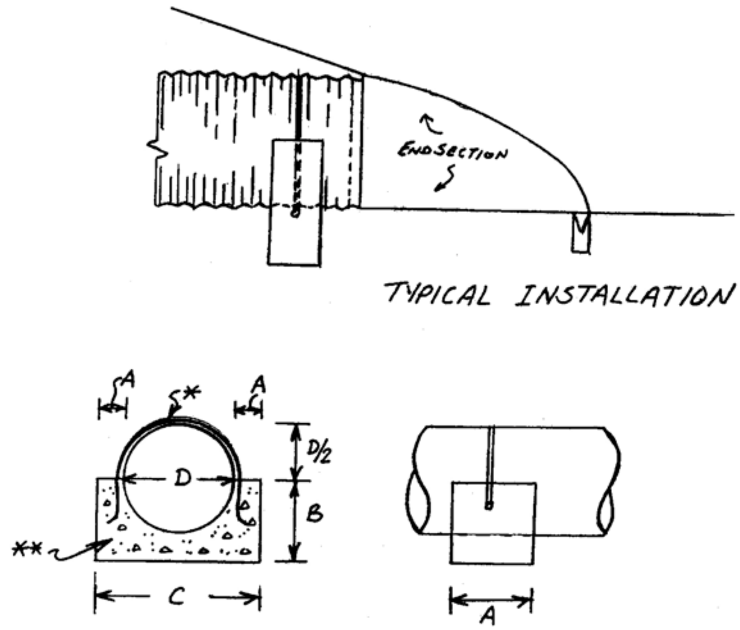


Figure 8-17. Counterweight Details for Pipes Subject to Uplift Forces

*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-inch minimum
B=D/2+12 inch
C=Variable as needed, D+12 inch minimum
D=Pipe diameter

8.6 References

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Appendix 8A-1 Definitions and Abbreviations

Definitions:

Culvert	A structure which is usually designed hydraulically to take advantage of submergence to increase hydraulic capacity. A structure used to convey surface runoff through embankments. 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. A structure which is 20 ft 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 in this chapter, regardless of length.
Critical Depth	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. Appendix 8C contains critical depth charts for different shapes.
Flow Type	The USGS has established seven culvert flow types which assist in determining the flow conditions at a particular culvert site. Diagrams of these flow types are provided in the design methods section.
Free Outlet	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.
Improved Inlet	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).
Normal Flow	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.
Slope	A steep slope occurs where critical depth is greater than normal depth. A mild slope occurs where critical depth is less than normal depth.
Submerged	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.

Abbreviations:

AASHTO	American Association of State Highway and Transportation Officials
BLM	Bureau of Land Management
DCR	Department of Conservation and Recreation
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
NRCS	National Resource Conservation Service; formerly Soil Conservation Service (SCS)
HDS	Hydraulic Design Series
HEC	Hydraulic Engineering Circular
HIRE	Highways in the River Environment
HW	Headwater
NFIA	National Flood Insurance Act
NFIP	National Flood Insurance Program
NOAA	National Oceanic and Atmospheric Administration
RDM	Road Design Manual
TVA	Tennessee Valley Authority
TW	Tailwater
USBR	United States Bureau of Reclamation
USCOE/USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
VDOT	Virginia Department of Transportation

Appendix 8A-2 Symbols

Symbol	Definition	Units
A	Area of cross section of flow	ft^2
B	Barrel or box width	in or ft
C_d	Overtopping coefficient (Weir coefficient)	-
C_r	Discharge coefficient	-
D	Culvert diameter or barrel height	in or ft
d	Depth of flow	ft
d_{50}	Mean stone size diameter	in or ft
d_B	Critical depth at riprap basin overflow	ft
d_c	Critical depth	ft
d_E	Equivalent brink depth	ft
d_n or d_o	Normal depth	ft
F_r	Froude Number	-
g	Acceleration due to gravity	ft/s^2
H	Total headloss	ft
H_b	Bend headloss	ft
H_E	Entrance headloss	ft
H_f	Friction losses	ft
H_g	Grate losses	ft
H_j	Junction losses	ft
H_L	Total energy losses	ft
H_o	Outlet or exit headloss	ft
h_s	Depth of riprap basin	ft
H_v	Velocity head	ft
h_o	Hydraulic grade line height above outlet invert	ft
HW	Headwater depth (subscript indicates section)	ft
HW_i	Headwater depth as a function of inlet control	ft
HW_o	Headwater depth above outlet invert	ft
HW_{oi}	Headwater depth as a function of outlet control	ft
HW_r	Headwater depth above roadway	ft
K_e	Entrance loss coefficient	-
k_t	Submergence coefficient	-
L	Length of culvert or length of roadway crest	ft
L_B	Length of riprap basin	ft
L_s	Length of dissipating pool	ft
n	Manning’s roughness coefficient	-
P_w	Wetted perimeter	ft
Q	Discharge	cfs
Q_d	Discharge through the culvert	cfs

Symbol	Definition	Units
Q_t	Design or check discharge at culvert	cfs
R	Hydraulic radius (A/P)	ft
S_o	Slope of culvert	ft/ft
TW	Tailwater depth above invert of culvert	ft
V	Average velocity of flow	fps
V_B	Average velocity at riprap basin overflow	fps
V_d	Average velocity in downstream channel	fps
V_L	Average velocity at length (L) downstream from brink	fps
V_o	Average velocity of flow at culvert outlet	fps
V_u	Average velocity in upstream channel	fps
W_B	Width of riprap basin at overflow	ft
W_o	Width dimension of culvert shape	ft
γ	Unit weight of water	lbs/ft^3

Appendix 8B-1 Culvert Design Form LD-269

LD-269
Rev. 3-83

Project ____________ Plan Sheet No. ____ Designer ____________ Sheet ____ of ____
Rev. Date Date

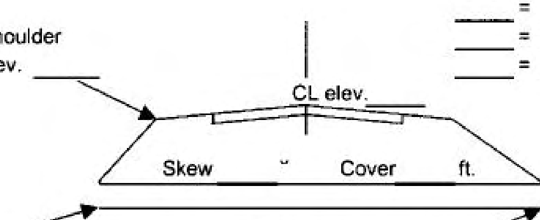
HYDROLOGICAL DATA:
D.A.= ______ AC.

STATION: ____________

AHW Controls
100 yr. Flood plain ______ elev. ______
Design AHW depth ______ elev. ______
Structures ______ elev. ______

freq.		TWelev.
____	=	____
____	=	____
____	=	____
____	=	____
____	=	____

Shoulder elev. ____



Inv. El. ____ S_o = ____ Inv. El.
Orig. Gr. Elev. ____ L = ____ Orig. Gr. Elev. ____

DISCHARGES USED

Q ____ = ____ CFS
Q ____ = ____ CFS
Q ____ = ____ CFS
Q ____ = ____ CFS
Q ____ = ____ CFS

RISK ASSESSMENT

ADT ______
Detours Available ______, Length ______
Overtopping Stage ______
Flood Plain Management ______
Criteria and Significant Impact ______

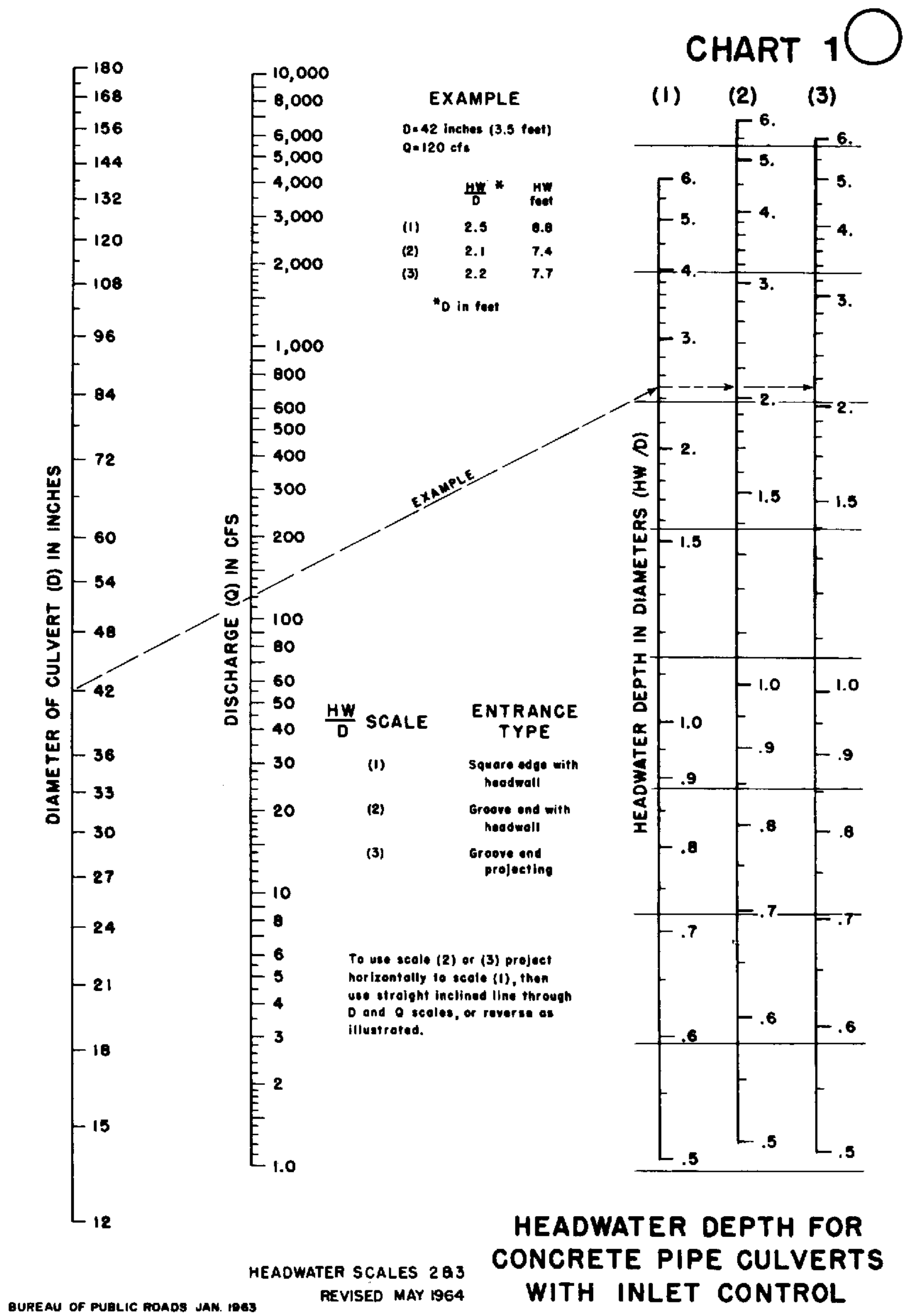
CULVERT TYPE & SIZE	Q	Q/B	HEADWATER COMPUTATIONS									CONT. HW.	OUTLET VELOCITY		End Treat.	COMMENTS
			INLET CONT.		OUTLET CONTROL											
			HW/D	HW	K_e	d_c	$(d_c+D)/2$	h_o	H	LS_o	HW	ELEV.	C.M.	Smooth		

SUMMARY & RECOMMENDATIONS:

Design Flood Exceed. Prob. ____ Elev. ____
Overtop Flood Exceed. Prob. ____ Elev. ____
Base Flood 1% Exceed. Prob. ____ Elev. ____

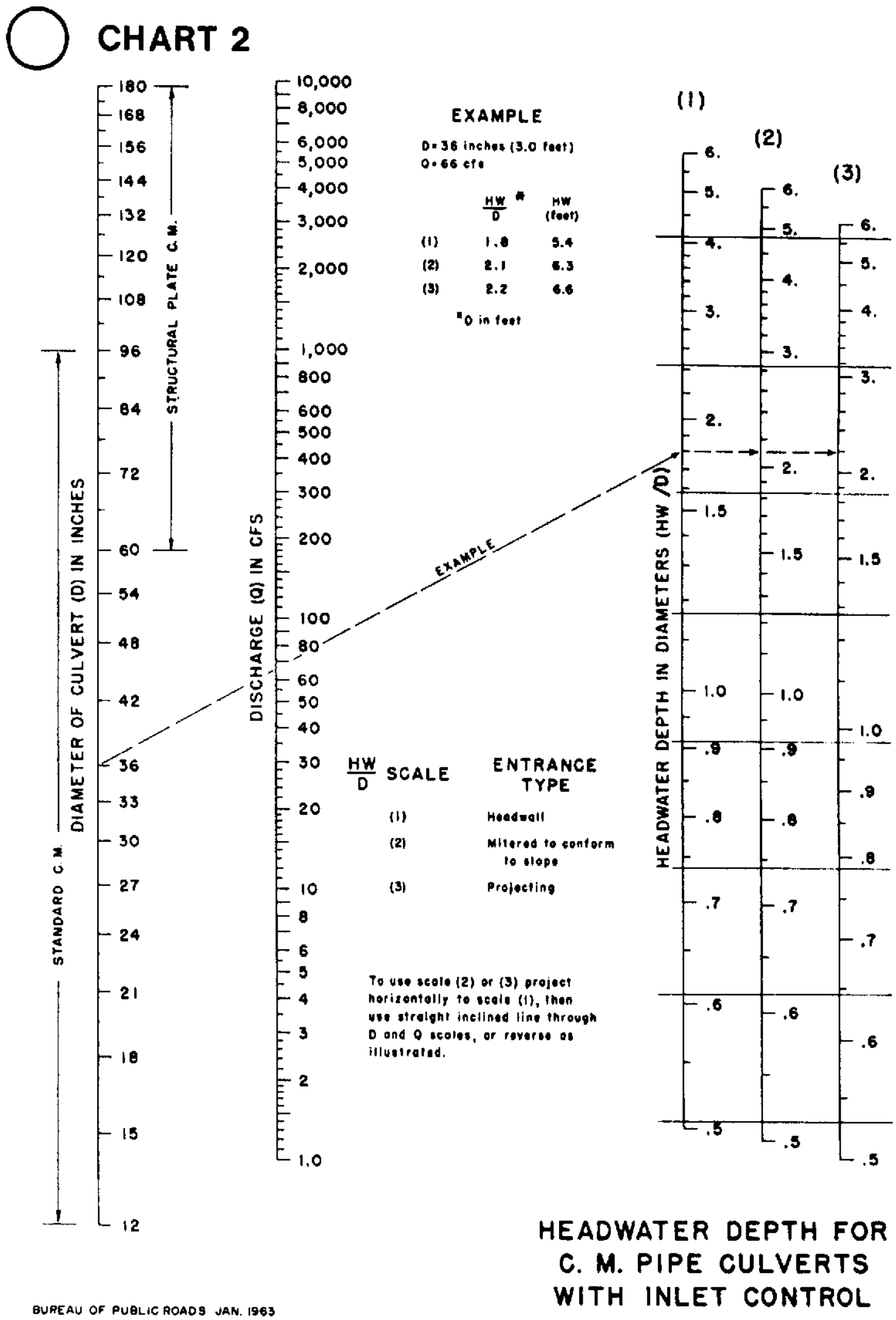
Source: VDOT

Appendix 8C-1 Inlet Control, Circular Concrete



Source: HDS -5

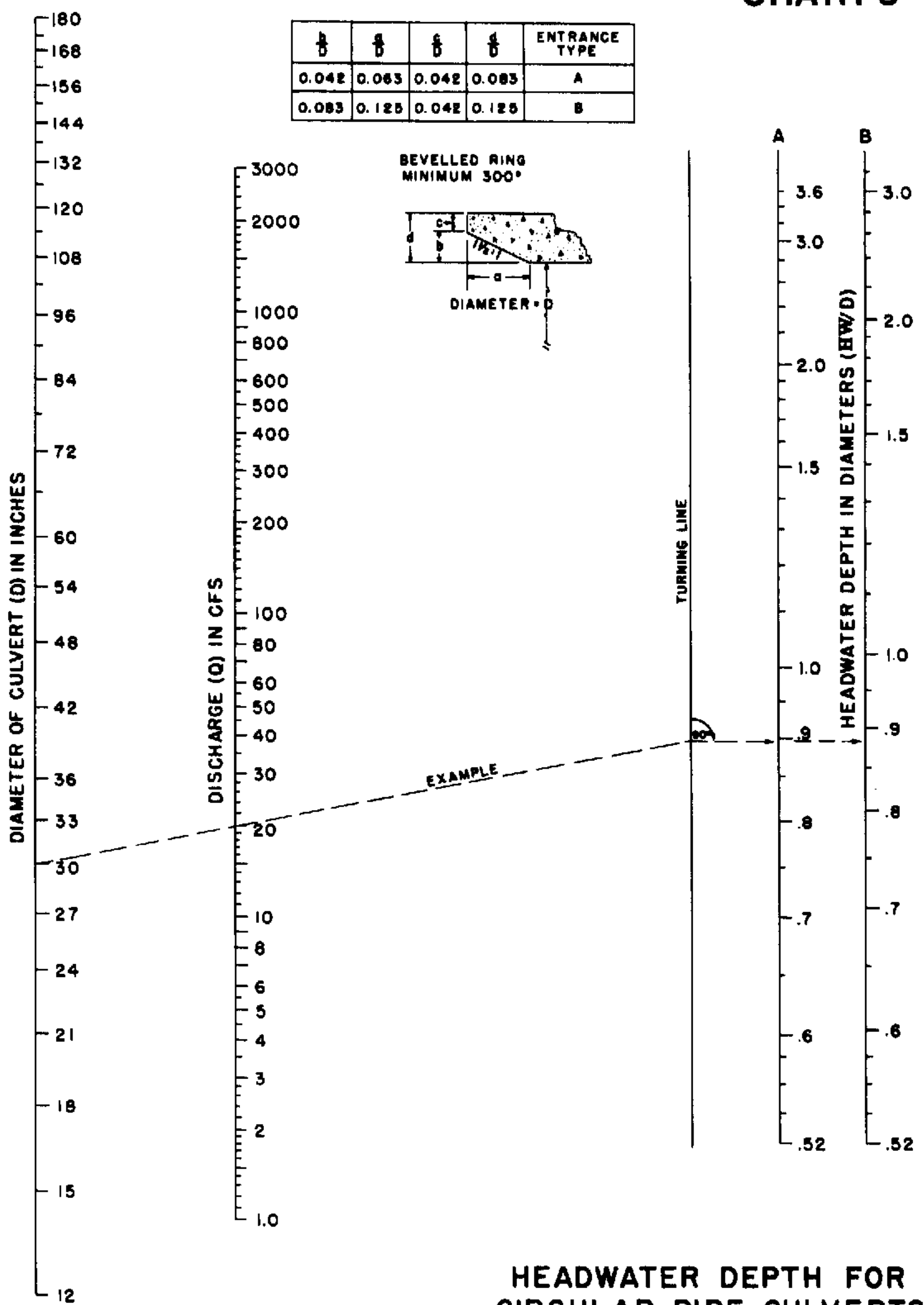
Appendix 8C-2 Inlet Control, Circular Corrugated Metal



Source: HDS-5

Appendix 8C-3 Inlet Control, Circular with Beveled Ring

CHART 3

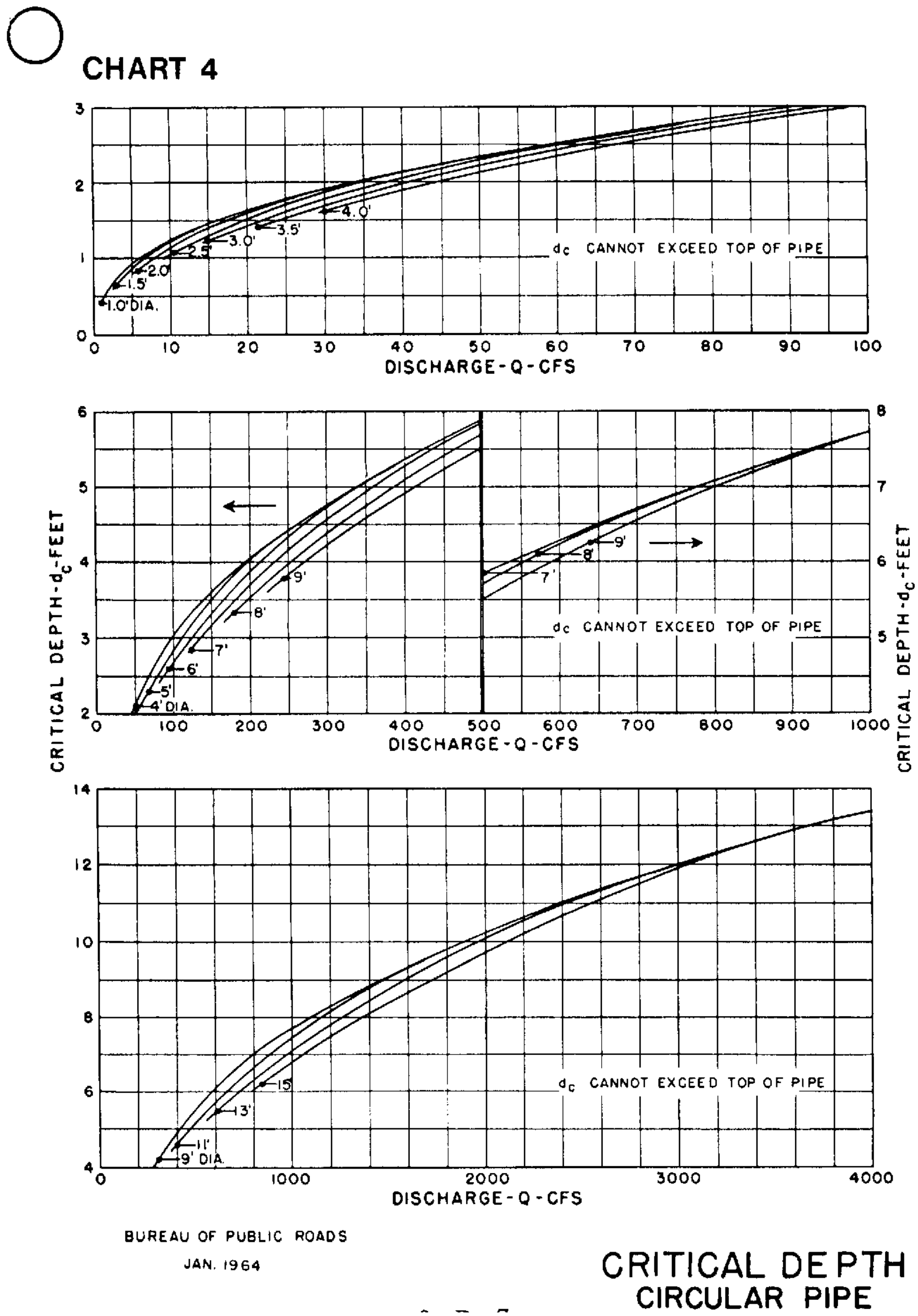


)ERAL HIGHWAY ADMINISTRATION
MAY 1973

HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL

Source: HDS-5

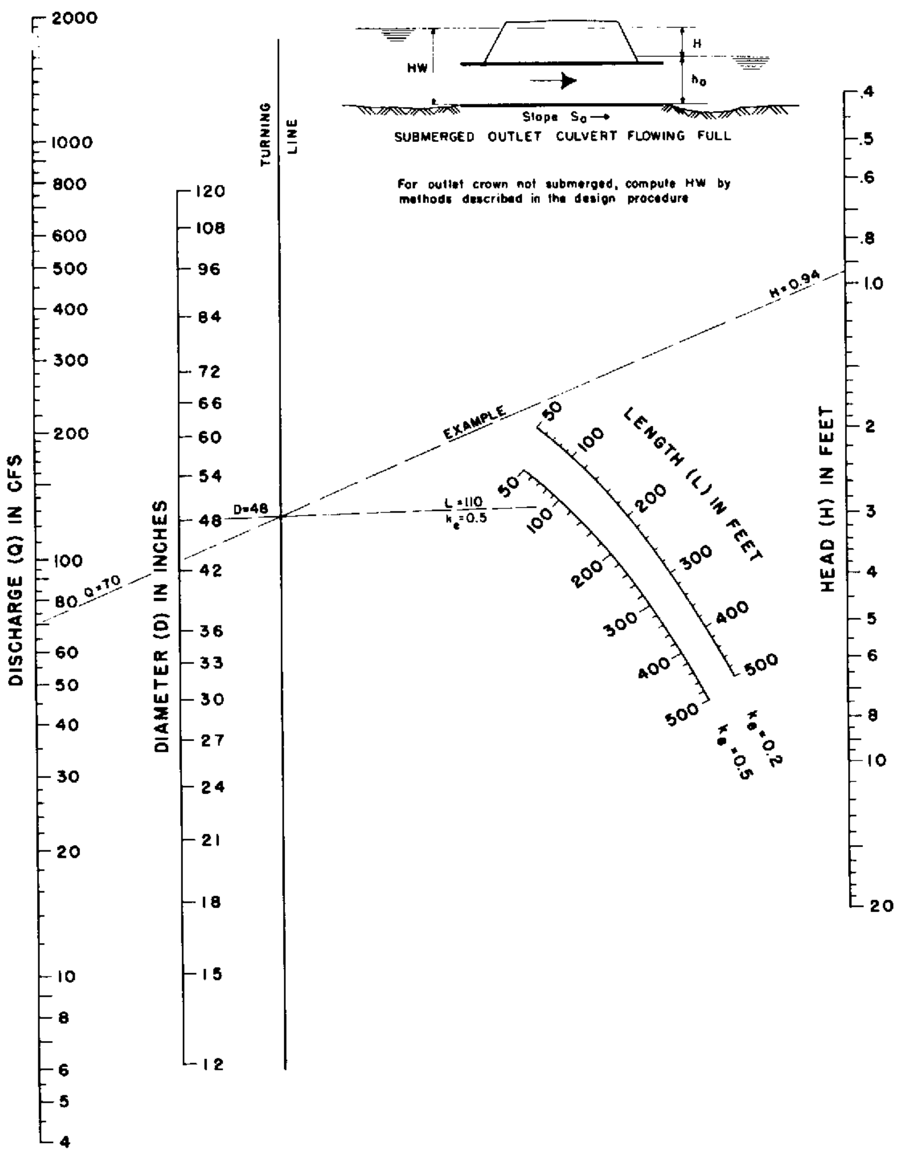
Appendix 8C-4 Critical Depth, Circular



Source: HDS-5

Appendix 8C-5 Outlet Control, Circular Concrete

CHART 5

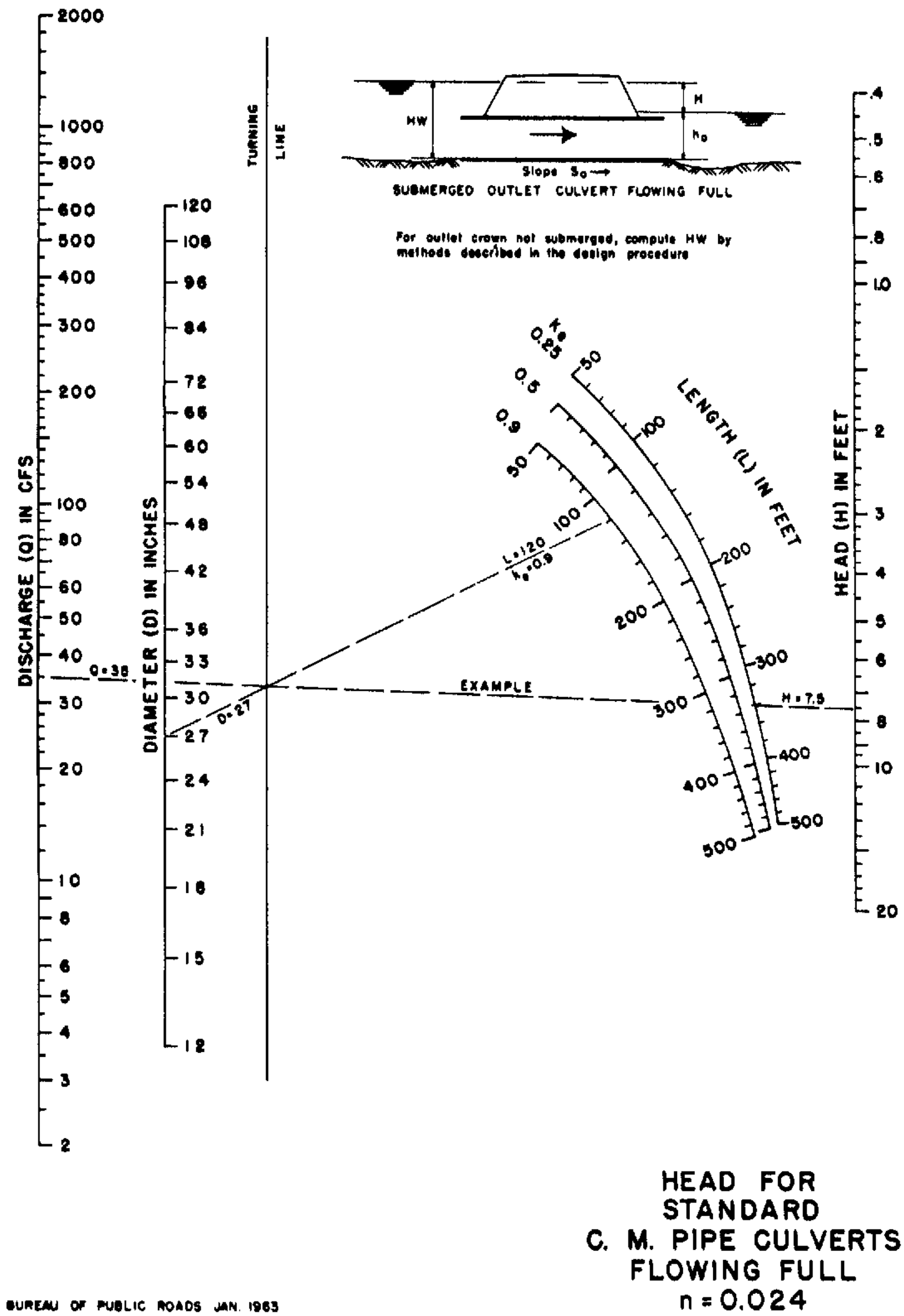


HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

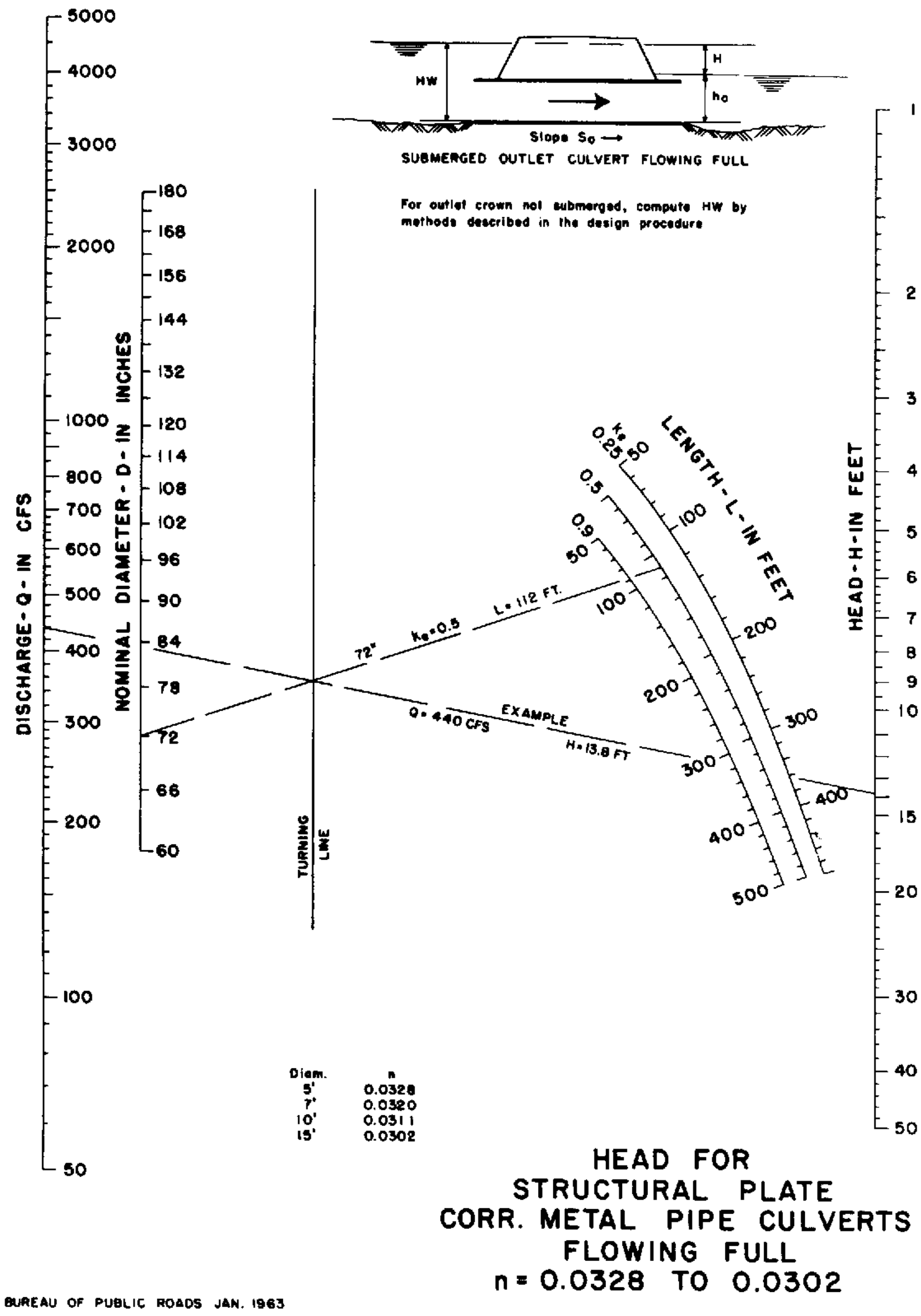
Source: HDS-5

Appendix 8C-6 **Outlet Control, Circular Corrugated Metal**



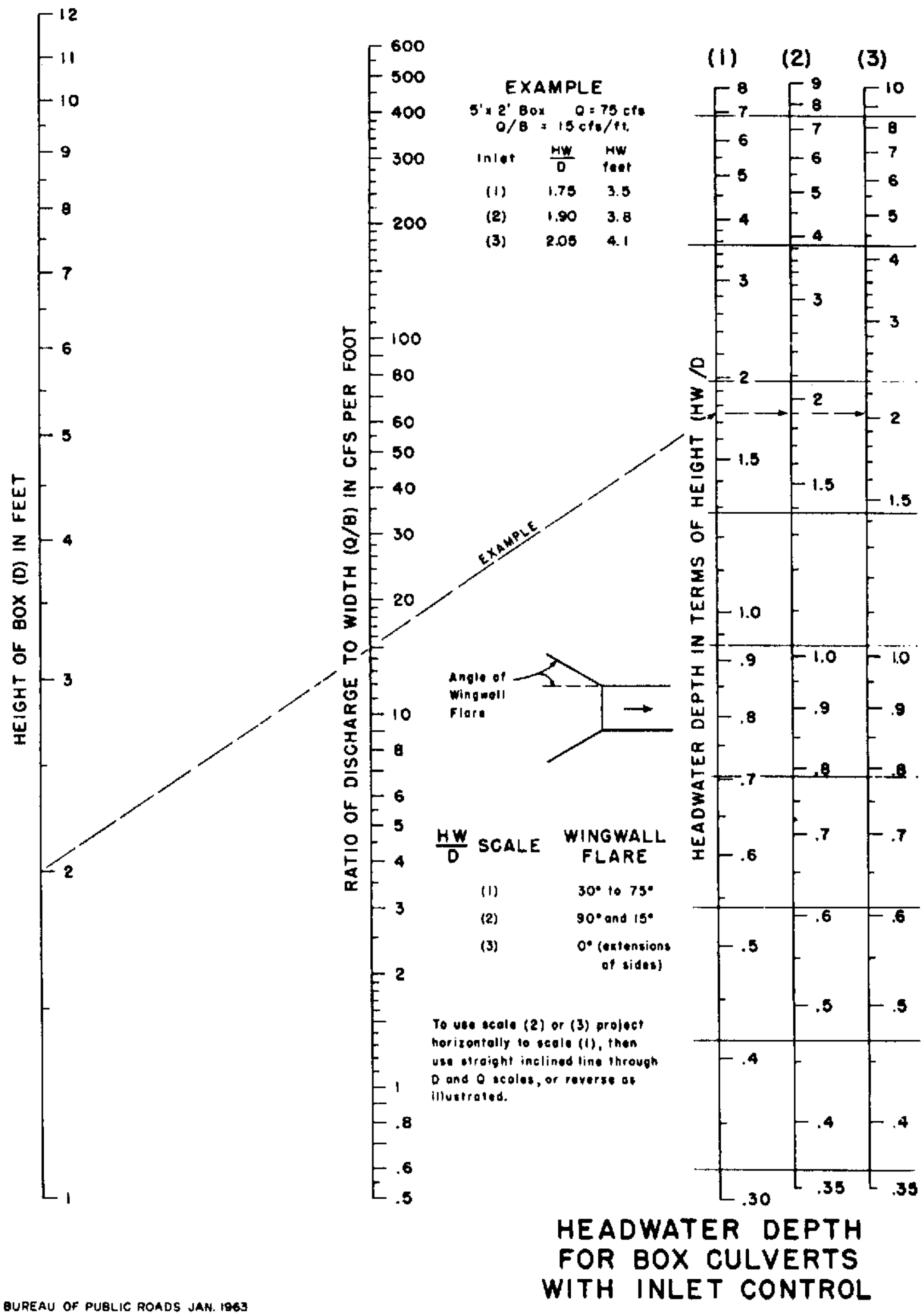
Source: HDS-5

Appendix 8C-7 Outlet Control, Circular Structural Plate Corrugated Metal



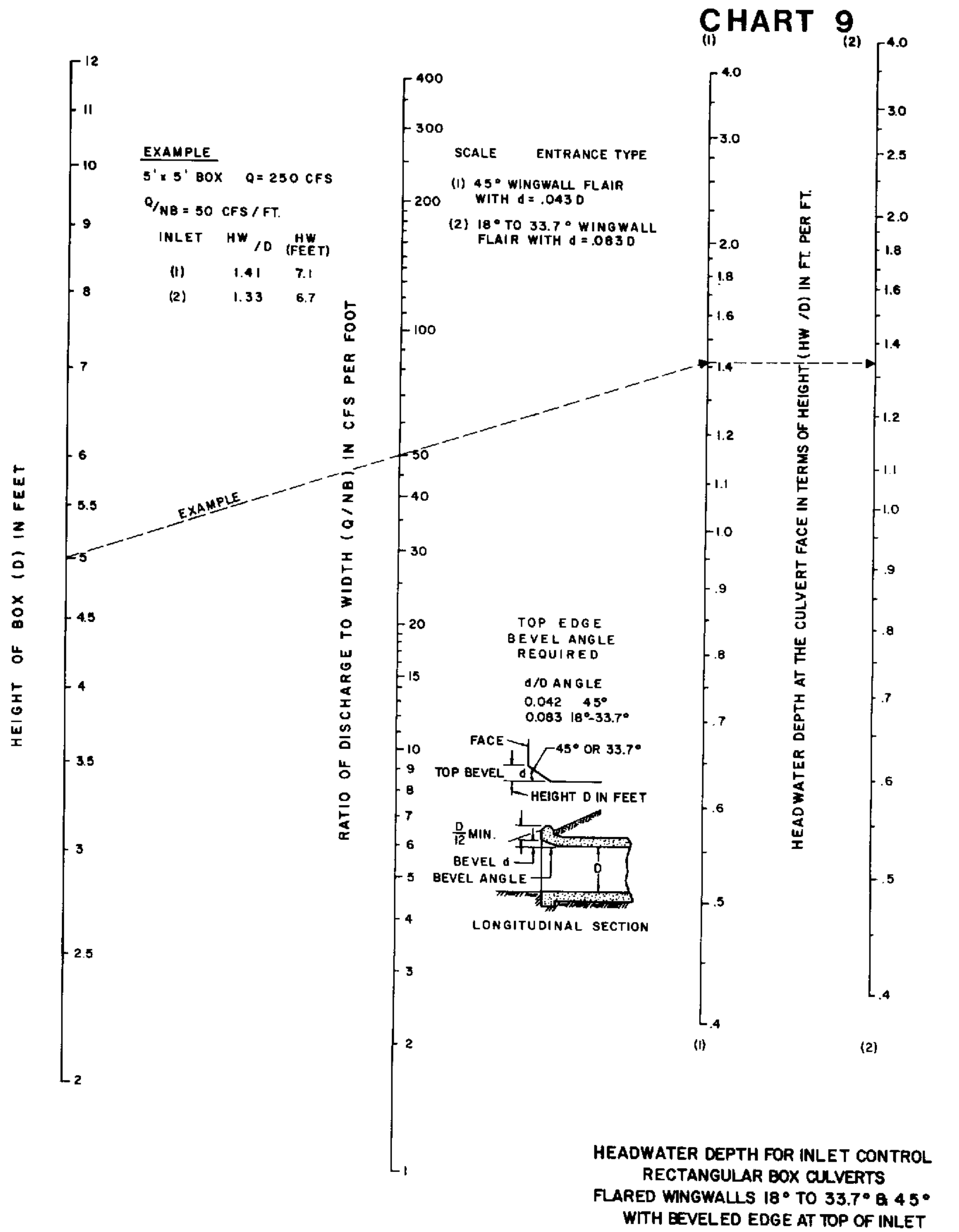
Source: HDS-5

Appendix 8C-8 Inlet Control, Concrete Box



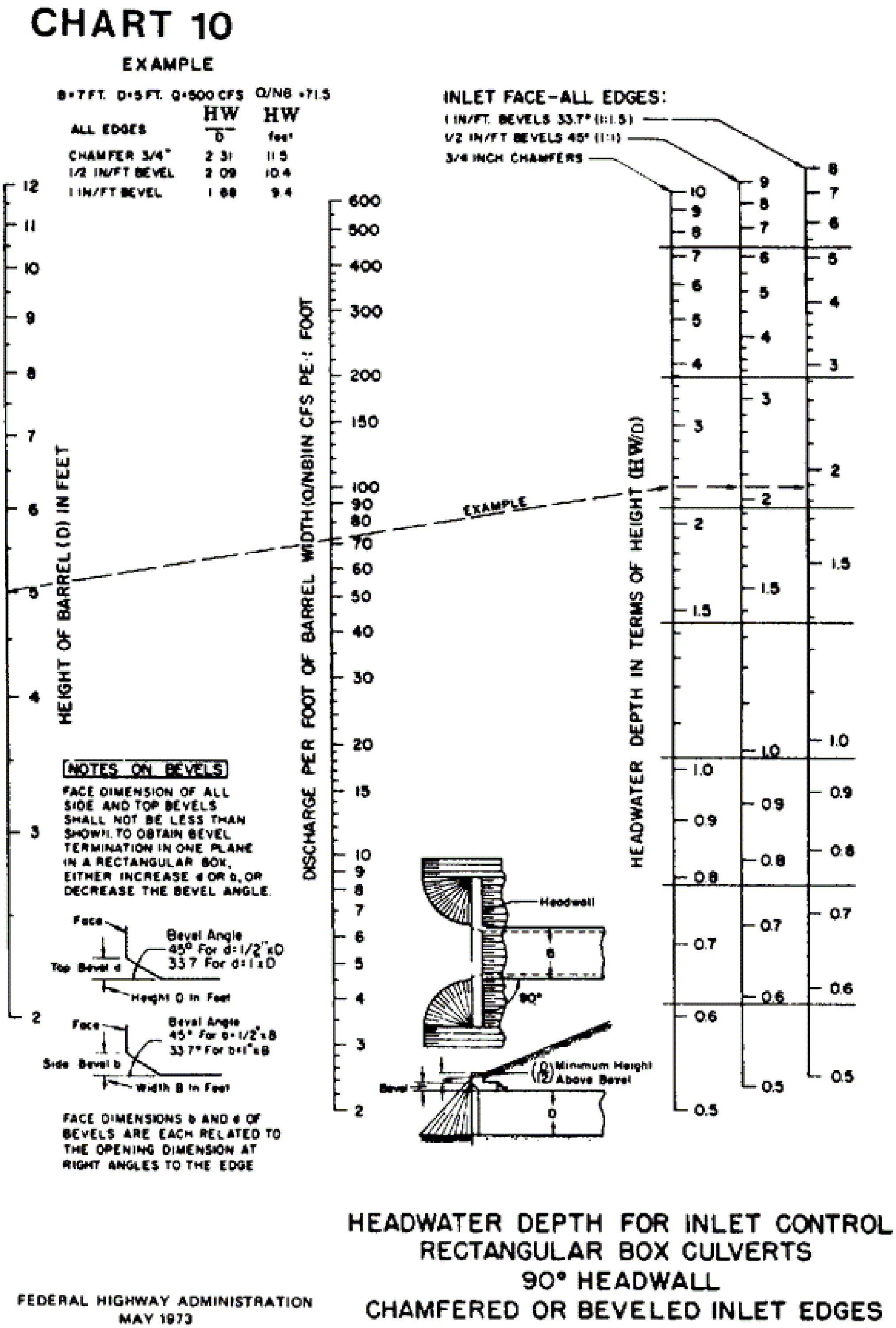
Source: HDS-5

Appendix 8C-9 Inlet Control, Concrete Box, Flared Wingwalls at 18° to 33.7°and 45°, Beveled Top Edge



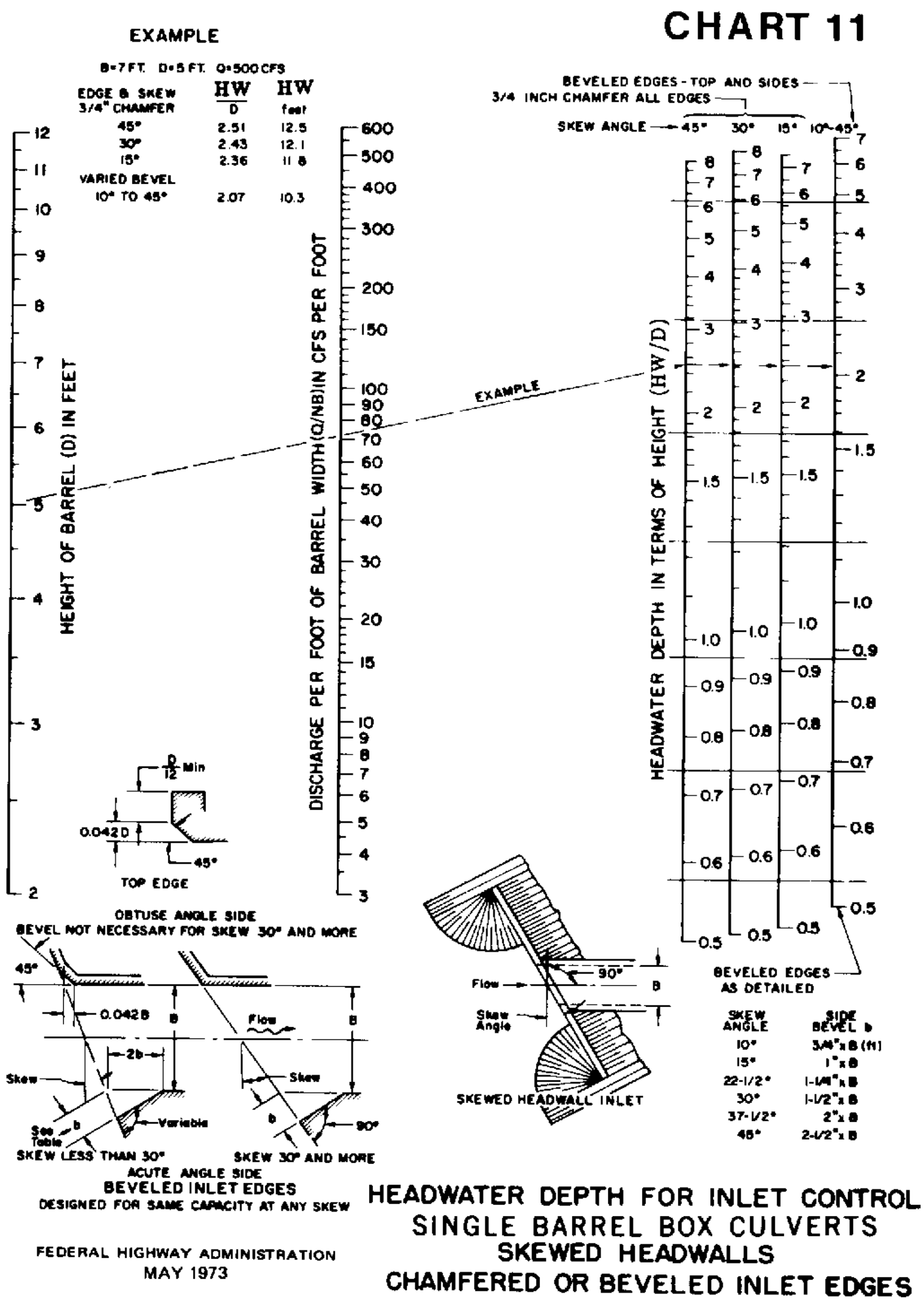
Source: HDS-5

Appendix 8C-10 Inlet Control, Concrete Box, 90^{o} Headwall, Chamfered or Beveled Edges



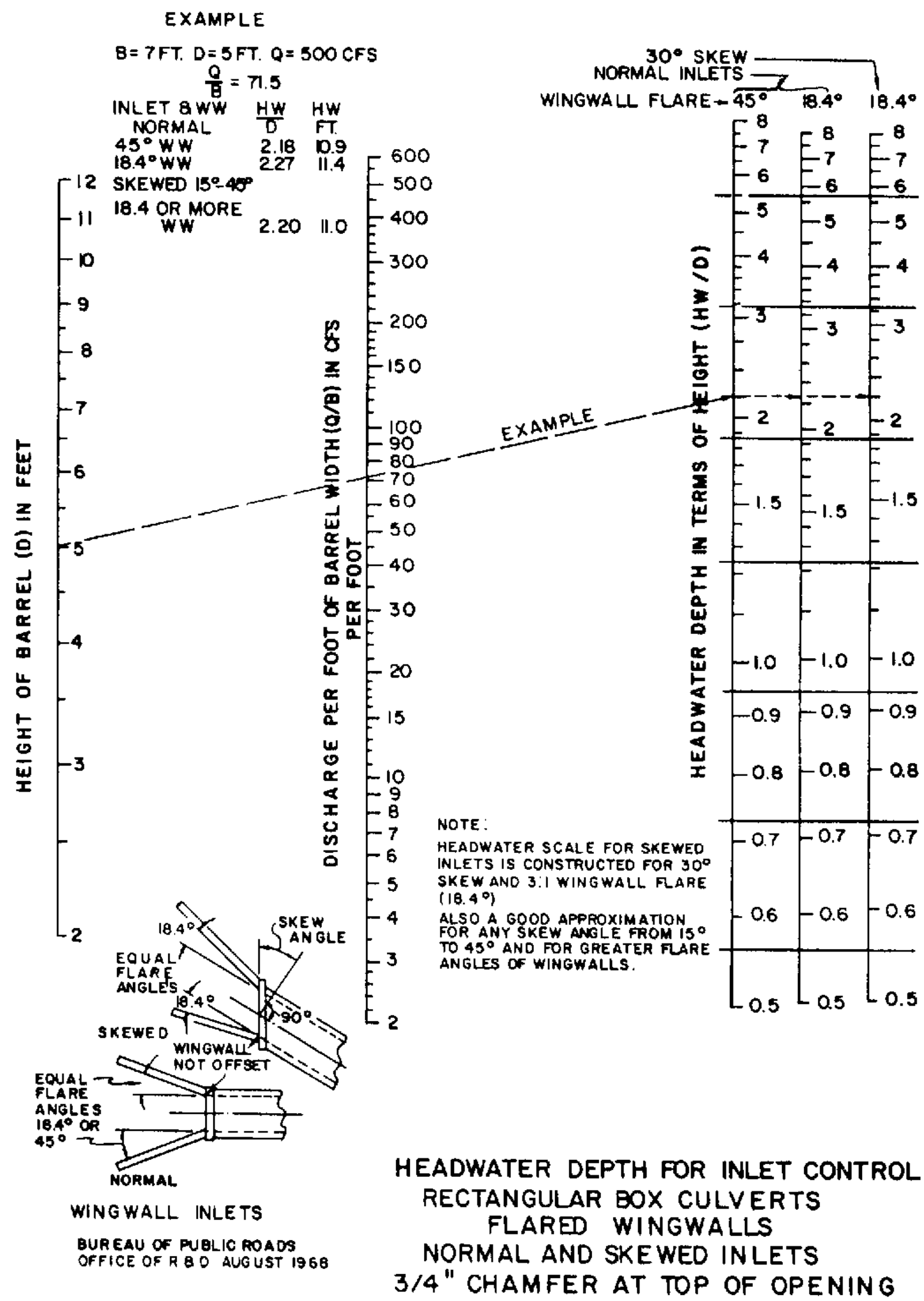
Source: HDS-5

Appendix 8C-11 Inlet Control, Single Barrel Concrete Box, Skewed Headwalls Chamfered or Beveled Edges



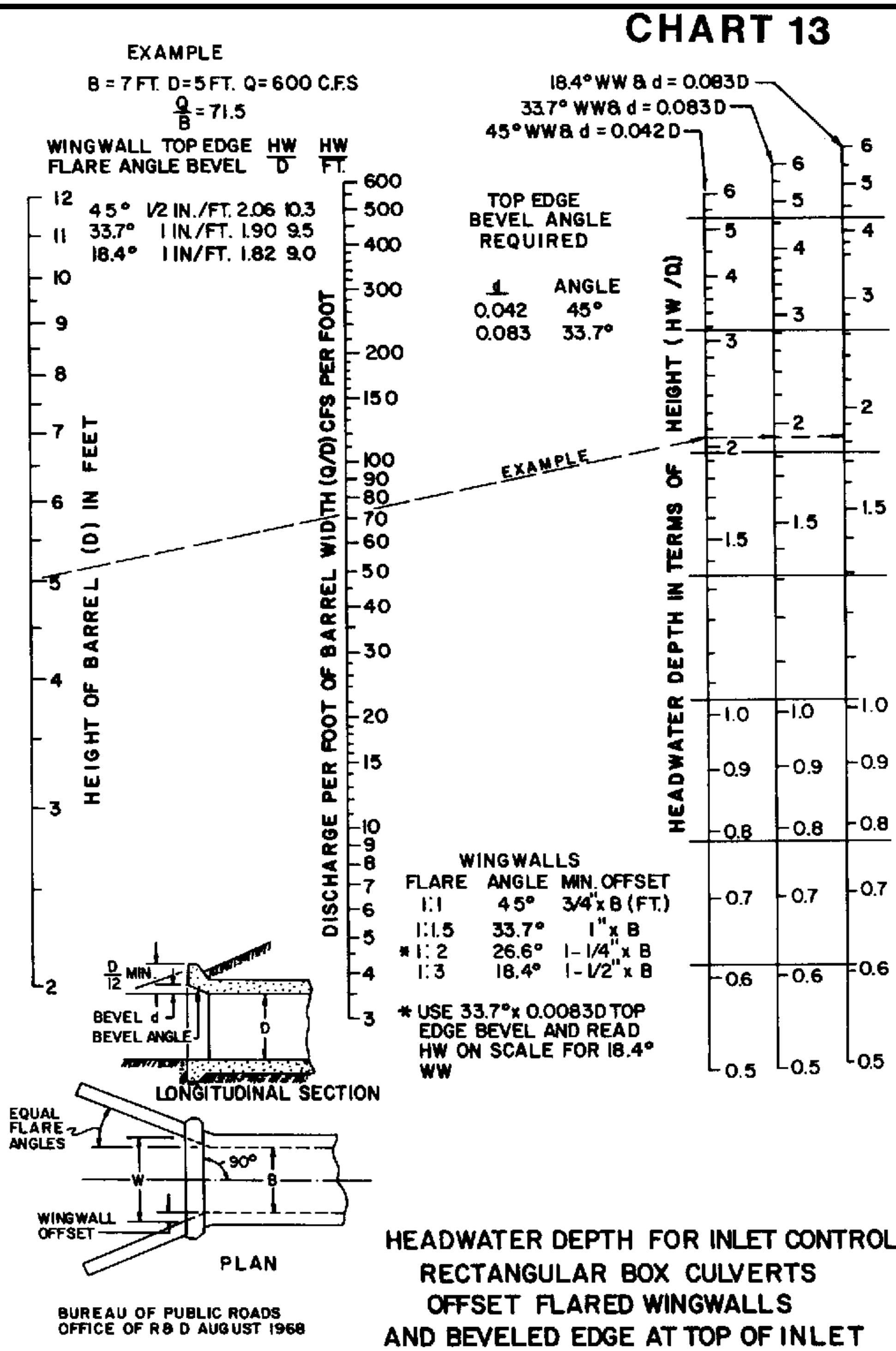
Source: HDS-5

Appendix 8C-12 Inlet Control, Concrete Box, Flared Wingwalls, Normal and Skewed Inlets, Chamfered Top Edge



Source: HDS-5

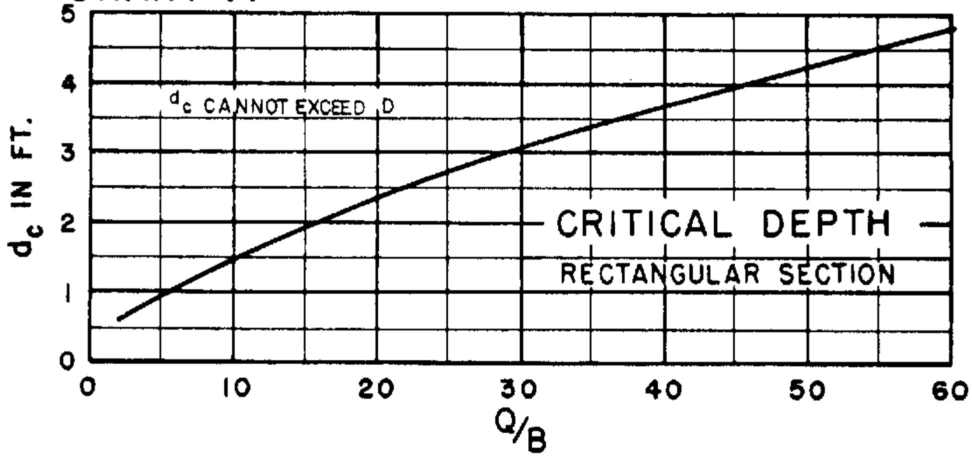
Appendix 8C-13 Inlet Control, Concrete Box with Offset Flared Wingwalls, Beveled Top Edge

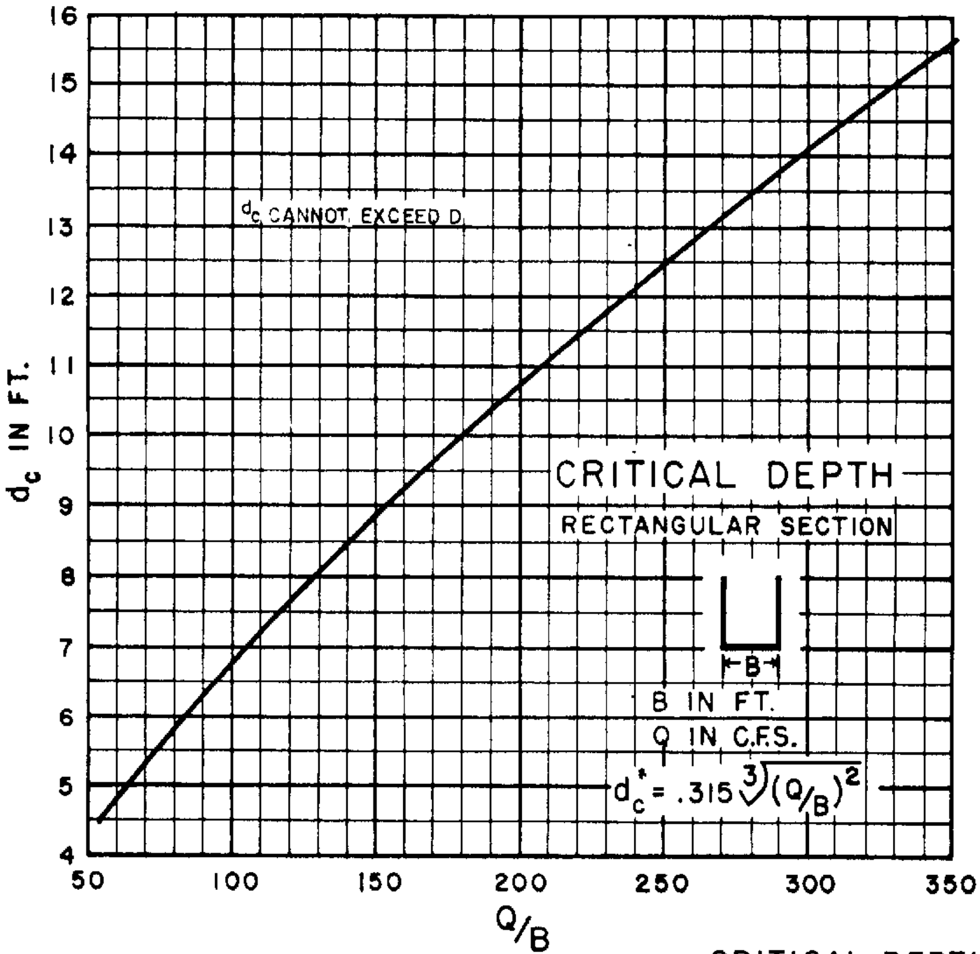


Source: HDS-5

Appendix 8C-14 Critical Depth, Concrete Box

CHART 14





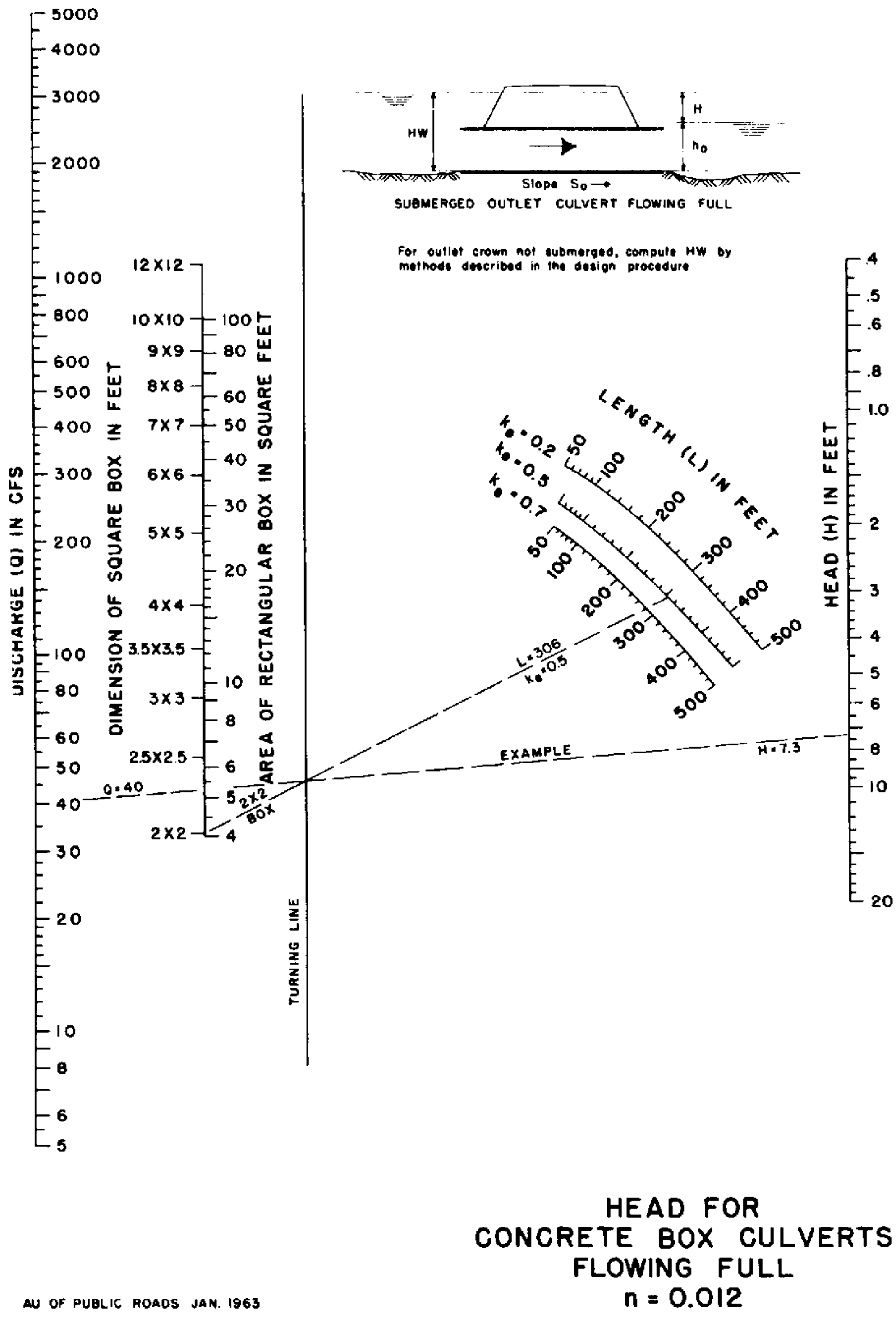
BUREAU OF PUBLIC ROADS JAN 1963

5-38

CRITICAL DEPTH
RECTANGULAR SECTION

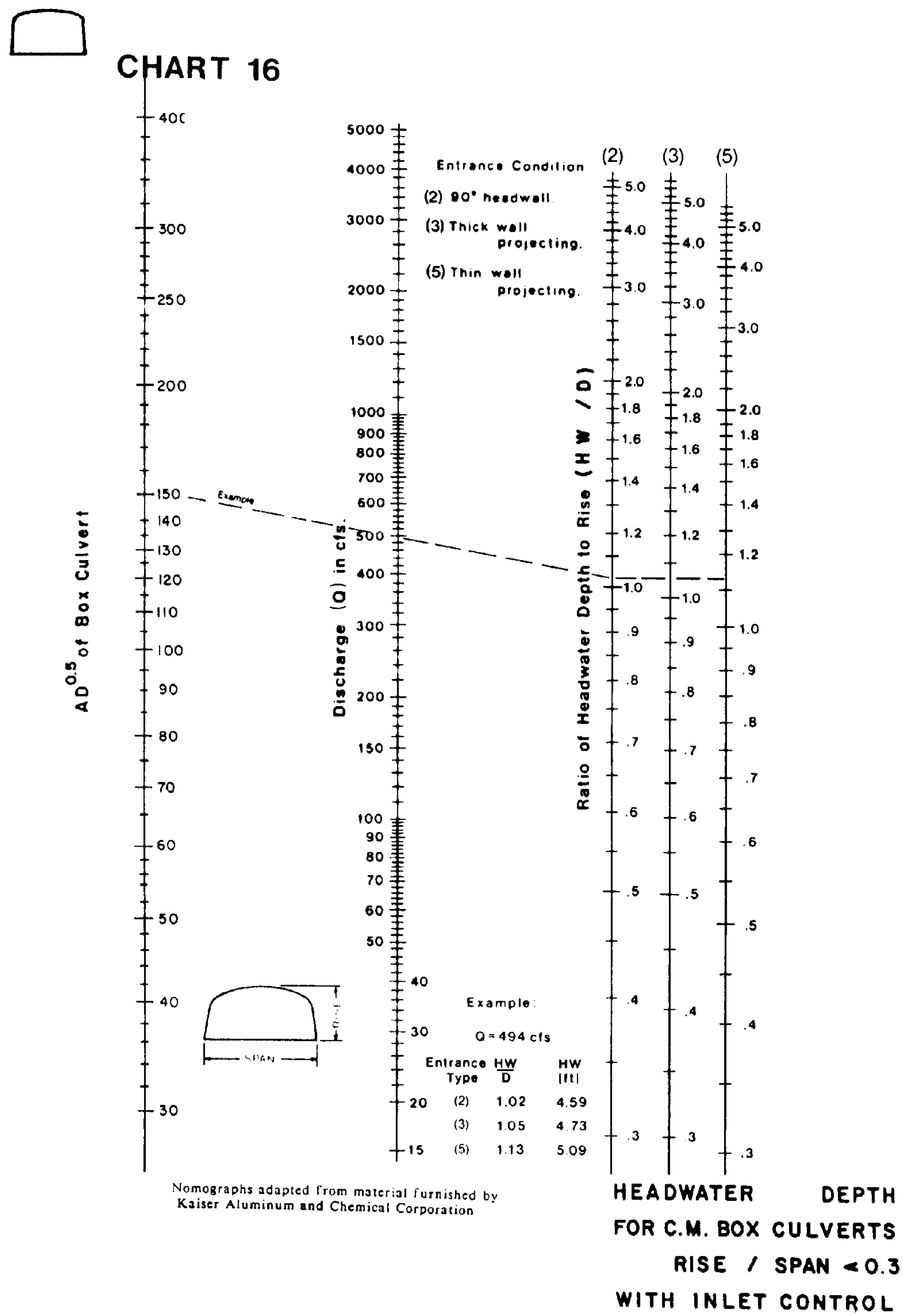
Source: HDS-5

Appendix 8C-15 Outlet Control, Concrete Box



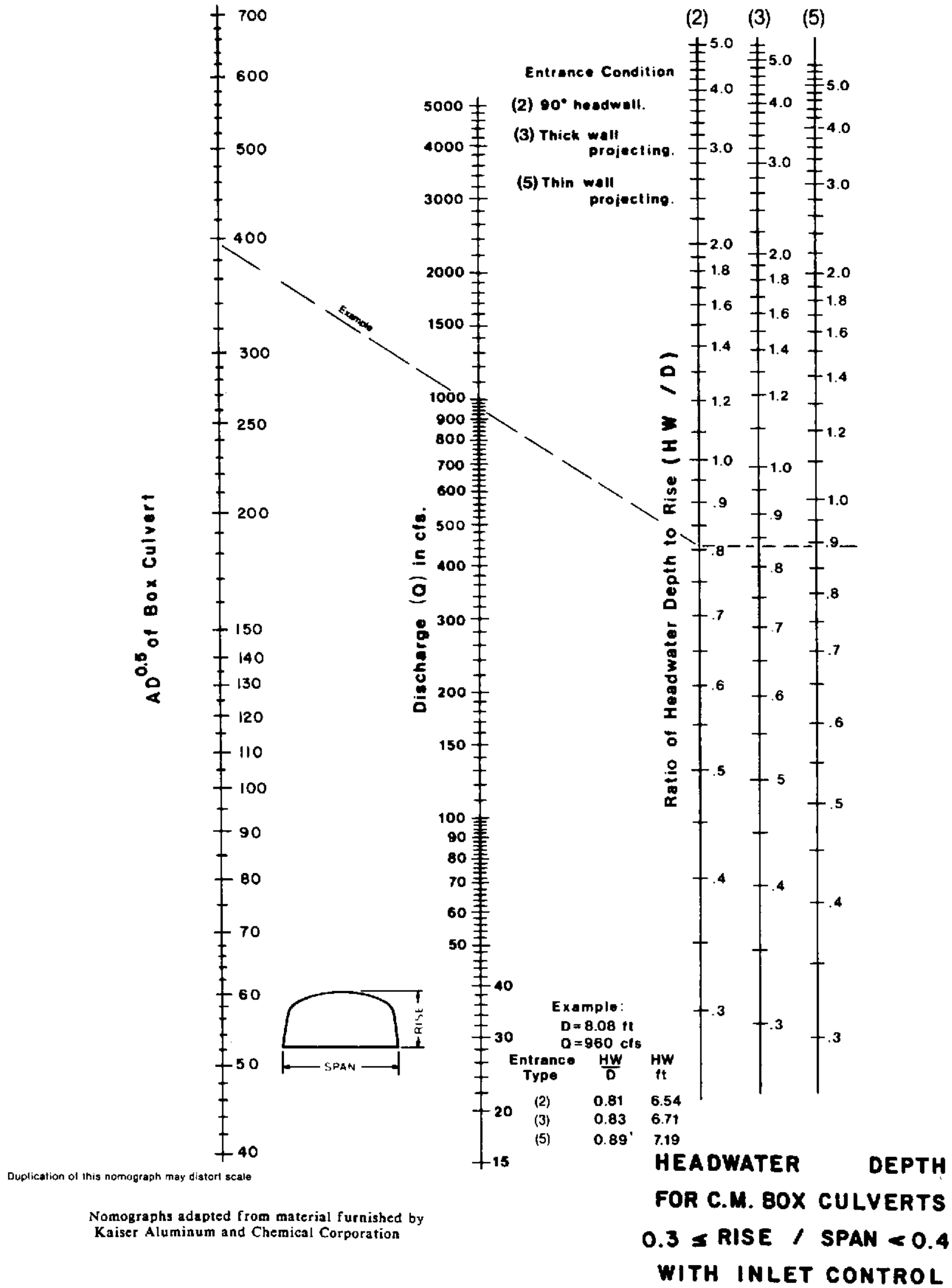
Source: HDS-5

Appendix 8C-16 **Inlet Control, Corrugated Metal Box, Rise/Span <0.3**



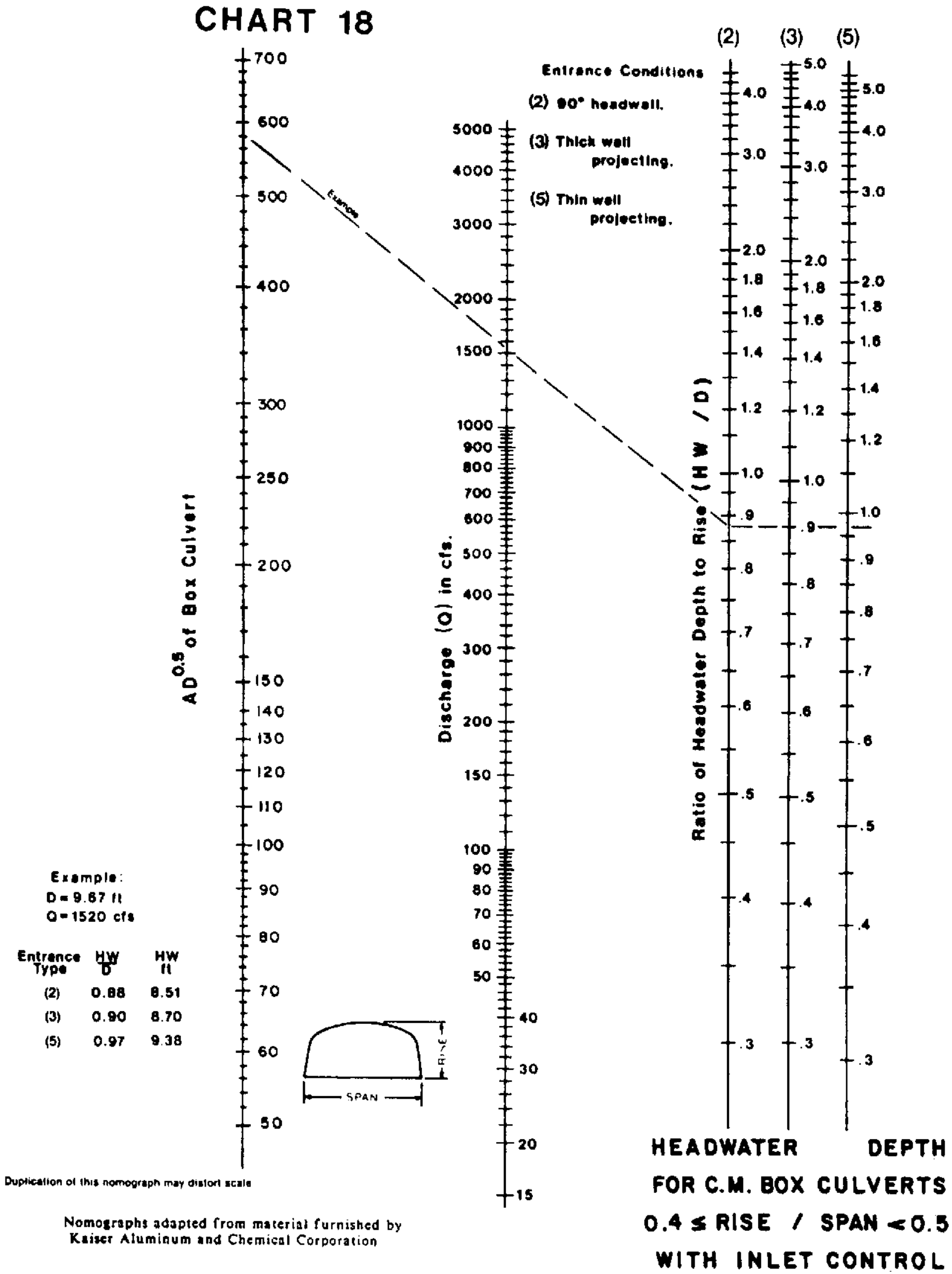
Source: HDS-5

Appendix 8C-17 Inlet Control, Corrugated Metal Box, 0.3≤ Rise/Span <0.4



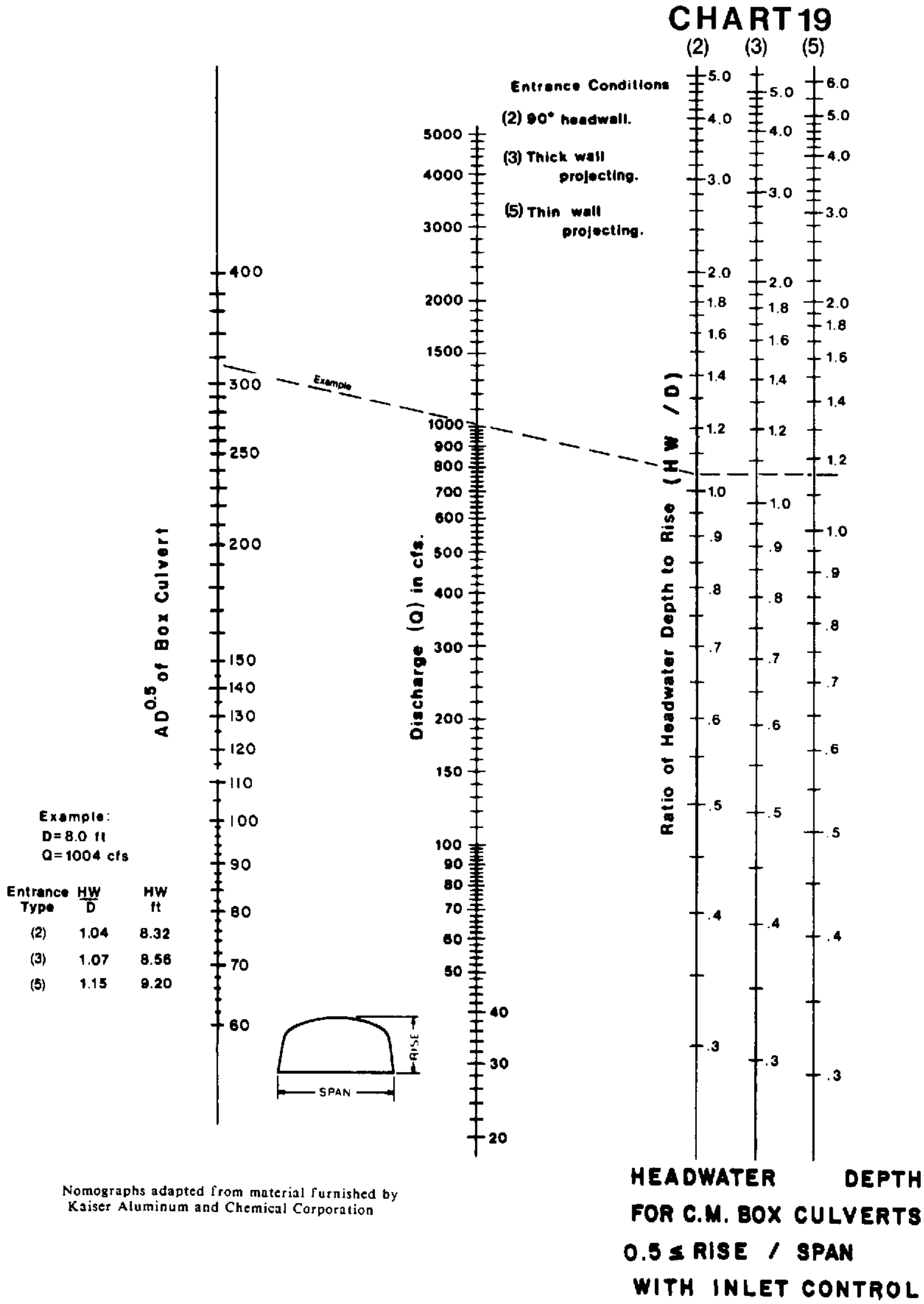
Source: HDS-5

Appendix 8C-18 **Inlet Control, Corrugated Metal Box, 0.4≤Rise/Span <0.5**



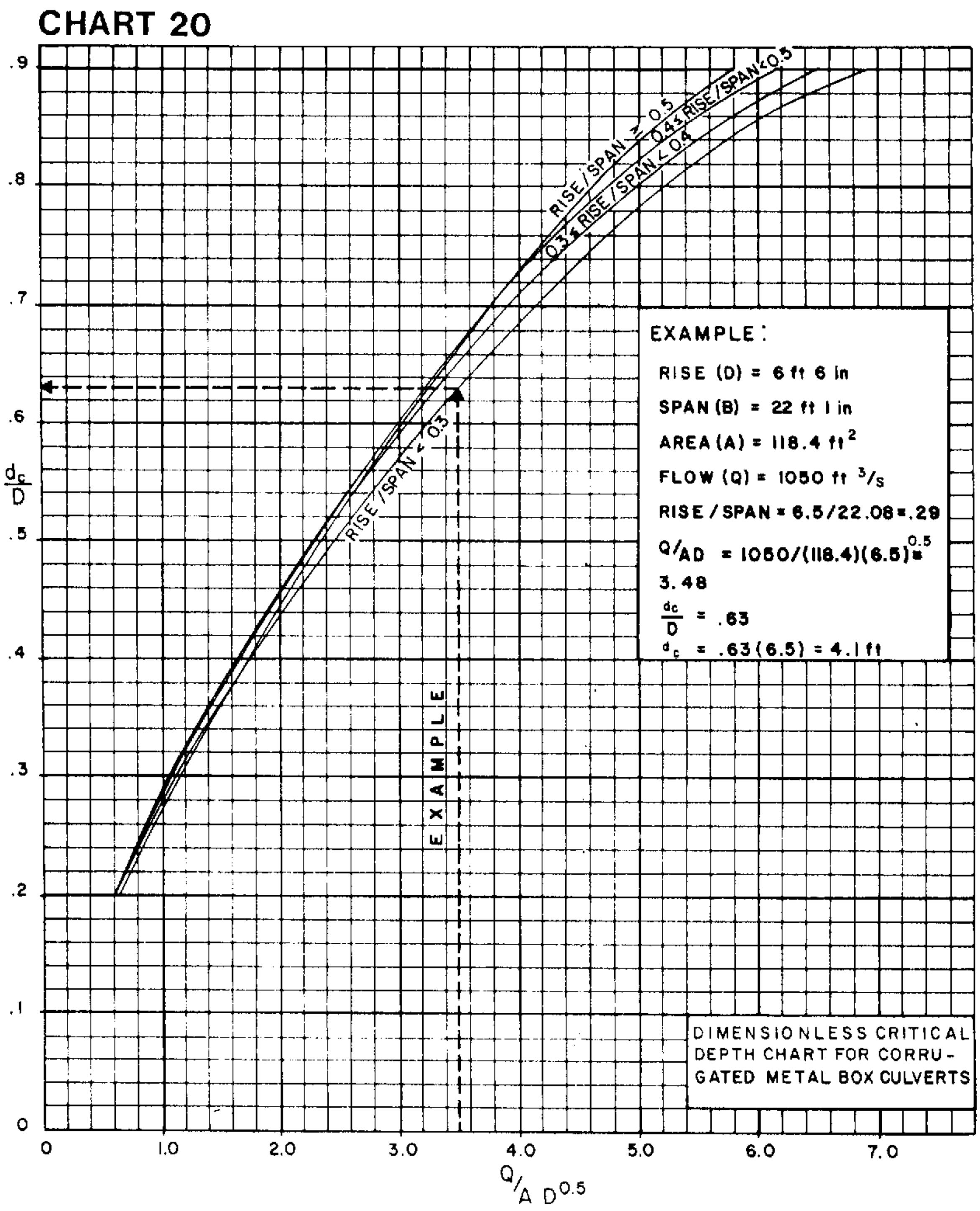
Source: HDS-5

Appendix 8C-19 **Inlet Control, Corrugated Metal Box, 0.5≤ Rise/Span**



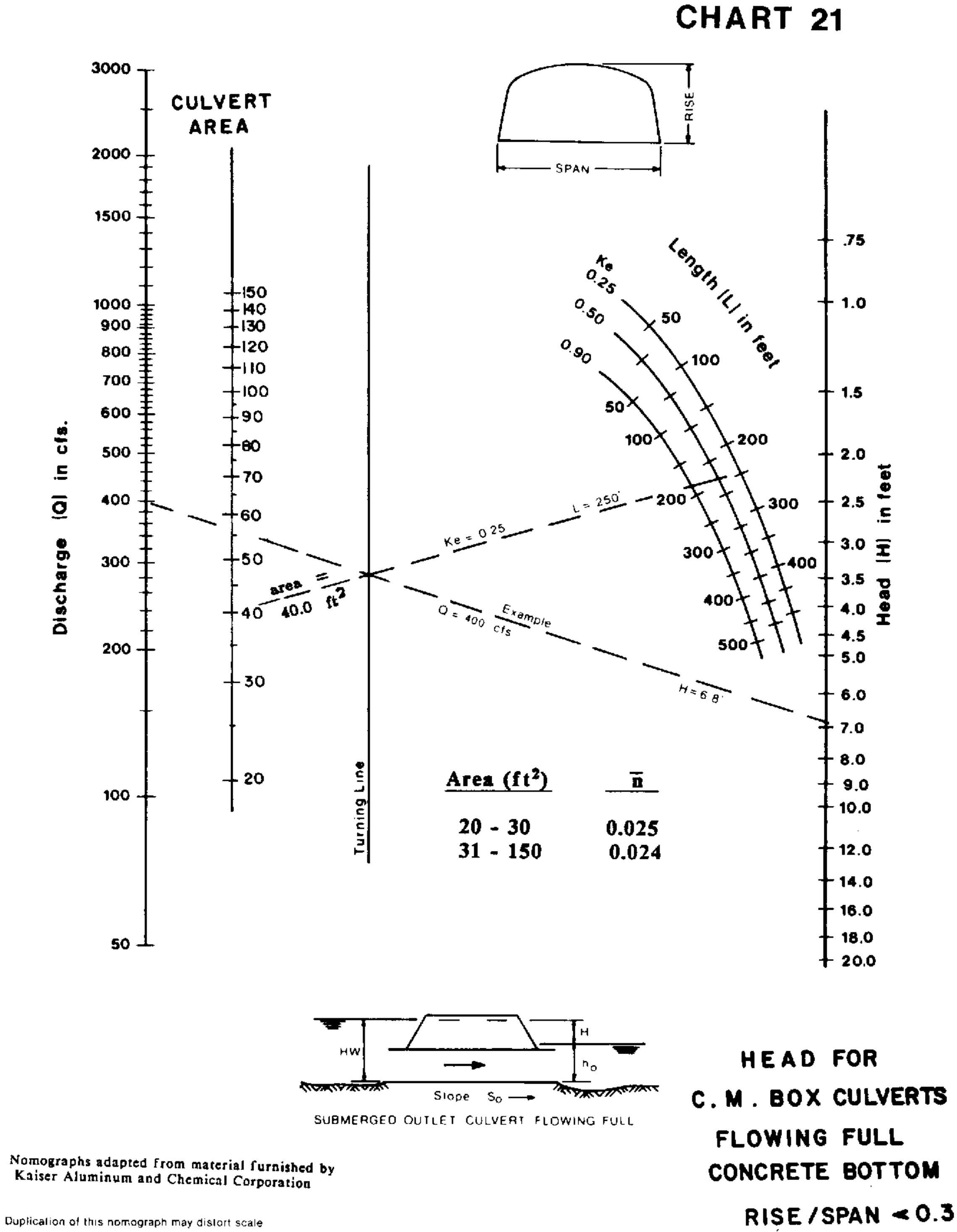
Source: HDS-5

Appendix 8C-20 Critical Depth, Corrugated Metal Box



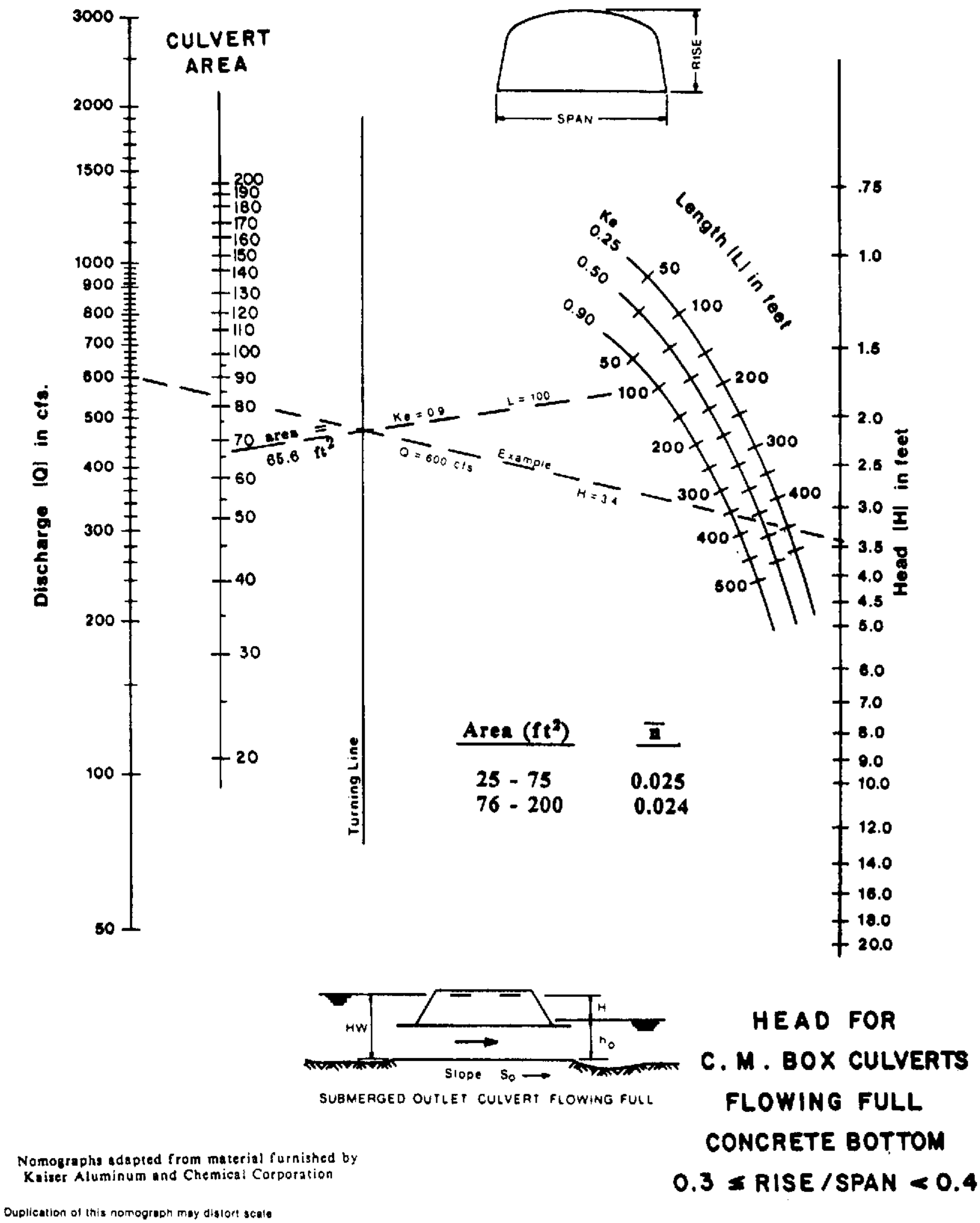
Source: HDS-5

Appendix 8C-21 Outlet Control, Corrugated Metal Box, Concrete Bottom Rise/Span <0.3



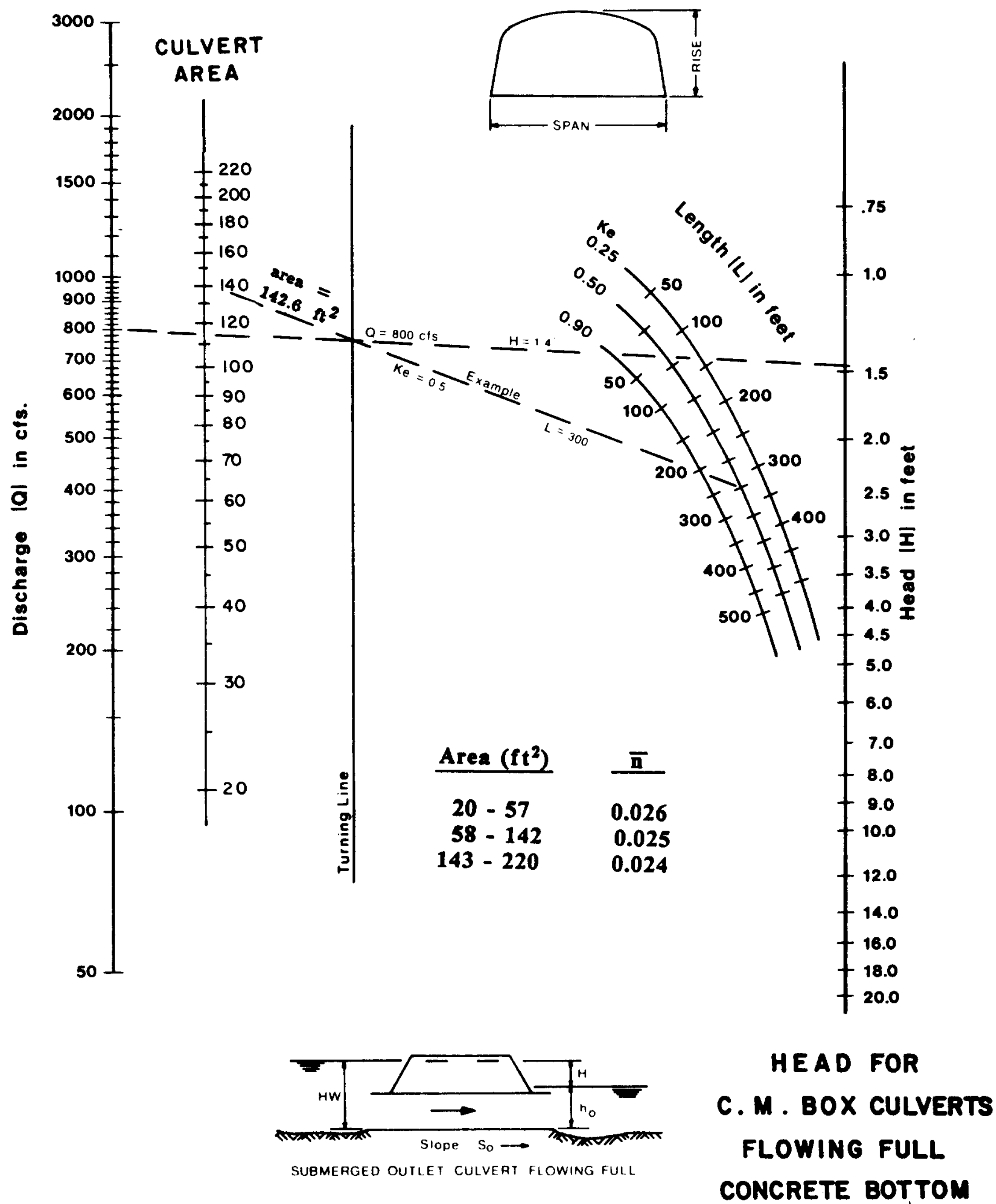
Source: HDS-3

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



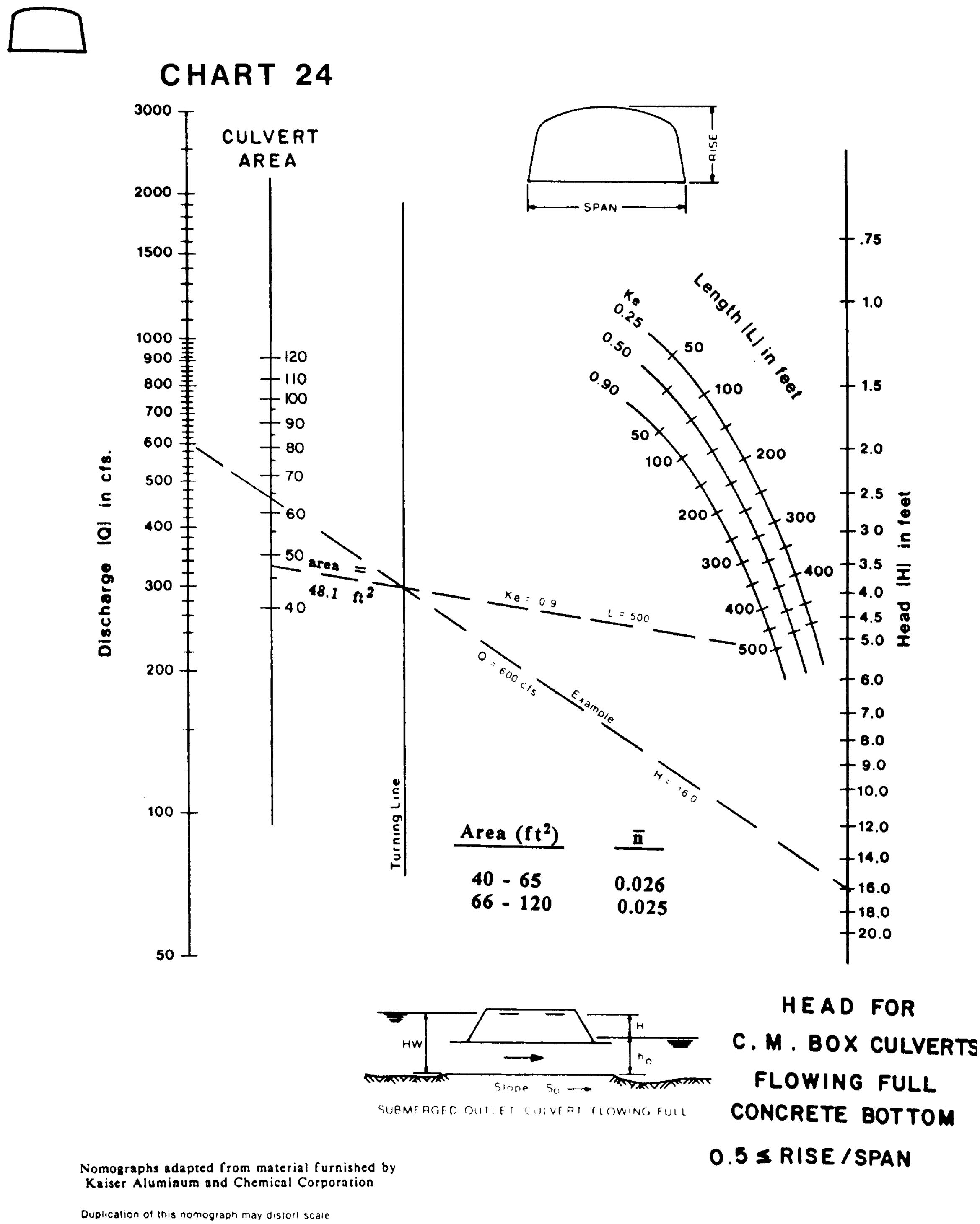
Area (ft²)	$\bar{n}$
20 - 57	0.026
58 - 142	0.025
143 - 220	0.024

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation

Duplication of this nomograph may distort scale

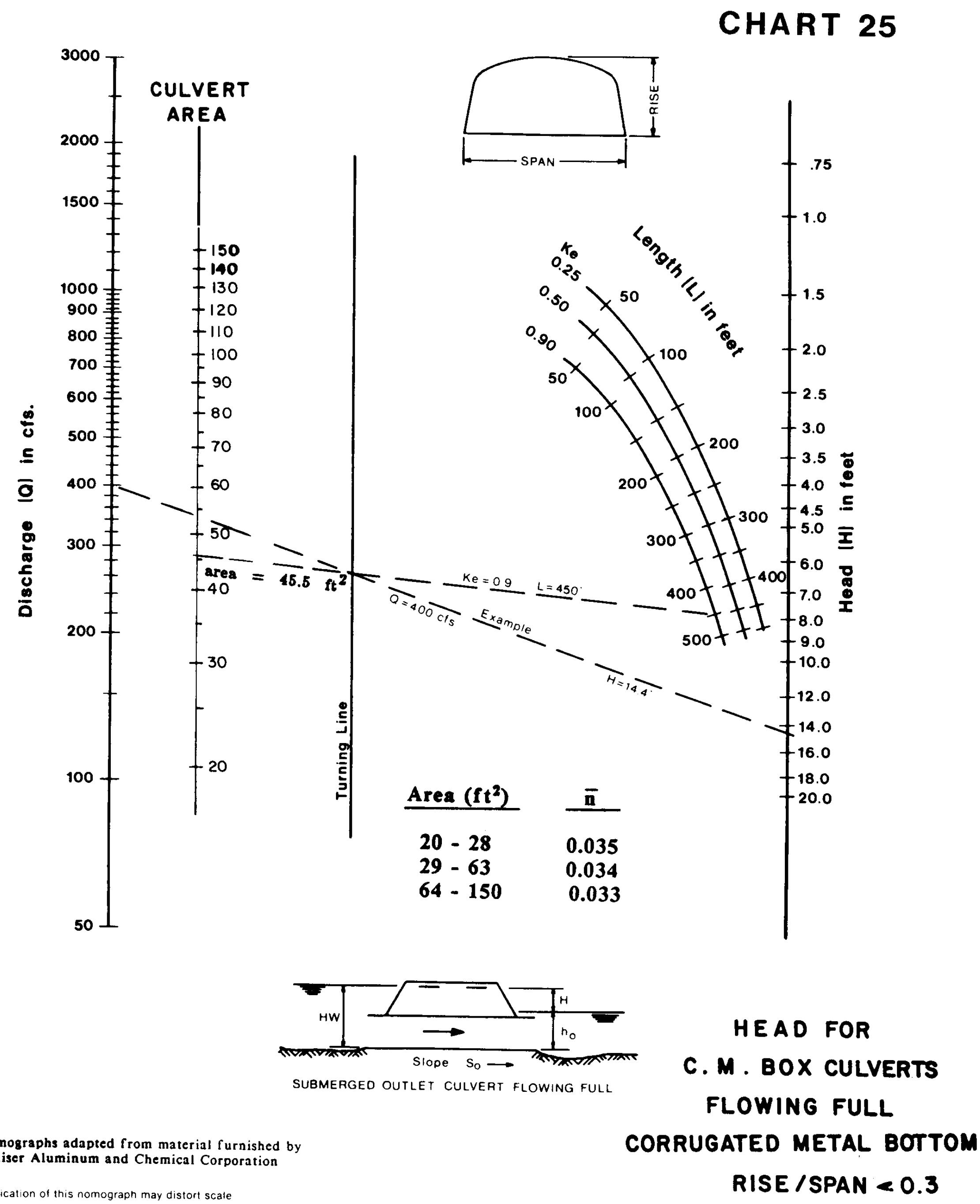
Source: HDS-5

Appendix 8C-24 Outlet Control, Corrugated Metal Box, Concrete Bottom 0.5≤ Rise/Span



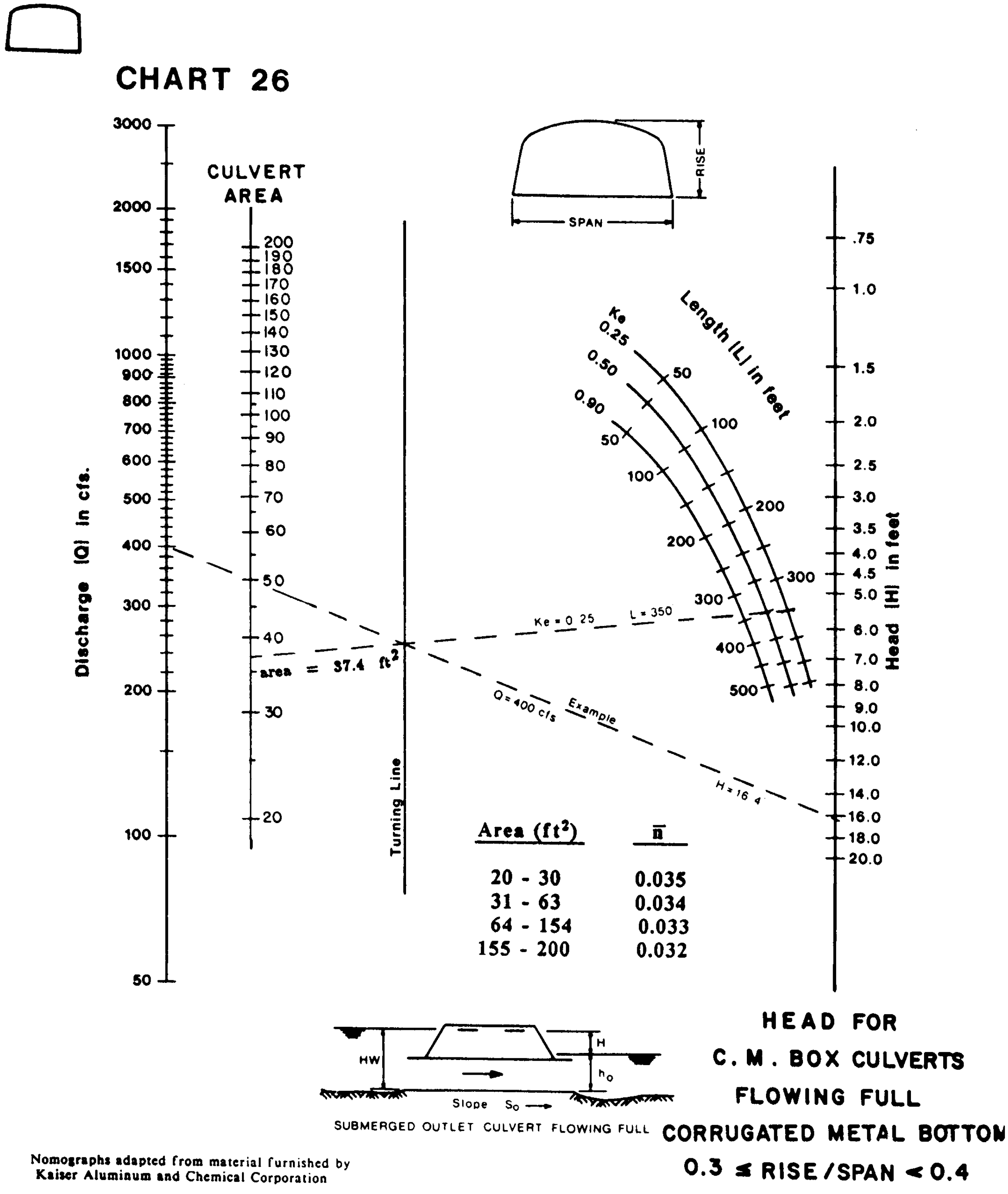
Source: HDS-5

Appendix 8C-25 Outlet Control, Corrugated Metal Box, Corrugated Metal Bottom, Rise/Span <0.3



Source: HDS-5

Appendix 8C-26 Outlet Control, Corrugated Metal Box, Corrugated Metal Box, 0.3≤ Rise/Span <0.4



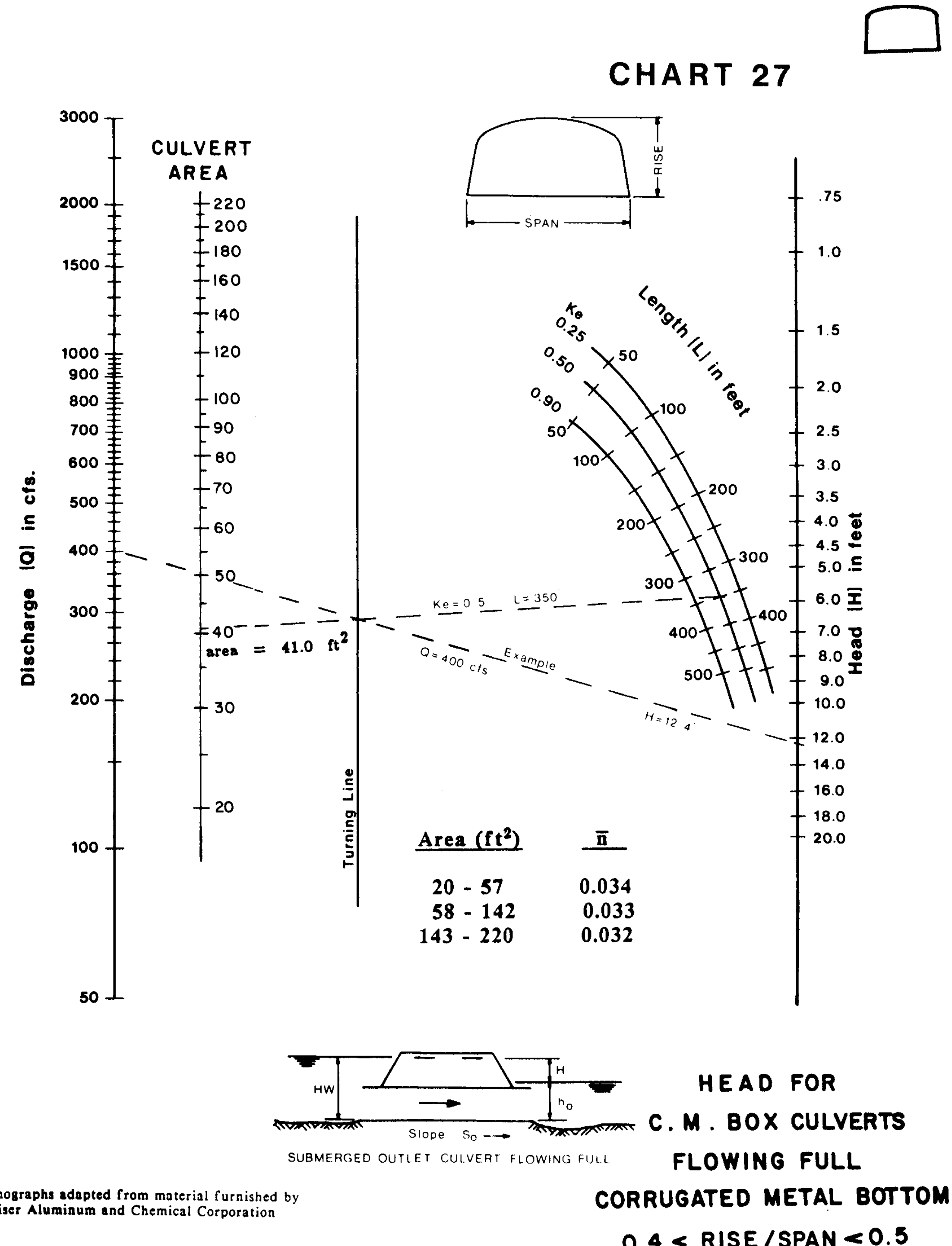
Area (ft²)	n
20 - 30	0.035
31 - 63	0.034
64 - 154	0.033
155 - 200	0.032

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation

Duplication of this nomograph may distort scale

Source: HDS-5

Appendix 8C-27 Outlet Control, Corrugated Metal Box, Corrugated Metal Bottom, 0.4≤ Rise/Span <0.5

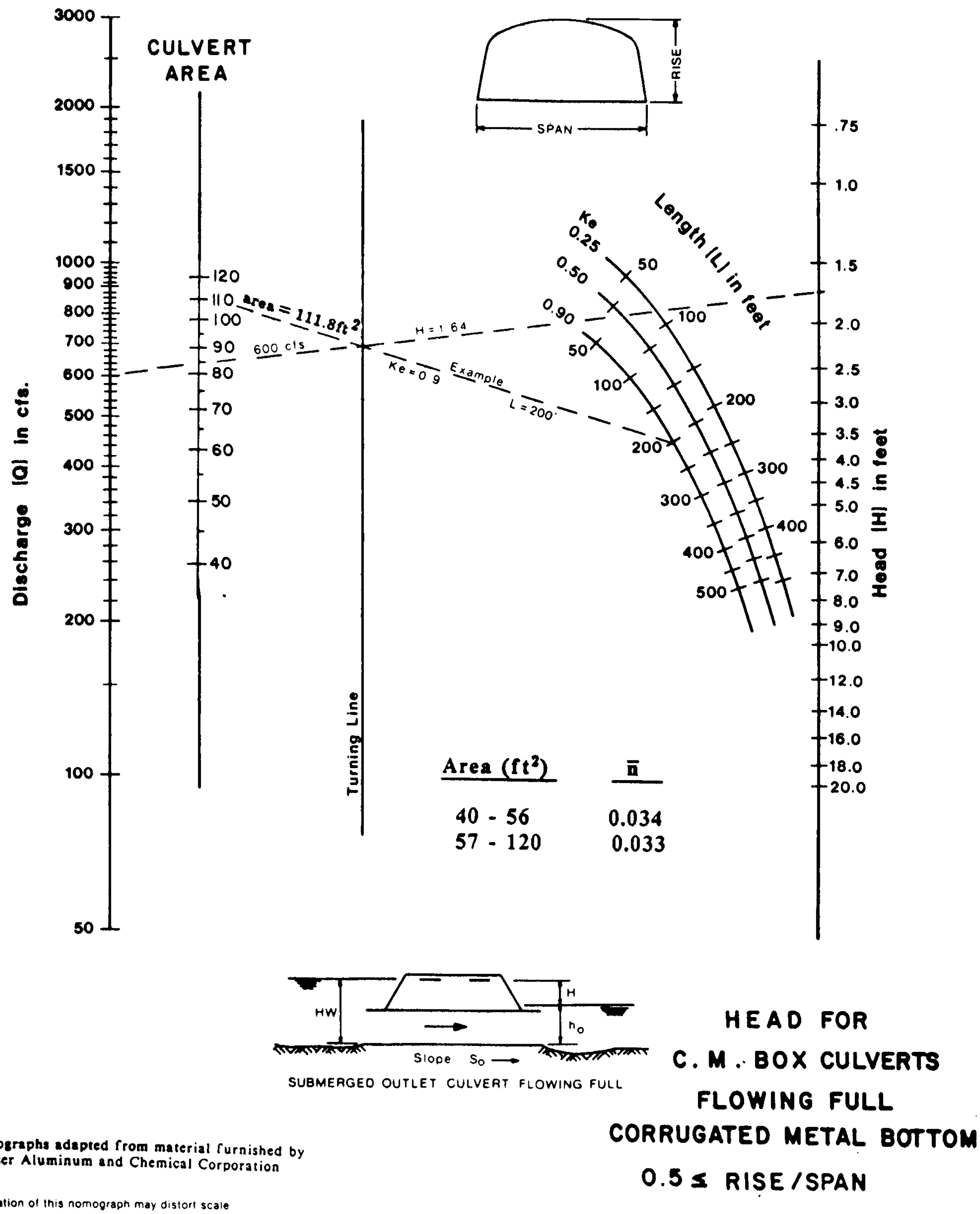


Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation

Duplication of this nomograph may distort scale

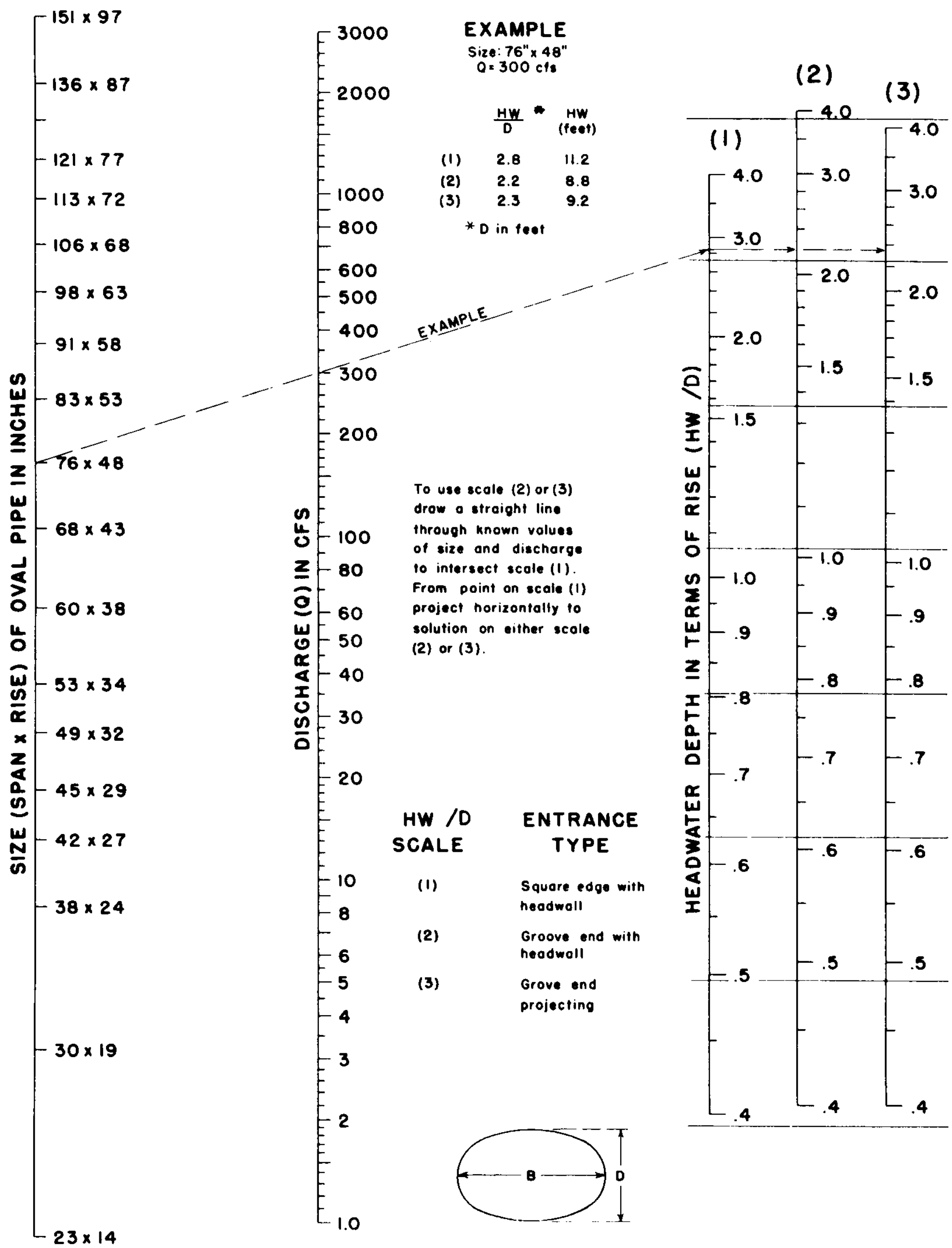
Source: HDS-5

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

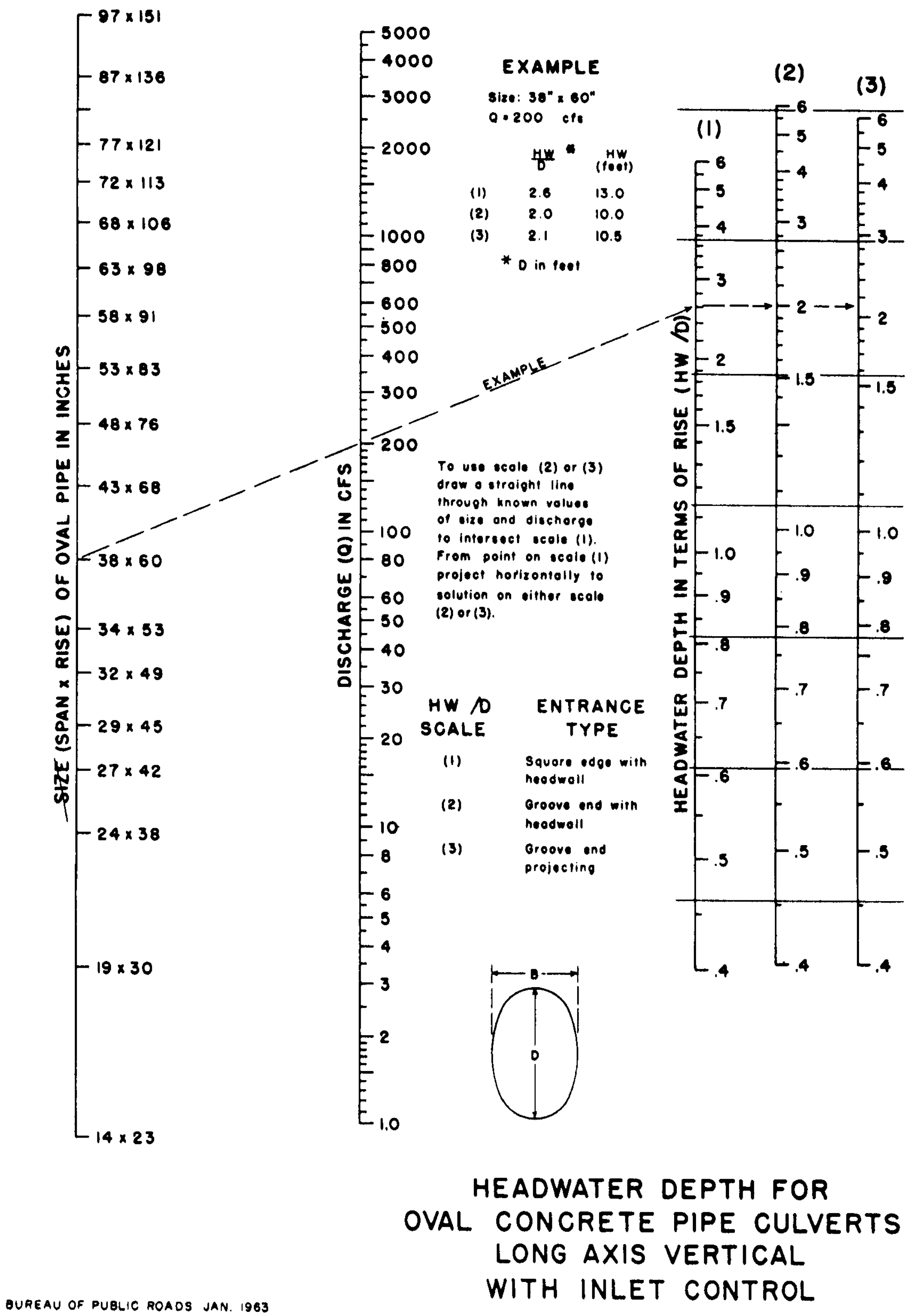


HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

Source: HDS-5

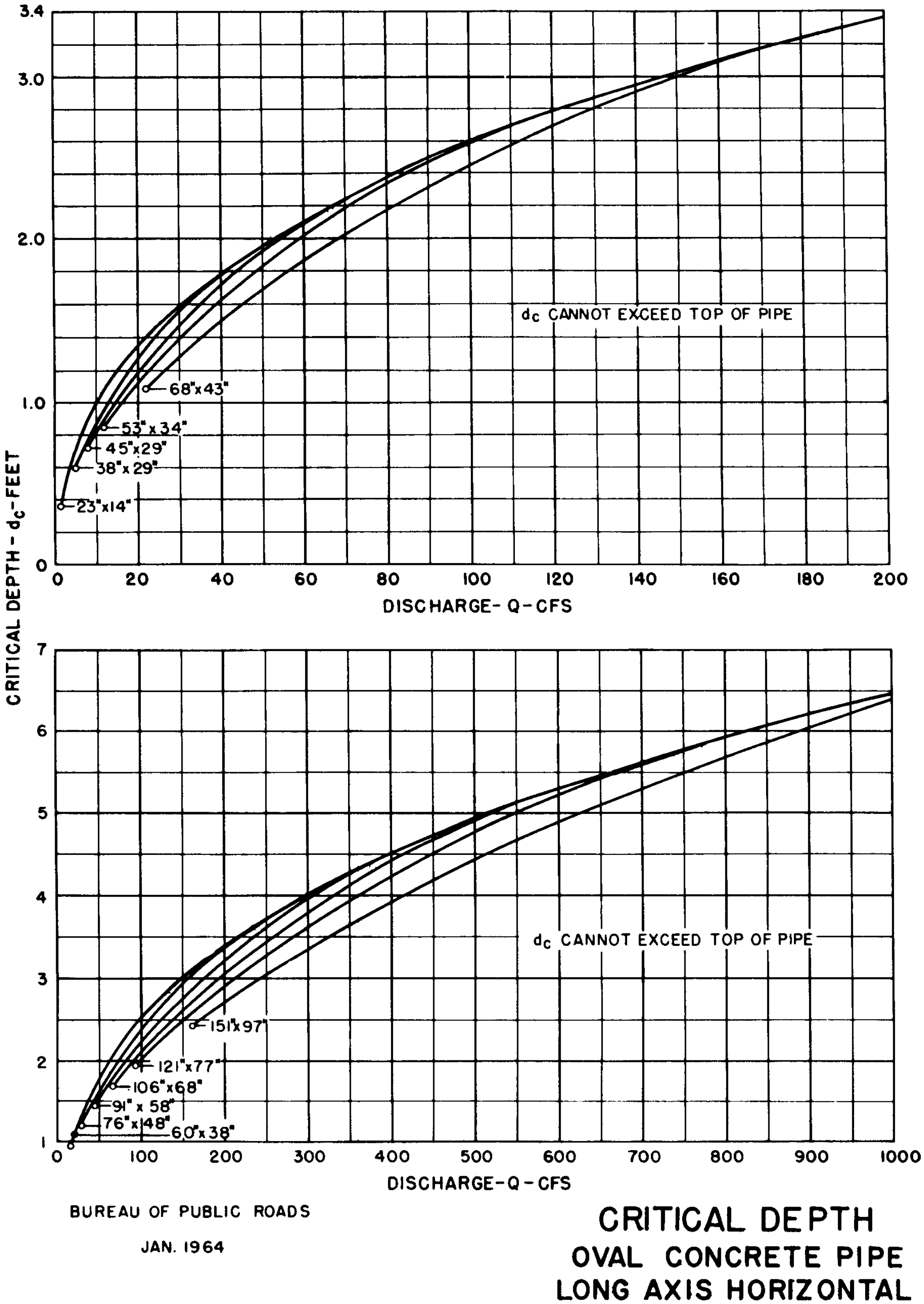
Appendix 8C-30 Inlet Control, Oval Concrete, Long Axis Vertical



HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS VERTICAL
WITH INLET CONTROL

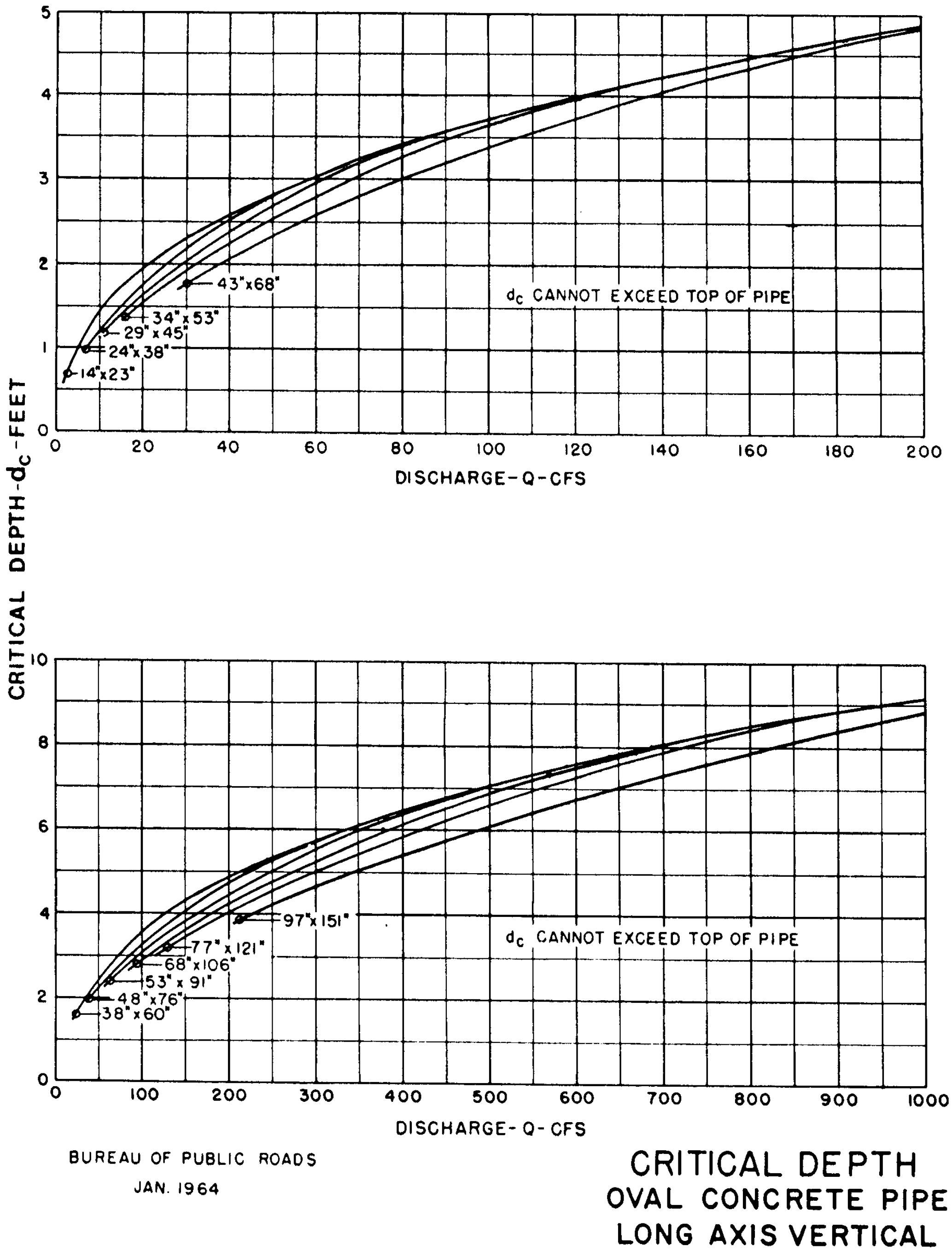
Source: HDS-5

Appendix 8C-31 **Critical Depth, Oval Concrete, Long Axis Horizontal**



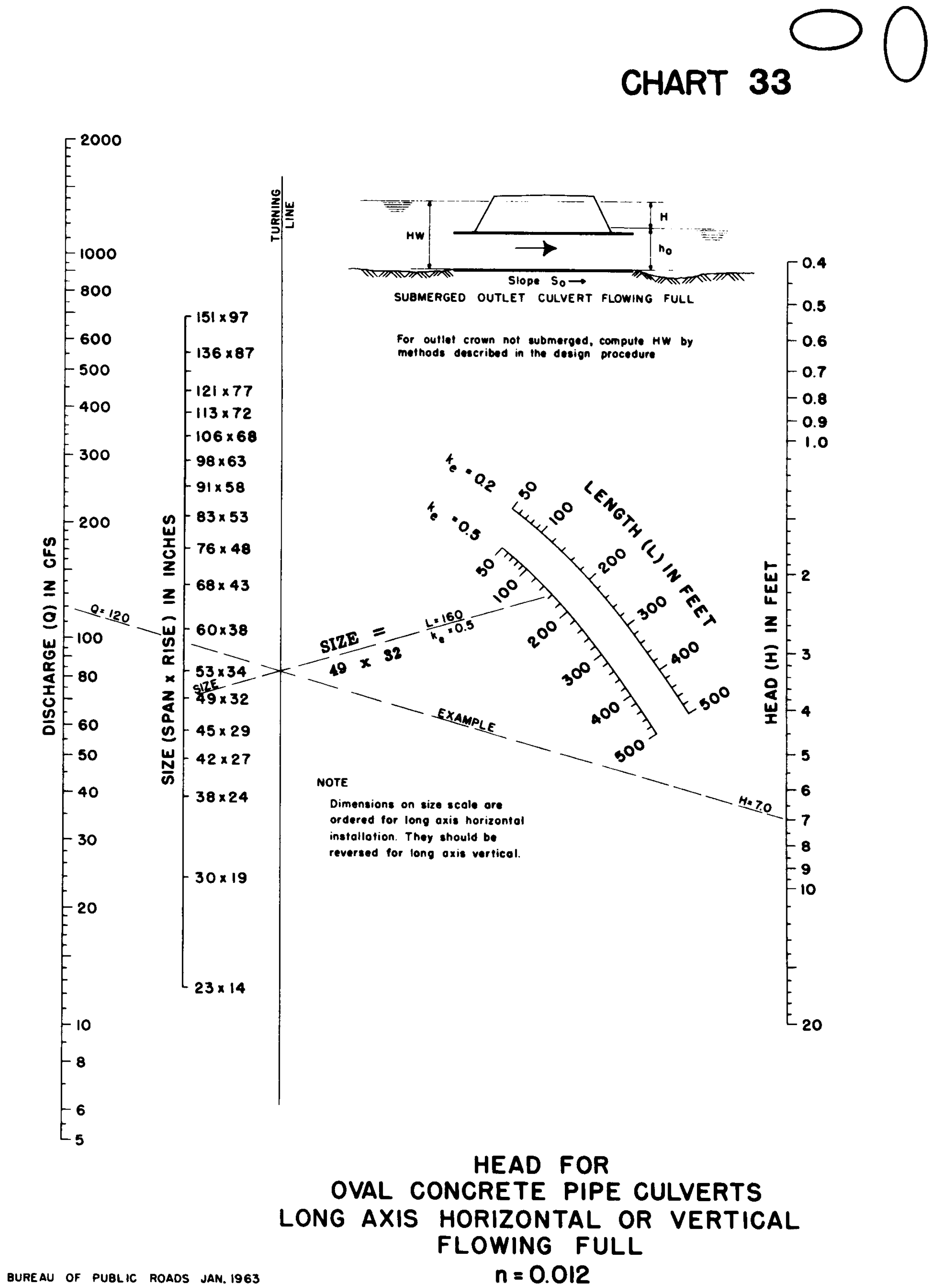
Source: HDS-5

Appendix 8C-32 Critical Depth, Oval Concrete, Long Axis Vertical



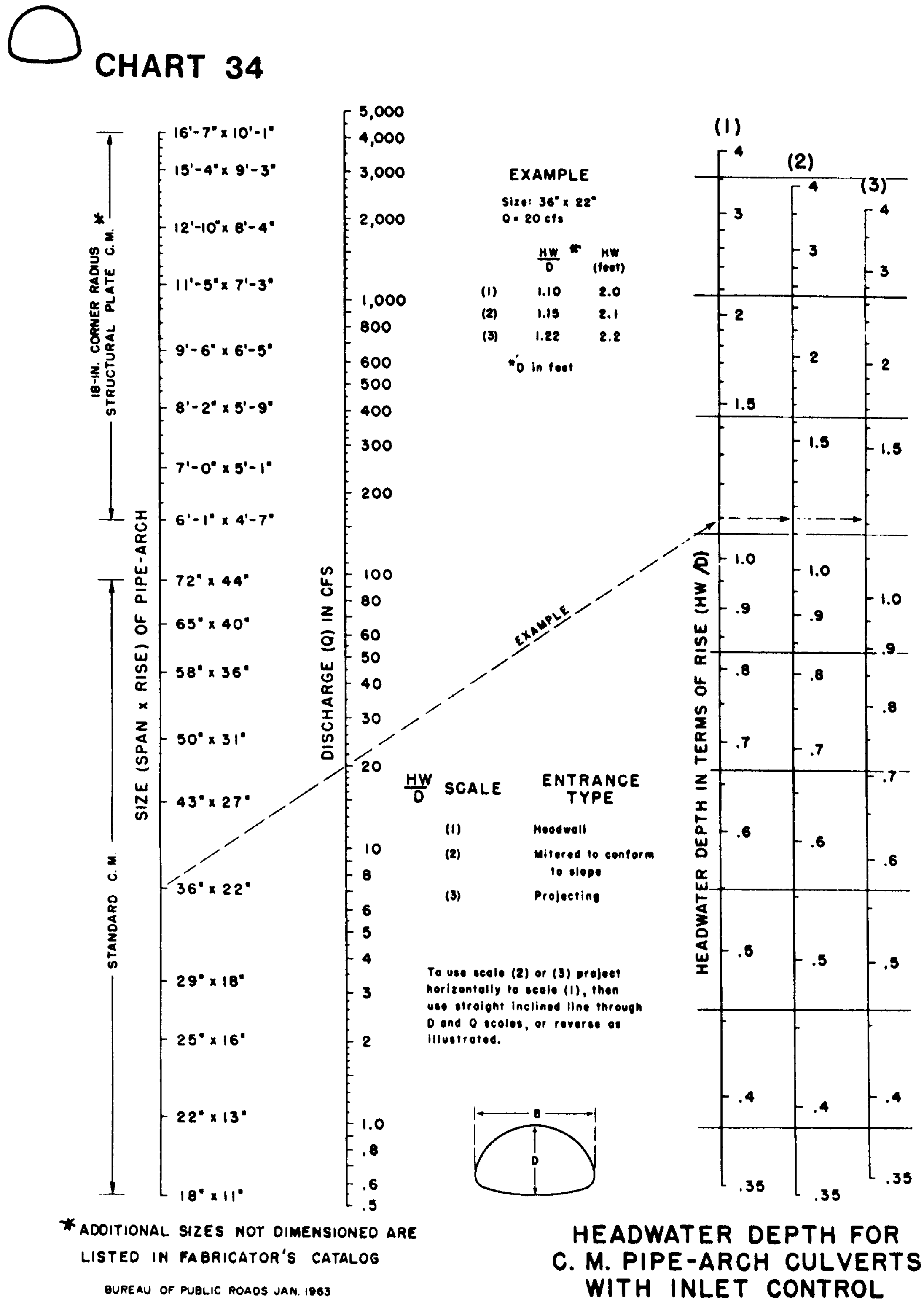
Source: HDS-5

Appendix 8C-33 Outlet Control, Oval Concrete, Long Axis Horizontal or Vertical



Source: HDS-5

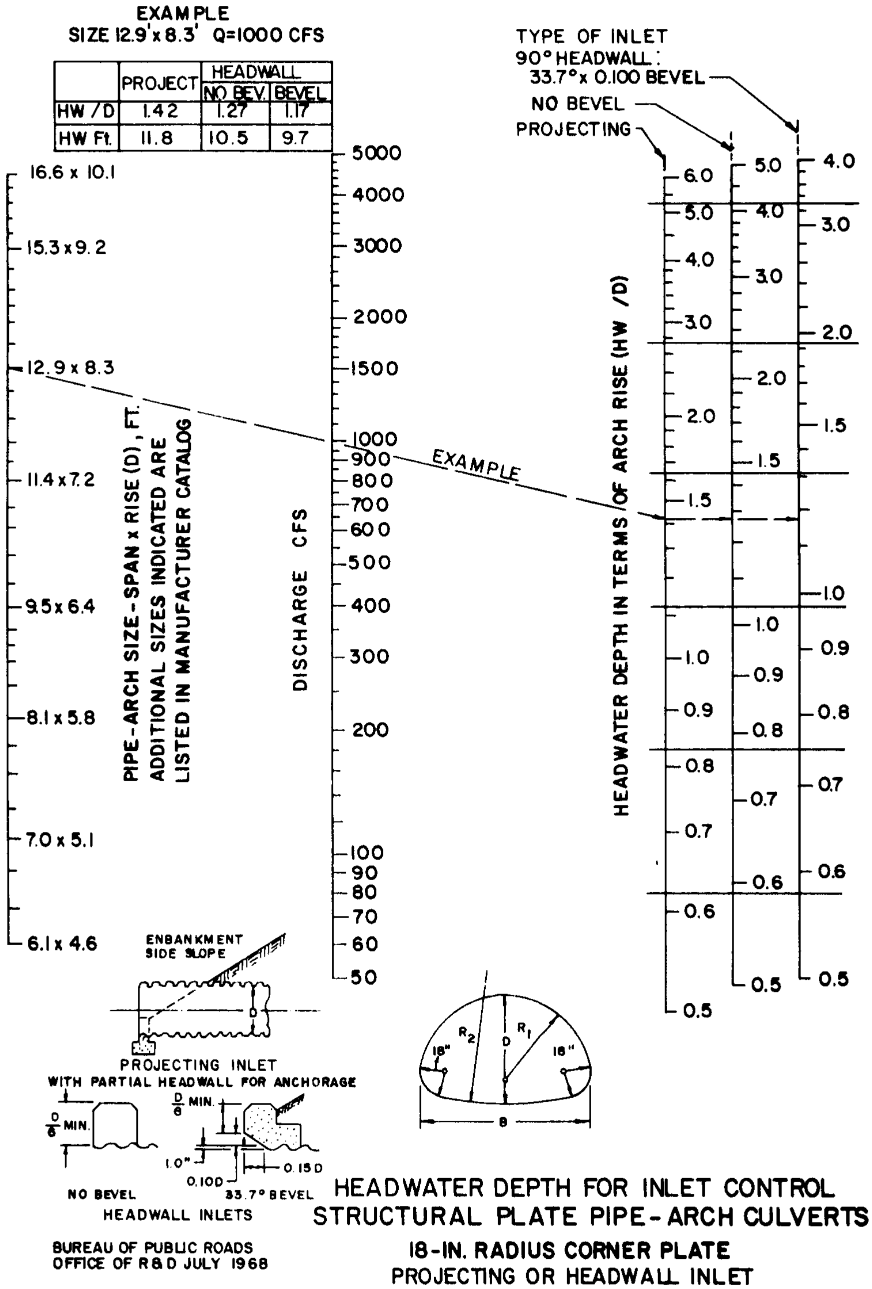
Appendix 8C-34 Inlet Control, Corrugated Metal Pipe-Arch



Source: HDS-5

Appendix 8C-35 **Inlet Control, Structural Plate Pipe-Arch, 18" Corner Radius**

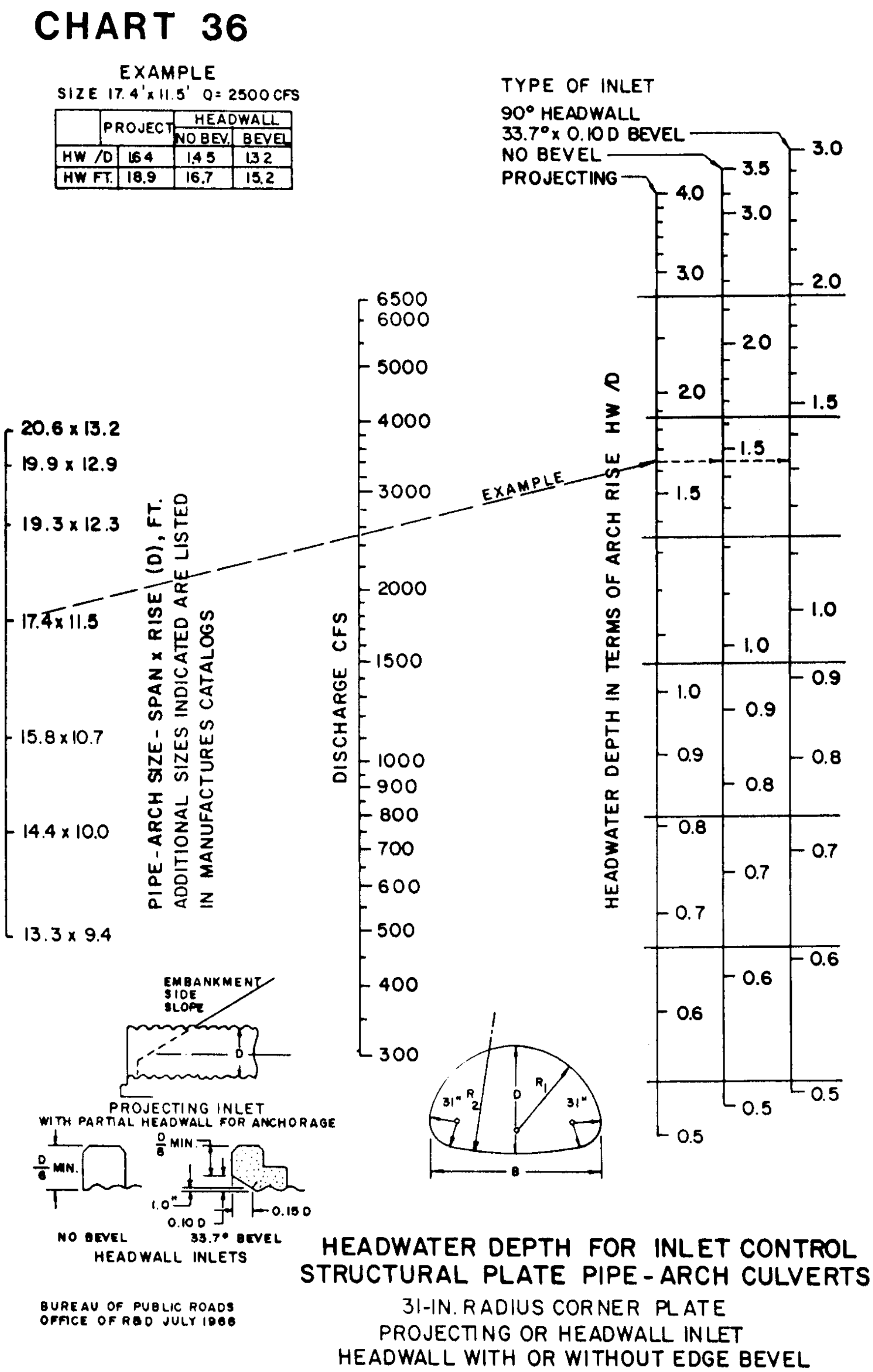
CHART 35



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

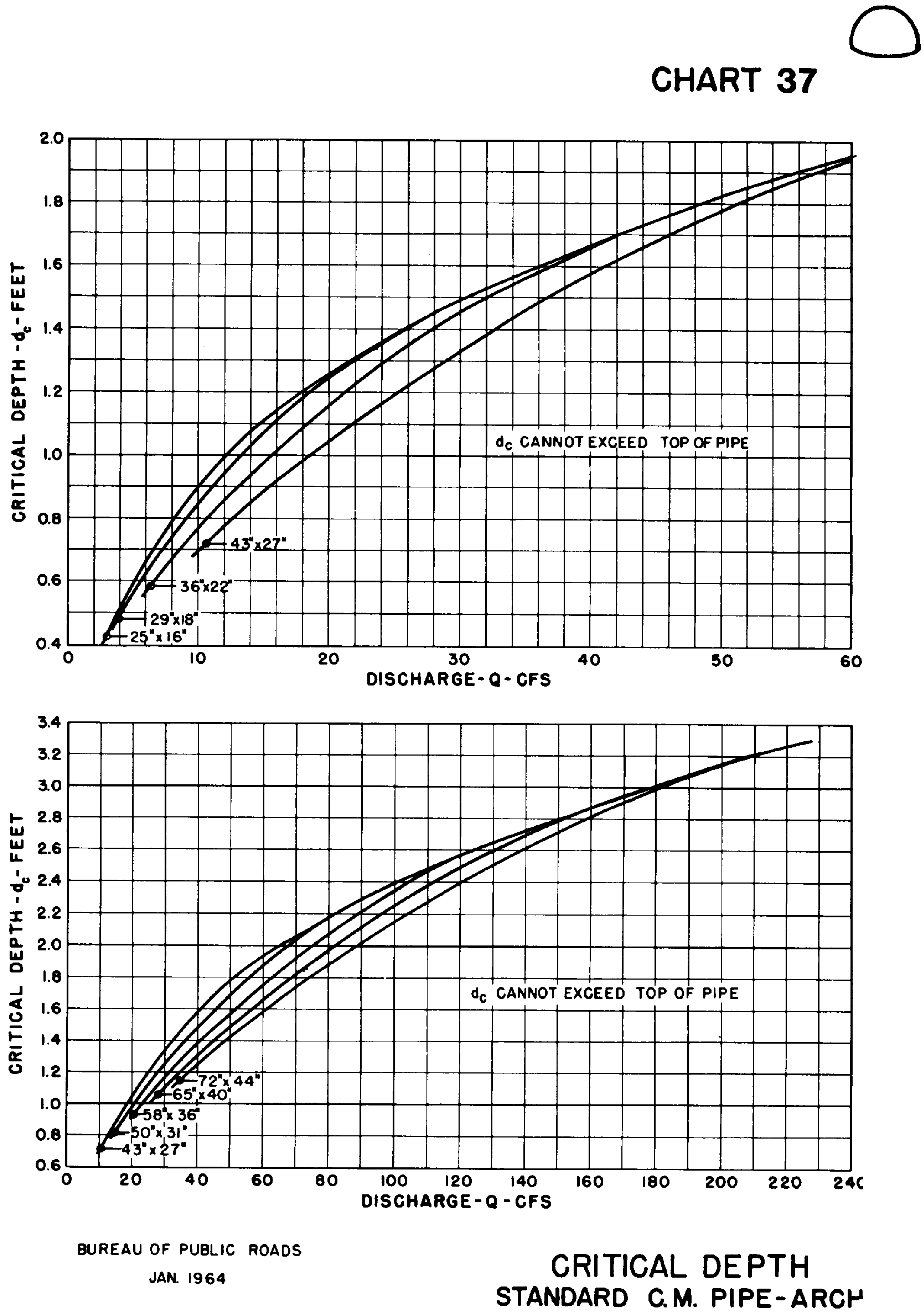
Source: HDS-5

Appendix 8C-36 Inlet Control, Structural Plate Pipe-Arch, 31" Corner Radius



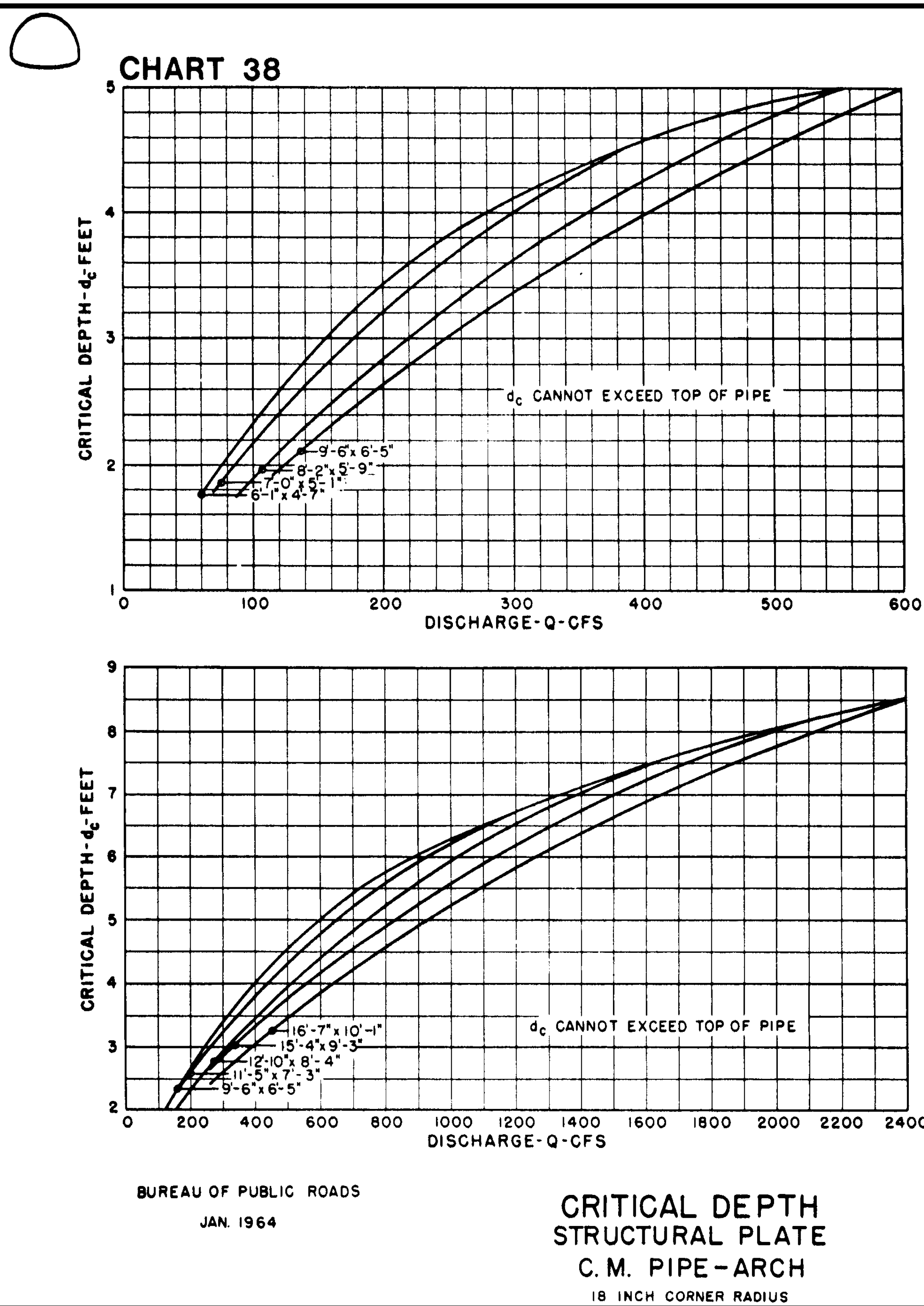
Source: HDS-5

Appendix 8C-37 **Critical Depth, Standard Corrugated Metal Pipe-Arch**



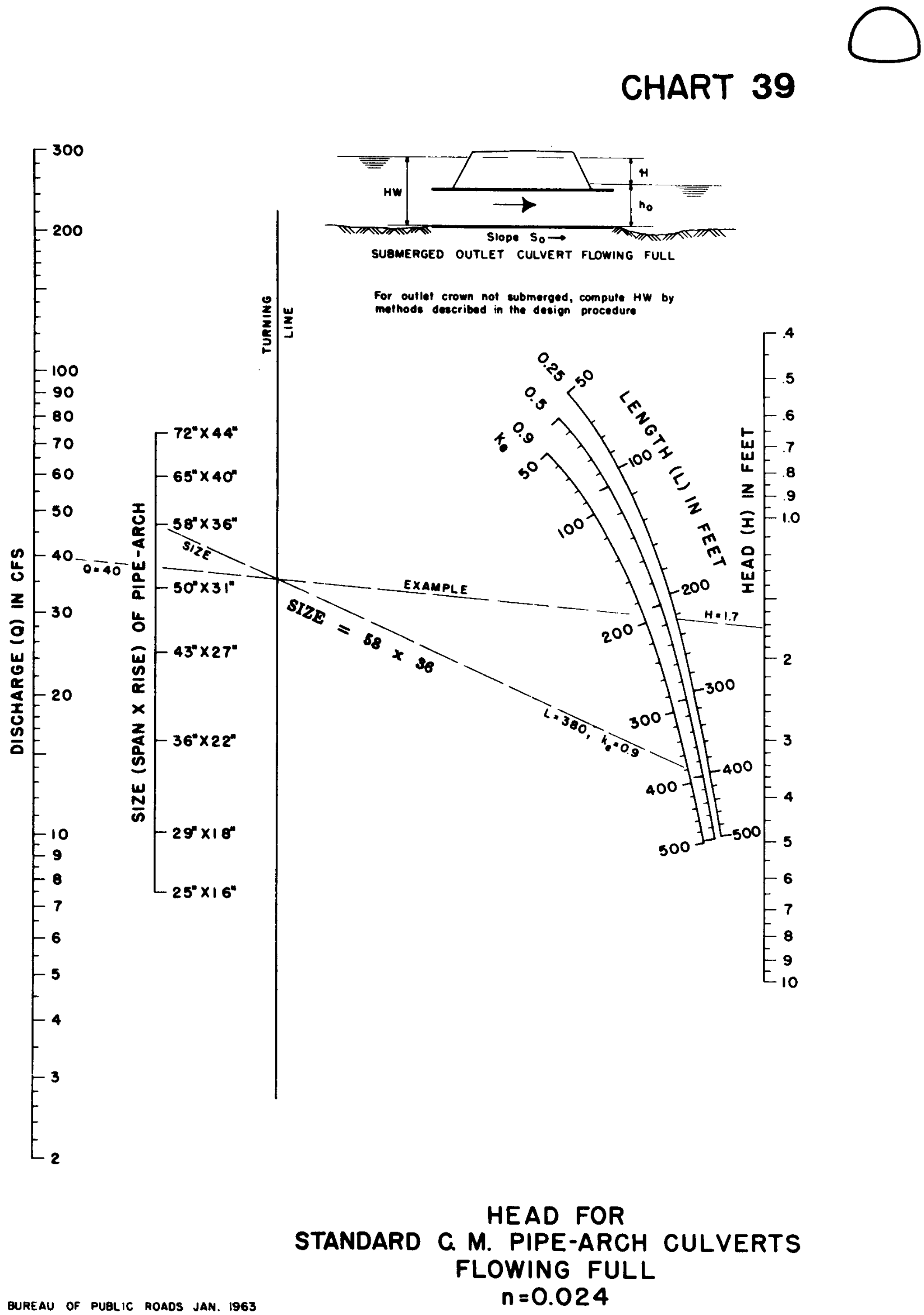
Source: HDS-5

Appendix 8C-38 **Critical Depth, Structural Plate Corrugated Metal Pipe-Arch, 18” Corner Radius**



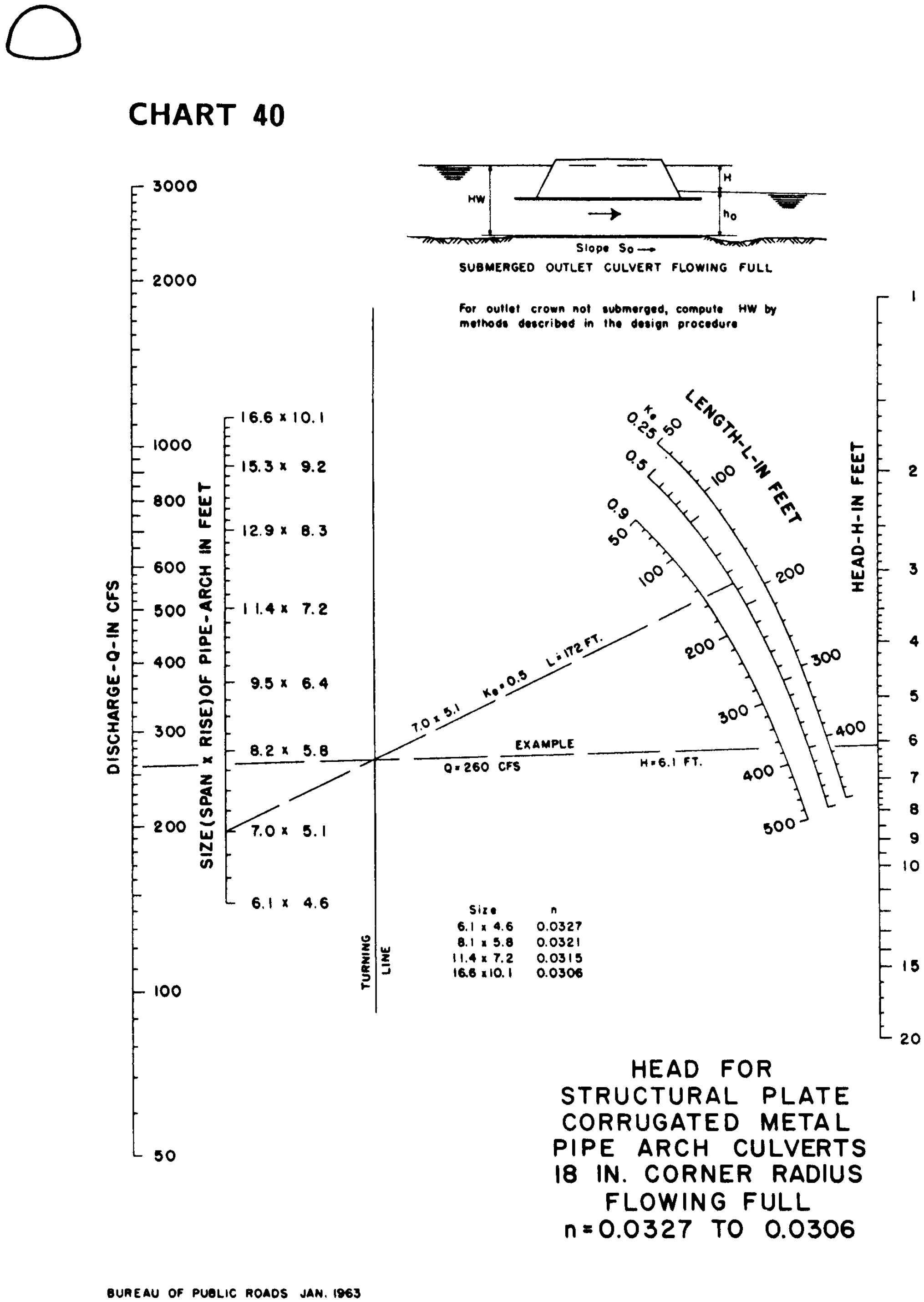
Source: HDS-5

Appendix 8C-39 Outlet Control, Standard Corrugated Metal Pipe-Arch



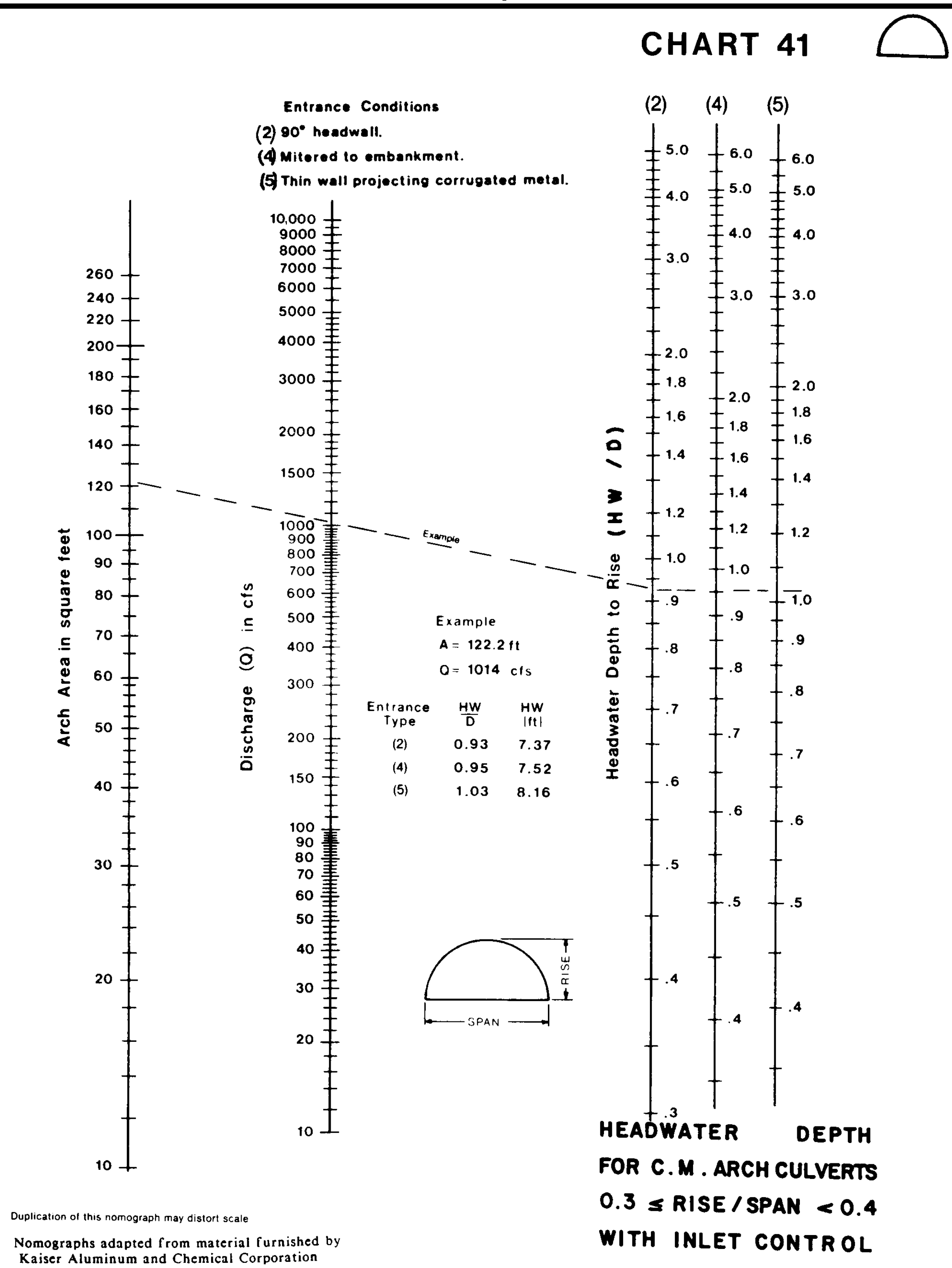
Source: HDS-5

Appendix 8C-40 Outlet Control, Structural Plate Corrugated Metal Pipe-Arch, 18" Corner Radius



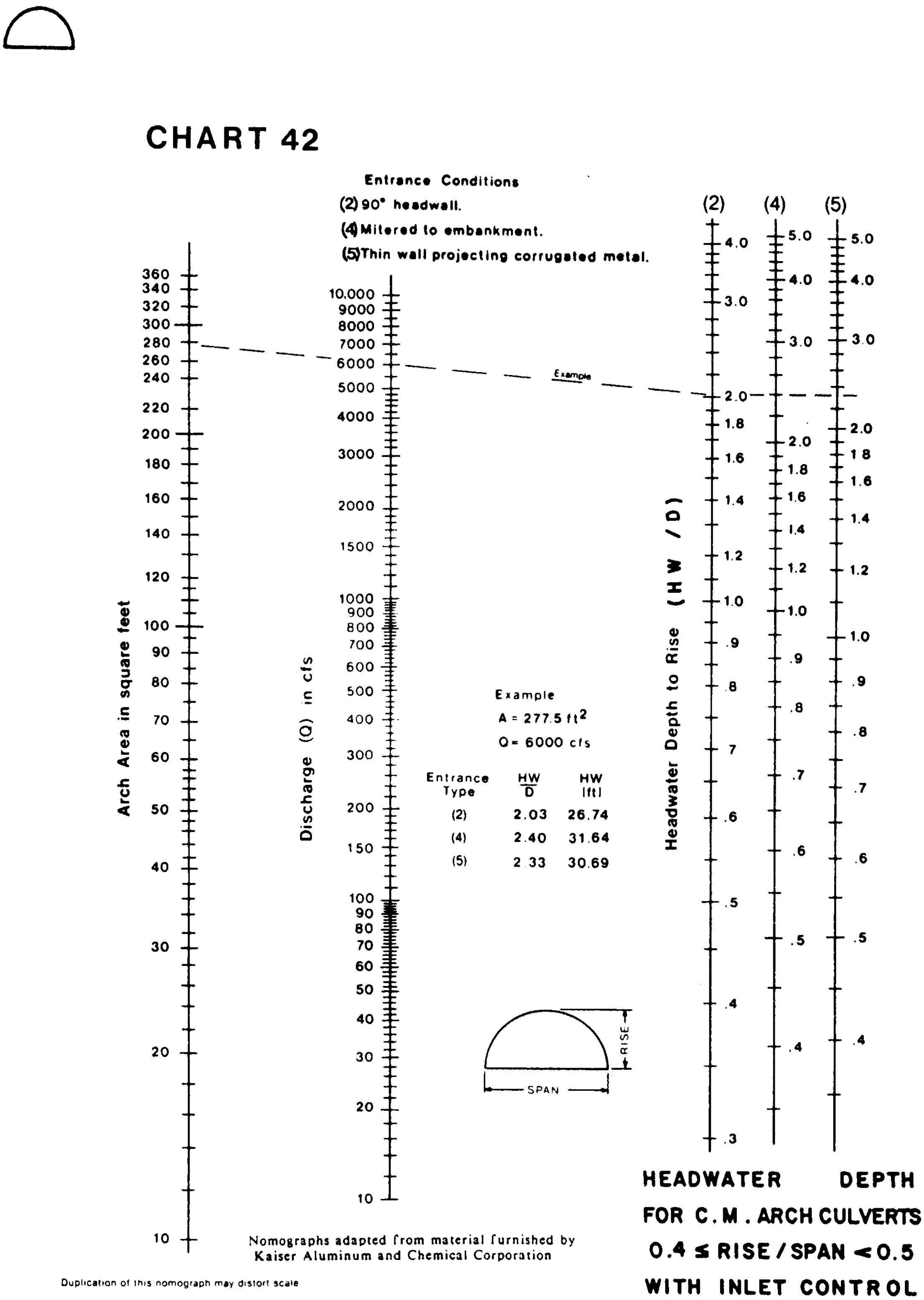
Source: HDS-5

Appendix 8C-41 Inlet Control, Corrugated Metal Arch, 0.3≤ Rise/Span <0.4



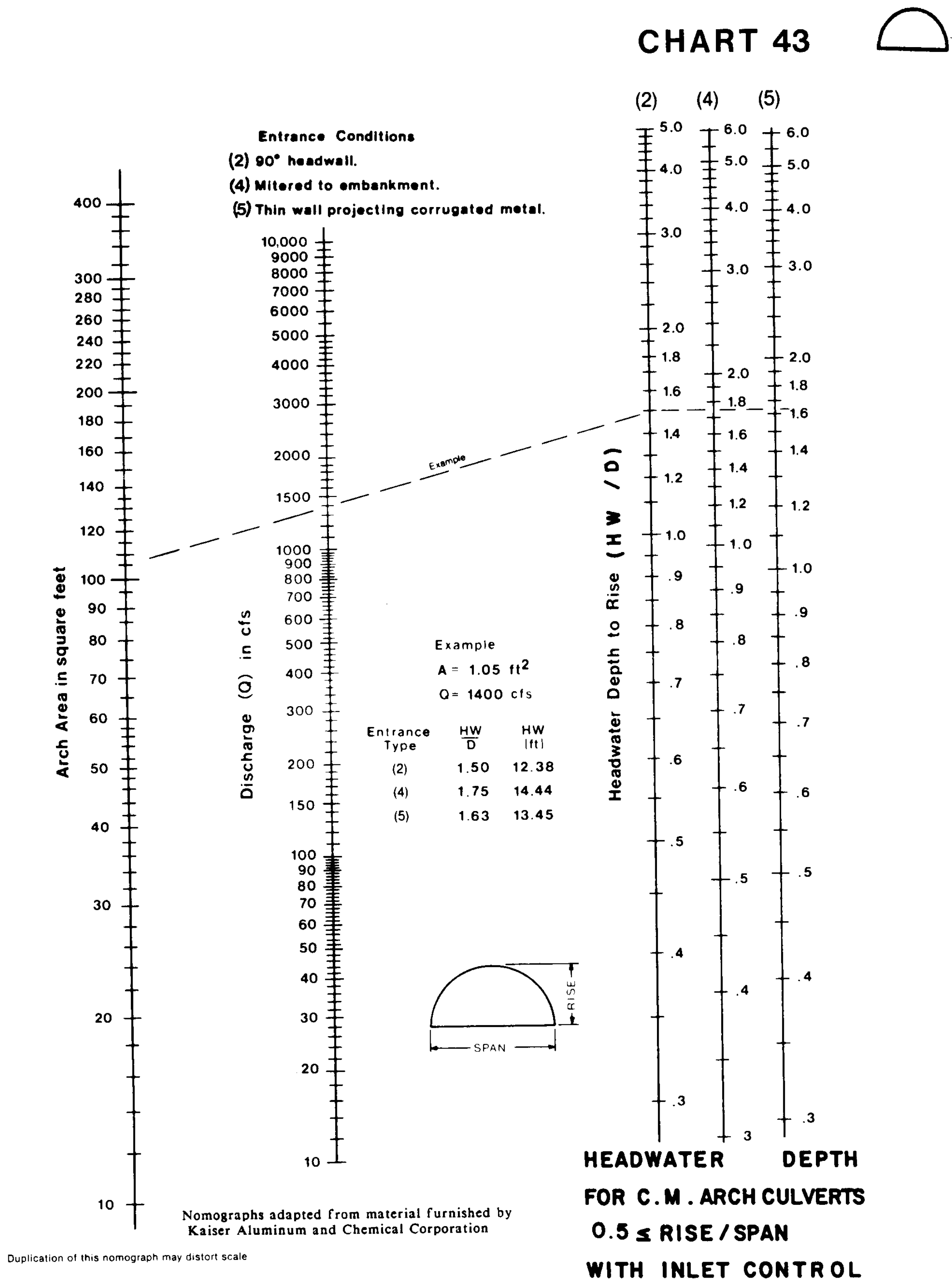
Source: HDS-5

Appendix 8C-42 Inlet Control, Corrugated Metal Arch, 0.4≤ Rise/Span <0.5



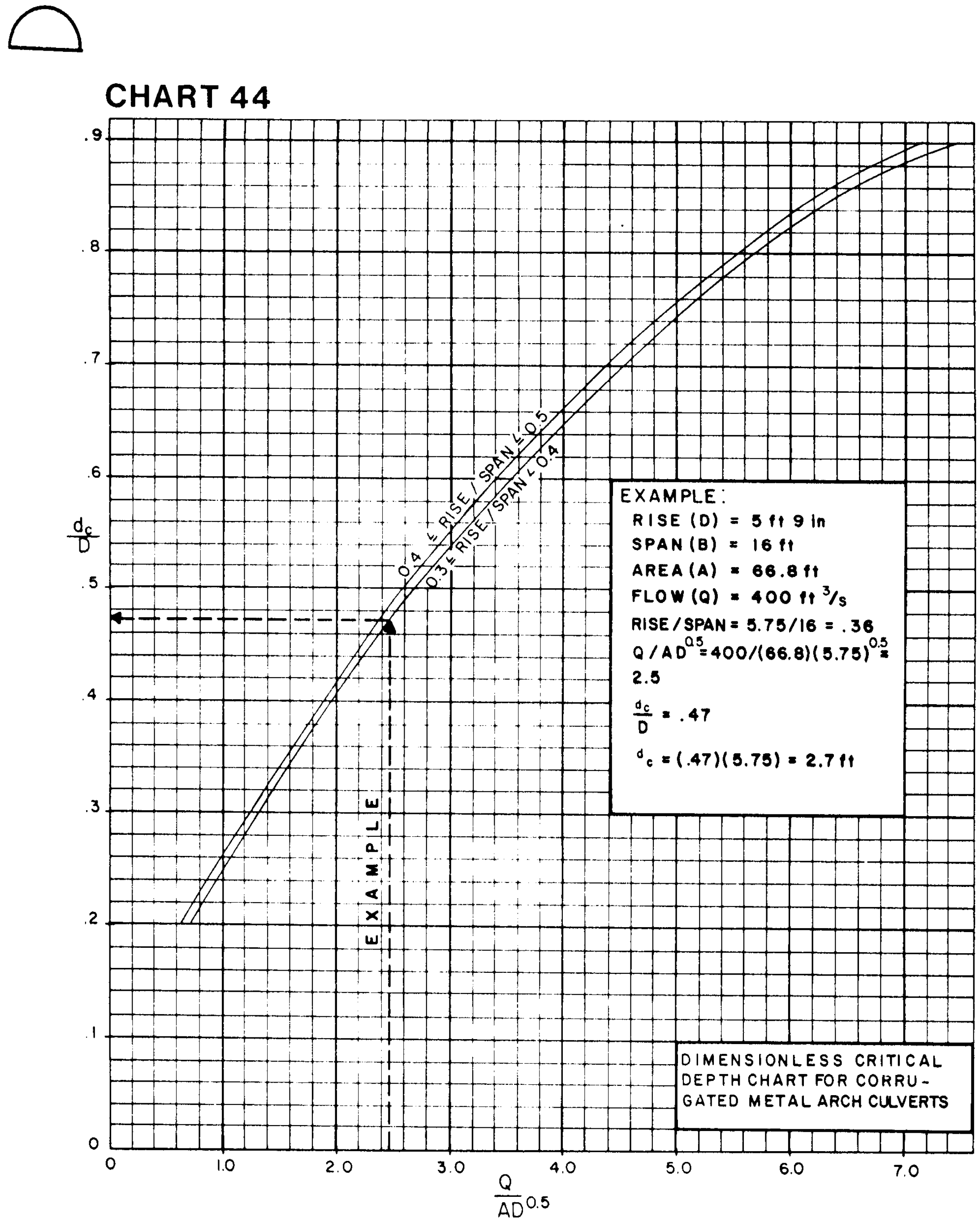
Source: HDS-5

Appendix 8C-43 Inlet Control, Corrugated Metal Arch, 0.5≤ Rise/Span



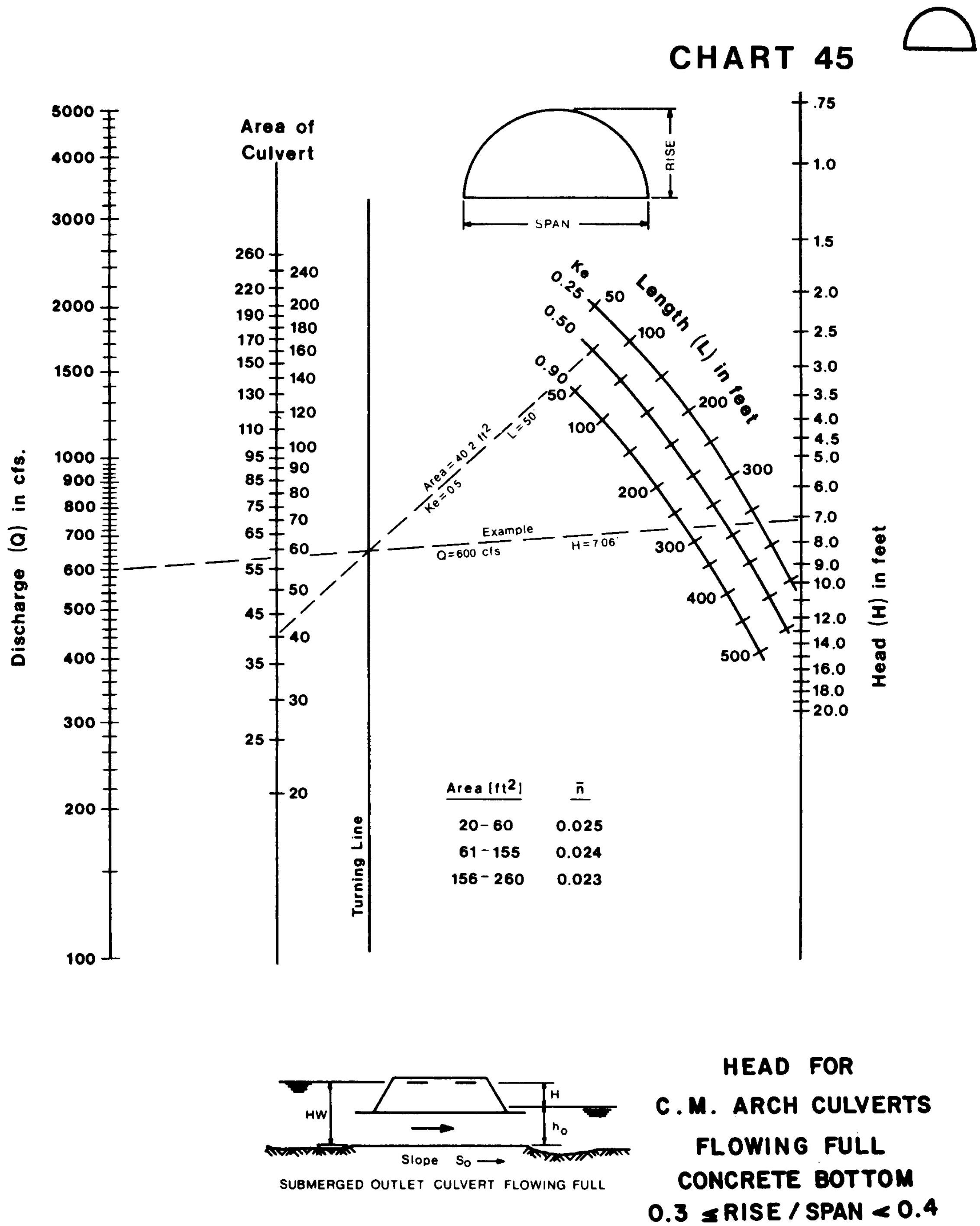
Source: HDS-5

Appendix 8C-44 Critical Depth, Corrugated Metal Arch



Source: HDS-5

Appendix 8C-45 Outlet Control, Corrugated Metal Arch, Concrete Bottom, 0.3≤ Rise/Span <0.4

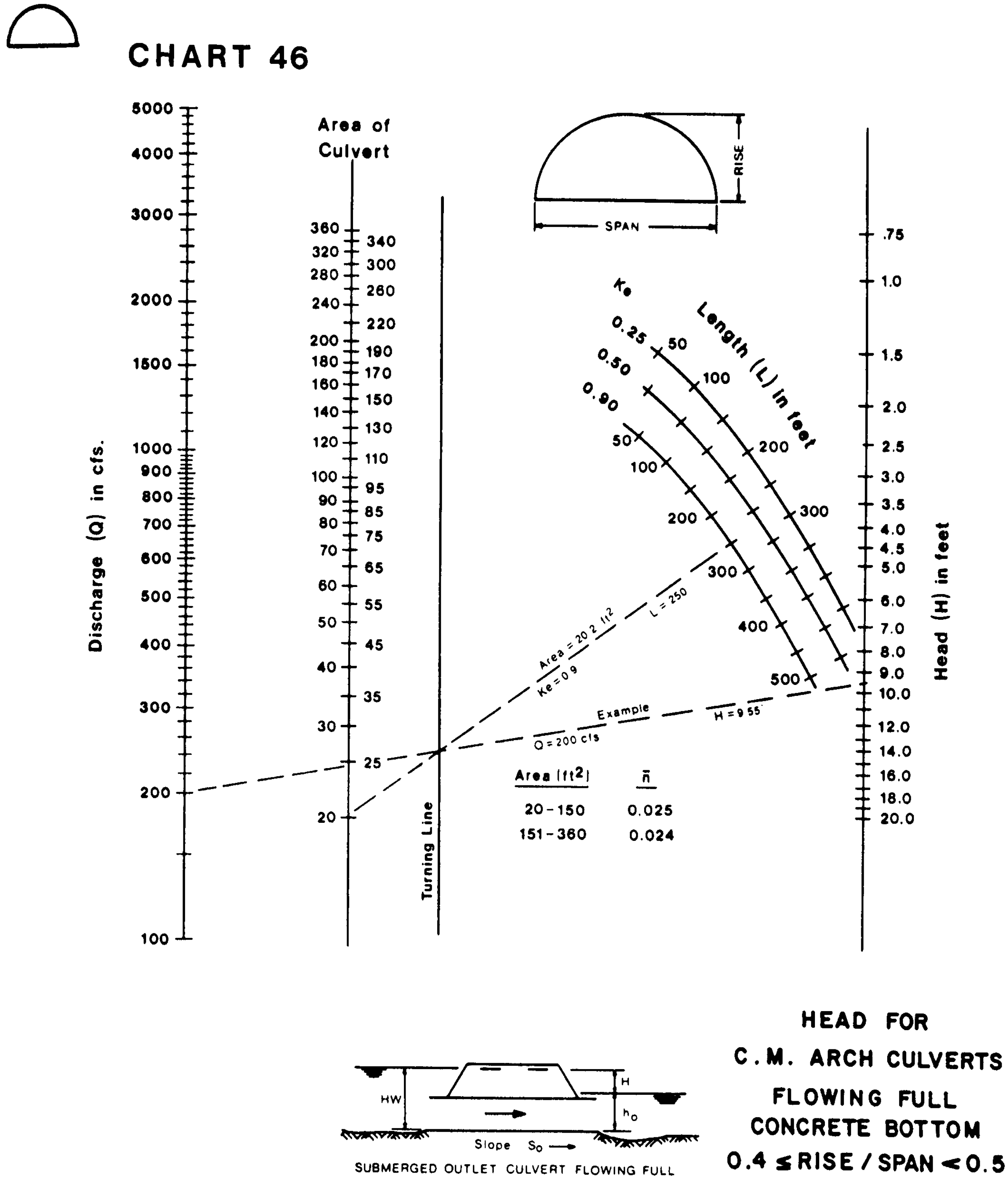


Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation

Duplication of this nomograph may distort scale

Source: HDS-5

Appendix 8C-46 Outlet Control, Corrugated Metal Arch, Concrete Bottom, 0.4≤ Rise/Span <0.5

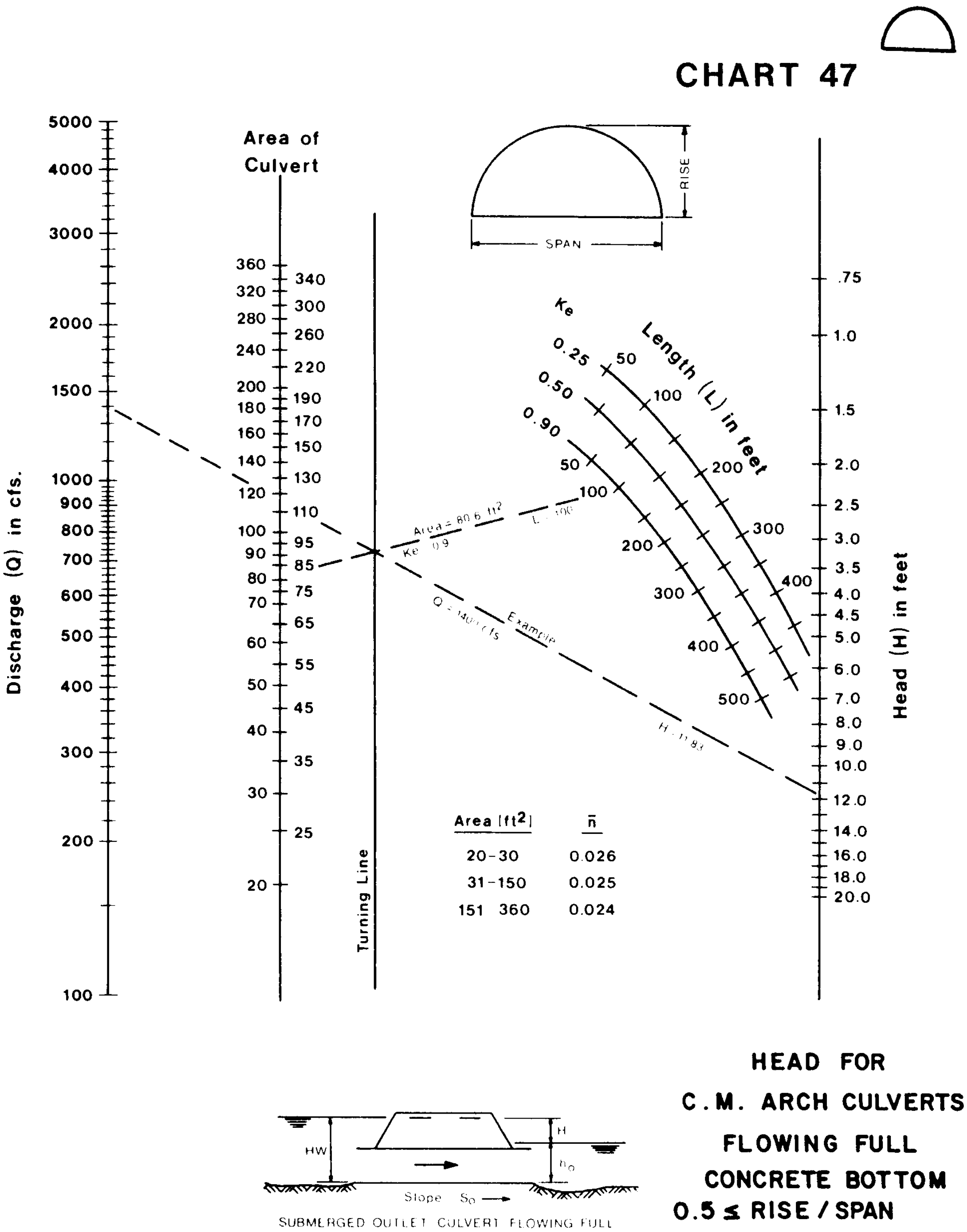


Nomographs adapted from material furnished by
Kaiser Aluminum and Chemical Corporation

Duplication of this nomograph may distort scale

Source: HDS-5

Appendix 8C-47 Outlet Control, Corrugated Metal Arch, Concrete Bottom, 0.5≤ Rise/Span

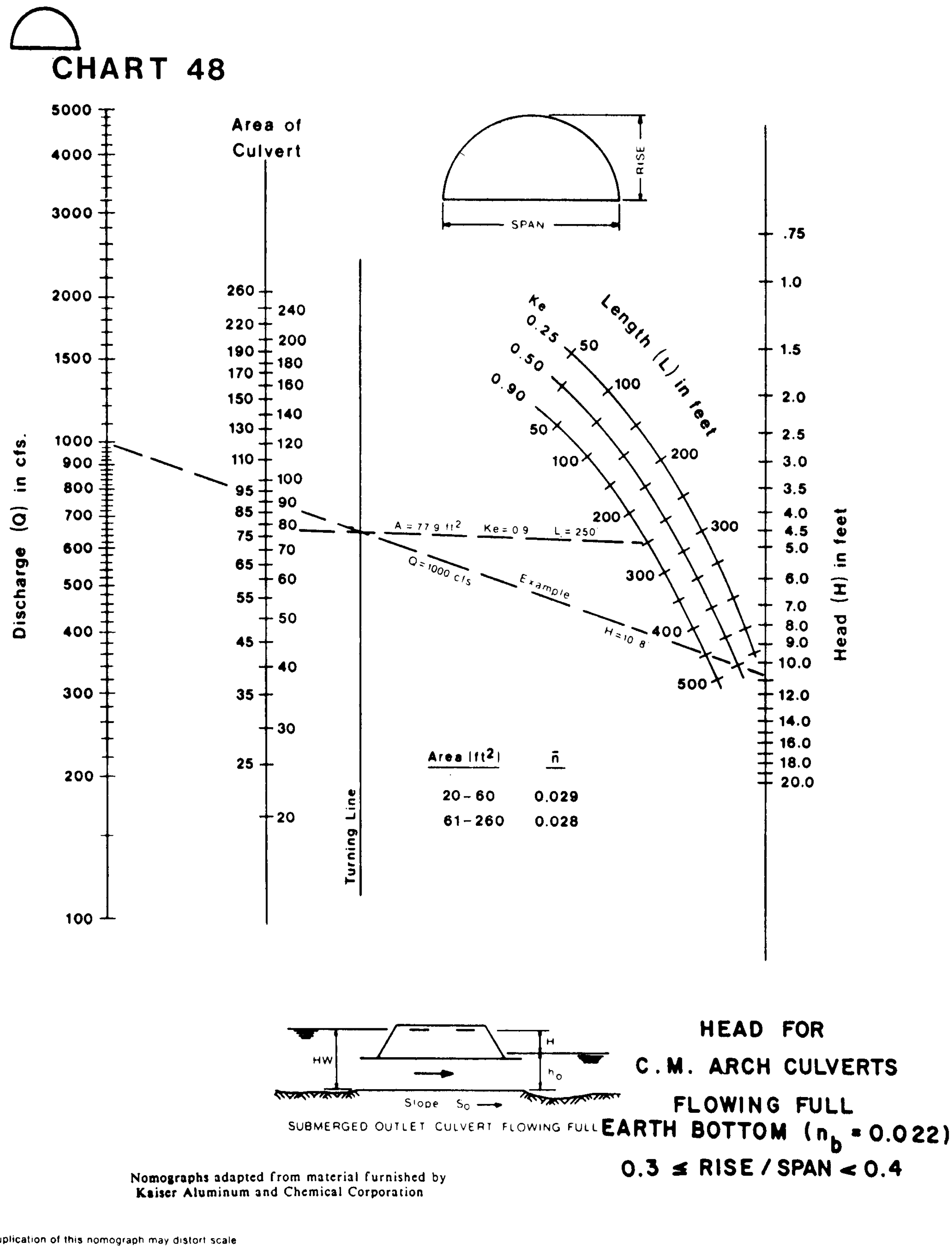


Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation

Duplication of this nomograph may distort scale

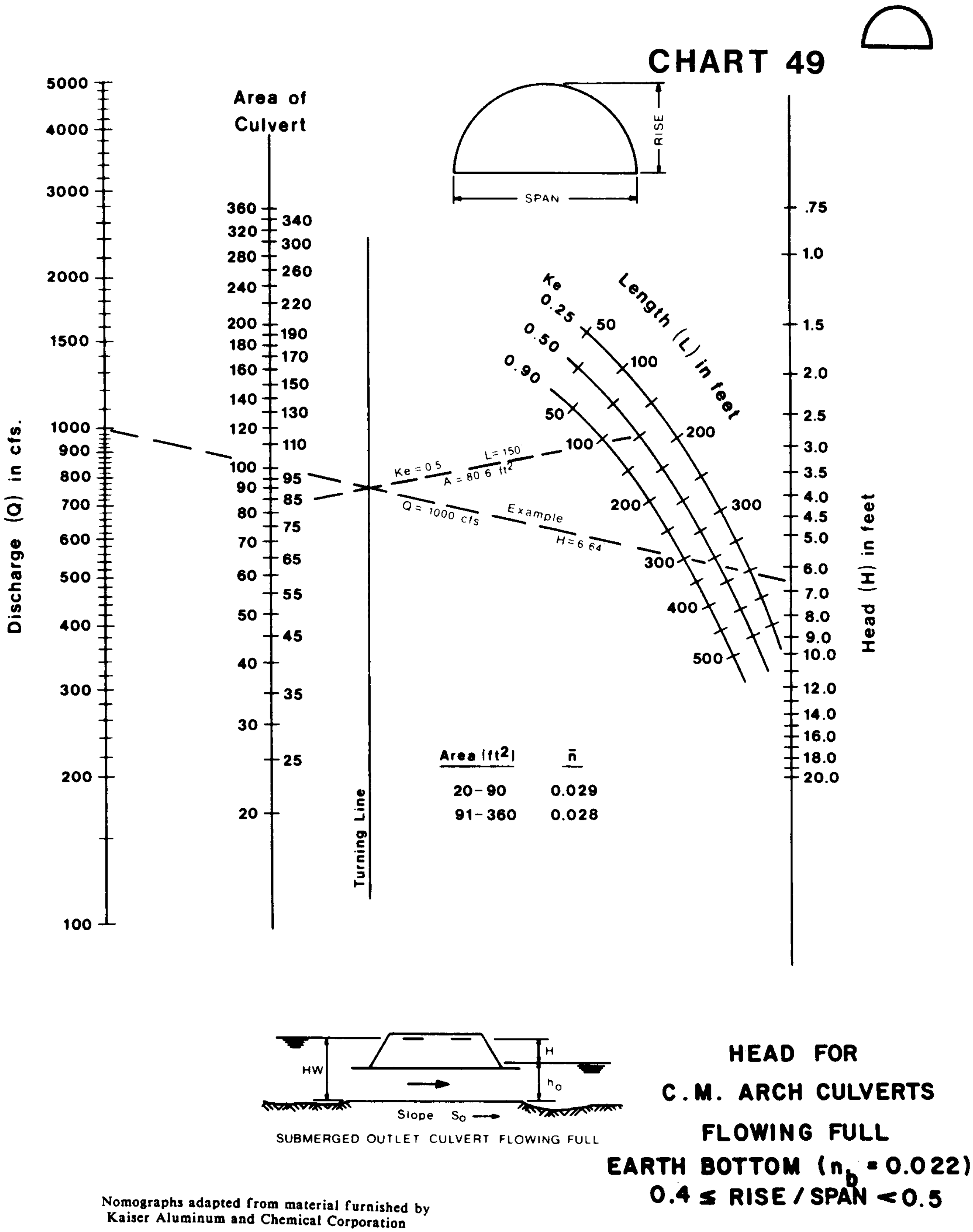
Source: HDS-5

Appendix 8C-48 Outlet Control, Corrugated Metal Arch, Earth Bottom, 0.3≤ Rise/Span <0.4



Source: HDS-5

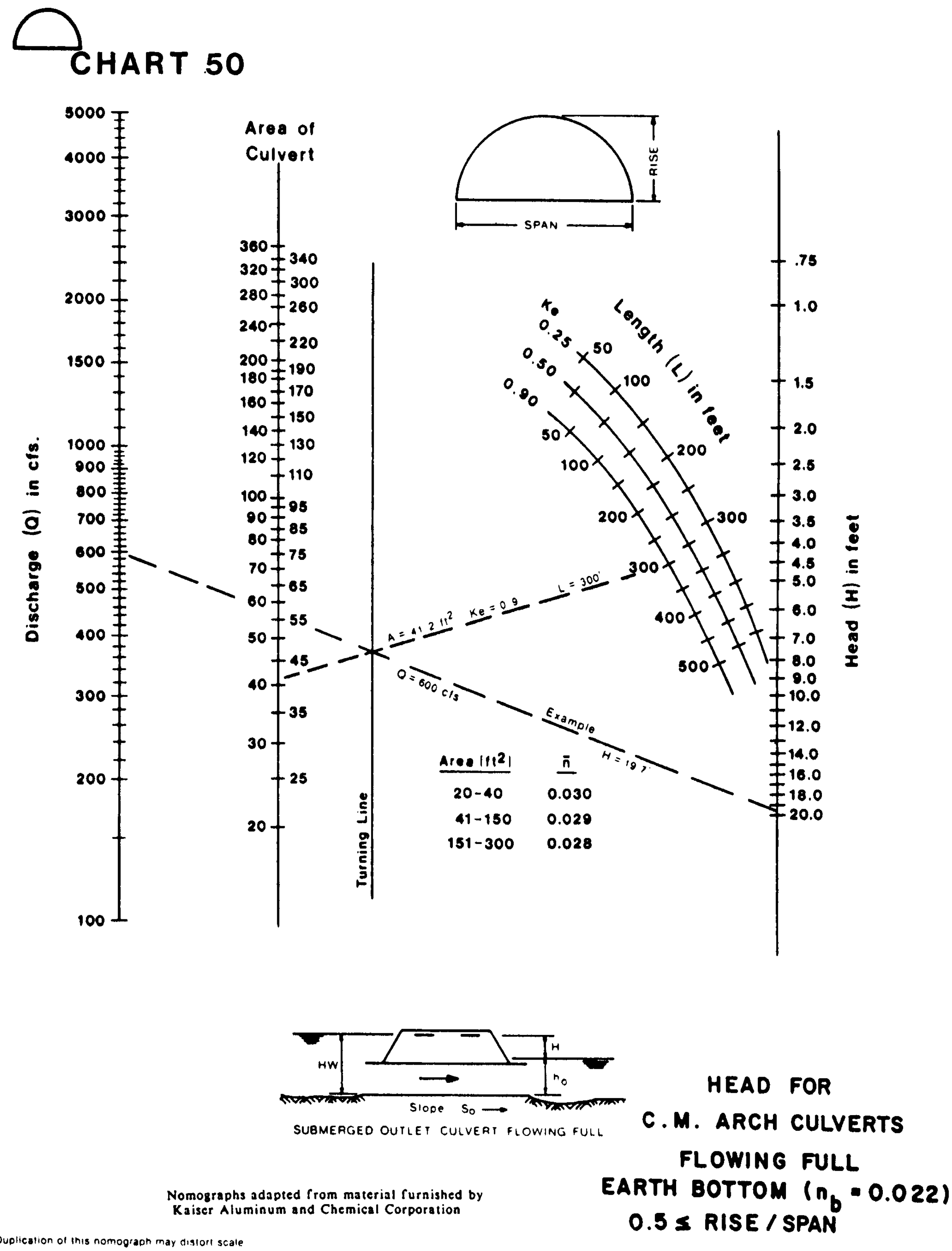
Appendix 8C-49 Outlet Control, Corrugated Metal Arch, Earth Bottom, 0.4≤ Rise/Span <0.5



Duplication of this nomograph may distort scale

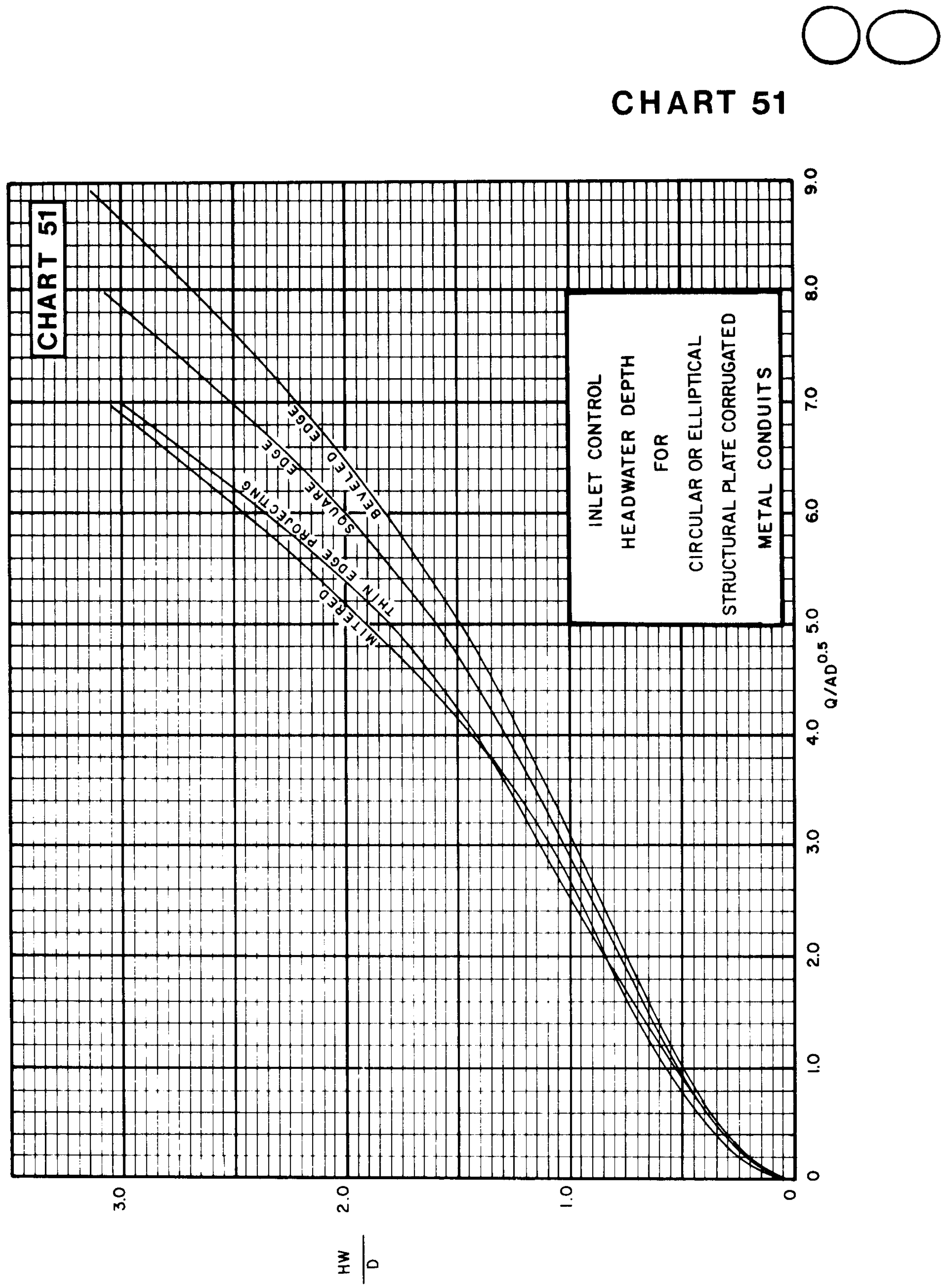
Source: HDS-5

Appendix 8C-50 Outlet Control, Corrugated Metal Arch, Earth Bottom, 0.5≤ Rise/Span



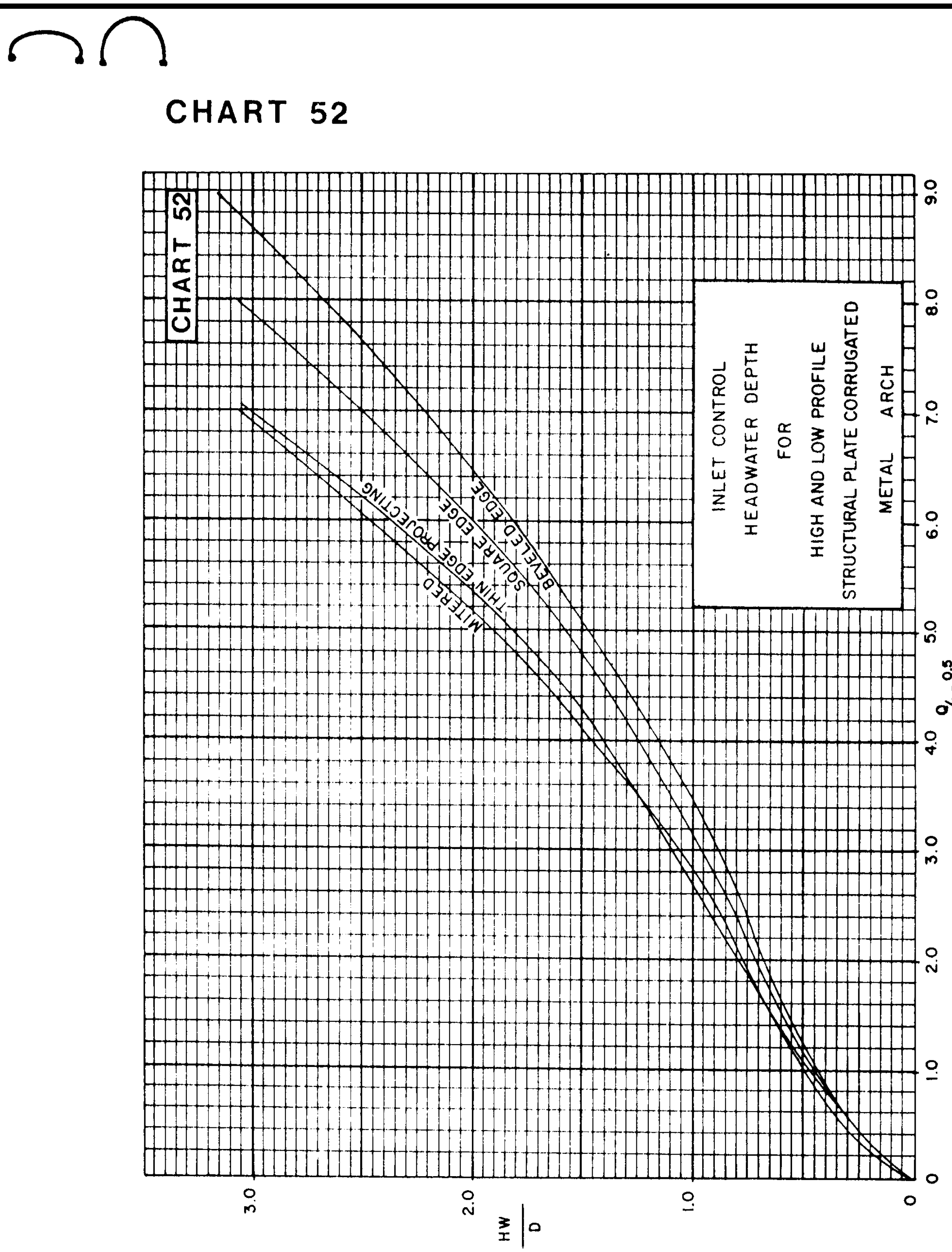
Source: HDS-5

Appendix 8C-51 **Inlet Control, Structural Plate Corrugated Metal, Circular or Elliptical**



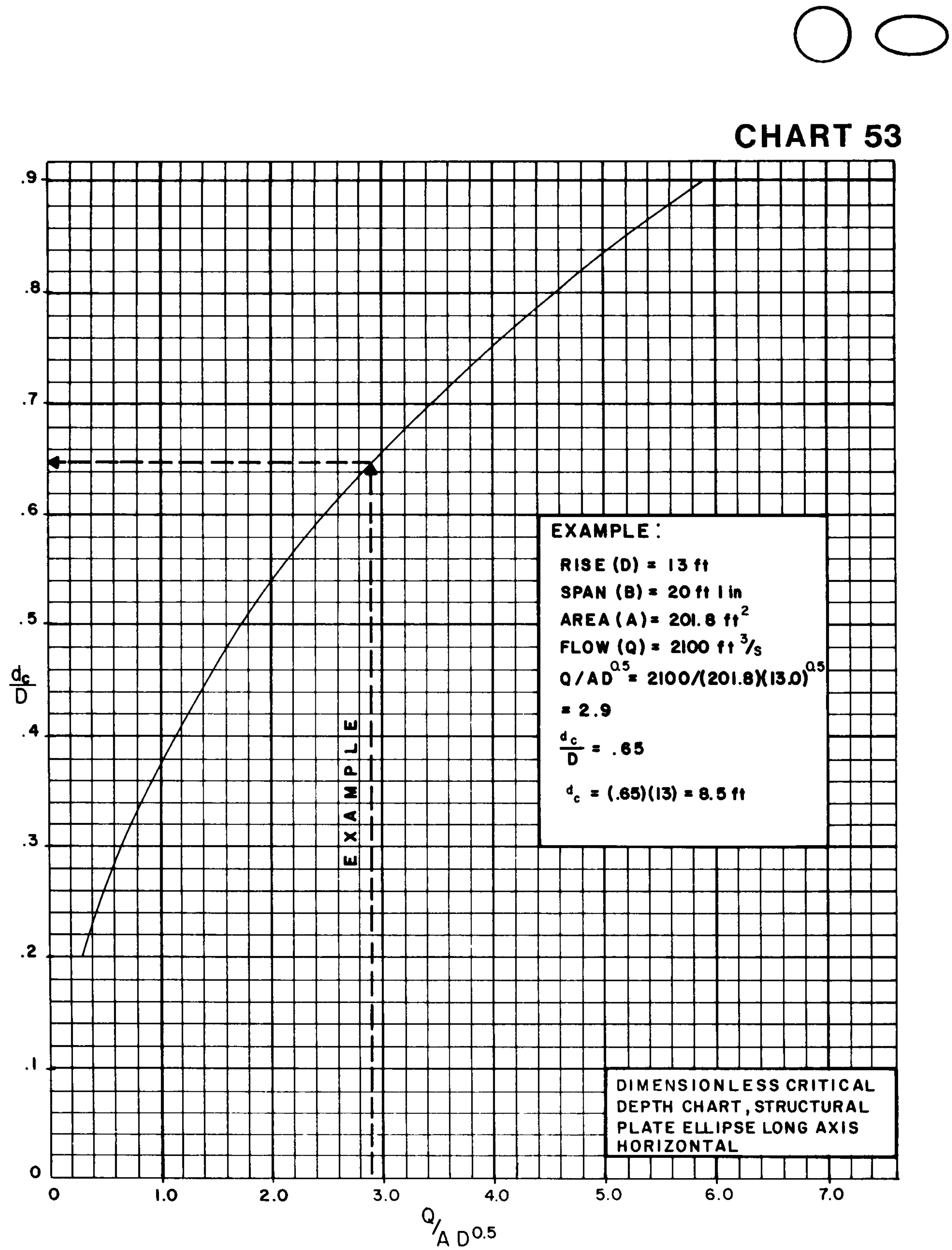
Source: HDS-5

Appendix 8C-52 **Inlet Control, Structural Plate Corrugated Metal Arch, High and Low Profile**



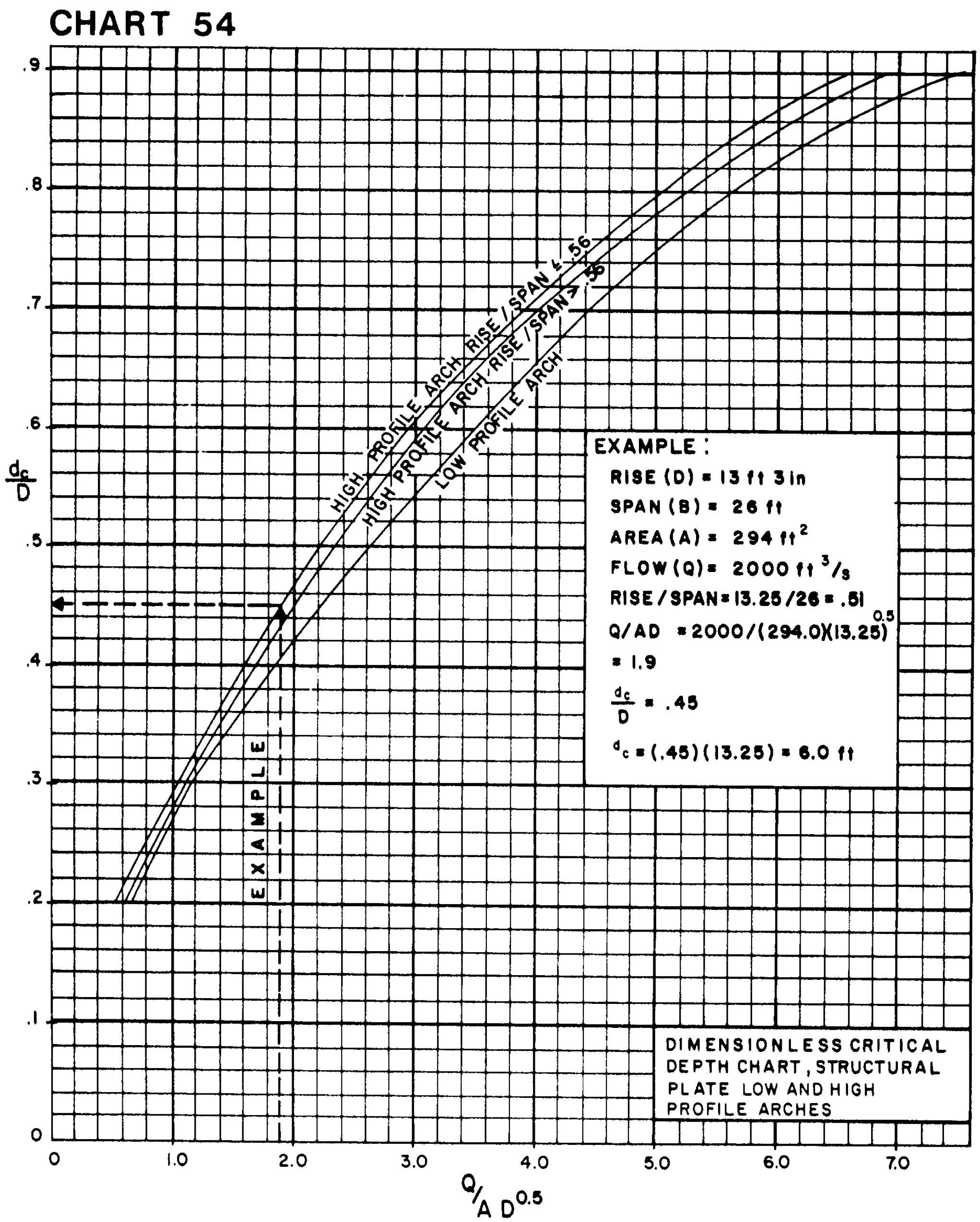
Source: HDS-5

Appendix 8C-53 Critical Depth, Structural Plate Ellipse, Long Axis Horizontal



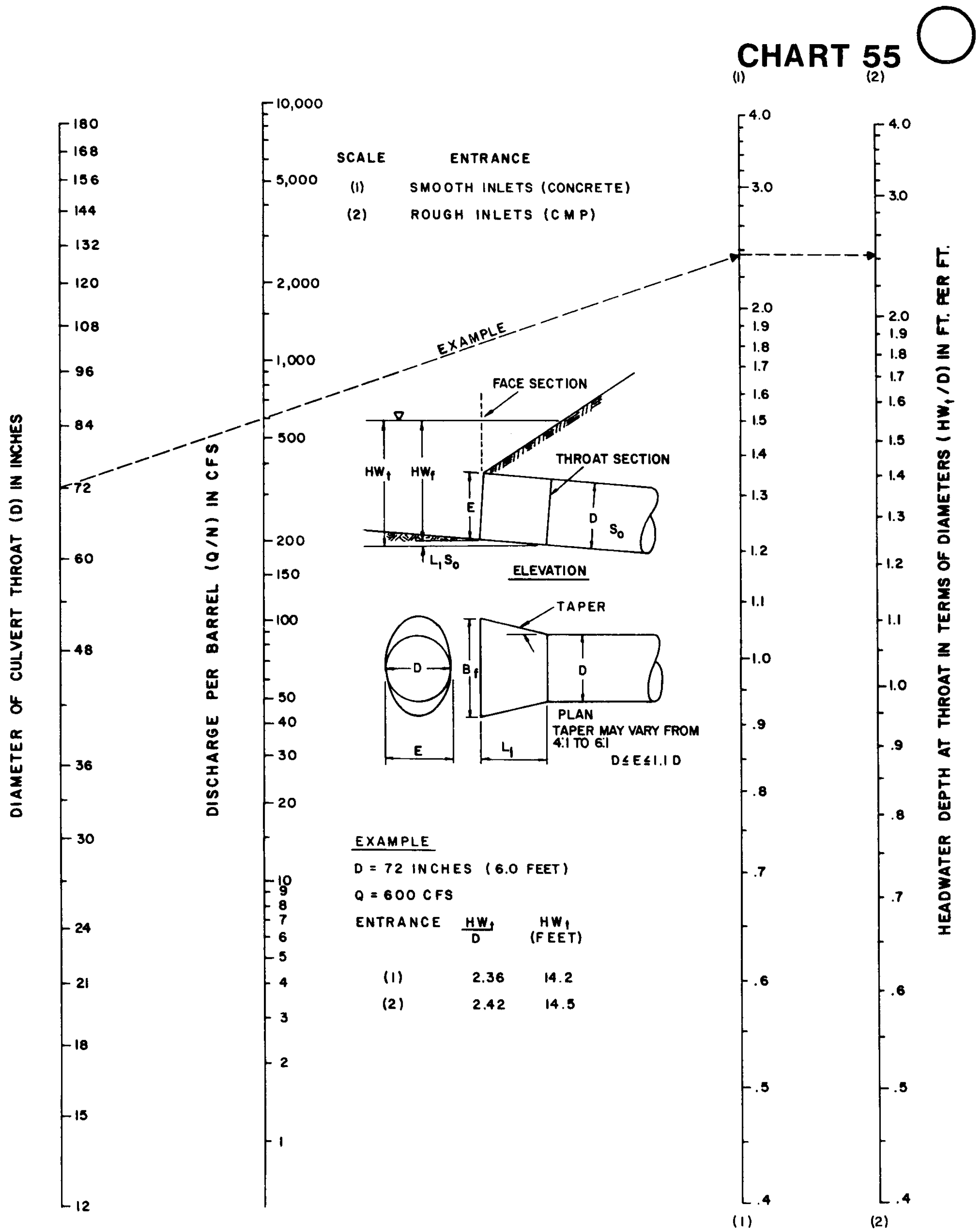
Source: HDS-5

Appendix 8C-54 Critical Depth, Structural Plate Arch, Low and High Profile



Source: HDS-5

Chart 8C-55 Throat Control, Circular Section, Side-Tapered

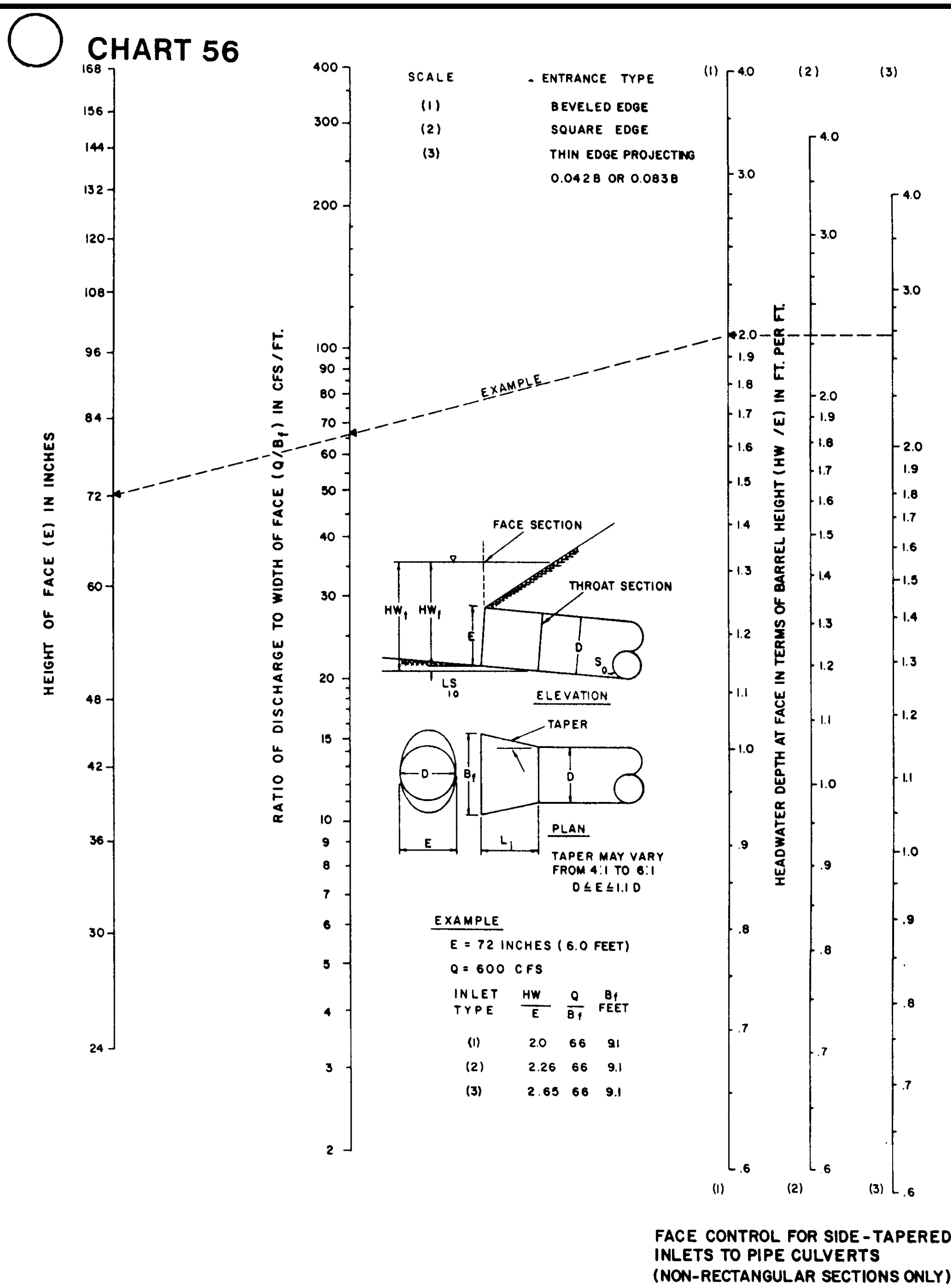


THROAT CONTROL
FOR SIDE-TAPERED INLETS TO PIPE CULVERT
(CIRCULAR SECTION ONLY)

9 - D - 58

Source: HDS-5

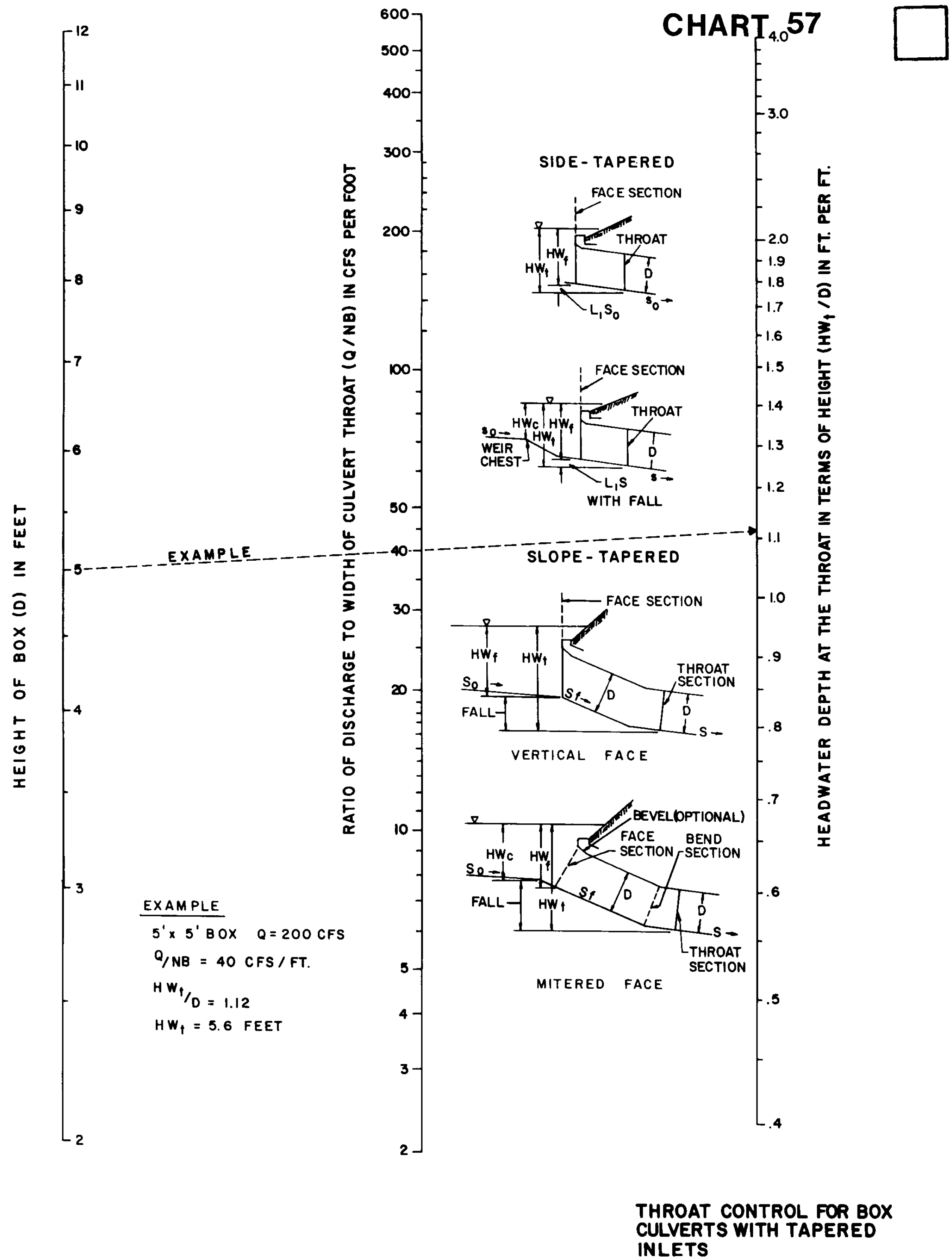
Chart 8C-56 Face Control, Non-Rectangular Section, Side-Tapered to Circular



9 - D - 59

Source: HDS-5

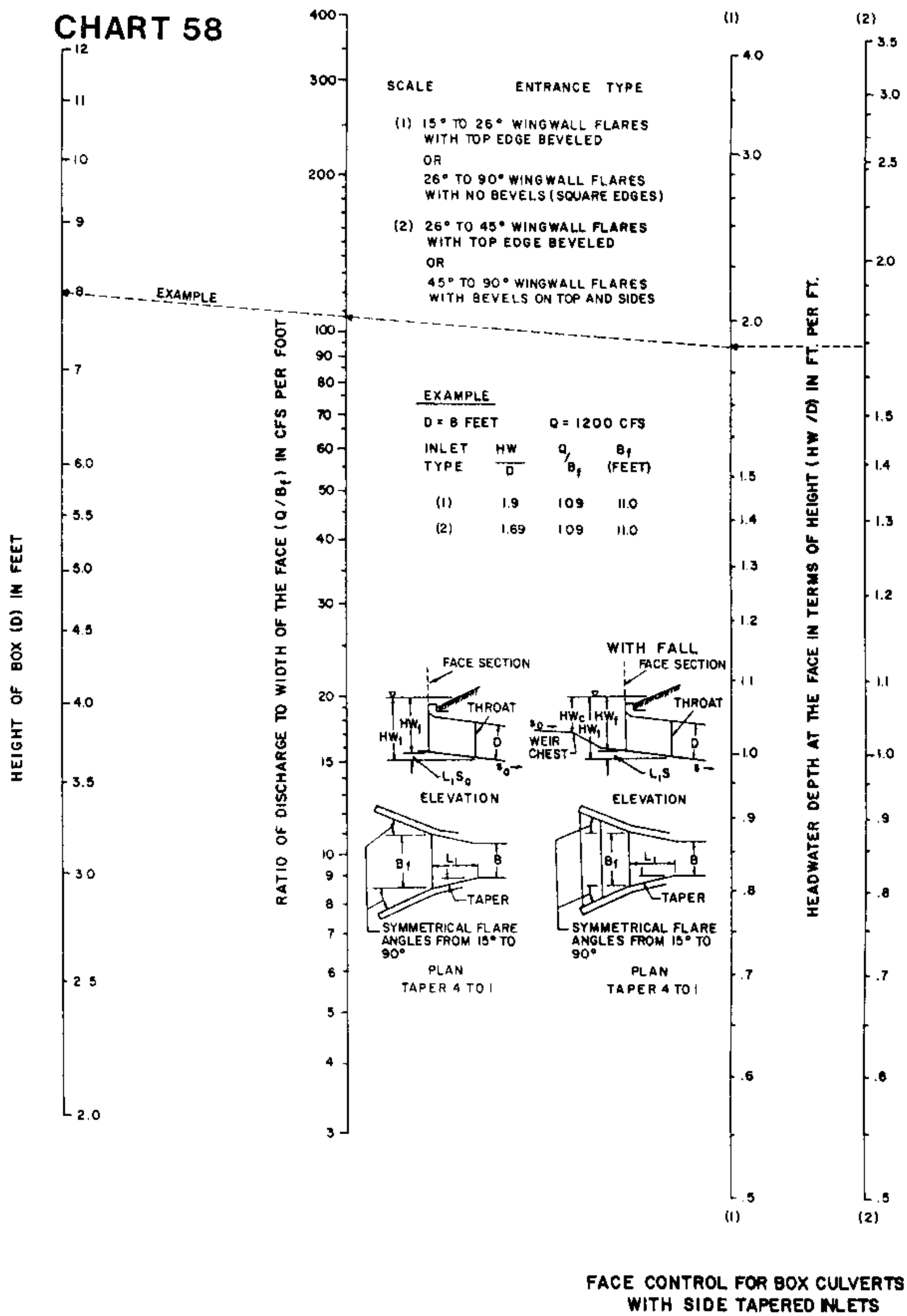
Chart 8C-57 Throat Control, Box Section, Tapered Inlet



9 - D - 60

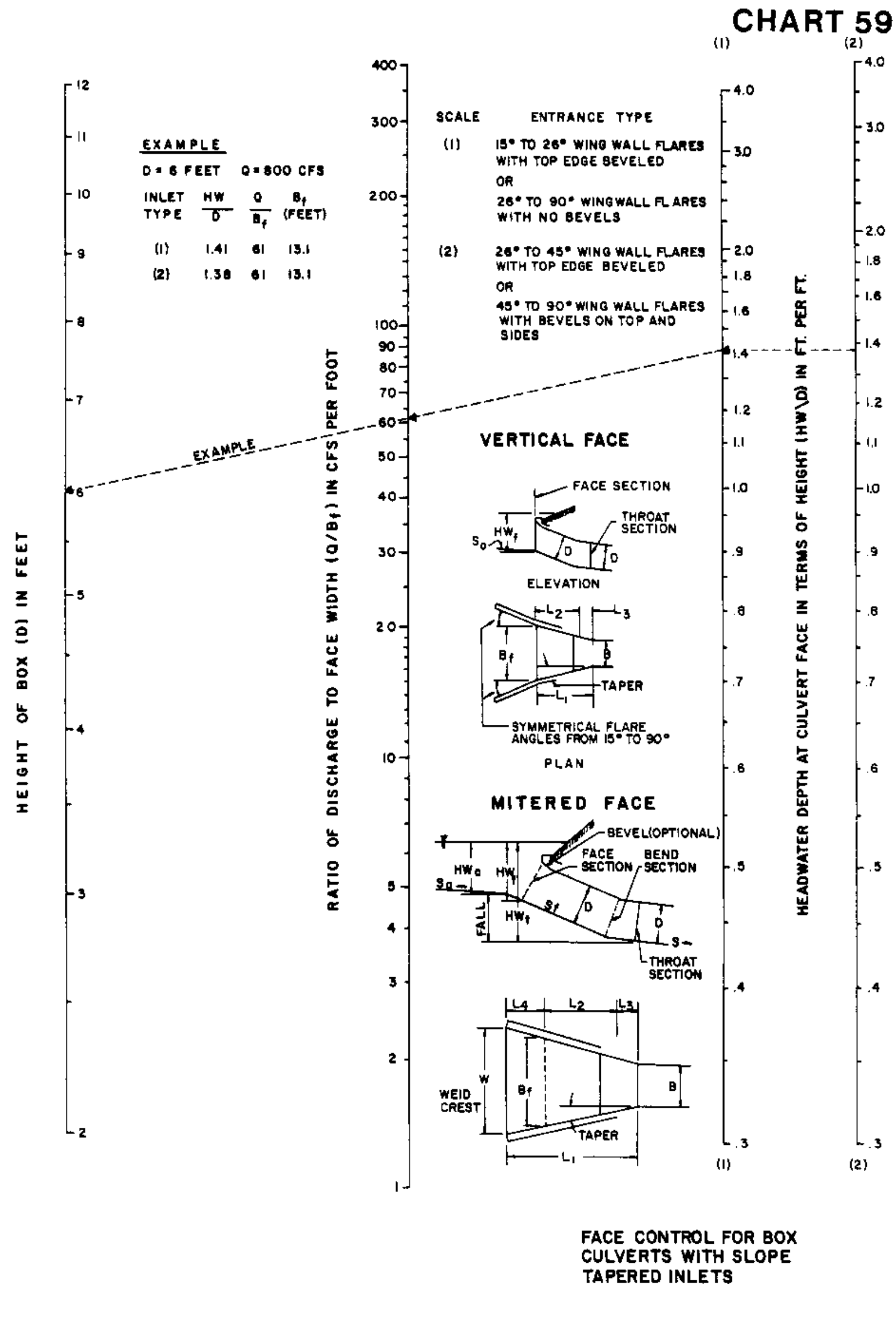
Source: HDS-5

Appendix 8C-58 Face Control, Box Section, Side-Tapered



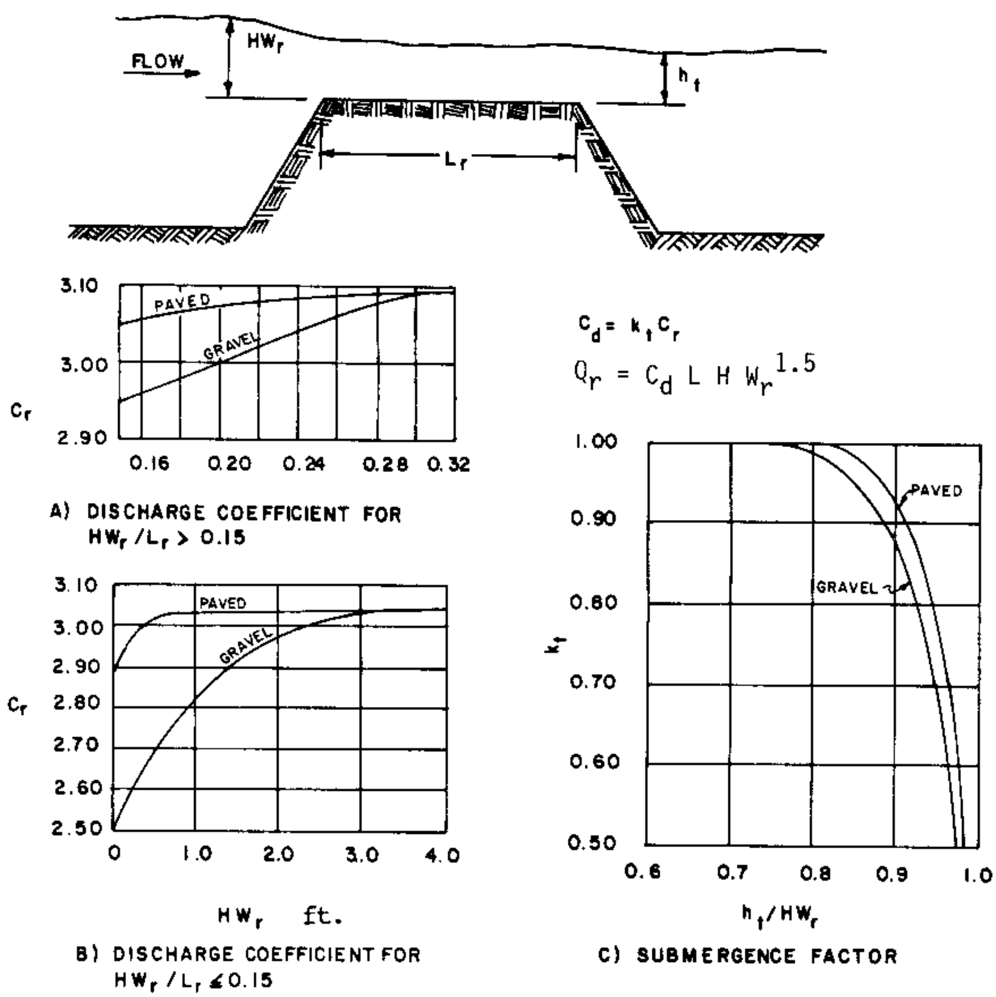
Source: HDS-5

Appendix 8C-59 Face Control, Box Section, Slope-Tapered



Source: HDS-5

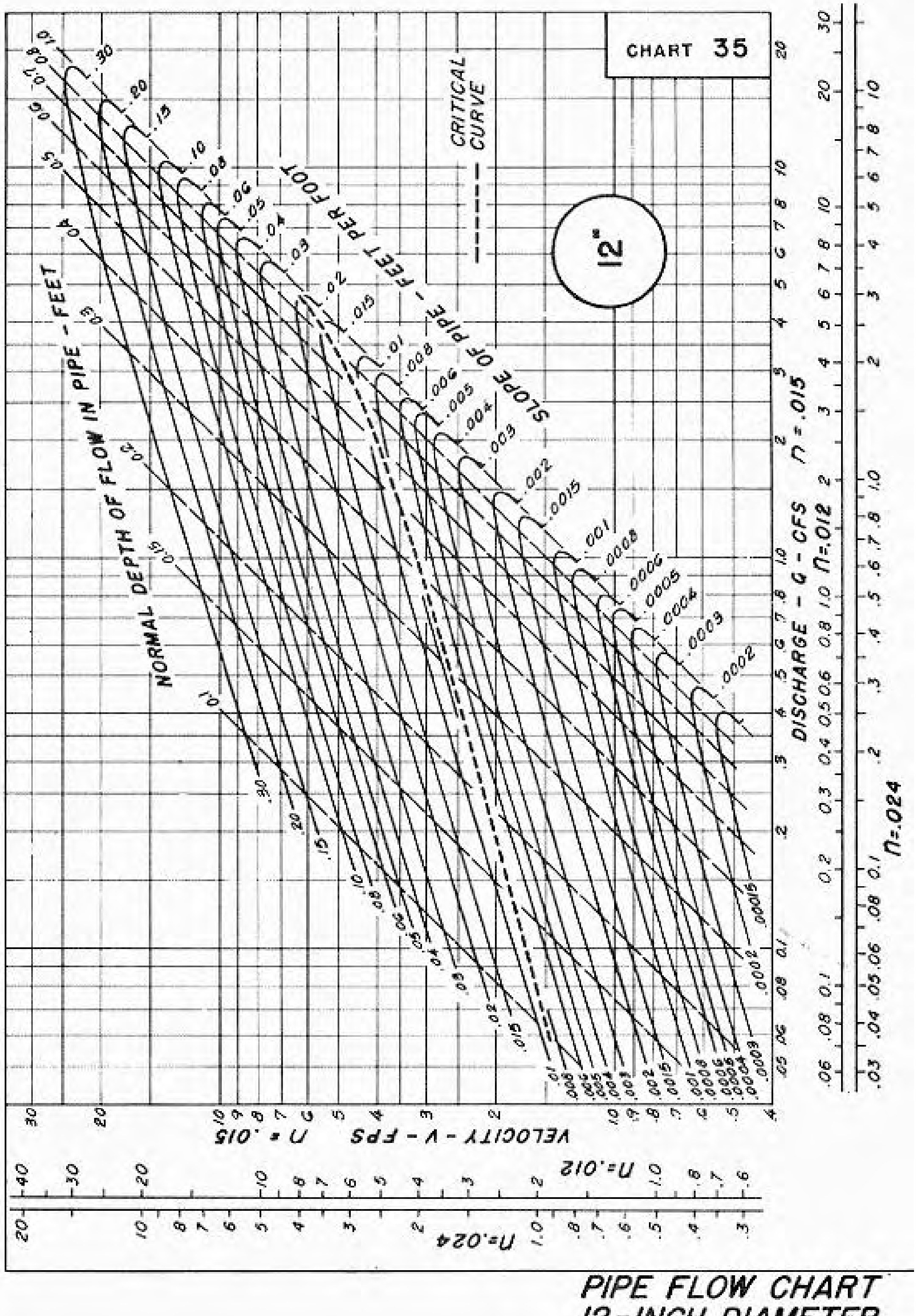
Chart 8C-60 Discharge Coefficients for Roadway Overtopping



DISCHARGE COEFFICIENTS
FOR ROADWAY OVERTOPPING

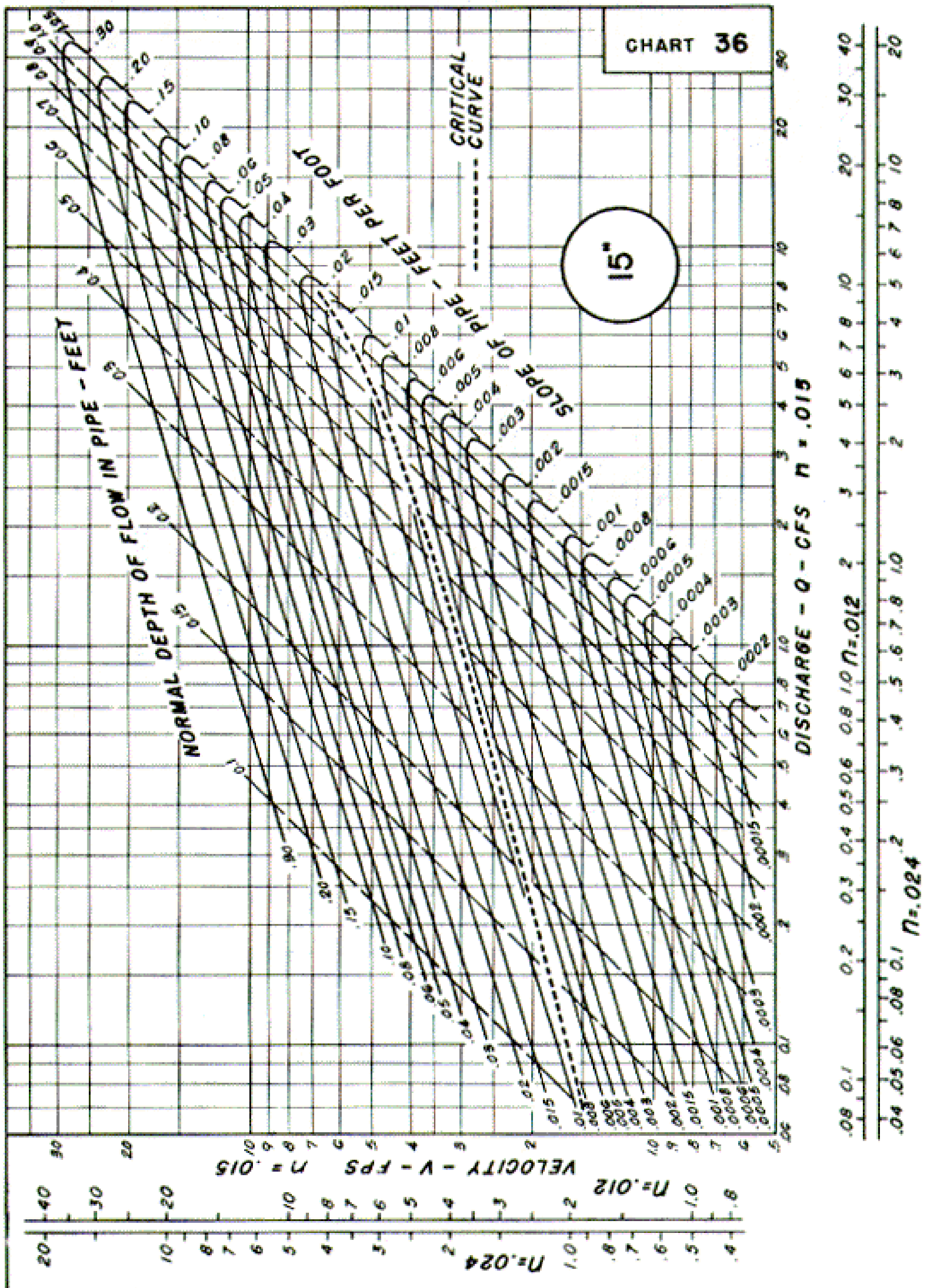
Source: HDS-5

Appendix 8C-61 Circular Pipe Flow Chart (Diameter = 12")



Source: HDS-3

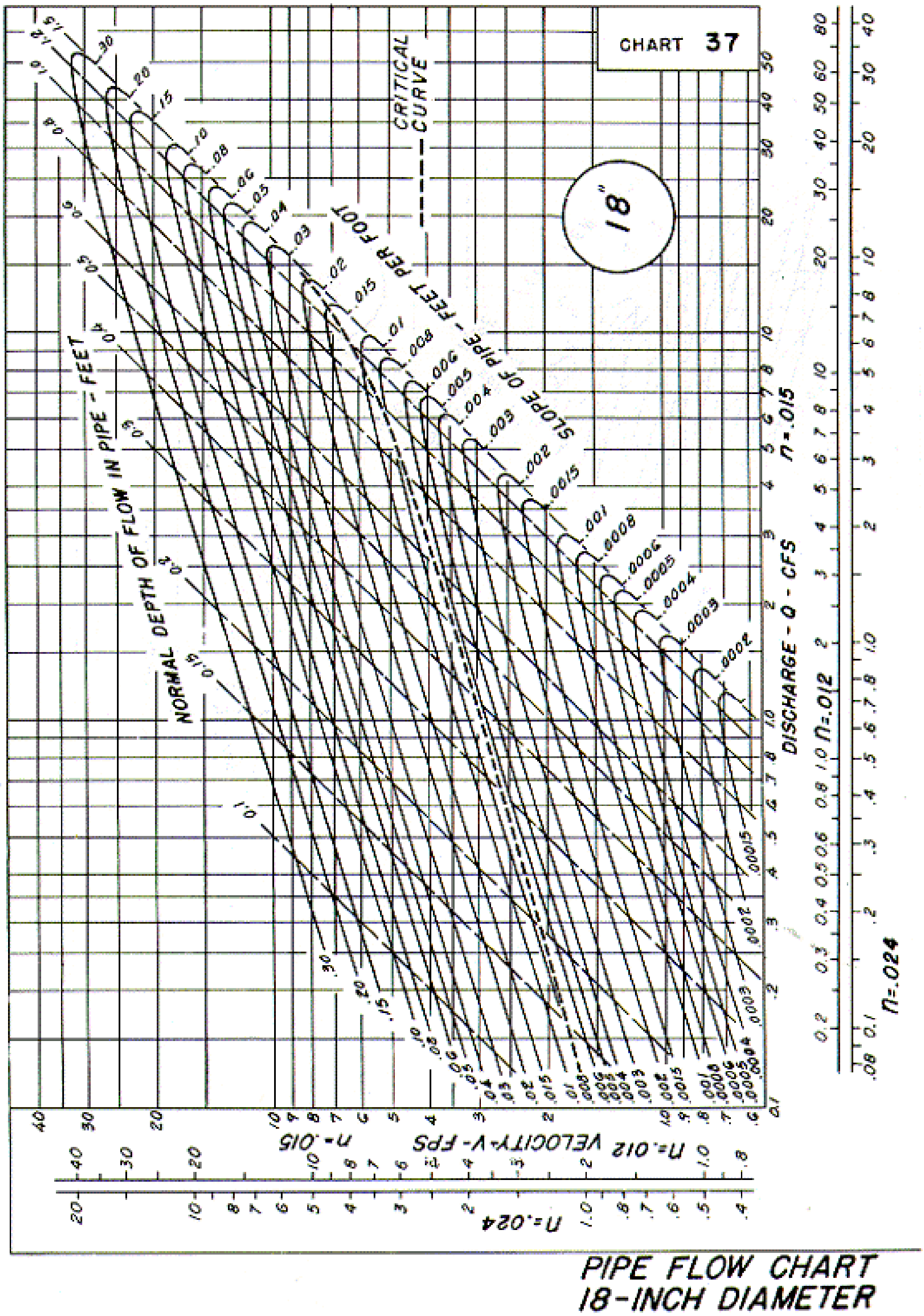
Appendix 8C-62 Circular Pipe Flow Chart (Diameter = 15”)



PIPE FLOW CHART
15-INCH DIAMETER

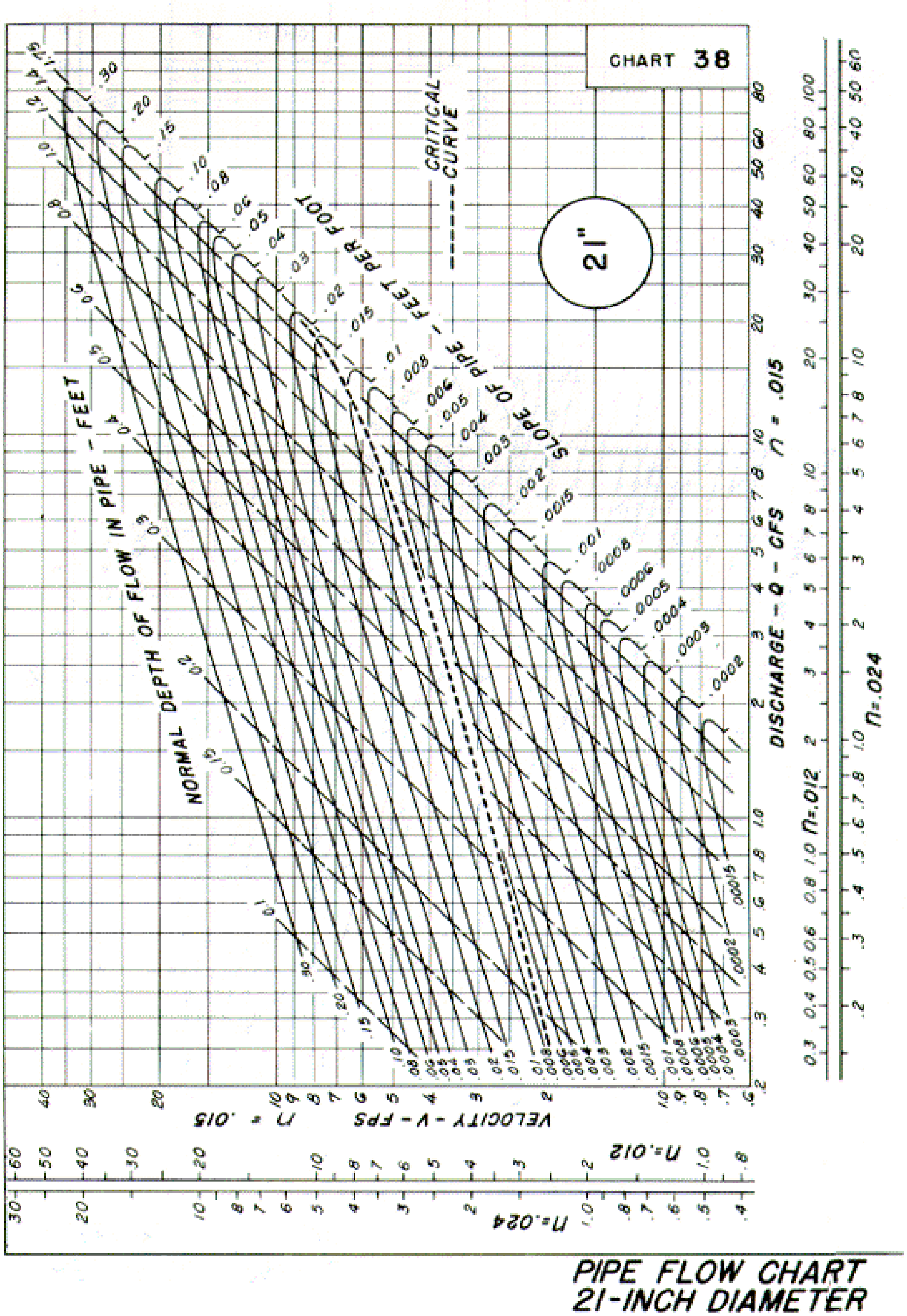
Source: HDS-3

Appendix 8C-63 Circular Pipe Flow Chart (Diameter = 18")



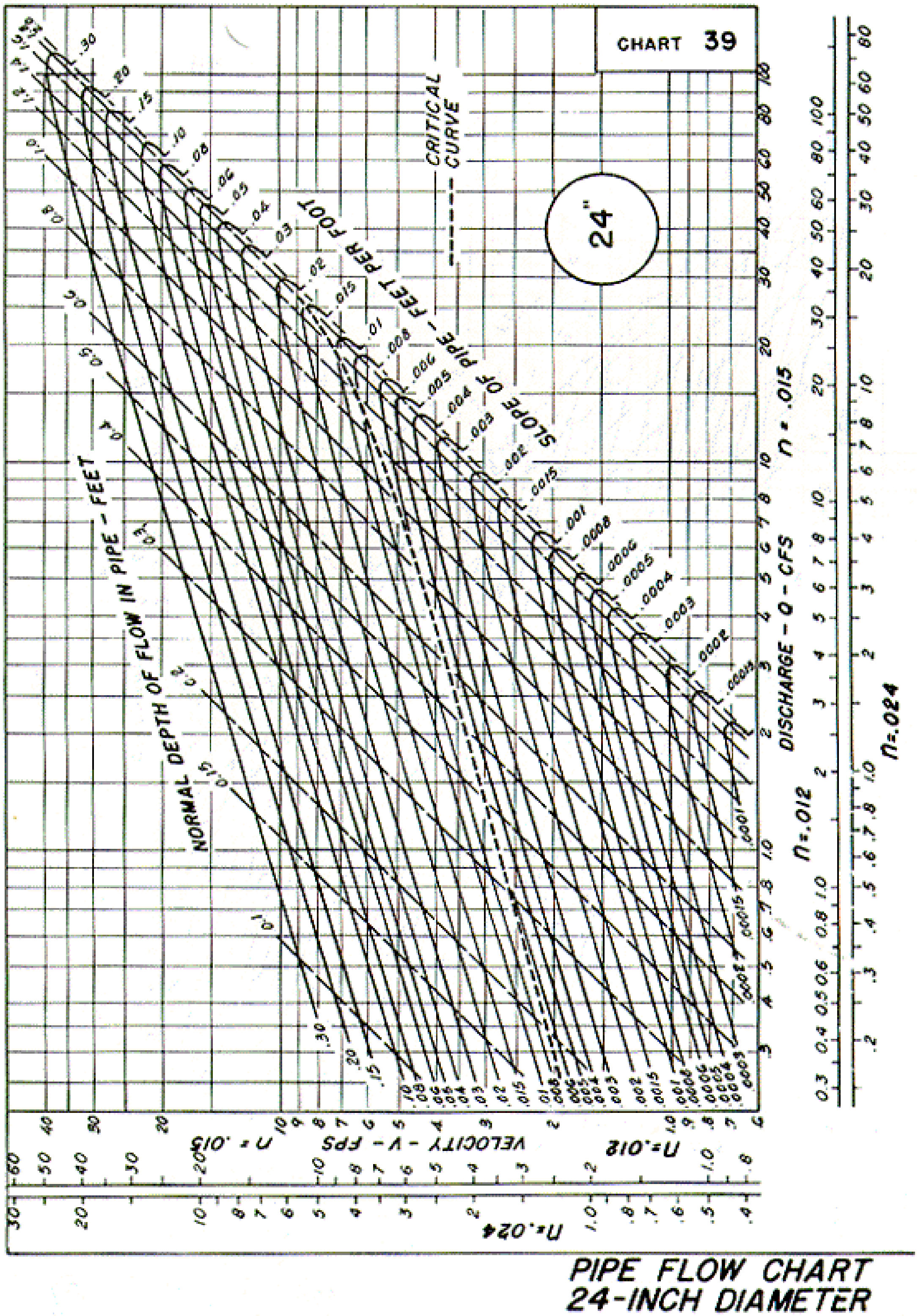
Source: HDS-3

Appendix 8C-64 Circular Pipe Flow Chart (Diameter = 21")



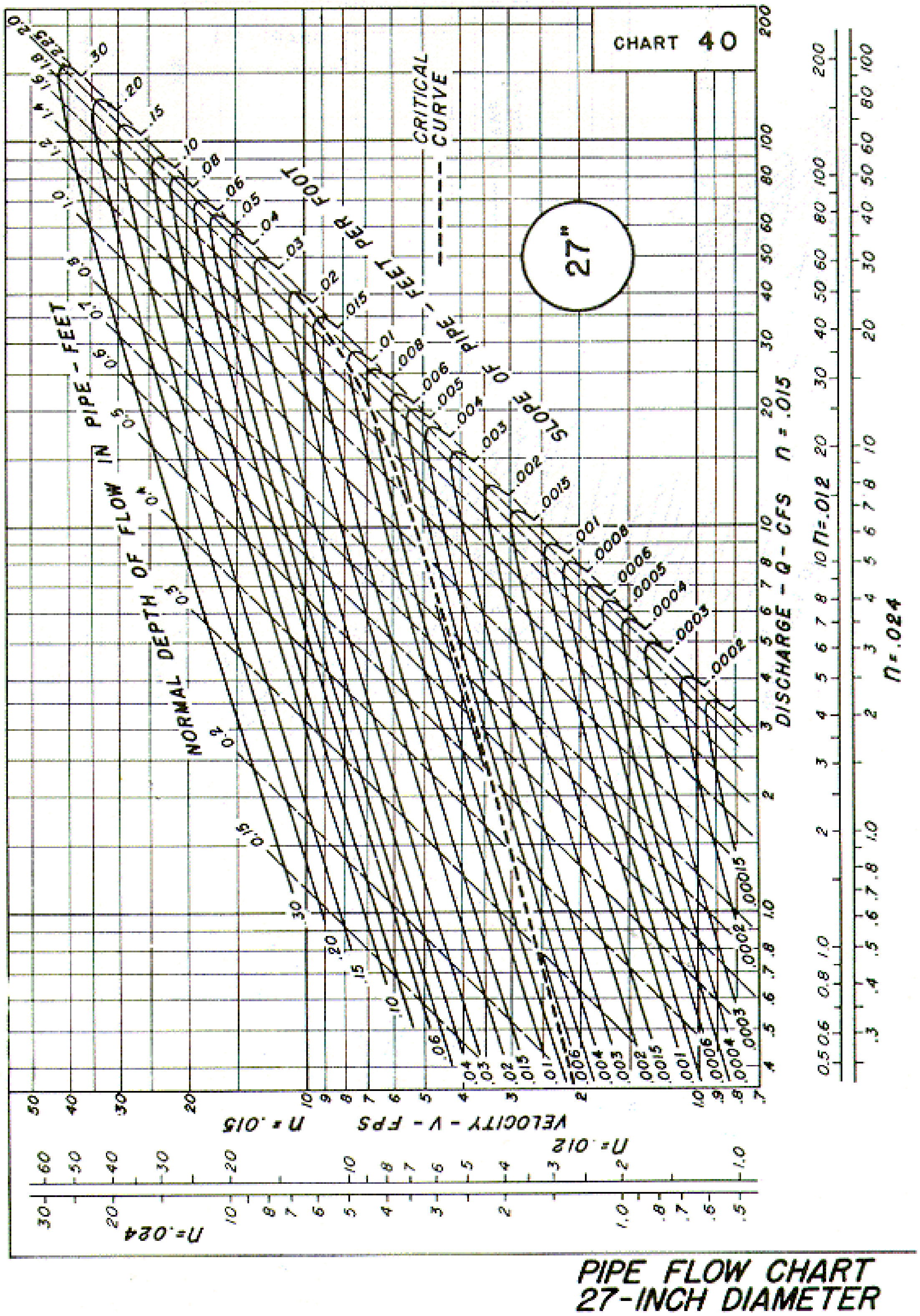
Source: HDS-3

Appendix 8C-65   Circular Pipe Flow Chart (Diameter = 24")



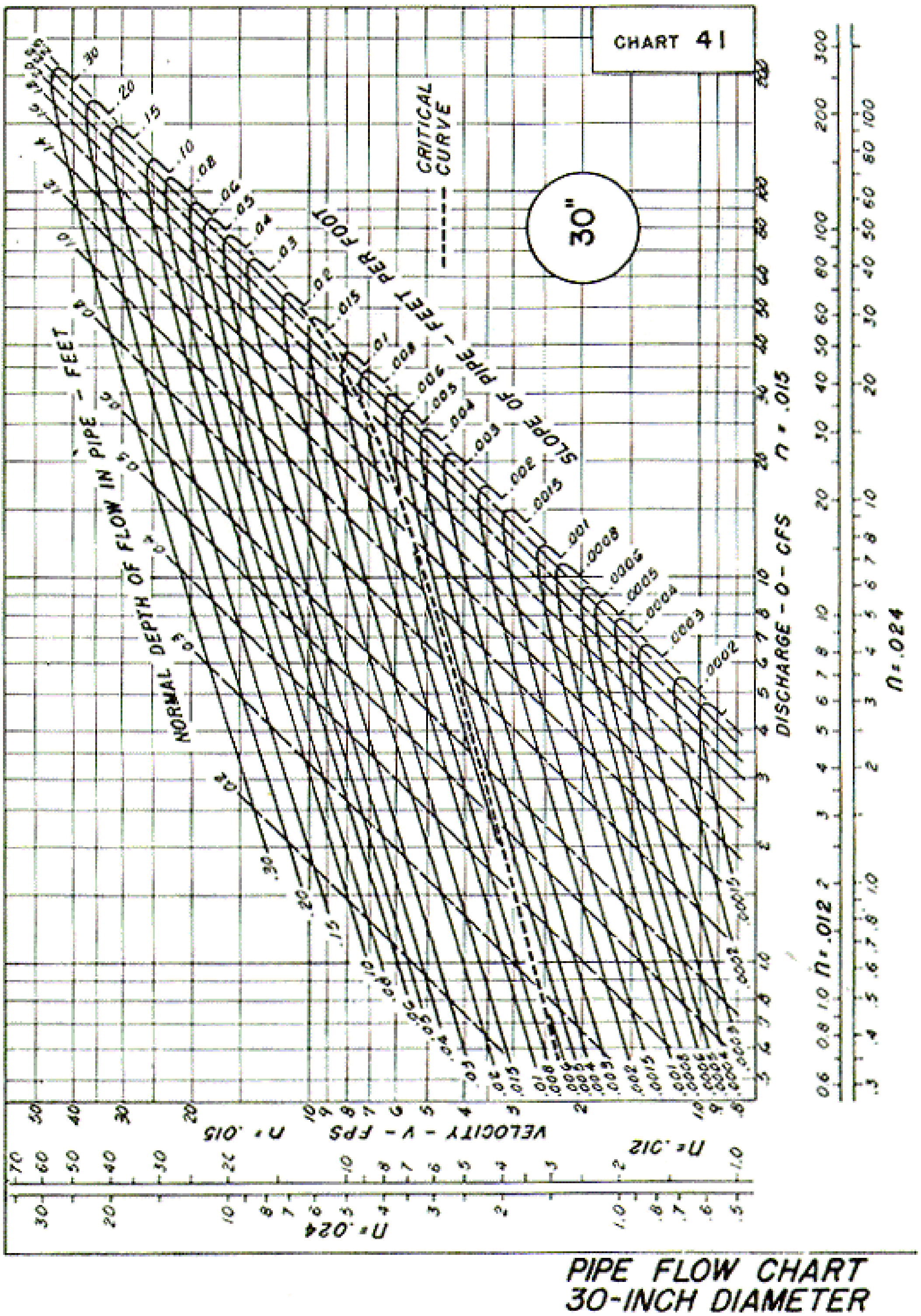
Source: HDS-3

Appendix 8C-66 Circular Pipe Flow Chart (Diameter = 27")



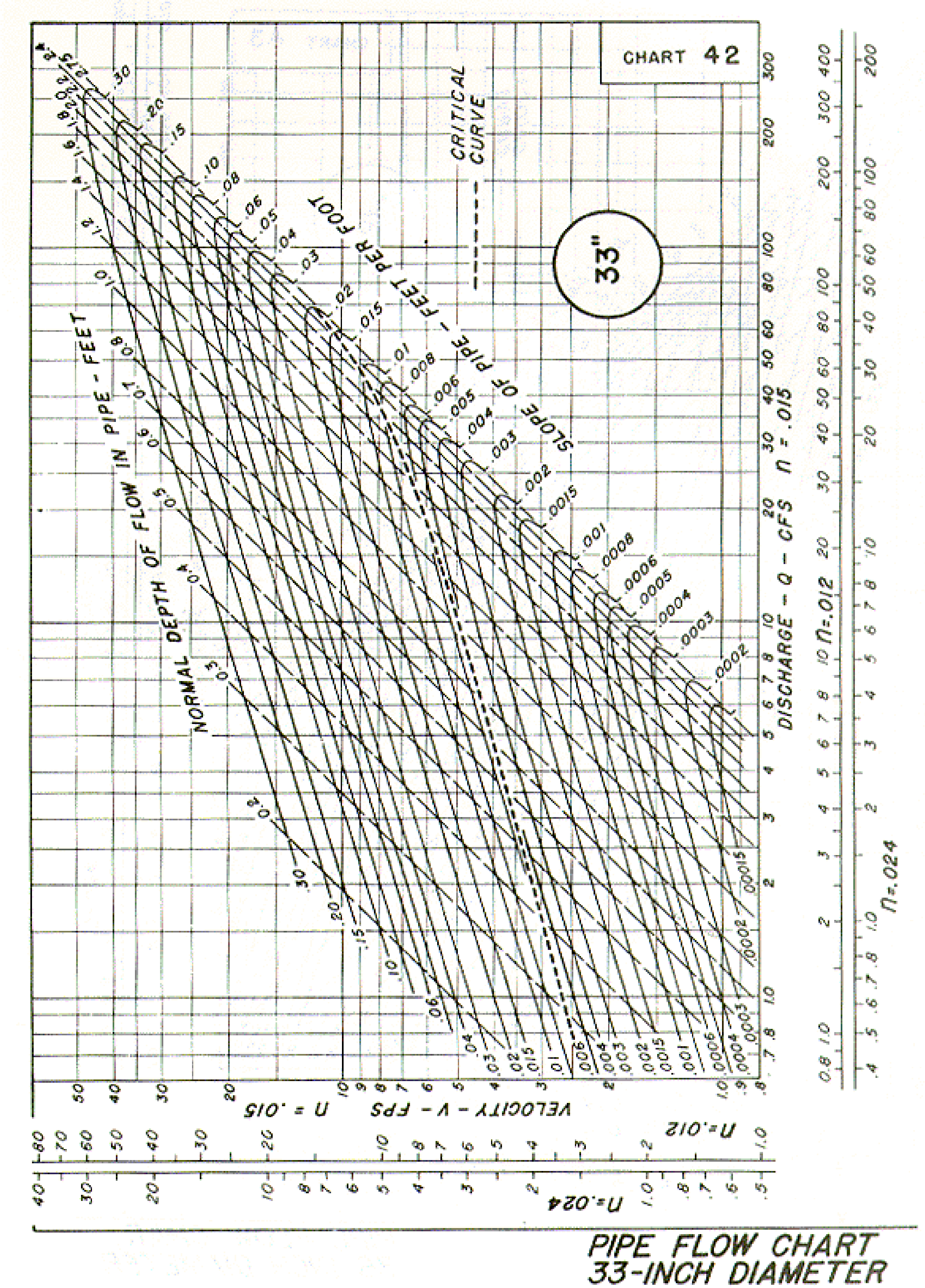
Source: HDS-3

Appendix 8C-67 Circular Pipe Flow Chart (Diameter = 30")



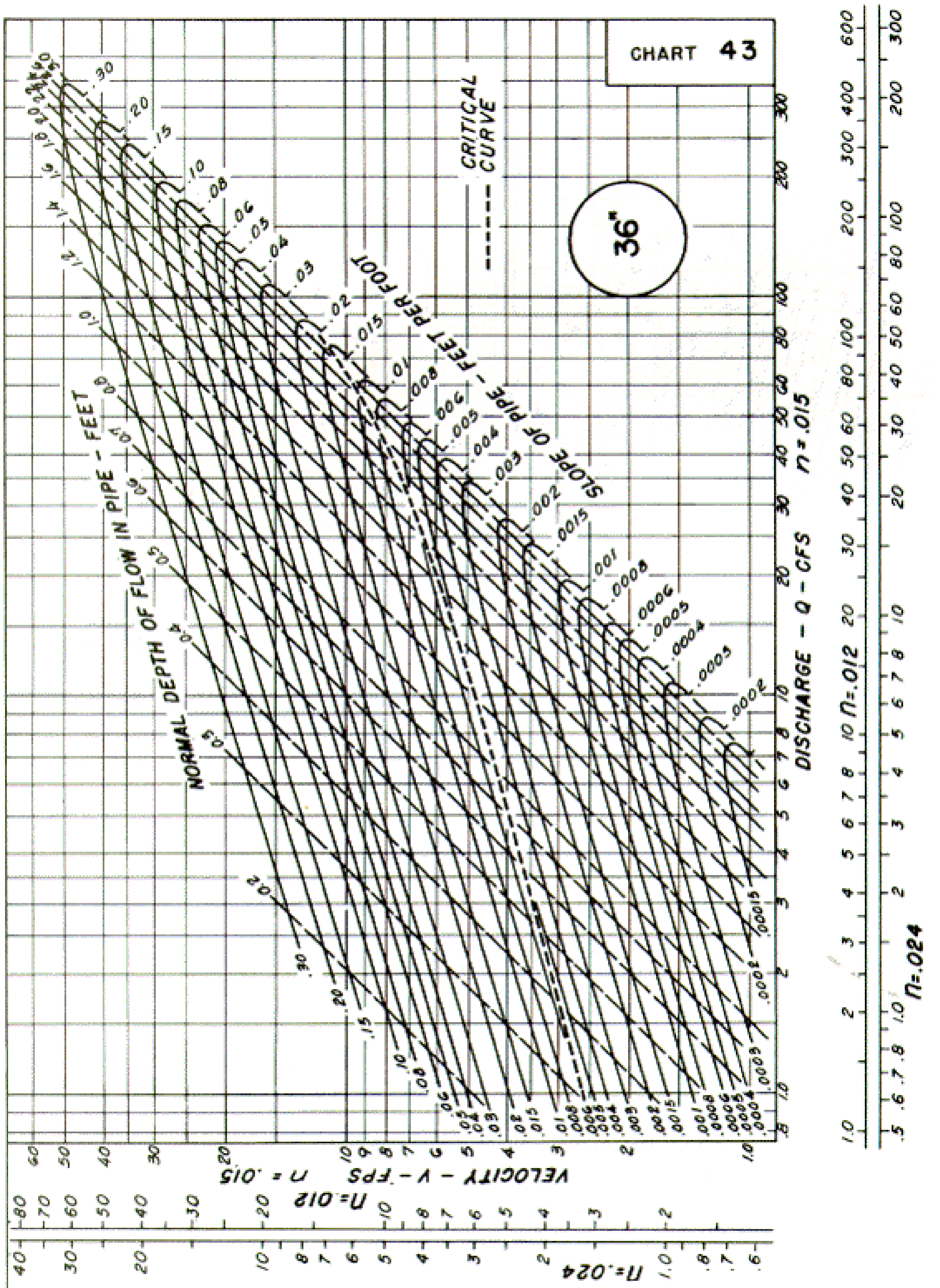
Source: HDS-3

Appendix 8C-68 Circular Pipe Flow Chart (Diameter = 33")



Source: HDS-3

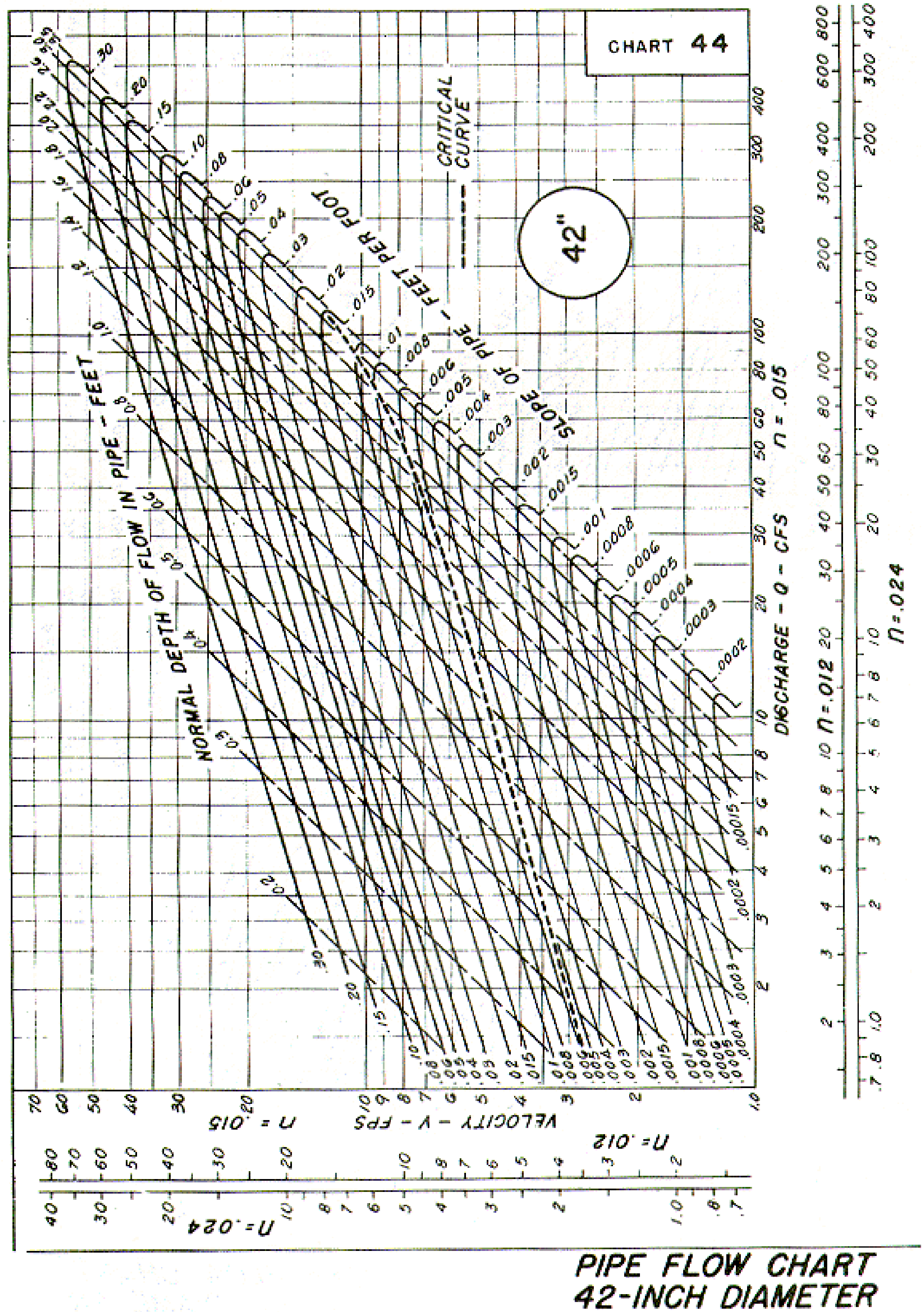
Appendix 8C-69 Circular Pipe Flow Chart (Diameter = 36")



PIPE FLOW CHART
36-INCH DIAMETER

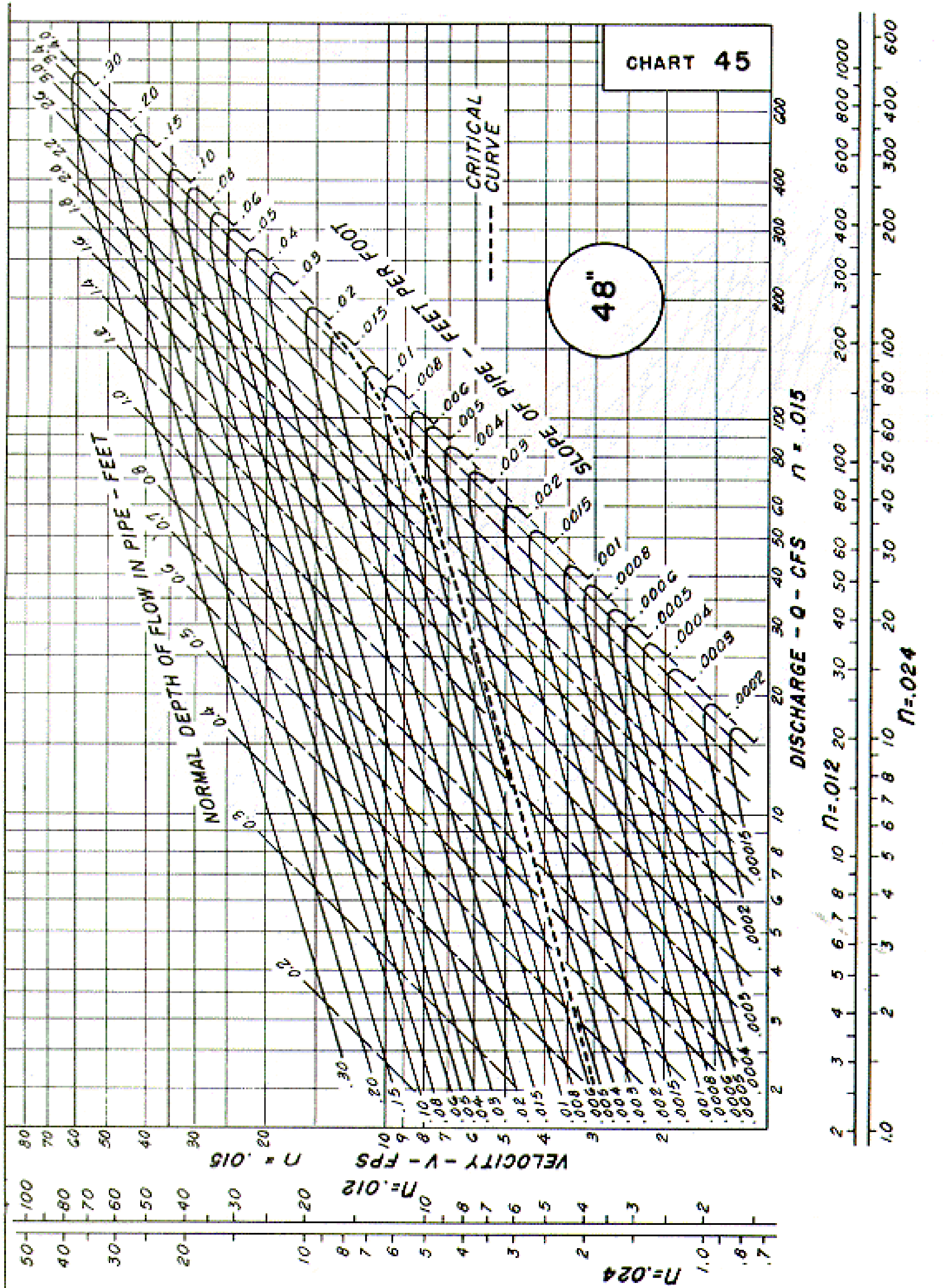
Source: HDS-3

Appendix 8C-70 Circular Pipe Flow Chart (Diameter = 42”)



Source: HDS-3

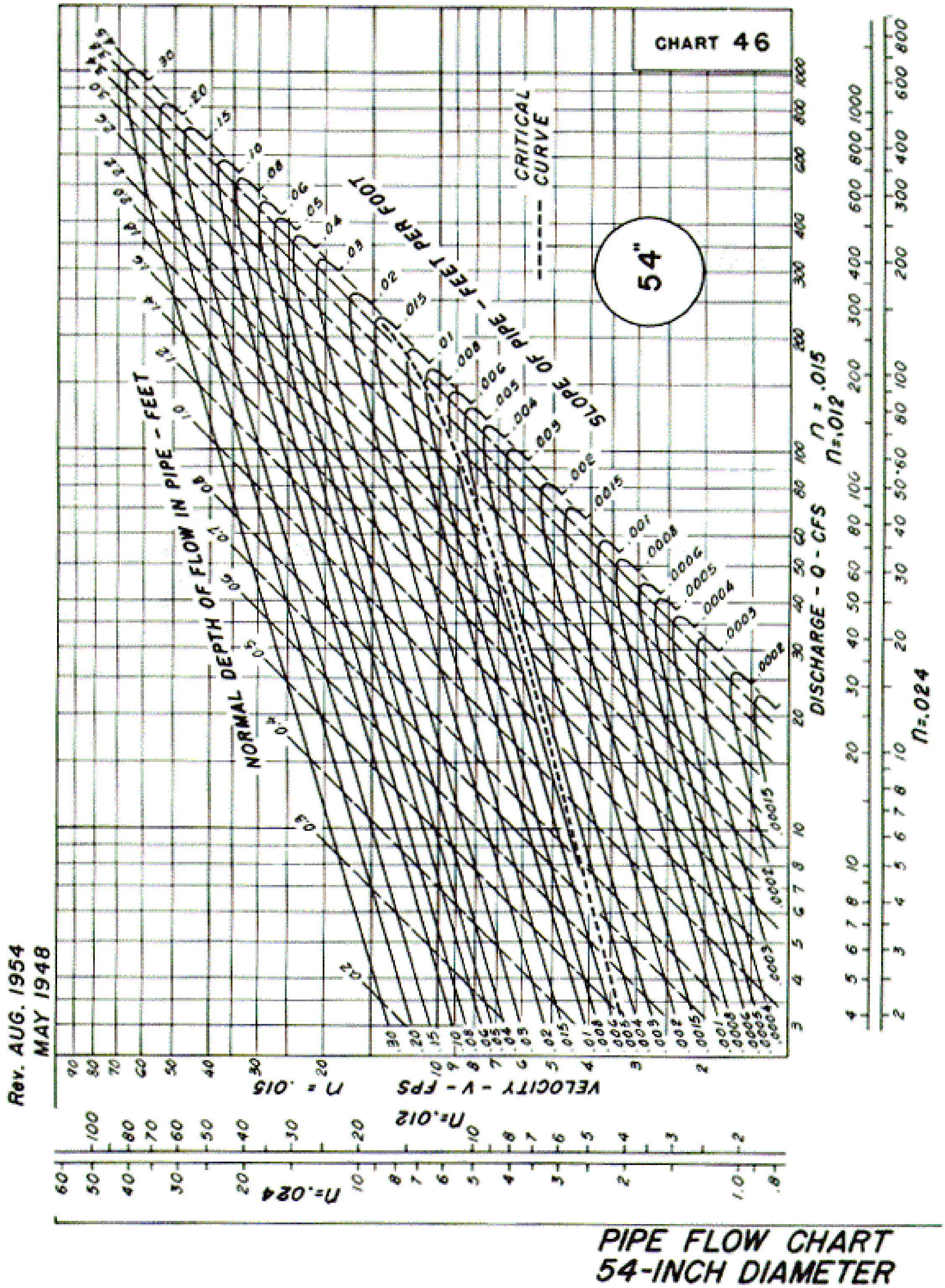
Appendix 8C-71 Circular Pipe Flow Chart (Diameter 48")



PIPE FLOW CHART
48-INCH DIAMETER

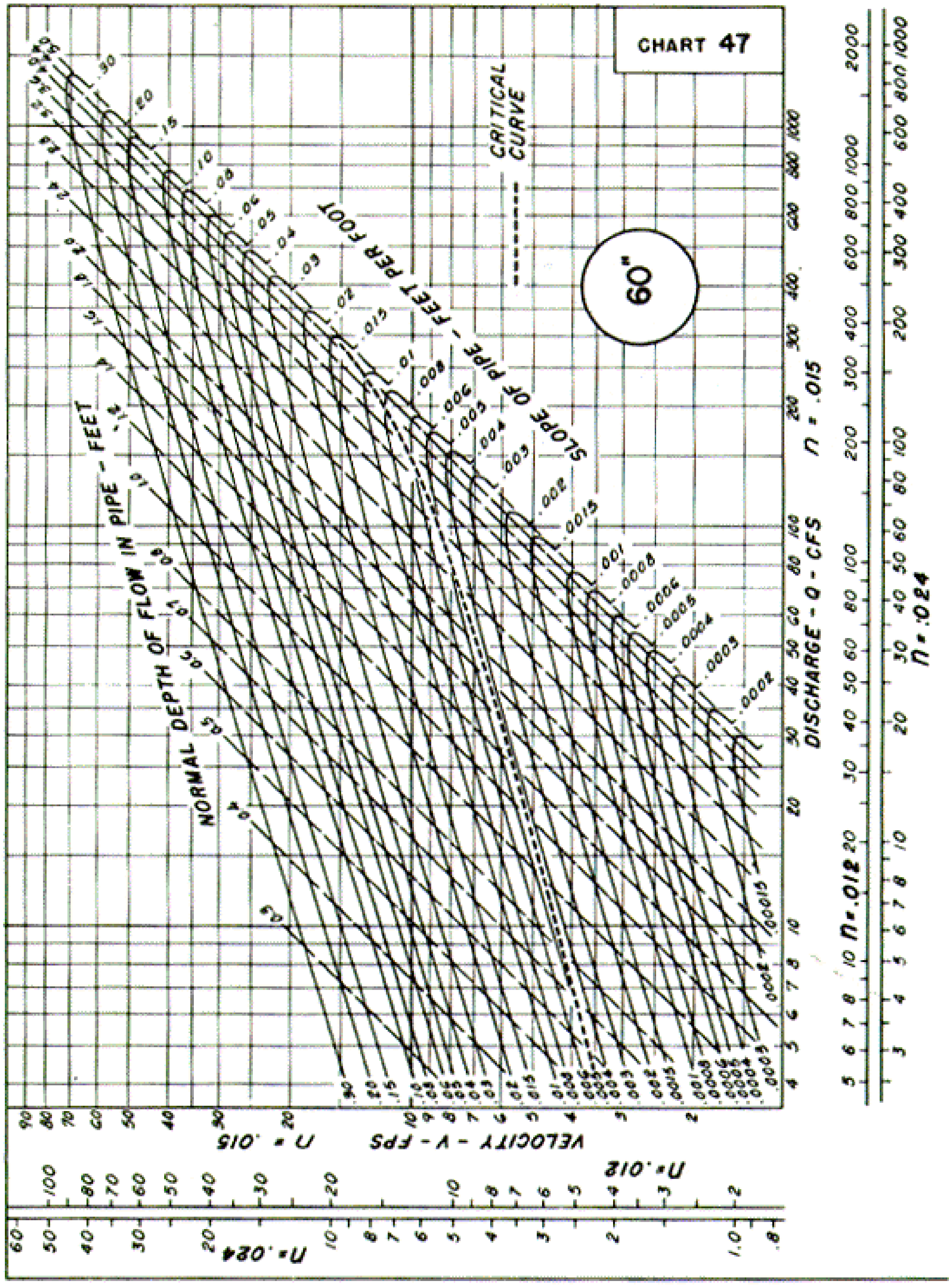
Source: HDS-3

Appendix 8C-72 Circular Pipe Flow Chart (Diameter = 54")



Source: HDS-3

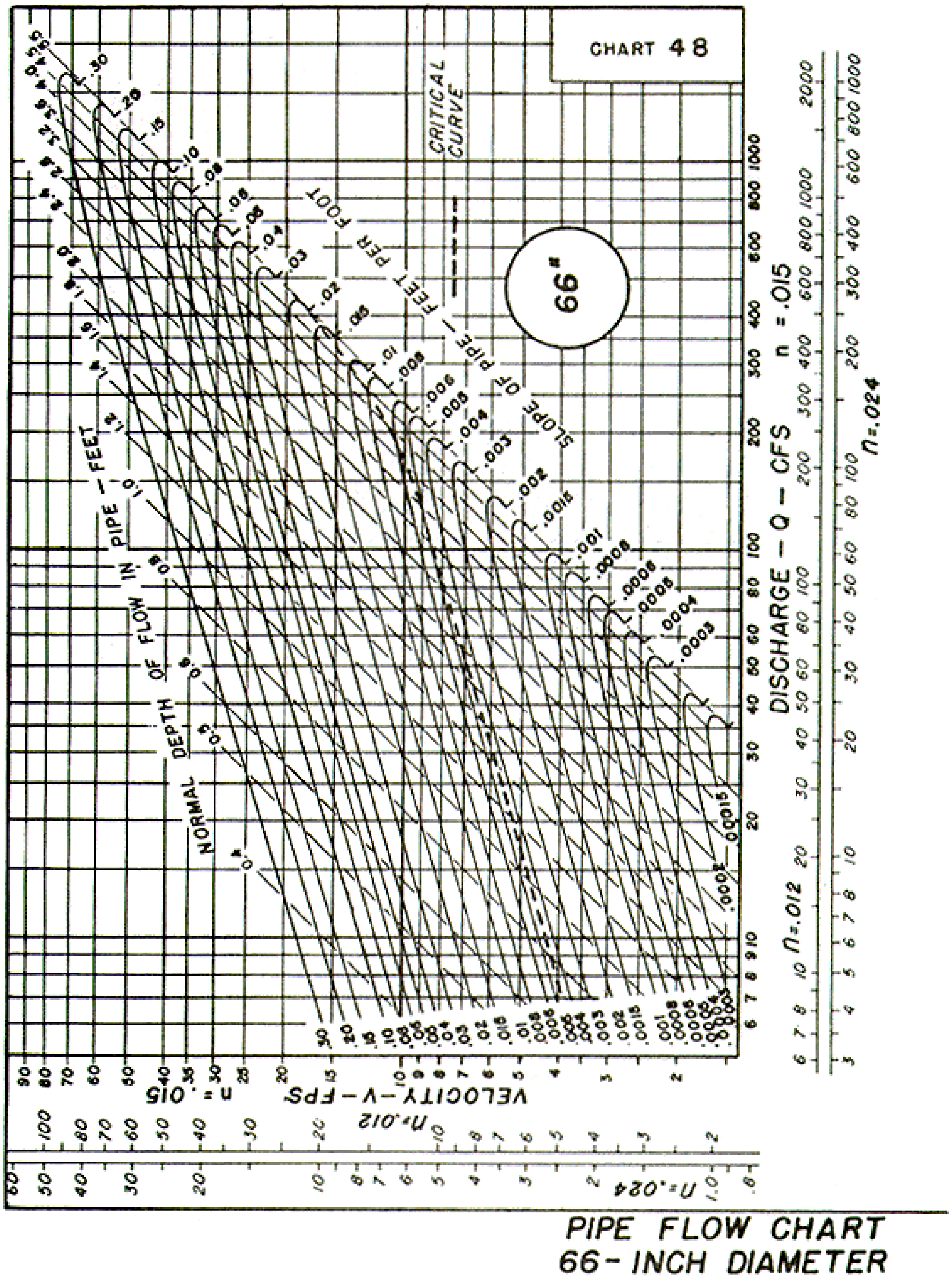
Appendix 8C-73 Circular Pipe Flow Chart (Diameter = 60")



PIPE FLOW CHART
60-INCH DIAMETER

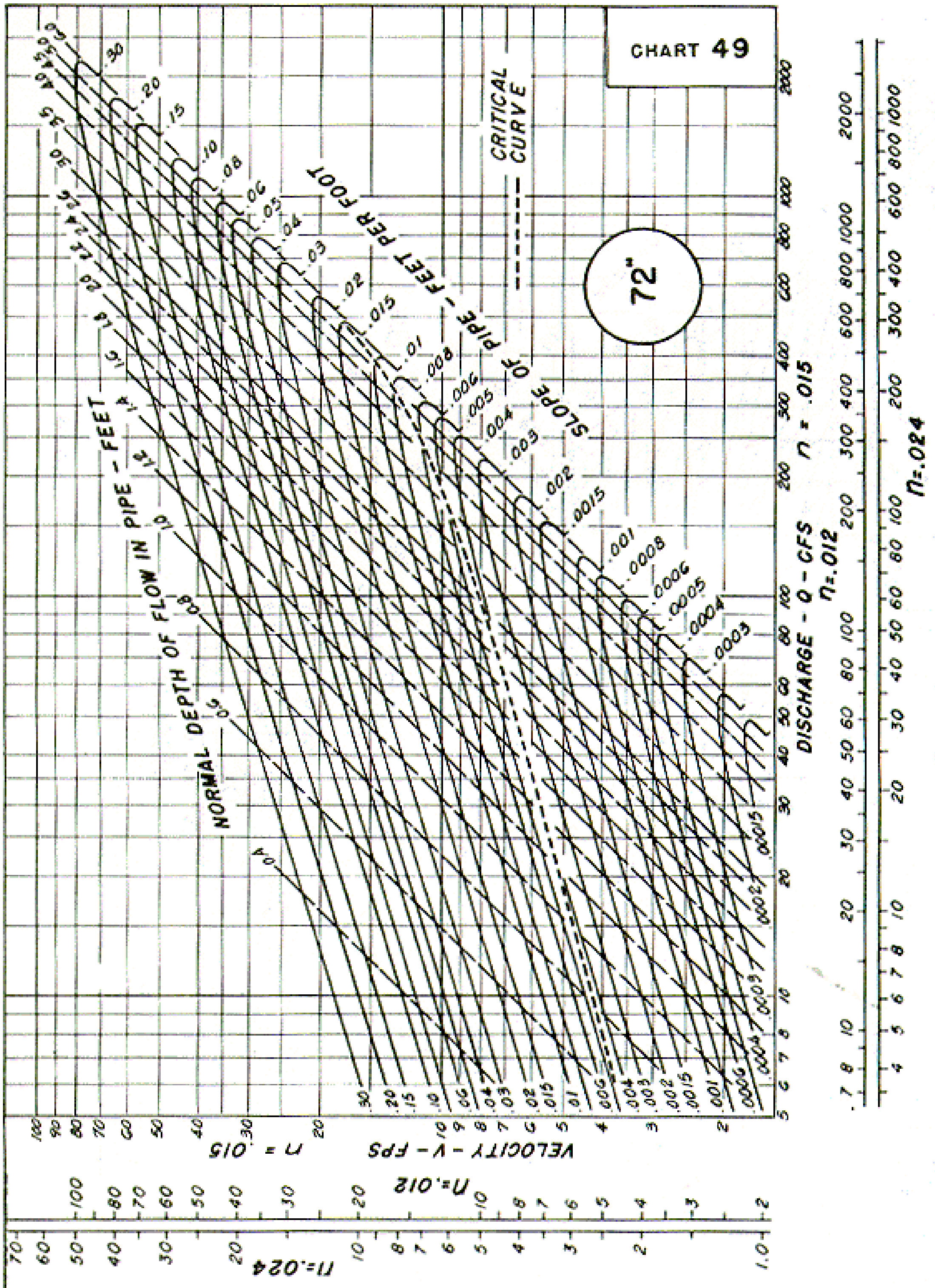
Source: HDS-3

Appendix 8C-74 Circular Pipe Flow Chart (Diameter = 66”)



Source: HDS-3

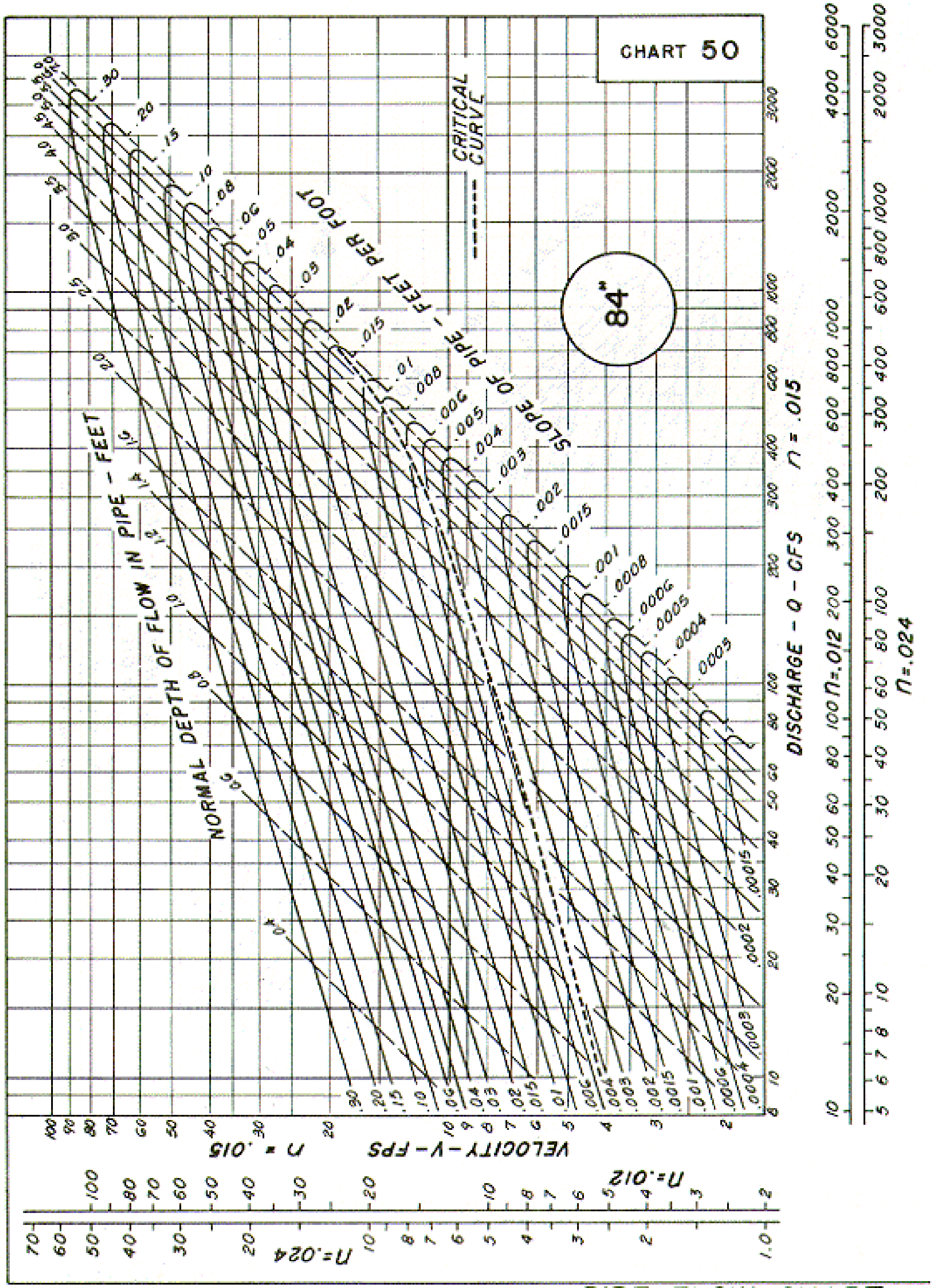
Appendix 8C-75 Circular Pipe Flow Chart (Diameter = 72")



PIPE FLOW CHART
72-INCH DIAMETER

Source: HDS-3

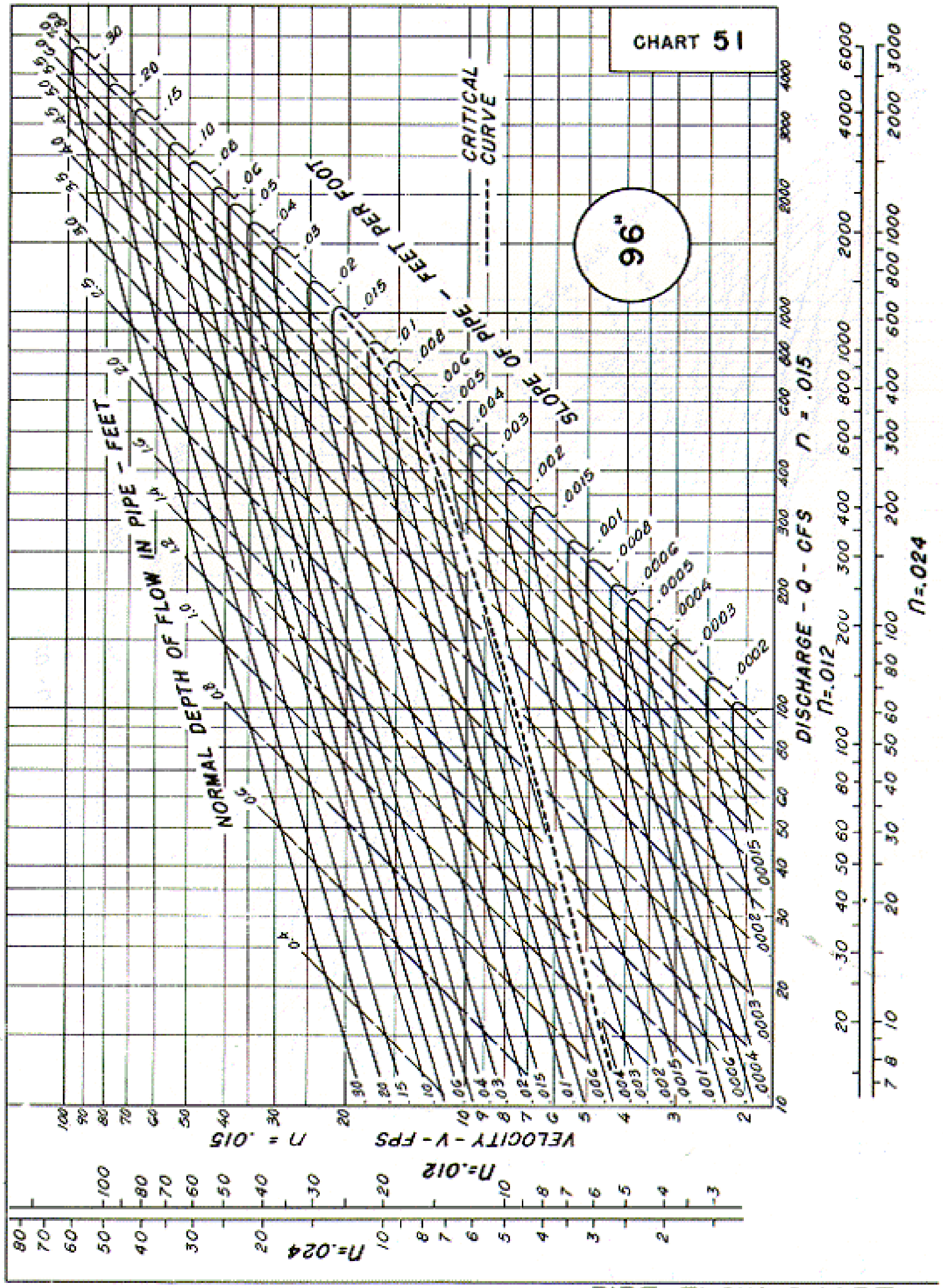
Appendix 8C-76 Circular Pipe Flow Chart (Diameter = 84")



PIPE FLOW CHART
84-INCH DIAMETER

Source: HDS-3

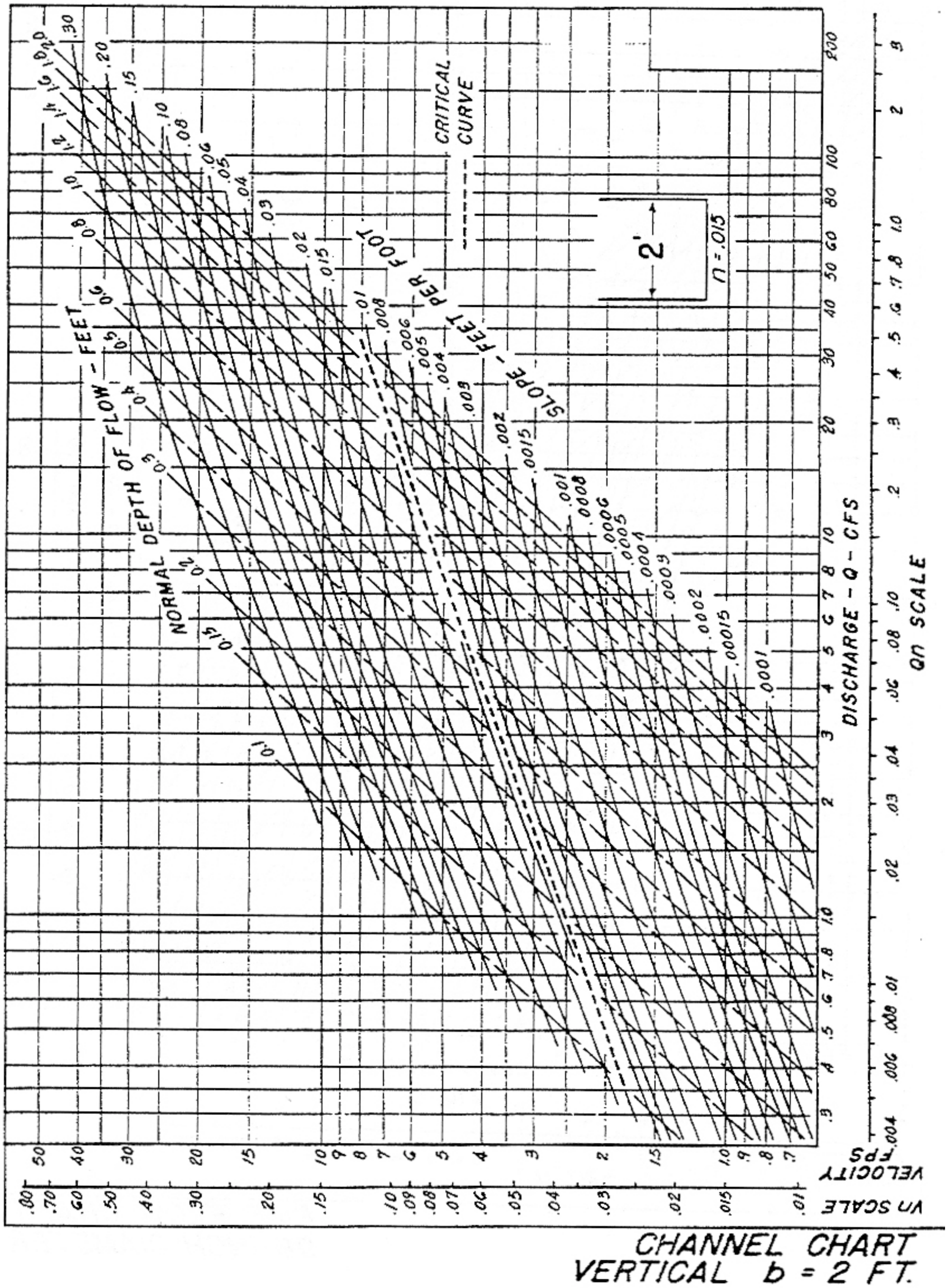
Appendix 8C-77 Circular Pipe Flow Chart (Diameter = 96”)



PIPE FLOW CHART
96-INCH DIAMETER

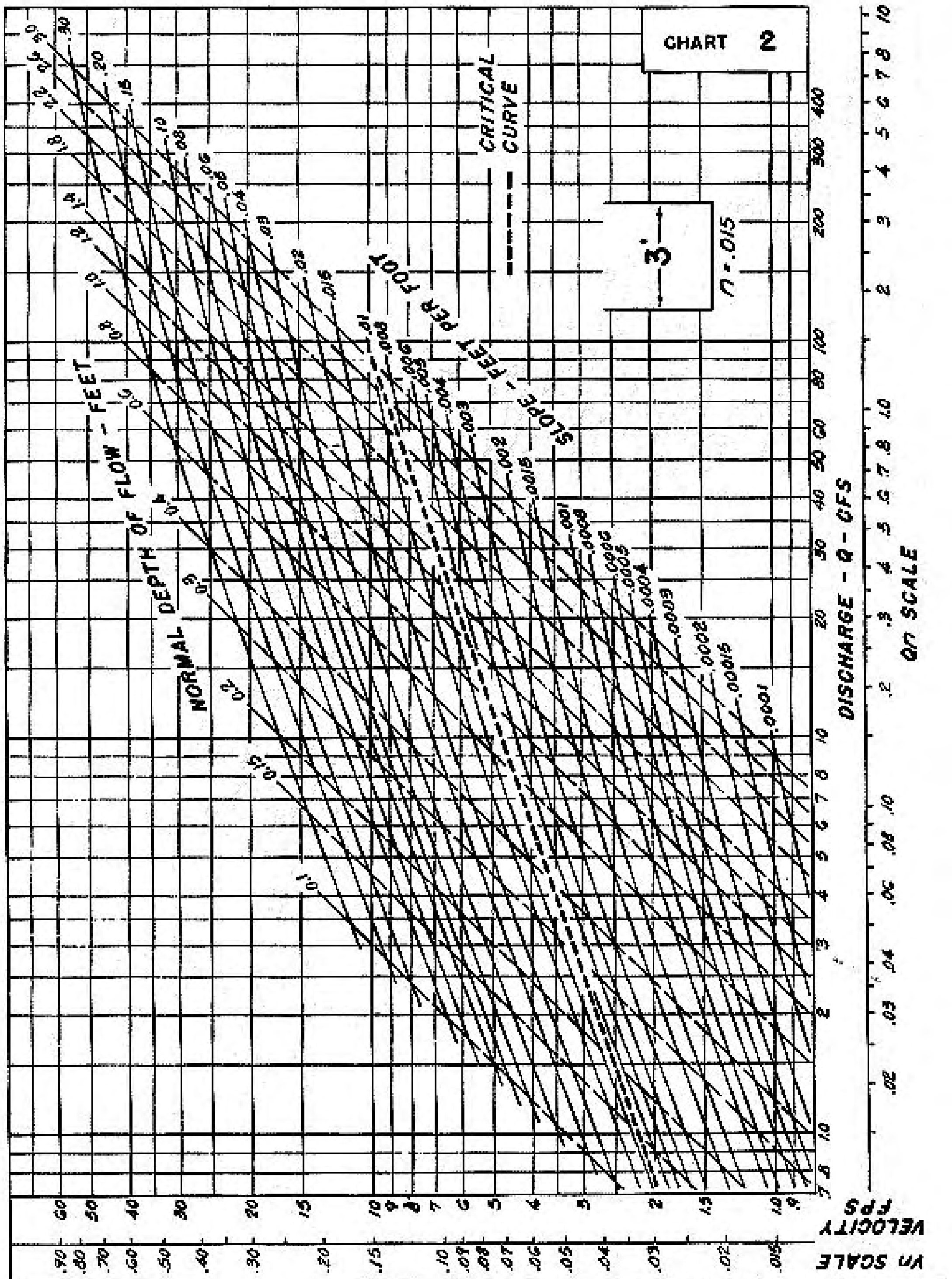
Source: HDS-3

Appendix 8C-78 Rectangular Channel Flow Chart (B=2')



Source: HDS-3

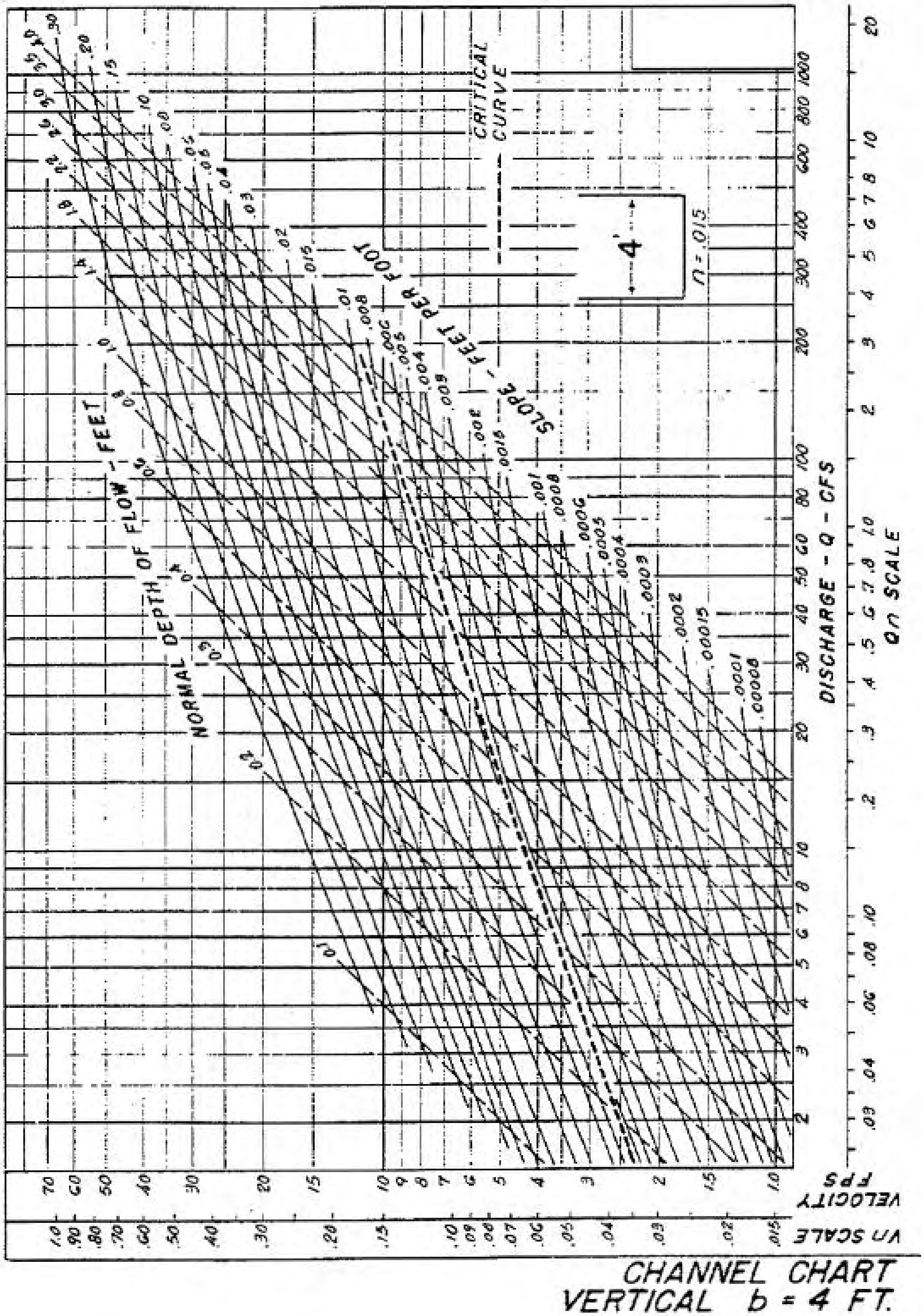
Appendix 8C-79 Rectangular Channel Flow Chart (B=3')



CHANNEL CHART
VERTICAL b = 3 FT.

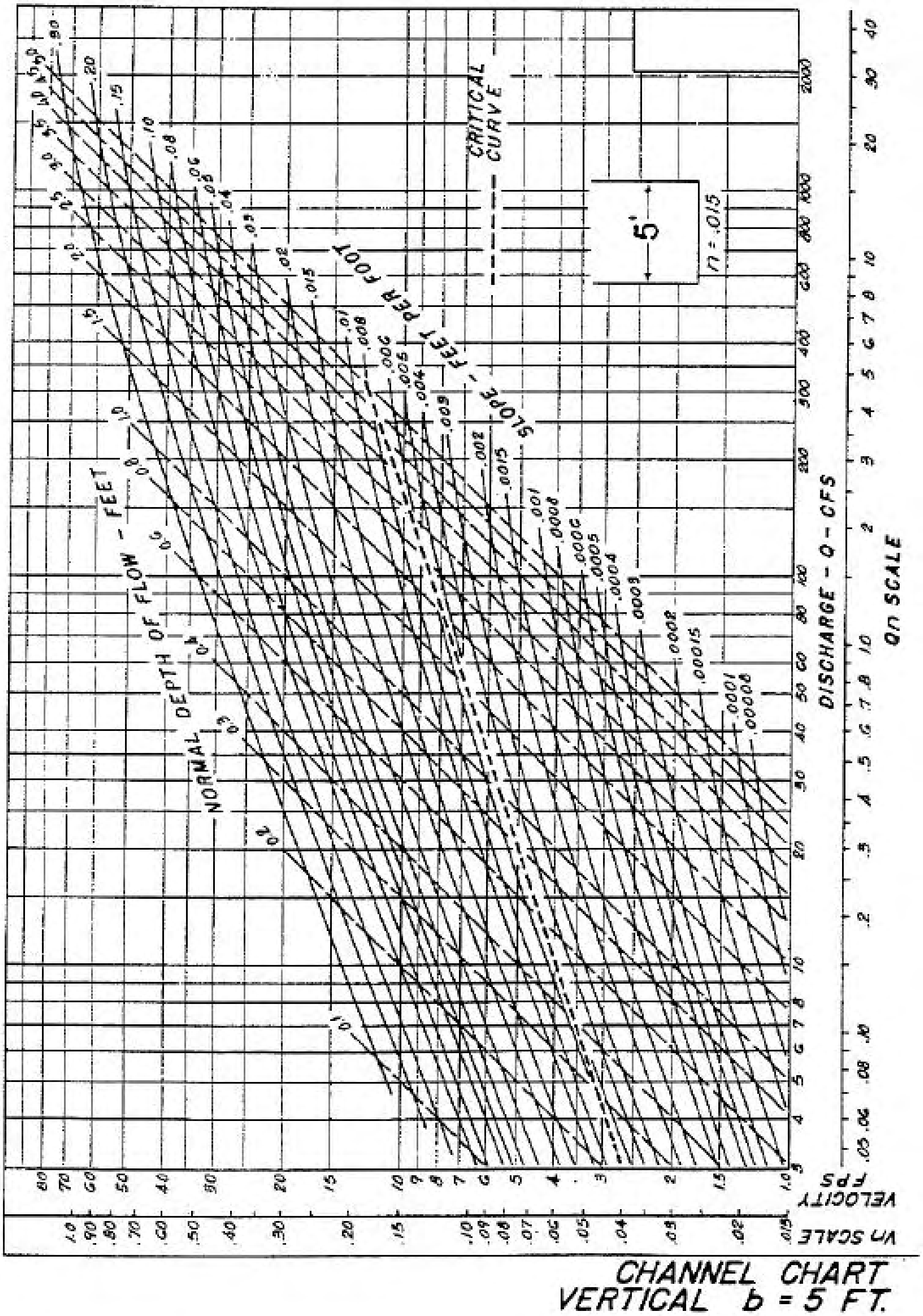
Source: HDS-3

Appendix 8C-80 Rectangular Channel Flow Chart (B=4')



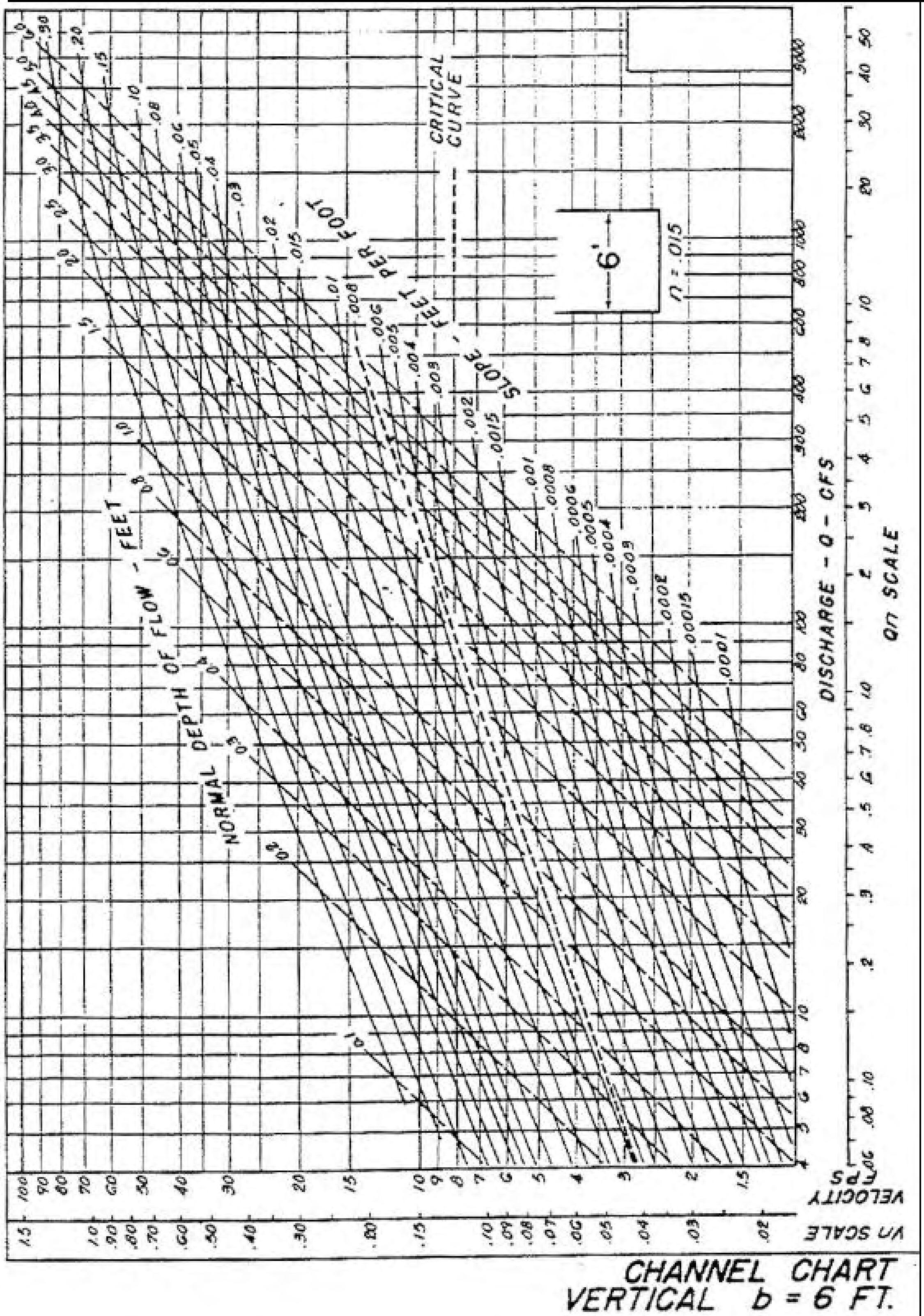
Source: HDS-3

Appendix 8C-81 Rectangular Channel Flow Chart (B=5')



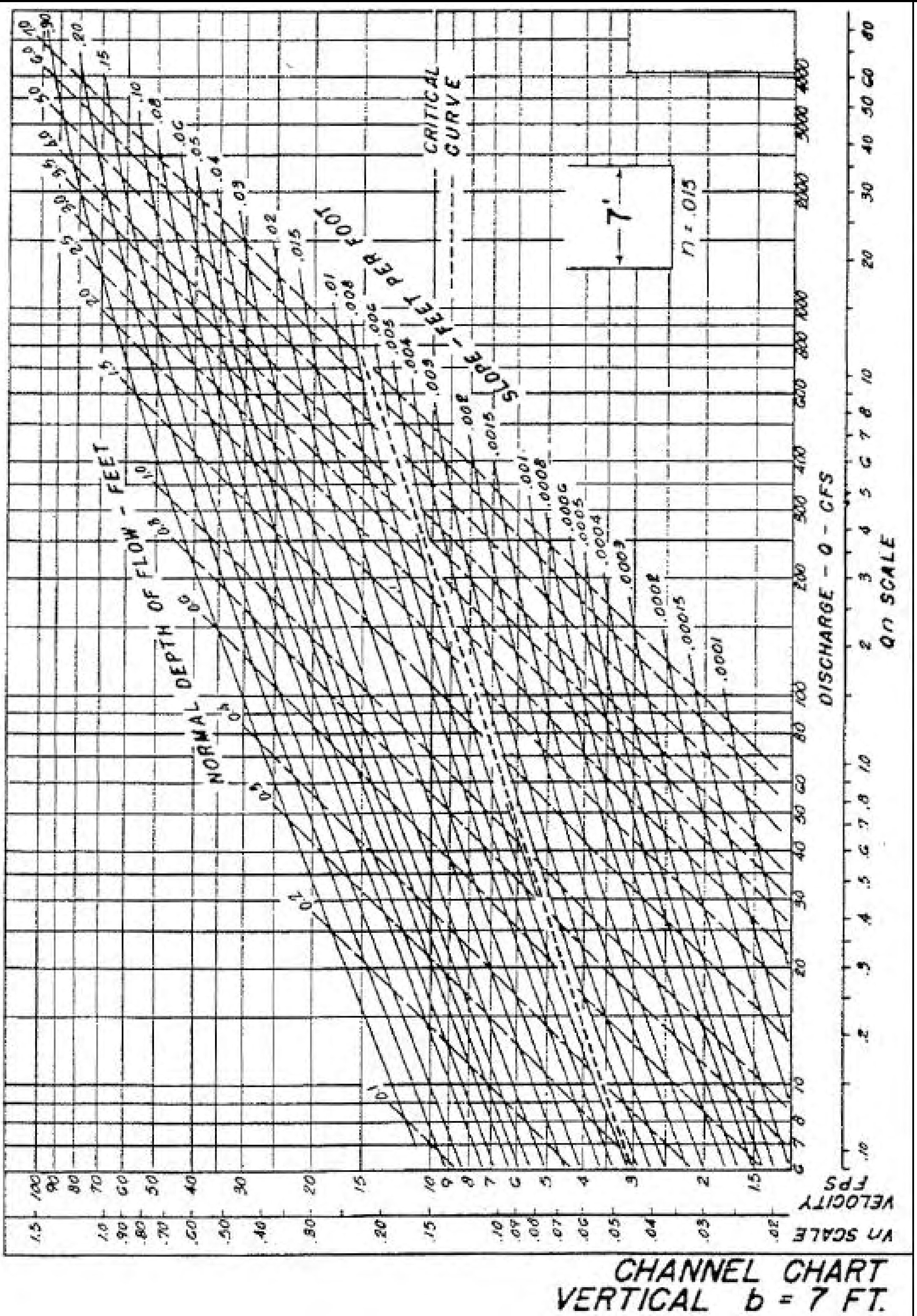
Source: HDS-3

Appendix 8C-82 Rectangular Channel Flow Chart (B=6')



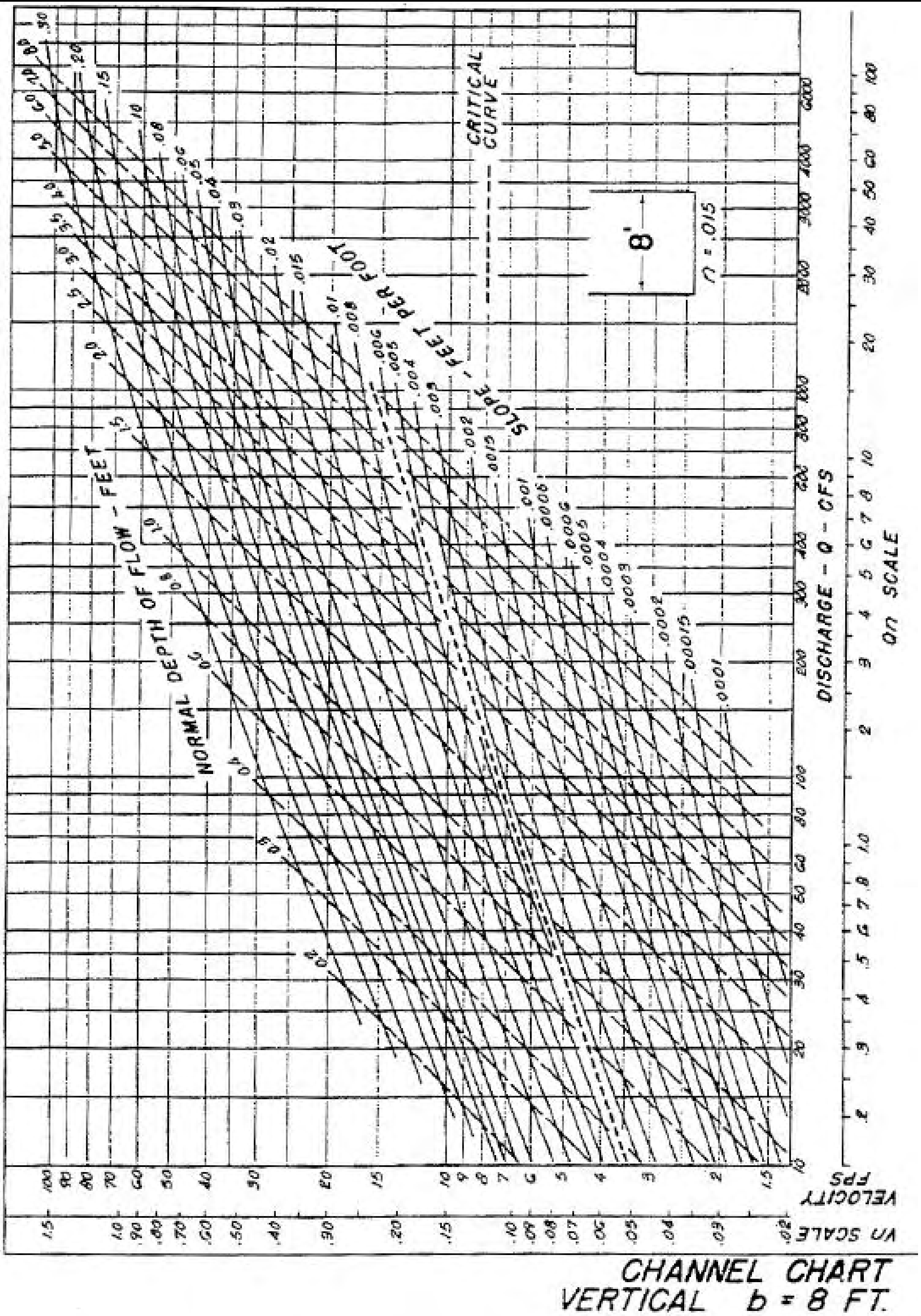
Source: HDS-3

Appendix 8C-83 Rectangular Channel Flow Chart (B=7')



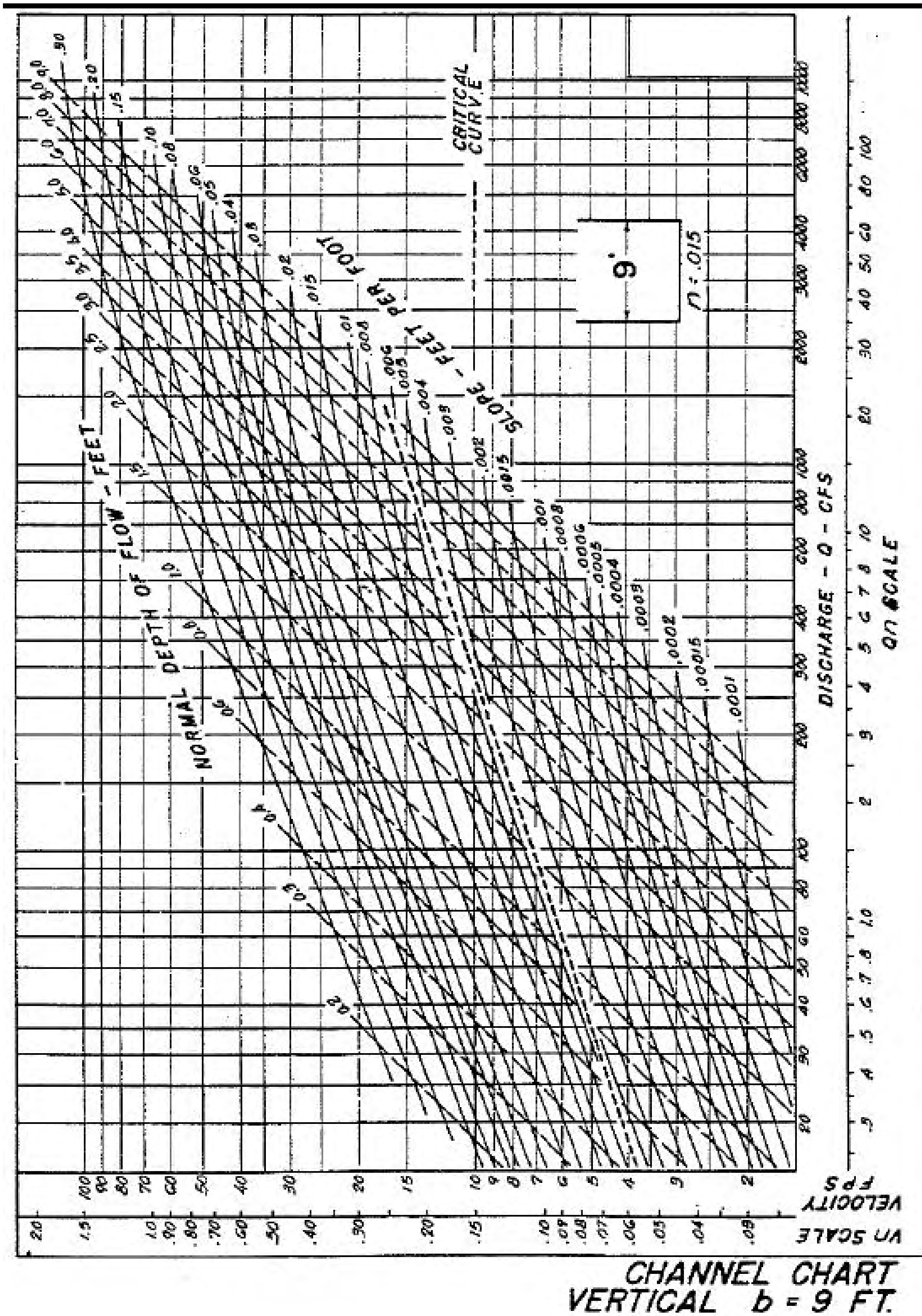
Source: HDS-3

Appendix 8C-84 Rectangular Channel Flow Chart (B=8')



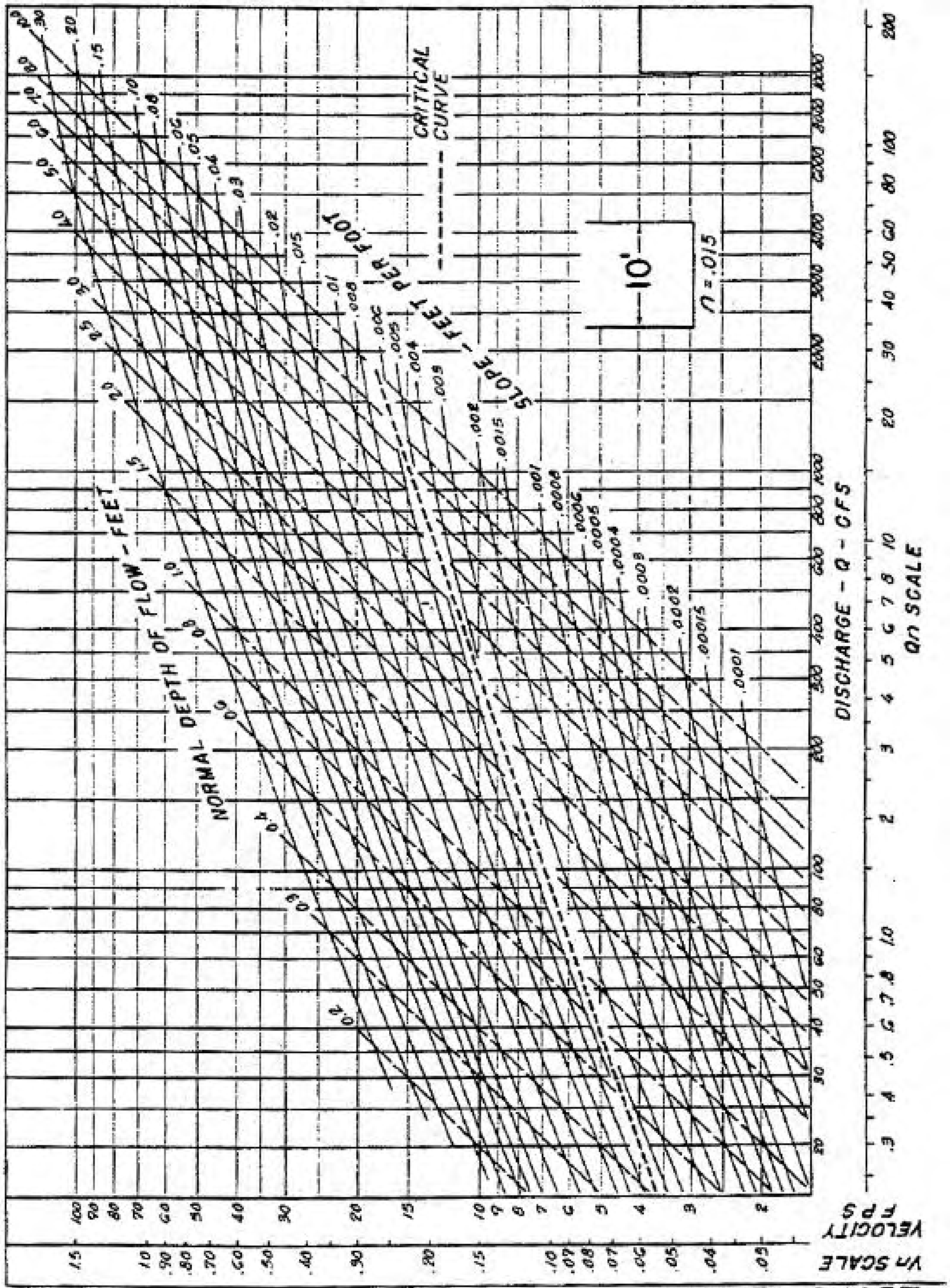
Source: HDS-3

Appendix 8C-85 Rectangular Channel Flow Chart (B=9')



Source: HDS-3

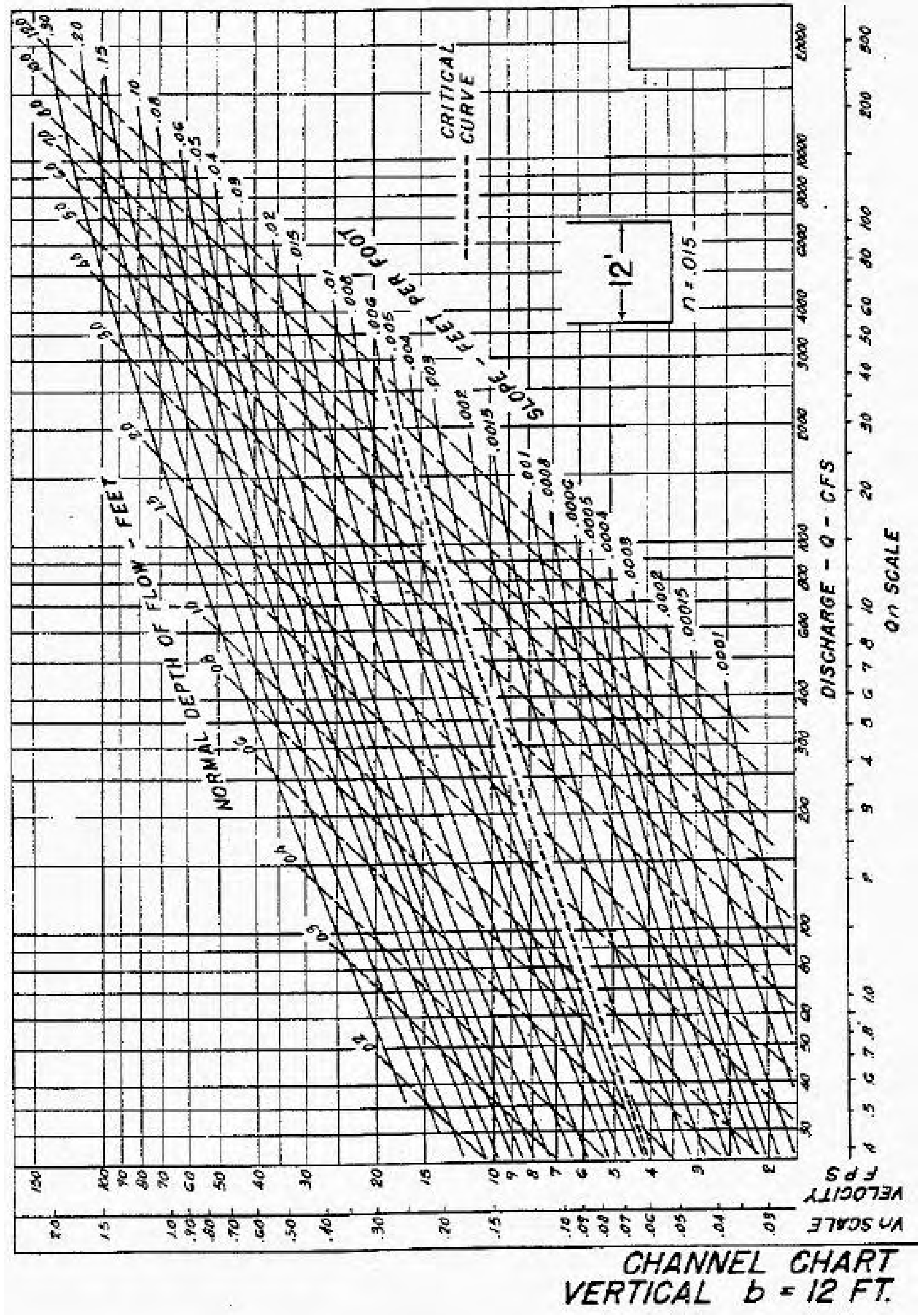
Appendix 8C-86 Rectangular Channel Flow Chart (B=10')



CHANNEL CHART
VERTICAL b = 10 FT.

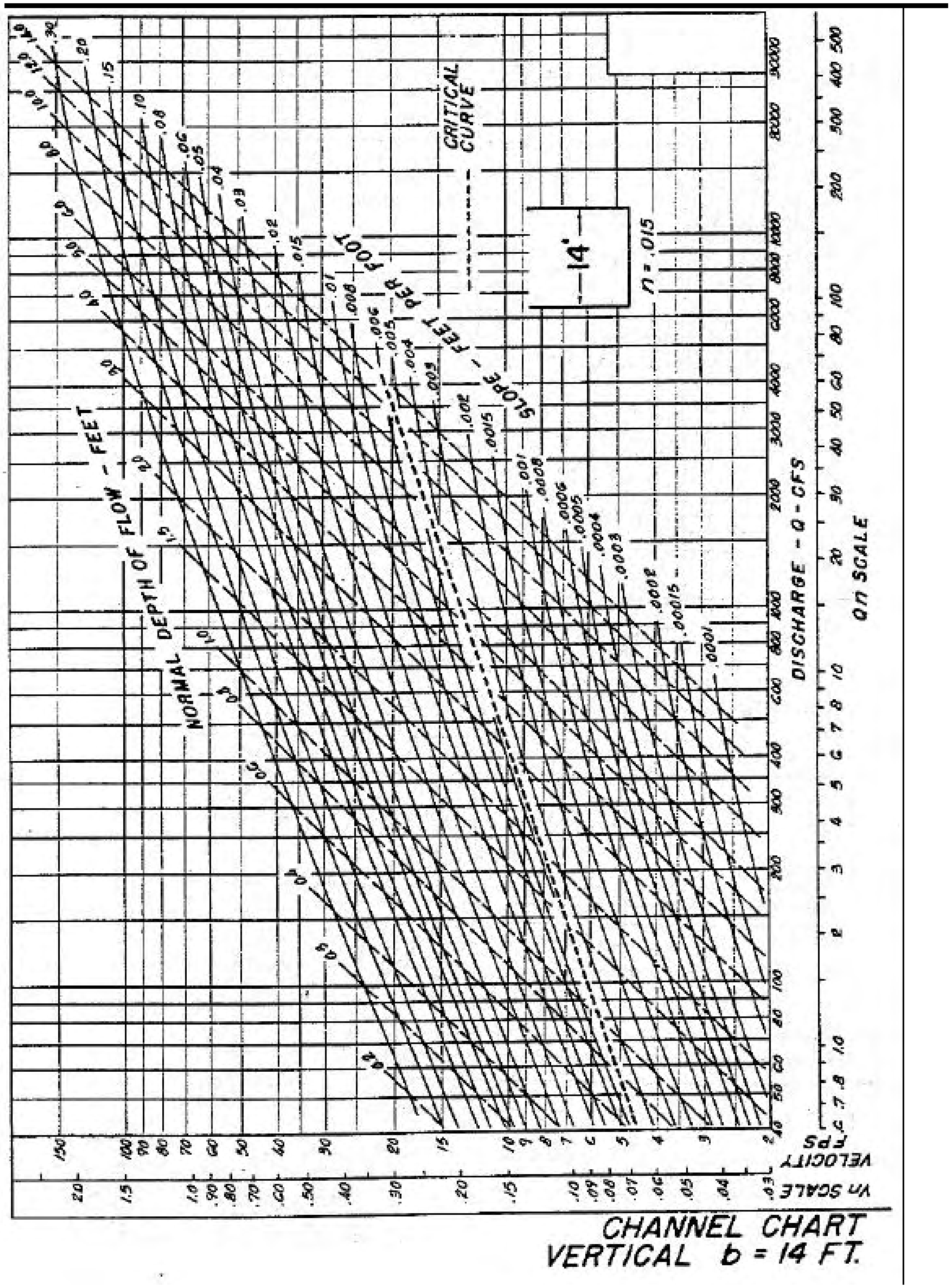
Source: HDS-3

Appendix 8C-87 Rectangular Channel Flow Chart (B=12')



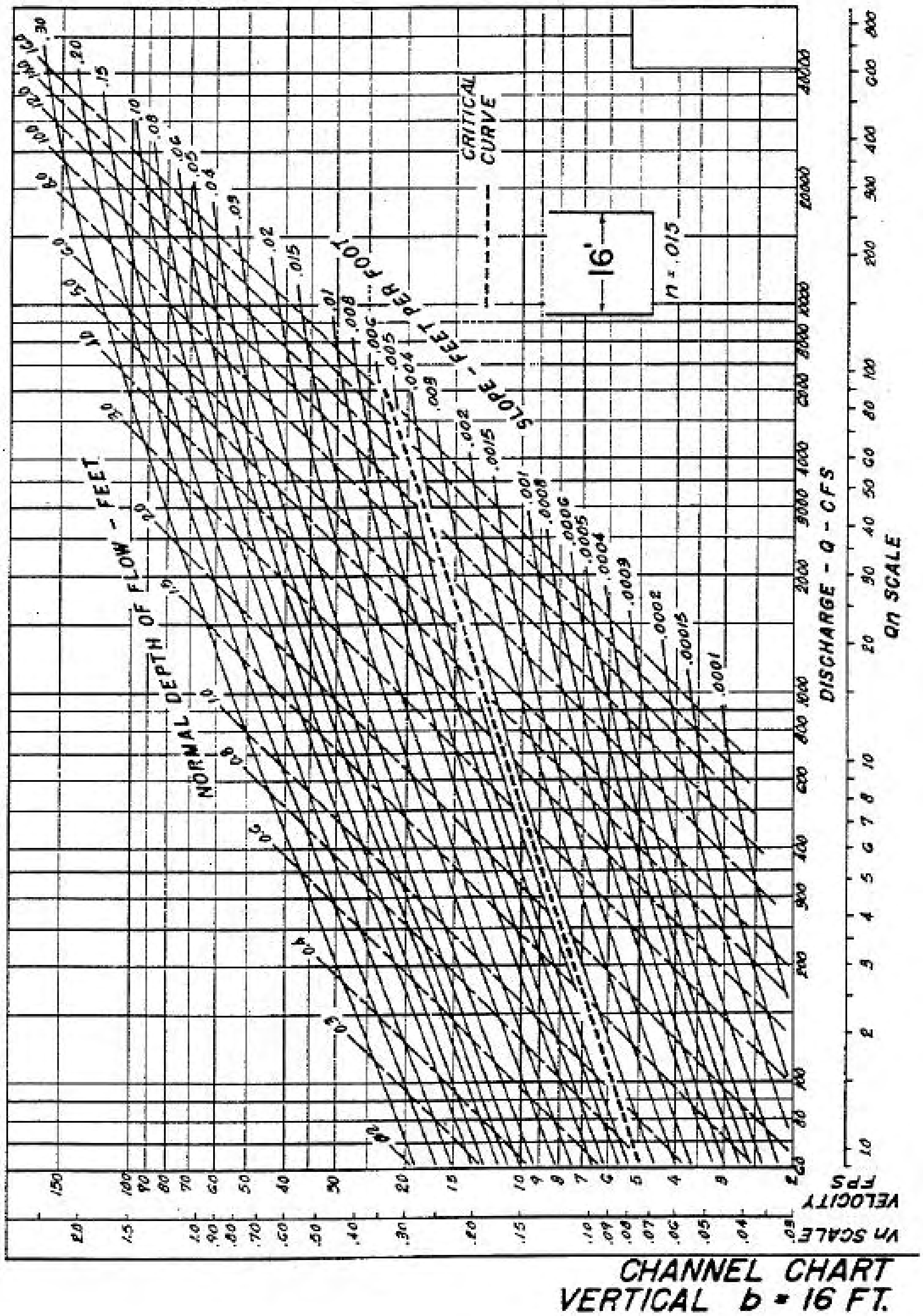
Source: HDS-3

Appendix 8C-88 Rectangular Channel Flow Chart (B=14')



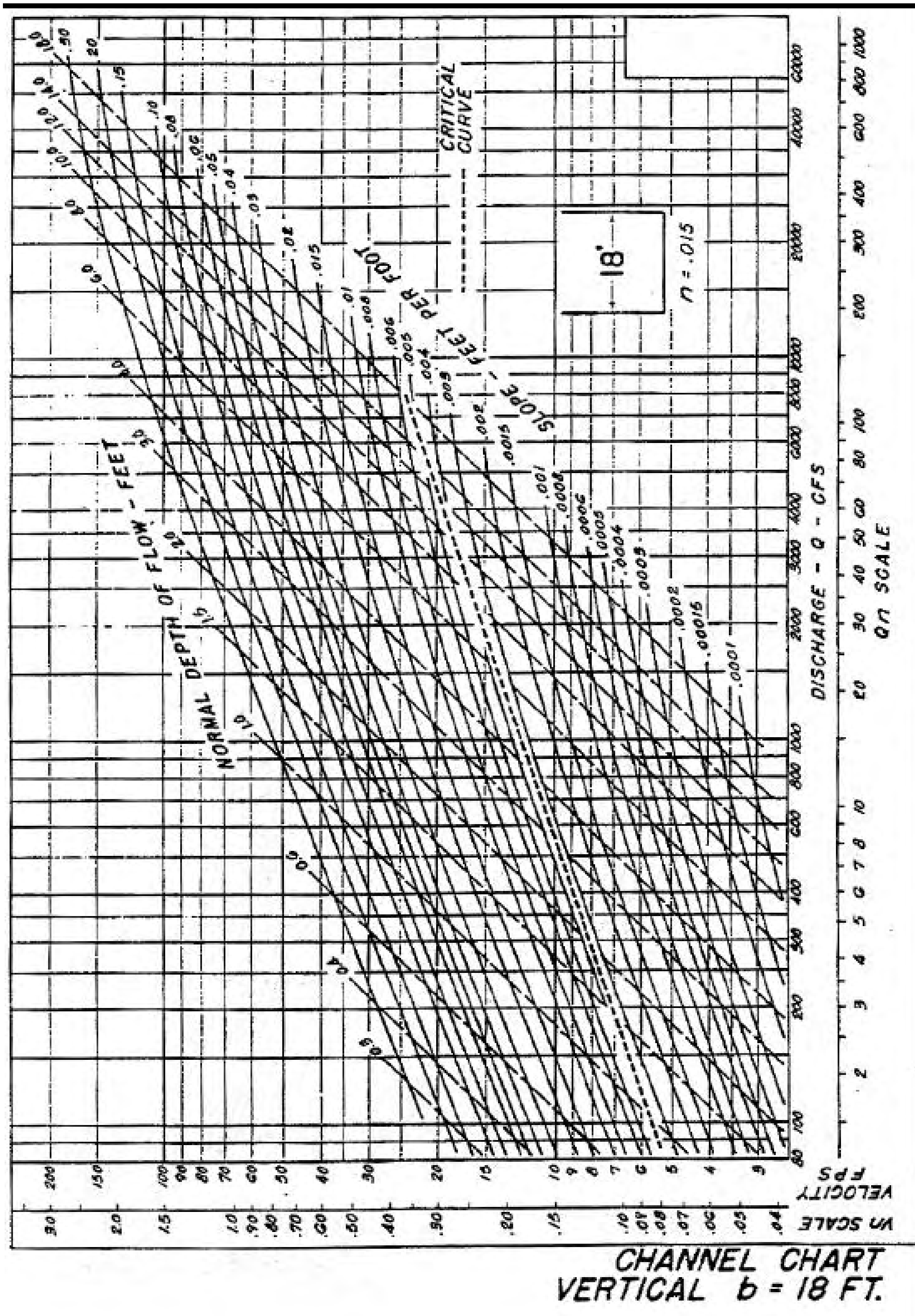
Source: HDS-3

Appendix 8C-89 Rectangular Channel Flow Chart (B=16')



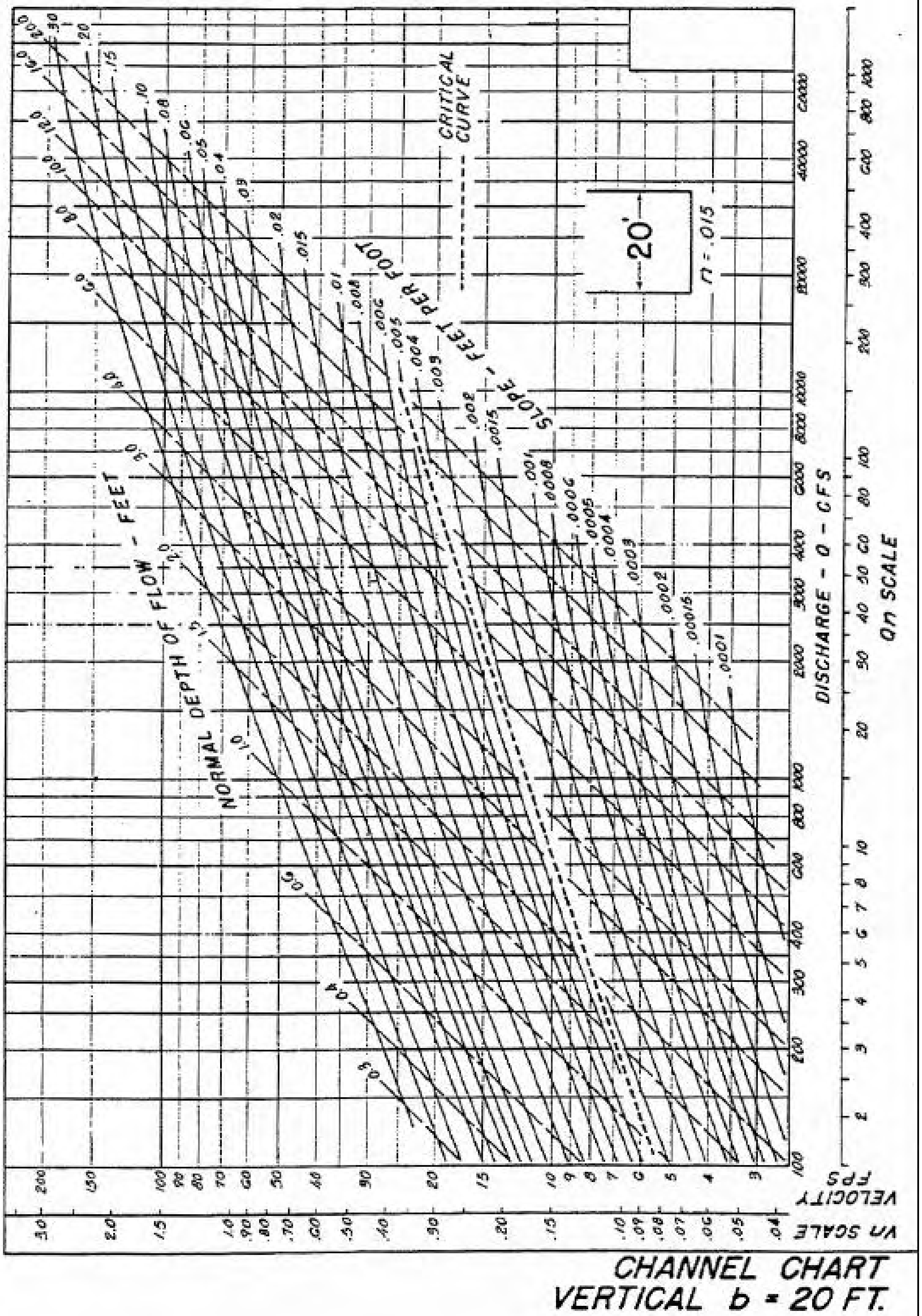
Source: HDS-3

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

Type of Conduit	Wall Description	Manning's n
Concrete Pipe	Smooth walls	0.010-0.013
Concrete Boxes	Smooth walls	0.12-0.015
Corrugated Metal Pipes and Boxes Annular or Helical Pipe (n varies Barrel size) See HDS5	2 2/3 by 1/2 inch corrugations	0.022-0.027
	6 by 1 inch corrugations	0.022-0.025
	5 by 1 inch corrugations	0.25-0.026
	3 by 1 inch corrugations	0.027-0.028
	6 by 2 inch structural plate	0.033-0.035
	9 by 2 1/2 inch structural plate	0.033-0.037
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	2 2/3 by 1/2 inch corrugations	0.12-0.024
Spiral Rib Metal	Smooth walls	0.012-0.013

*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, page 163.

Source: HDS-5

Appendix 8D-2 Entrance Loss Coefficients (K_e), Outlet Control, Full or Partly Full

Type of Structure and Design of Entrance	Coefficient
Pipe, Concrete	
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Socket end of pipe (groove-end)	0.2
Projecting from fill, socket end (groove-end)	0.2
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Mitered to conform to fill slope, paved or unpaved slope	0.7
Headwall or headwall and wingwalls square-edge	0.5
*End-Section conforming and to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Wingwalls at 10° to 25° or 30° to 75° to barrel	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel Dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Crown edge rounded to radius of 1/12 barrel Dimension, or beveled top edge	0.2
Side-or slope-tapered inlet	0.2

*Note : “End Section conforming to fill slope,” made of either metal or concrete, are the sections commonly available form manufacturers. From limited hydraulic test they are equivalent in operation to a headwall in both inlet and oulet 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.

Appendix 8E-1 Energy Dissipation

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 under Table 8-1 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 = 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 $1.5d_{max}$
Remainder = $2W_o$ or $1.5d_{max}$

Step 5: Determine exit velocity at brink (V_B)

a. Basin exit depth, d_B = critical depth at basin exit

b. Basin exit velocity, $V_B = \dfrac{Q}{W_B d_B}$

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:

$$A = \frac{\pi D_E^{\ 2}}{4} = d_o(W_o)$$

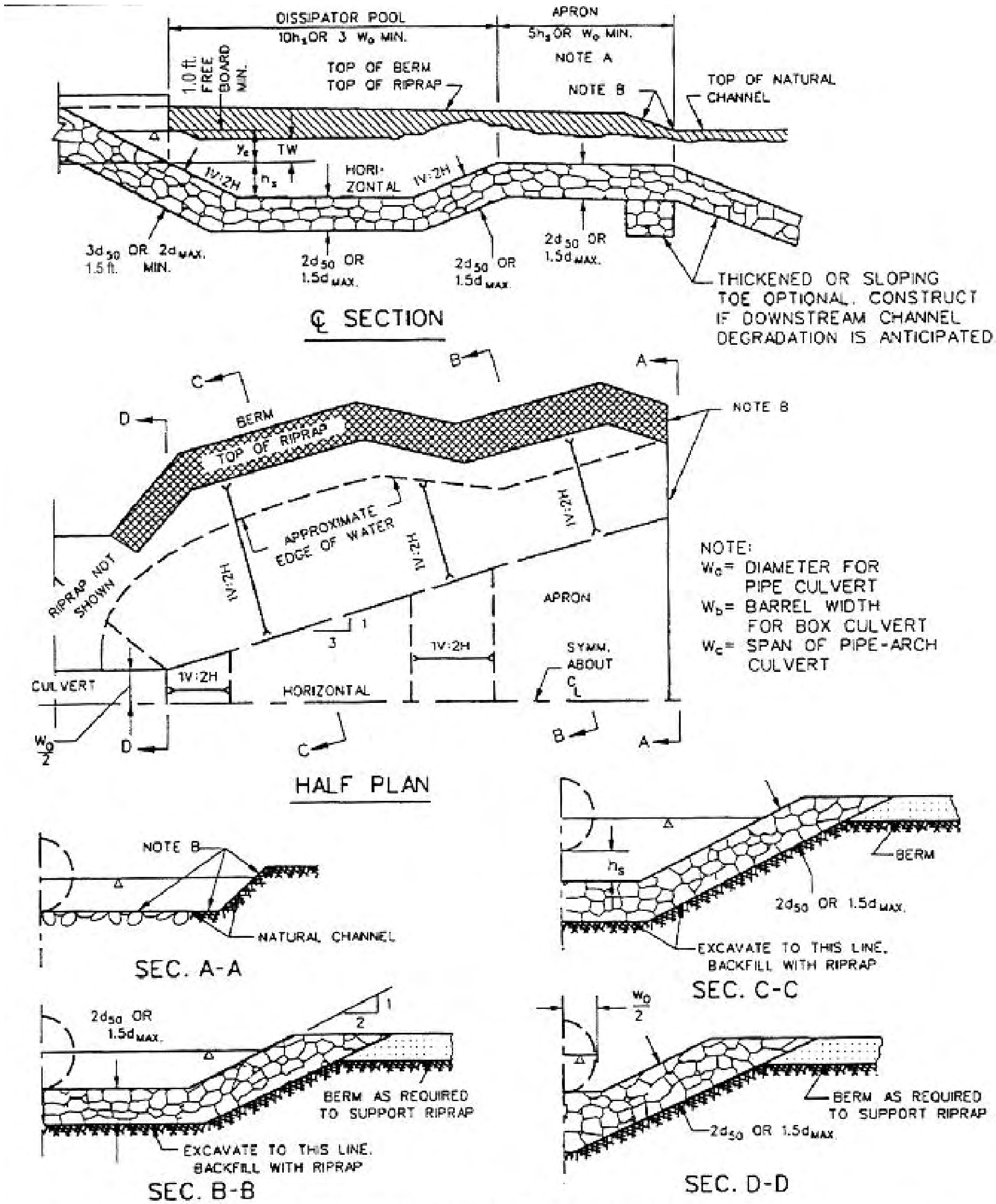
c. Estimate centerline velocity at a series of downstream cross sections using Figure 8E-4.

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 deign 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.



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_{des}/(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- 1. Details of Riprap Basin Energy Dissipator

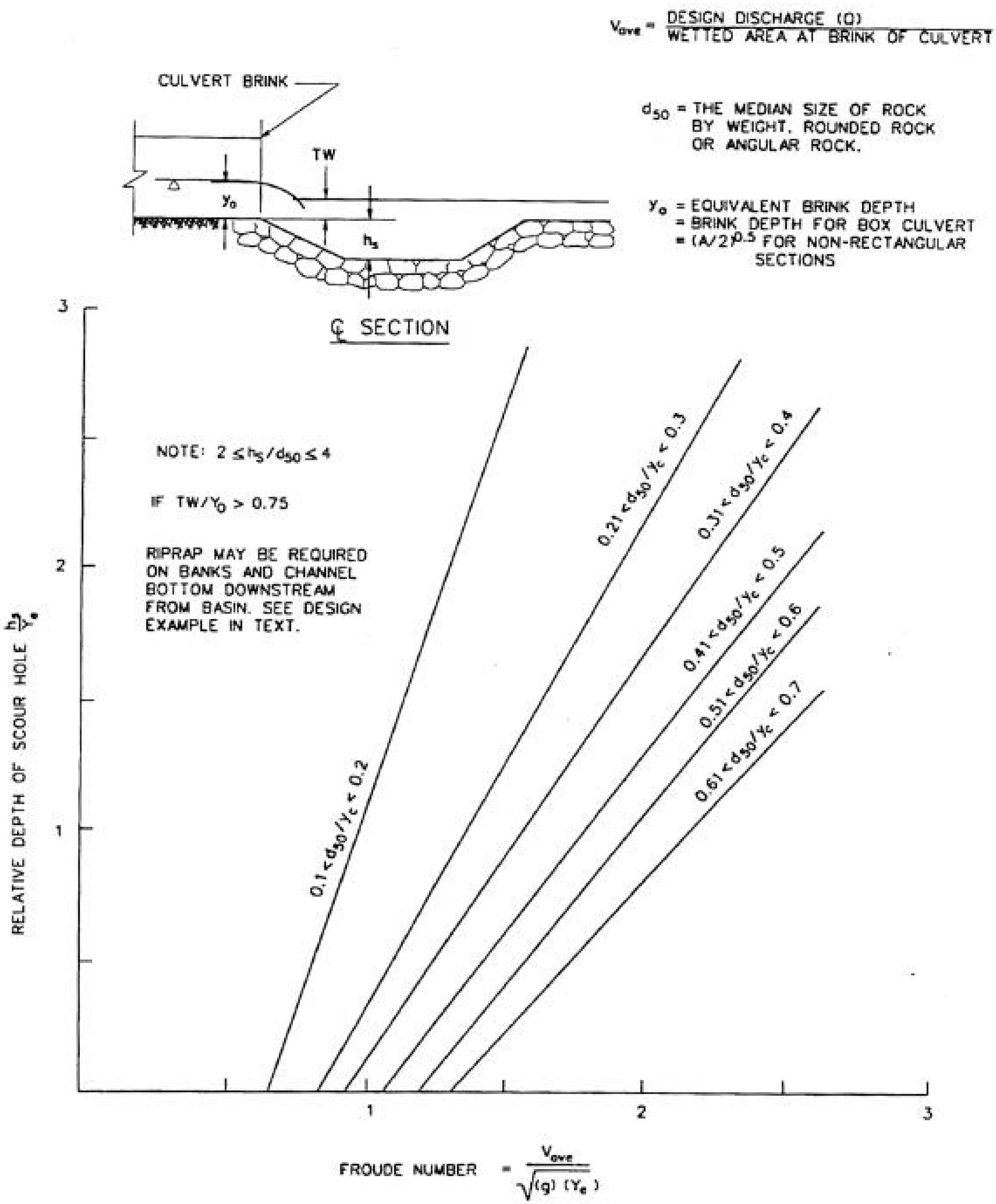


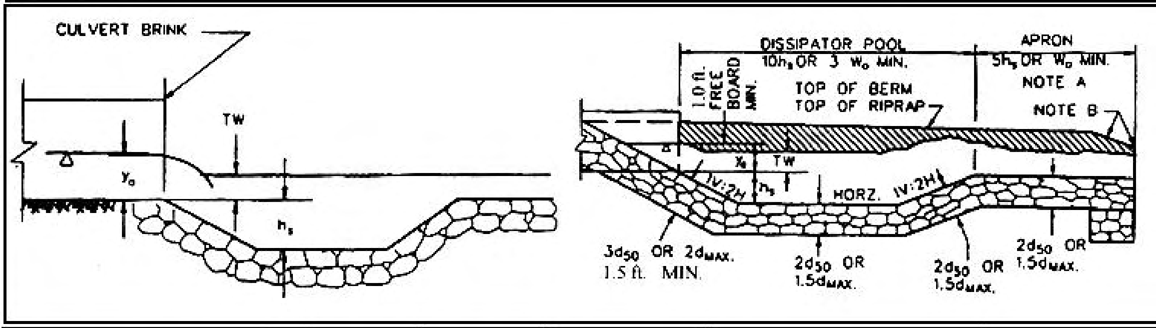
Figure 8E- 2. Riprap Basin Depth of Scour

RIPRAP BASIN

Project No. ______________________________

Designer ______________________________ Date ______________

Reviewer ______________________________ Date ______________



DESIGN VALUES (Figure 8E-2)	TRIAL 1	FINAL . TRIAL
Equi. Depth, d_E		
D_{50}/d_E		
D_{50}		
Froude No., Fr		
h_s/d_E		
h_s		
h_s/D_{50}		
$2 < h_s/D_{50} < 4$		

BASIN DIMENSIONS		FEET	
Pool length is the larger of:	$10h_s$		
	$3W_o$		
Basin length is the larger of:	$15h_s$		
	$4W_o$		
Approach Thickness	$3D_{50}$		
Basin Thickness	$2D_{50}$		

TAILWATER CHECK	
Tailwater, TW	
Equivalent depth, d_E	
TW/d_E	
IF $TW/d_E > 0.75$, calculate riprap downstream using Figure 8E-4	
$D_E = (4A_c/\pi)^{0.5}$	

DOWNSTREAM RIPRAP (Figure 8E-4)				
L/D_E	L	V_L/V_o	V_L	D_{50}
10				
15				
20				
21				

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_e 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.

$$V_o = \frac{Q}{A} = \frac{800}{4.0(8.0)} = 25\text{fps}$$

$$F_r = \frac{V_o}{\sqrt{gd_E}} = \frac{25}{\sqrt{32.2(4.0)}} = 2.2 < 3.0$$

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 $d_{50}/d_E = 0.45$

$$d_{50} = \left(\frac{d_{50}}{d_E}\right)d_E = 0.45(4.0) = 1.8 \text{ ft.}$$

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 $2<h_S/d_{50}<4$:

$$h_s = \left(\frac{h_s}{d_E}\right)d_E = 1.6(4.0) = 6.4\text{ft.}$$

$$\frac{h_s}{d_{50}} = \frac{6.4}{1.8} = 3.55\text{ft.}$$

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:

$L_S = 10h_S = 10(6.4) = 64$ ft.
L_S min.= $3W_o = 3(8) = 24$ ft
Therefore, use $L_S = 64$ ft

c. Determine length of basin, L_B:

$L_B = 15h_S = 15(6.4) = 96$ ft

L_B min. = $4W_o = 4(8) = 32$ ft

Therefore, use $L_B = 96$ ft

d. Thickness of riprap:

Approach = $3d_{50} = 3(1.80) = 5.4$ ft
Remainder = $2d_{50} = 2(1.80) = 3.6$ 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 ft.)

b. $$V_B = \frac{Q}{W_B d_B} = \frac{800}{24(3.3)} = 10\text{fps}$$

c. V_B =10 fps $<V_d$ = 18 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 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 ft, h_S = 6.4 ft
L_S = 64 ft, L_B = 96 ft

b. Compute equivalent circular diameter, D_E, for brink area from:

$$A = \frac{\pi D_E^{\ 2}}{4} = d_o(W_o) = 4.0(8.0) = 32 ft^2$$

$$D_E = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4(32)}{\pi}} = 6.4 ft.$$

V_o = 25 fps (Sample Problem 8E.2.1).

c. Estimate centerline velocity at a series of downstream cross sections using Figure 8E-5.

$\frac{L}{D_E}$ [1]	L	$\frac{V_L}{V_O}$	V_L	D_{50} [2]
10	64	0.59	14.7	1.4
15[3]	96	0.36	9.0	0.6
20	128	0.30	7.5	0.4
21	135	0.28	7.0	0.4

[1] Use $W_o = D_E$ in Figure 8E- 5.
[2] From Figure 8E- 6.
[3] Is on a logarithmic scale so interpolations must be performed logarithmically.

c. 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.

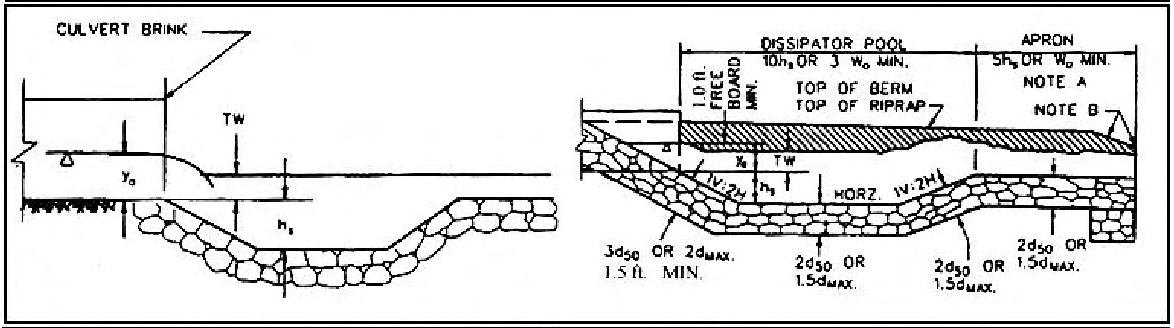
This information is summarized in the worksheet for riprap basin design, Figure 8E- 4.

RIPRAP BASIN

Project No. ______________________________

Designer ______________________________ Date ______________

Reviewer ______________________________ Date ______________



DESIGN VALUES (Figure 8E-2)	TRIAL 1	FINAL TRIAL
Equi. Depth, d_E	4.0 ft.	4.0 ft.
D_{50}/d_E	0.45	0.45
D_{50}	1.80 ft.	1.80 ft.
Froude No., Fr	2.20	2.20
h_s/d_E	1.60	1.60
h_s	6.40 ft.	6.40 ft.
h_s/D_{50}	3.55	3.55
$2 < h_s/D_{50} < 4$	OK	OK

BASIN DIMENSIONS		FEET	
Pool length is the larger of:	$10h_s$	64	64
	$3W_o$	24	
Basin length is the larger of:	$15h_s$	96	96
	$4W_o$	32	
Approach Thickness	$3D_{50}$	5.4	
Basin Thickness	$2D_{50}$	3.6	

TAILWATER CHECK	
Tailwater, TW	4.2
Equivalent depth, d_E	4.0
TW/d_E	1.05
IF $TW/d_E > 0.75$, calculate riprap downstream using Figure 8E-4	
$D_E = (4A_c/\pi)^{0.5}$	

DOWNSTREAM RIPRAP (Figure 8E-4)				
L/D_E	L	V_L/V_o	V_L	D_{50}
10	64	0.59	14.7	1.4
15	96	0.37	9.0	0.6
20	128	0.30	7.5	0.4
21	135	0.28	7.0	0.4

Figure 8E- 4. Riprap Basin Design Worksheet, Sample Problem

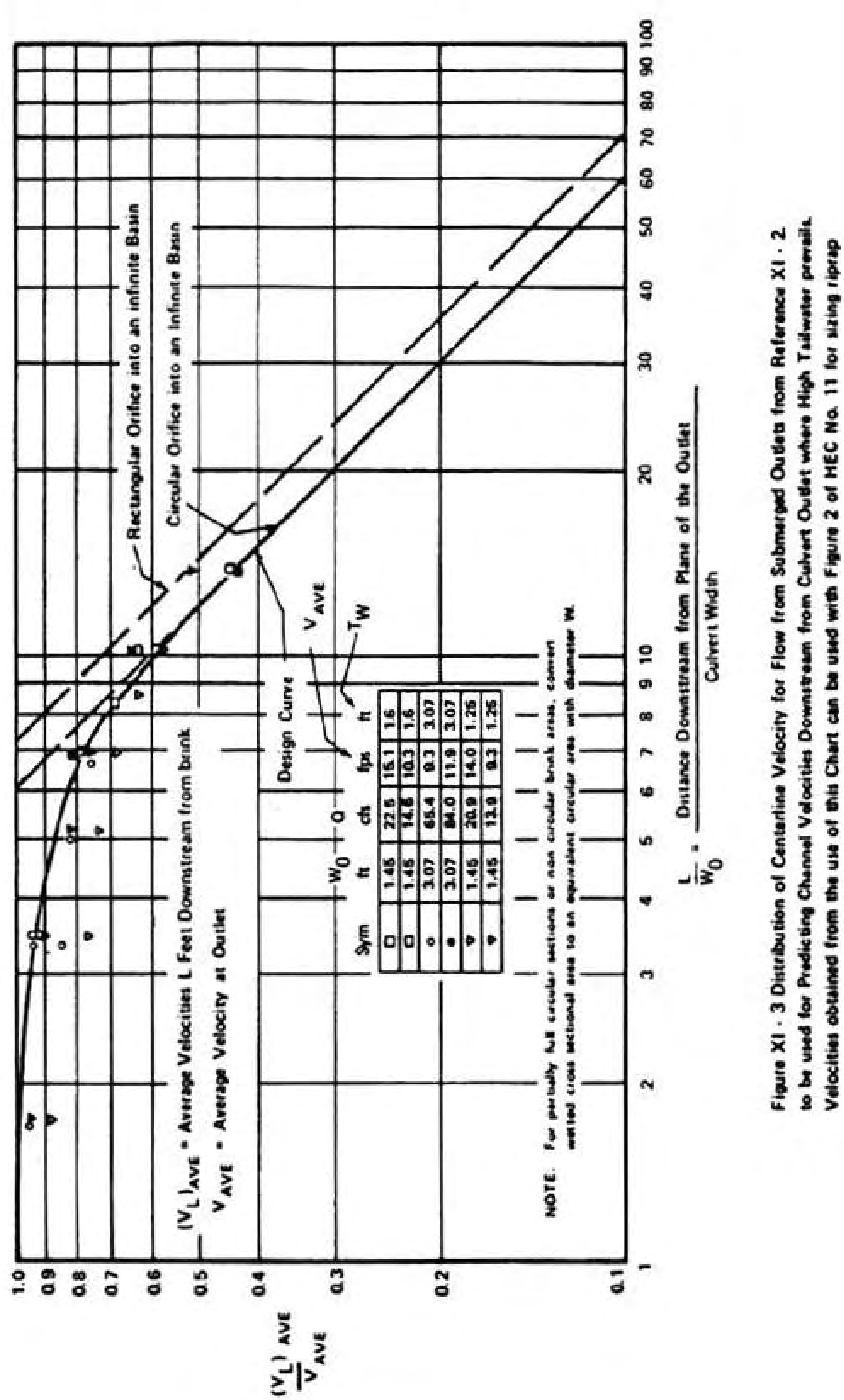


Figure 8E- 5. Distribution of Centerline Velocity for Flow from Submerged Outlets

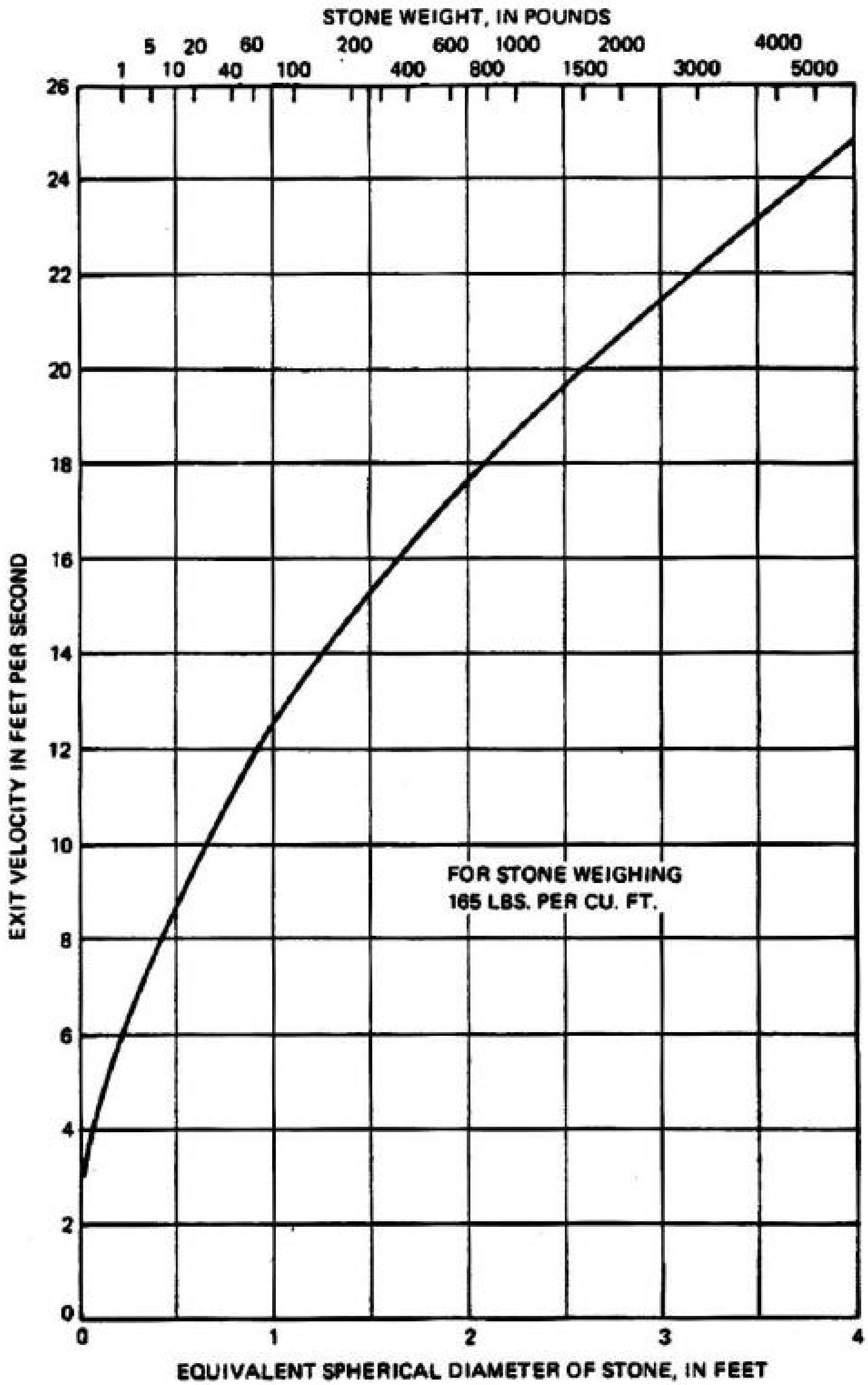


Figure 8E- 6. Riprap Size Versus Exit Velocity

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.

FHWA CULVERT ANALYSIS, HY-8, VERSION 6.0

CURRENT DATE	CURRENT TIME	FILE NAME	FILE DATE
06-02-1997	15:23:59	ENERGY3	06-02-1997

CULVERT AND CHANNEL DATA

CULVERT NO. 1	DOWNSTREAM CHANNEL
CULVERT TYPE: 8.0 ft X 6.0 ft, BOX	CHANNEL TYPE: IRREGULAR
CULVERT LENGTH = 300 ft	BOTTOM WIDTH = 8.0 ft
NO. OF BARRELS = 1.0	TAILWATER DEPTH = 2.8 ft
FLOW PER BARREL= 400 cfs	TOTAL DESIGN FLOW = 400 cfs
INVERT ELEVATION = 172.5 ft	BOTTOM ELEVATION = 172.5 ft
OUTLET VELOCITY = 25 fps	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 (2⅔"x½" Corrugations)

Table 1-3 Handling Weight of Corrugated Steel Pipe (2 2/3 x 1/2 in)
Estimated Average Weights – Not for Specification Use*

Inside Diameter In Inches	Specified Thickness In Inches	Approximate Pounds per Lineal Foot ** Galvanized	Full-Coated	Full-Coated and Invert Paved	Full-Coated and Full Paved
12	.052	8	10	13	
	.064	10	12	15	
	.079	12	14	17	
15	.052	10	12	15	
	.064	12	15	18	
	.079	15	18	21	
18	.052	12	14	17	
	.064	15	19	22	
	.079	18	22	25	
21	.052	14	16	19	
	.064	17	21	26	
	.079	21	25	30	
24	.052	15	17	20	
	.064	19	24	30	45
	.079	24	29	35	60
30	.052	20	22	25	
	.064	24	30	36	55
	.079	30	36	42	60
36	.052	24	26	29	
	.064	29	36	44	65
	.079	36	43	51	75
42	.052	28	30	33	
	.064	34	42	51	
	.079	42	50	59	85
48	.052	31	33	36	
	.064	38	48	57	
	.079	48	58	67	95
54	.064	44	55	66	95
	.079	54	65	76	105
60	.079	60	71	85	
	.109	81	92	106	140
66	.109	89	101	117	160
	.138	113	125	141	180
72	.109	98	112	129	170
	.138	123	137	154	210
78	.109	105	121	138	200
	.138	133	149	166	230
84	.109	113	133	155	225
	.138	144	161	179	240
90	.109	121	145	167	
	.138	154	172	192	
	.168	186	204	224	
96	.138	164	191	217	
	.168	198	217	239	

* 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, 2⅔ x ½ 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)

Table 1-4 **Handling Weight of Corrugated Steel Pipe (3 x 1 In or 125 x 25 mm)*****
Estimated Average Weights—Not for Specification Use*

Inside Diameter In Inches	Specified Thickness In Inches	Approximate Pounds per Lineal Foot ** Galvanized	Full-Coated	Full-Coated and Invert Paved	Full-Coated and Full Paved
54	.064	50	66	84	138
	.079	61	77	95	149
60	.064	55	73	93	153
	.079	67	86	105	165
66	.064	60	80	102	168
	.079	74	94	116	181
72	.064	66	88	111	183
	.079	81	102	126	197
78	.064	71	95	121	198
	.079	87	111	137	214
84	.064	77	102	130	213
	.079	94	119	147	230
90	.064	82	109	140	228
	.079	100	127	158	246
96	.064	87	116	149	242
	.079	107	136	169	262
102	.064	93	124	158	258
	.079	114	145	179	279
108	.064	98	131	166	273
	.079	120	153	188	295
114	.064	104	139	176	289
	.079	127	162	199	312
120	.064	109	146	183	296
	.079	134	171	210	329
	.109	183	220	259	378
126	.079	141	179	220	346
	.109	195	233	274	400
132	.079	148	188	231	363
	.109	204	244	287	419
138	.079	154	196	241	379
	.109	213	255	300	438
144	.109	223	267	314	458
	.138	282	326	373	517

* Lock seam construction only; weights will vary with other fabrication practices.

** For other coatings or linings the weights may be interpolated.

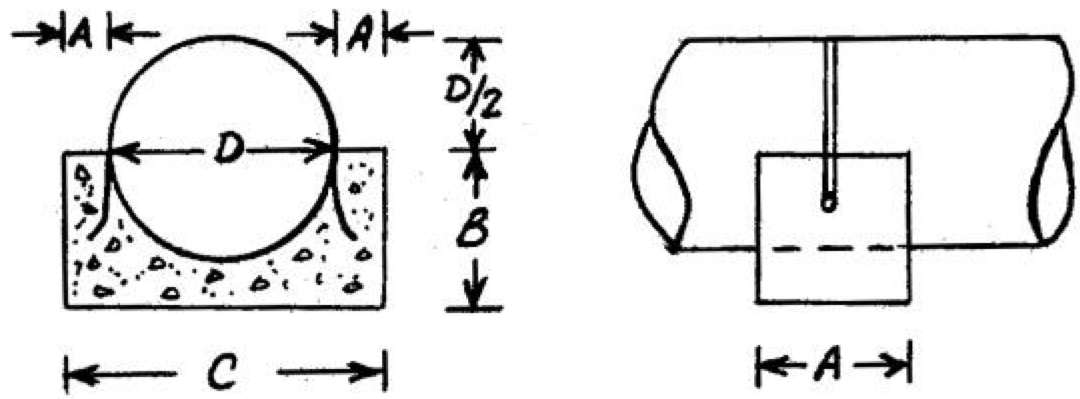
*** 125 x 25mm may be referred to as 5 x 1in. and weighs approximately 12% less than 3 x 1in.

Note: Pipe arch weights will be the same as the equivalent round pipe.
For example; for 42 x 29, 2⅔ x ½ 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-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 = PIPE DIAMETER

* WEIGHT OF CONCRETE @ 150 LBS. PER CU. FT.

Pipe Diameter (inches)	Dimensions (inches)			Concrete	
D	A	B	C	Volume (cu. ft.)	Weight* (lbs.)
12	6	18	24	1.30	195
15	6	19.5	27	1.52	228
18	6	21	30	1.75	263
24	6	24	36	2.22	333
30	6	27	42	2.71	407
36	6	30	48	3.23	485
42	6	33	54	3.78	567
48	6	36	60	4.36	654
54	6	39	66	4.96	744
60	6	42	72	5.59	839
66	6	45	78	6.25	938
72	6	48	84	6.93	1040

Source:

Appendix 8F-4 Diameter Dimensions and $D^{2.5}$ Values for Structural Plate Corrugated Circular Pipe (9"x2½" Aluminum Corrugations)

Diameter (feet)		$D^{2.5}$	Plates per Ring
Nominal	Actual		
6.5	6.42	104.4	2
7.0	6.93	126.4	2
7.5	7.44	151.0	3
8.0	7.96	178.8	3
8.5	8.46	208.2	3
9.0	8.97	241.0	3
9.5	9.48	276.7	3
10.0	9.99	315.4	3
10.5	10.50	357.2	3
11.0	11.01	402.2	4
11.5	11.52	450.4	4
12.0	12.04	503.0	4
12.5	12.54	556.9	4
13.0	13.05	615.2	4
13.5	13.57	678.3	4
14.0	14.08	743.9	4
14.5	14.59	813.1	5
15.0	15.10	886.0	5

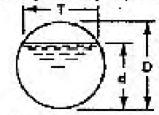
Source:

Appendix 8F-5 Geometric Properties and Critical Flow Factors for Circular Conduits Flowing Full and Partly Full

Table 4. – Geometric properties and critical flow factors for circular conduits flowing full and partly full

d = Depth of flow
d_c = Critical depth
d_m = Mean depth
D = Diameter of pipe
A = Area of flow
R = Hydraulic radius
T = Top width of flow

Q_c = Discharge at a critical flow condition
H_c = Specific head at critical flow
$H_c = d_c + (\alpha V_c^2)/(2g)$ (invariant with α)
V_c = Critical velocity
α = Kinetic energy correction factor
g = Acceleration due to gravity = 32.16 ft./sec./sec.



$\frac{d}{D}$ or $\frac{d_c}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{T}{D}$	$\frac{d_m}{D}$	$\frac{Q_c}{D^{5/2}}$ $\alpha=1.00$	$\alpha=1.04$	$\alpha=1.12$	$\frac{\alpha V_c^2}{2gD}$ all α	$\frac{H_c}{D}$ all α
1.00	0.7854	0.2500	—	—	—	—	—	—	—
0.99	.7841	.2666	0.1990	—	—	—	—	—	—
.98	.7817	.2735	.2800	—	—	—	—	—	—
.97	.7785	.2787	.3412	2.2817	6.6695	6.5400	6.3021	1.1410	2.1110
.96	.7749	.2829	.3919	1.9773	6.1785	6.0585	5.8381	0.9883	1.9483
.95	.7707	.2865	.4359	1.7681	5.8119	5.6991	5.4917	.8840	1.8840
.94	.7662	.2895	.4750	1.6131	5.5182	5.4111	5.2142	.8063	1.7463
.93	.7612	.2921	.5103	1.4917	5.2727	5.1703	4.9822	.7459	1.6759
.92	.7560	.2944	.5426	1.3933	5.0602	4.9620	4.7814	.6965	1.6165
.91	.7504	.2963	.5724	1.3119	4.8724	4.7778	4.6040	.6555	1.5655
.90	.7445	.2980	.6000	1.2408	4.7033	4.6120	4.4442	.6205	1.5205
.89	.7384	.2995	.6258	1.1799	4.5486	4.4603	4.2980	.5899	1.4799
.88	.7320	.3007	.6499	1.1263	4.4057	4.3202	4.1630	.5633	1.4433
.87	.7254	.3018	.6726	1.0785	4.2722	4.1893	4.0369	.5393	1.4093
.86	.7186	.3026	.6940	1.0354	4.1466	4.0661	3.9182	.5177	1.3777
.85	.7115	.3033	.7142	0.9962	4.0276	3.9494	3.8057	.4982	1.3482
.84	.7043	.3038	.7332	.9606	3.9144	3.8384	3.6988	.4803	1.3202
.83	.6969	.3041	.7513	.9276	3.8062	3.7323	3.5965	.4637	1.2937
.82	.6893	.3043	.7684	.8971	3.7021	3.6302	3.4982	.4484	1.2684
.81	.6815	.3043	.7846	.8686	3.6020	3.5321	3.4036	.4343	1.2443
.80	.6736	.3042	.8000	.8420	3.5051	3.4370	3.3120	.4209	1.2209
.79	.6655	.3039	.8146	.8170	3.4111	3.3449	3.2232	.4084	1.1984
.78	.6573	.3036	.8285	.7934	3.3200	3.2555	3.1371	.3966	1.1766
.77	.6489	.3031	.8417	.7709	3.2314	3.1687	3.0534	.3855	1.1555
.76	.6405	.3024	.8542	.7498	3.1450	3.0839	2.9717	.3749	1.1349
0.75	0.6319	0.3017	0.8660	0.7297	3.0606	3.0012	2.8920	0.3648	1.1148
.74	.6231	.3008	.8773	.7102	2.9783	2.9206	2.8142	.3552	1.0952
.73	.6143	.2998	.8879	.6919	2.8977	2.8414	2.7581	.3459	1.0759
.72	.6054	.2987	.8980	.6742	2.8188	2.7641	2.6635	.3371	1.0571
.71	.5964	.2975	.9075	.6572	2.7416	2.6884	2.5906	.3285	1.0385
.70	.5872	.2962	.9155	.6407	2.6656	2.6138	2.5188	.3204	1.0204
.69	.5780	.2948	.9250	.6249	2.5912	2.5409	2.4485	.3125	1.0025
.68	.5687	.2933	.9330	.6095	2.5182	2.4693	2.3795	.3048	0.9848
.67	.5594	.2917	.9404	.5949	2.4465	2.3990	2.3117	.2974	.9674
.66	.5499	.2900	.9474	.5804	2.3760	2.3299	2.2451	.2902	.9502
.65	.5404	.2882	.9539	.5665	2.3068	2.2620	2.1797	.2833	.9333
.64	.5308	.2862	.9600	.5529	2.2386	2.1951	2.1153	.2765	.9165
.63	.5212	.2842	.9656	.5398	2.1717	2.1295	2.0521	.2699	.8999
.62	.5115	.2821	.9708	.5299	2.1058	2.0649	1.9898	.2635	.8835
.61	.5018	.2799	.9755	.5144	2.0410	2.0014	1.9286	.2572	.8672
.60	.4920	.2776	.9798	.5021	1.9773	1.9389	1.8684	.2511	.8511
.59	.4822	.2753	.9837	.4902	1.9147	1.8775	1.8092	.2451	.8351
.58	.4724	.2728	.9871	.4786	1.8531	1.8171	1.7510	.2393	.8153
.57	.4625	.2703	.9902	.4671	1.7924	1.7576	1.6937	.2335	.8035
.56	.4526	.2676	.9928	.4559	1.7328	1.6992	1.6373	.2279	.7879
.55	.4425	.2649	.9950	.4448	1.6741	1.6416	1.5819	.2224	.7724
.54	.4327	.2621	.9968	.4341	1.6166	1.5852	1.5275	.2170	.7570
.53	.4227	.2592	.9982	.4235	1.5598	1.5295	1.4739	.2117	.7417
.52	.4127	.2562	.9992	.4130	1.5041	1.4749	1.4212	.2065	.7266
.51	.4027	.2531	.9998	.4028	1.4494	1.4213	1.3696	.2014	.7114
.50	.3927	.2500	1.0000	.3927	1.3956	1.3685	1.3187	.1964	.6964
.49	.3827	.2468	.9998	.3828	1.3427	1.3166	1.2687	.1914	.6814
.48	.3727	.2435	.9992	.3730	1.2908	1.2657	1.2197	.1865	.6665
.47	.3627	.2401	.9982	.3634	1.2400	1.2159	1.1717	.1817	.6517
.46	.3527	.2366	.9968	.3538	1.1900	1.1669	1.1244	.1770	.6370
.45	.3428	.2331	.9950	.3445	1.1410	1.1188	1.0781	.1722	.6222
.44	.3328	.2295	.9928	.3352	1.0929	1.0717	1.0327	.1677	.6077
.43	.3229	.2258	.9902	.3261	1.0459	1.0256	0.9883	.1631	.5931
.42	.3130	.2220	.9871	.3171	0.9997	0.9803	.9446	.1586	.5786
.41	.3032	.2182	.9837	.3082	.9546	.9361	.9020	.1541	.5641
.40	.2934	.2142	.9799	.2994	.9104	.8927	.8602	.1497	.5497
.39	.2836	.2102	.9755	.2907	.8672	.8504	.8194	.1454	.5354
.38	.2739	.2062	.9708	.2821	.8249	.8089	.7795	.1410	.5210
.37	.2642	.2020	.9656	.2736	.7836	.7684	.7404	.1368	.5068
.36	.2546	.1978	.9600	.2652	.7433	.7289	.7024	.1325	.4925
.35	.2450	.1935	.9539	.2568	.7040	.6903	.6652	.1284	.4784
.34	.2355	.1891	.9474	.2486	.6657	.6528	.6290	.1242	.4642
.33	.2260	.1847	.9404	.2403	.6284	.6162	.5938	.1202	.4502
.32	.2167	.1802	.9330	.2323	.5921	.5806	.5595	.1151	.4361
.31	.2074	.1756	.9250	.2242	.5569	.5461	.5262	.1121	.4221
.30	.1982	.1709	.9165	.2163	.5226	.5125	.4958	.1031	.4081
.29	.1890	.1662	.9075	.2083	.4893	.4798	.4623	.1042	.3942
.28	.1800	.1514	.8980	.2004	.4571	.4482	.4319	.1003	.3803
.27	.1711	.1566	.8879	.1927	.4259	.4175	.4024	.0963	.3663
.26	.1623	.1516	.8773	.1850	.3957	.3880	.3739	.0924	.3524
.25	.1535	.1466	.8660	.1773	.3667	.3596	.3465	.0887	.3387
.24	.1449	.1416	.8542	.1696	.3386	.3320	.3199	.0849	.3249
.23	.1365	.1364	.8417	.1622	.3116	.3055	.2944	.0810	.3110
.22	.1281	.1312	.8285	.1546	.2857	.2802	.2700	.0773	.2973
.21	.1199	.1259	.8145	.1472	.2609	.2558	.2465	.0736	.2836
0.20	0.1118	0.1206	0.8000	0.1397	0.2371	0.2325	0.2240	0.0699	0.2699
.19	.1039	.1152	.7846	.1324	.2144	.2102	.2026	.0662	.2562
.18	.0961	.1097	.7684	.1251	.1928	.1891	.1822	.0626	.2426
.17	.0885	.1042	.7513	.1178	.1724	.1691	.1629	.0590	.2290
.16	.0811	.0986	.7332	.1106	.1530	.1500	.1446	.0553	.2153
.15	.0739	.0929	.7142	.1035	.1347	.1321	.1272	.0516	.2016
.14	.0668	.0871	.6940	.0963	.1176	.1153	.1111	.0482	.1882
.13	.0600	.0813	.6726	.0892	.1016	.0996	.0960	.0446	.1746
.12	.0534	.0755	.6499	.0822	.0868	.0851	.0820	.0411	.1611
.11	.0470	.0695	.6258	.0751	.0731	.0717	.0691	.0375	.1476

Source:

Appendix 8F-6 Velocity Head and Resistance Computations Factors for Circular Conduits Flowing Full and Partly Full

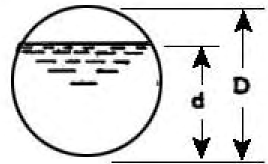
Table 3. -- Velocity head and resistance computation factors for circular conduits flowing full and partly full

Column A: Relative depth of flow, d/D

Column B: Relative velocity head
$h_v/D = \alpha V^2/2gD$, $\alpha = 1.00$, $Q/D^{2.5} = 1.0$
V = Mean flow velocity
α = Kinetic energy correction factor
g = Accel. due to gravity = 32.16 ft./sec./sec.

Column C: Resistance computation factor (K_n) for the Manning equation, $V = (1.486/n)(R)^{2/3}(S)^{1/2}$
$S_f = Q^2 n^2/2.208 R^{4/3} A^2 = K_n\ (n^2/D^{1/3})\ (Q/D^{2.5})^2$
$K_n = 0.4529/(R/D)^{4/3}\ (A/D^2)^2$
A = Flow area in conduit
S_f = Friction slope
R = Hydraulic radius
n = Manning coefficient

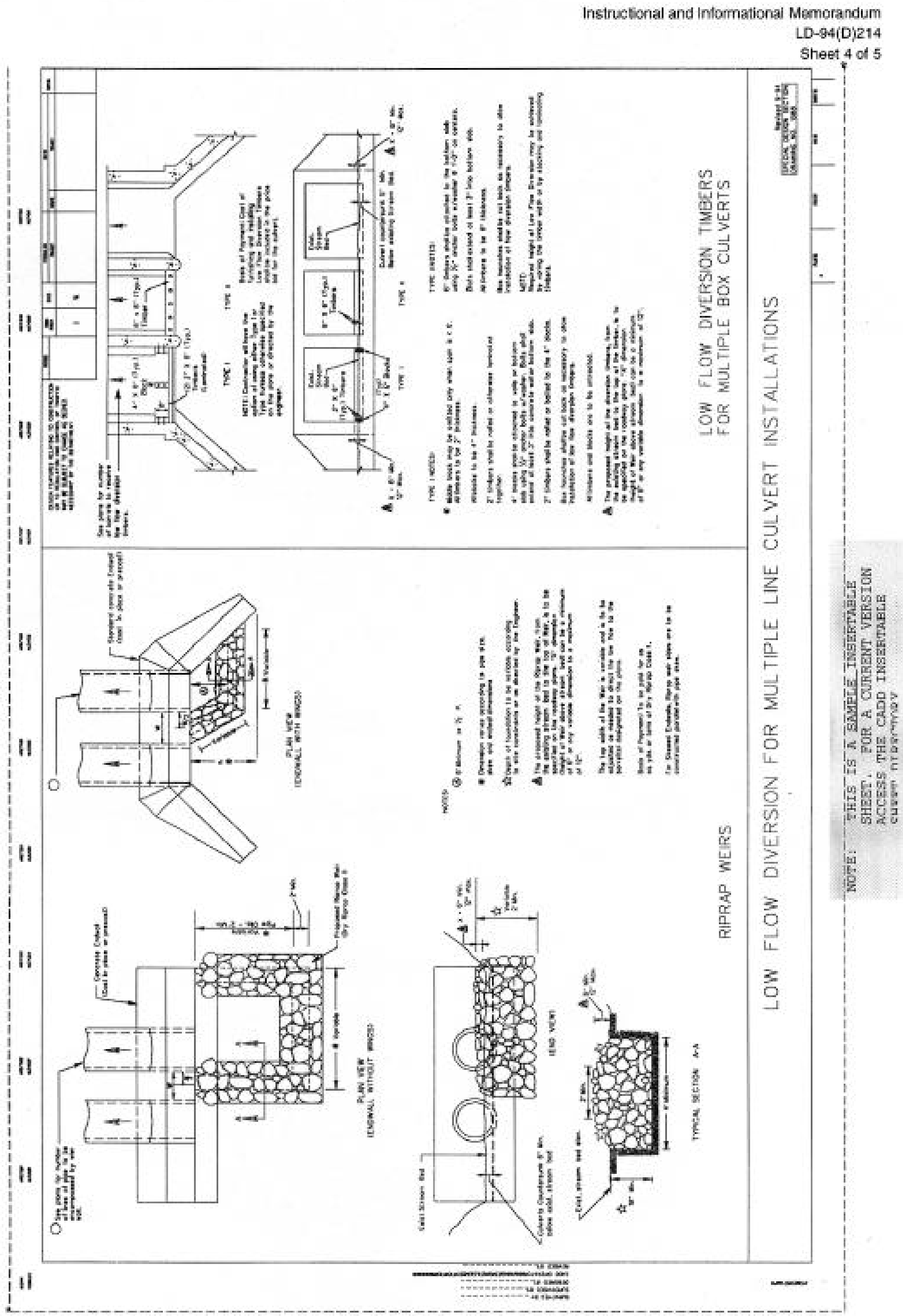
Column D: Resistance computation factor (K_f) for the Darcy equation, $h_f = (f)\ (L/4R)\ (V^2/2g)$
$S_f = Q^2 f/257.28 R A^2 = K_f(f)\ (Q/D^{2.5})^2$
$K_f = 0.003887/(R/D)\ (A/D^2)^2$
h_f = Friction head loss, ft.
f = Darcy coefficient
L = Length of conduit, ft.



(A) Relative depth d/D	(B) Relative velocity head $\alpha V^2/2gD$ $\alpha = 1.00$ $Q/D^{2.5} = 1.0$	(C) Manning Eq. resistance computation factor K_n	(D) Darcy Eq. resistance computation factor K_f	(A) Relative depth d/D	(B) Relative velocity head $\alpha V^2/2gD$ $\alpha = 1.00$ $Q/D^{2.5} = 1.0$	(C) Manning Eq. resistance computation factor K_n	(D) Darcy Eq. resistance computation factor K_f
1.00	0.02520	4.662	0.02520	0.85	0.03071	4.390	0.02532
0.99	.02529	4.293	.02371	.84	.03134	4.470	.02579
.98	.02544	4.174	.02326	.83	.03201	4.560	.02632
.97	.02565	4.104	.02301	.82	.03272	4.657	.02688
.96	.02589	4.061	.02288	.81	.03348	4.764	.02750
.95	.02618	4.037	.02284	.80	.03426	4.878	.02816
.94	.02648	4.028	.02287	.79	.03510	5.004	.02888
.93	.02683	4.033	.02296	.78	.03598	5.137	.02963
.92	.02720	4.046	.02310	.77	.03692	5.282	.03045
.91	.02761	4.071	.02330	.76	.03790	5.438	.03133
.90	.02805	4.105	.02353	.75	.03894	5.605	.03226
.89	.02852	4.145	.02380	.74	.04004	5.787	.03328
.88	.02902	4.195	.02412	.73	.04120	5.981	.03436
.87	.02955	4.251	.02448	.72	.04242	6.188	.03550
.86	.03011	4.317	.02487	.71	.04371	6.411	.03673

Source:

Appendix 8G-1 Low Flow Diversion for Multiple Line Culvert Installations



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Chapter 9 - Storm Drains

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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 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.

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.

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

Roadway Classification		Design Speed (mph)	Design Storm Frequency (year[1, 2])	Design Storm Intensity (in./hr.)	Maximum Design Spread Width[3] (ft)
Principal Arterial					
With Shoulder	On Grade	≤ 50	10	Actual	Sh. Width + 3
With Shoulder	On Grade	> 50	10	Actual	Sh. Width
With Shoulder	Sag Location[5]	All	10	Actual	Sh. Width + 3
Without Shoulder	On Grade	≤ 50	N/A[4]	4	½ Driving Lane + Gutter Width (If Any)
Without Shoulder	On Grade[5]	> 50	10	Actual	½ Driving Lane + Gutter Width (If Any)
Without Shoulder	Sag Location[5]	≤ 50	N/A[4]	4	½ Driving Lane + Gutter Width (If Any)
Without Shoulder	Sag Location[5]	> 50	50	Actual	½ Driving Lane + Gutter Width (If Any)
Minor Arterial, Collector, Local					
With Shoulder	On Grade	≤ 50	N/A[4]	4	Sh. Width + 3
With Shoulder	On Grade	> 50	N/A[4]	4	Sh. Width
With Shoulder	Sag Location	All	N/A[4]	4	Sh. Width + 3
Without Shoulder	On Grade	All	N/A[4]	4	½ Driving Lane + Gutter Width (If Any)
Without Shoulder	Sag Location	All	N/A[4]	4	½ Driving Lane + Gutter Width (If Any)

Table 9-2. Design Frequencies for Storm Drain Conduit

Roadway Classification	Design Speed (mph)	Design Storm Frequency (year[1, 2])
Principal Arterial		
With Shoulder	All	25
Without Shoulder	≤ 50	10
Without Shoulder	> 50	25
Minor Arterial, Collector, Local		
With or Without Shoulder	All	10

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.

3. 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 one (1) inch.

   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 inches per hour 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 inches per hour.

   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.

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 0.3 percent. The minimum gutter profile grade is 0.2 percent. The minimum pavement cross slope should not be less than 2 percent except during the occurrence of superelevation transition. The coincident occurrence of superelevation transitions and sag points or zero grades should be avoided.

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.

In locations where significant ponding may occur, such as 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.

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-inch diameter or its equivalent for non-circular shapes. Where necessary, it will be permissible to use a 12-inch diameter pipe for laterals or initial pipe runs of 50 feet or less.

Where feasible, the storm drains should be designed to avoid existing utilities. A minimum velocity of 3 feet per second 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.

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 9-3. Access Hole Spacing

Pipe Diameter (in)	Maximum Distance (ft)
12	50
15 - 42	300[1]
≥ 48	800

Note 1: This distance may be increased to 400 feet if the flow velocity for the design storm exceeds 5 feet per second and the flow depth for the design storm is at least 25 percent 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, they 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.

9.4 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 constructibility 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 inches.

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

Refer to FHWA Hydraulic Engineering Circular No. 22, “Urban Drainage Design Manual” for a discussion on 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 (Highway Drainage Guidelines, Volume 9, Storm Drain 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

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 percent for curbed pavements with an absolute minimum of 0.2 percent. 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 percent should be maintained within 50 feet 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 feet per percent (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.

The USDOT, FHWA (FHWA-RD-79-30, 31, 1979) reports that cross slopes of 2 percent have little effect on driver effort in steering, especially with power steering, or on friction demand for vehicle stability. Thus, the minimum recommended cross slope is 2 percent.

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 percent.

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.

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 Location and Design Instructional and Information Memo (I&IM) LD (D) 150 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 one (1) inch below the top of the curb.

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 G.R.E.A.T. or C.A.T. 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 system head on. 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.

9.4.4 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.

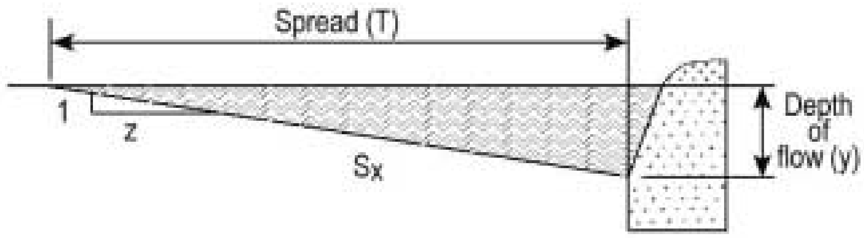
9.4.4.2 Manning's n for Pavement and Gutter Flow

It is recommended that an n-value of 0.015 be used in the computational analysis for pavement and gutter flow.

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.



$$\text{Cross Slope, } S_x = \frac{1}{z}$$

Figure 9-1. Uniform Cross Section

$$Q = \frac{0.56}{n} S_x^{\frac{5}{3}} S^{\frac{1}{2}} T^{\frac{8}{3}} \qquad (9.1)$$

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.

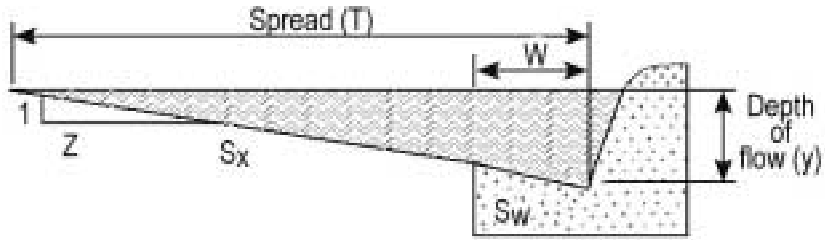


Figure 9-2. Composite Cross Section

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.

$$T = 1.243(Qn)^{\frac{3}{8}} S_x^{-\frac{5}{8}} S^{-\frac{3}{16}} \qquad (9.2)$$

The spread in various types of composite sections may be determined by using the appropriate chart in Appendices 9C-2 through 9C-6.

9.4.5 Inlets

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.

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.

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.

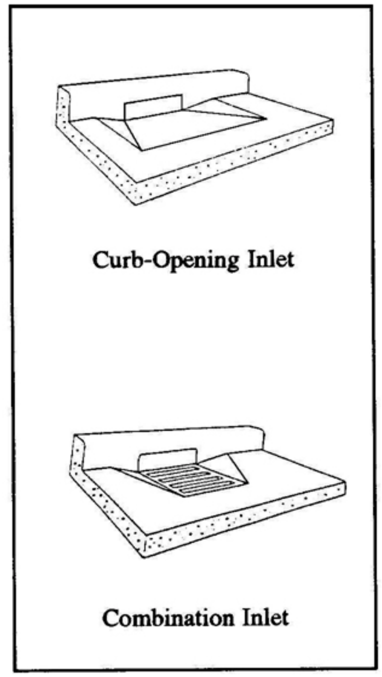


Figure 9-3. Curb Inlets

9.4.5.1.3 Slotted Drain Inlets and Trench Drain 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.

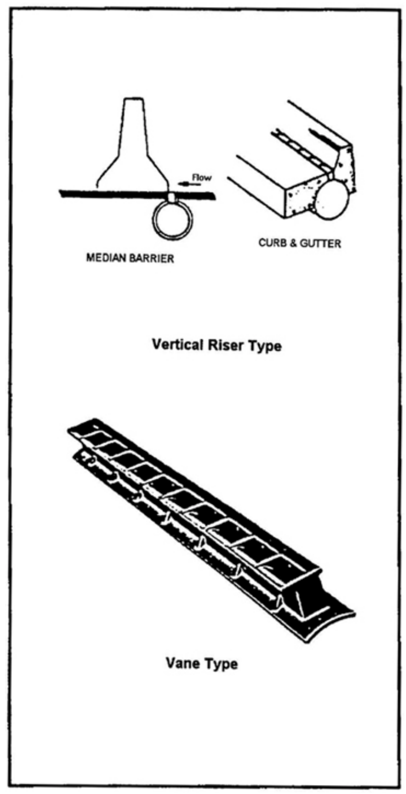


Figure 9-4. Slotted Drain Inlets

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.

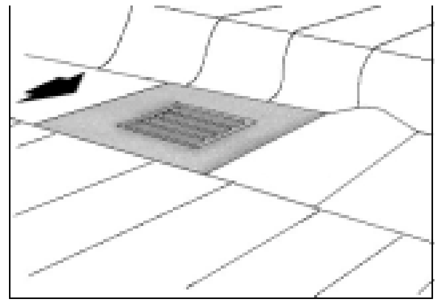


Figure 9-5. Grate Inlets

9.4.5.2 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.

9.4.6 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.

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:

$$L = \frac{43{,}560Q_t}{CiW} \qquad (9.3)$$

Where:

L = Distance from the crest, ft
Q_t = 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 grade 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:

$$L_T = \frac{0.6Q^{0.42}S^{0.3}}{(nS_e)^{0.6}} \qquad (9.4)$$

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.

$$S_e = S_x + S'_w E_o \qquad (9.5)$$

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 one inch of local depression for each one-foot width of the concrete gutter pan. For inlets used in curb only sections, a local depression of one inch is applied over a horizontal distance of one foot 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.

$$E_o = \frac{K_w}{K_w + K_o} \qquad (9.6)$$

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.

$$K = 1.486\frac{A^{\frac{5}{3}}}{nP^{\frac{2}{3}}} \tag{9.7}$$

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:

$$E = 1-\left(1-\frac{L}{L_T}\right)^{1.8} \tag{9.8}$$

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:

$$Q_i = EQ \tag{9.9a}$$

Where:

Q_i = Intercepted flow, cfs
E = Curb opening efficiency
Q = Total flow approaching inlet, cfs

The bypass flow (Q_b) is the difference between the total approach flow and the intercepted flow.

$$Q_i = Q - Q_b \quad (9.9b)$$

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:

$$Q_i = C_w(L+1.8W)d^{1.5} \quad (9.10)$$

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

$$Q_i = C_w L d^{1.5} \tag{9.11}$$

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 1.2h, where h is the height of the curb opening. The weir coefficient applies to the inlet throat.

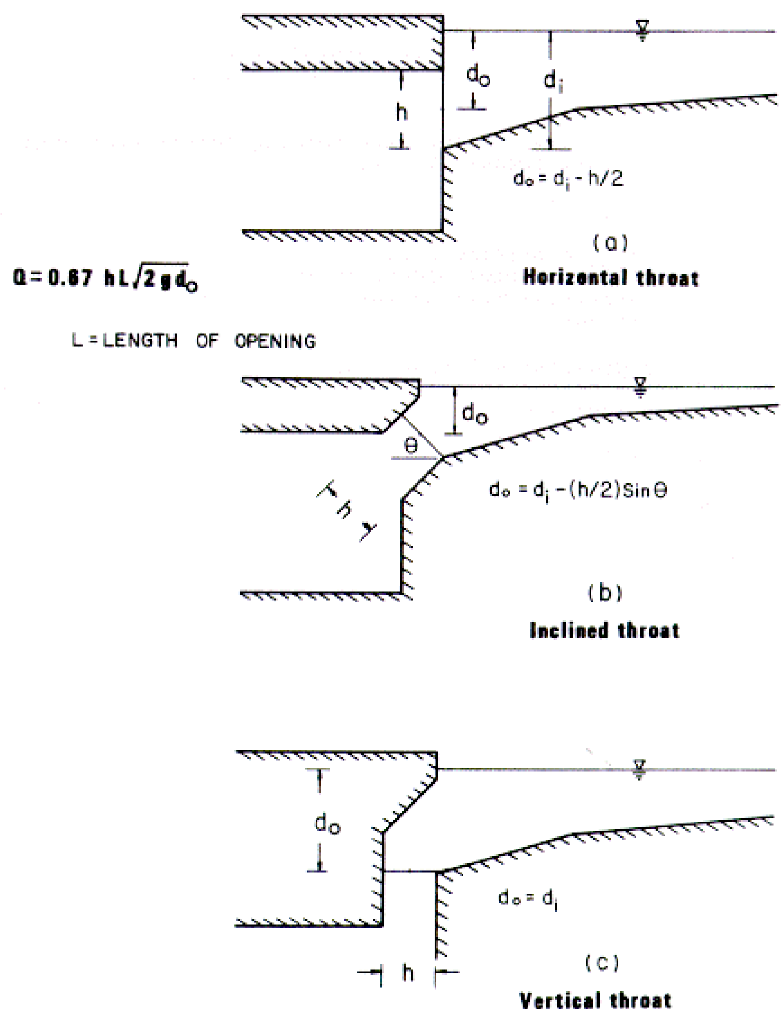


Figure 9-6. Curb Opening Inlets (Operating as an Orifice)

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:

$$Q_i = C_o A[2gd_o]^{0.5} \tag{9.12}$$

Where:

C_o = Orifice coefficient, use 0.67
h = Height of curb-opening orifice, ft
A = Clear area of opening, sq. ft.
d_o = Effective head on the center of the orifice throat, ft
For VDOT standard curb inlets, $d_o = \left(d_i - \frac{h}{2}\right)\sin\theta$
d_i = Depth of flow at the curb including inlet depression (if present), ft
h = Throat opening measured normal to the throat opening, ft

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 as shown in Figure 9-7. 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. Refer to Table 9-4 for recommended flanking inlet locations. The typical location for flanking inlets is 50 feet each side of the sag point.

Table 9-4. Flanking Inlet Locations

Distance to flanking inlet in sag vertical curve locations using depth at curb criteria (ft).											
d↓ K→	20	30	40	50	70	90	110	130	160	167	180	220
0.1	20	24	28	31	37	42	47	51	56	58	60	66
0.2	28	34	40	44	52	59	66	72	79	81	84	93
0.3	34	42	49	54	64	73	81	88	97	99	103	114
0.4	40	49	57	63	75	85	94	102	113	116	120	133
0.5	45	55	63	72	84	95	105	114	127	129	134	148
0.6	49	60	69	77	92	104	115	125	139	142	147	163
0.7	53	65	75	84	99	112	124	135	150	153	159	176
0.8	57	69	80	89	106	120	133	144	160	163	170	188

NOTES: 1. $\times = (200dK)^{0.5}$, where $\times$ = distance from the low point
2. Drainage maximum K = 167(A/%).
3. d = depth at curb (ft).

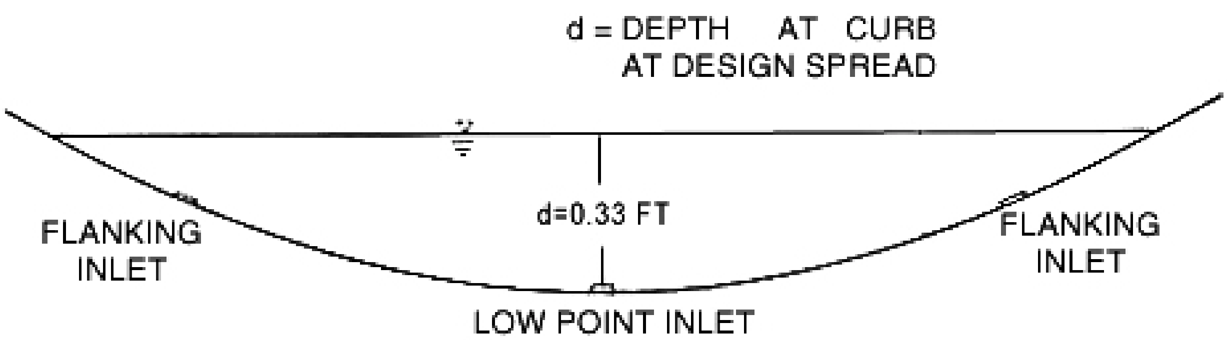


Figure 9-7. Flanking Inlets at a Sag Point

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.

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 five (5) inches and as an orifice for depths greater than about 17 inches. Between these depths, a transition from weir to orifice flow occurs. The capacity of a grate inlet operating as a weir is:

$$Q_i = C_w P d^{1.5} \qquad (9.13)$$

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:

$$Q_i = C_o A(2gd)^{0.5} \qquad (9.14)$$

Where:

C_o = Orifice coefficient, use 0.67
A = Effective clear open area of grate, sq. ft.
g = Gravitational acceleration, 32.2 ft/s^2

Appendices 9C-12 through 9C-16 provide nomograph solutions of Equations 9.13 and 9.14 for various grate sizes and provides dimensions for standard VDOT 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 feet 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 entire drainage area is not contributing. 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.

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 should 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.

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.

9.4.8.4 Conduit Material Selection

Refer to I&IM LD (D) 121 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.

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:

$$V = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \qquad (9.15)$$

Where:

- V = Mean velocity of flow, fps
- n = Manning's roughness coefficient
- R = Hydraulic radius, ft = $\left(\frac{A}{P}\right)$
- A = Flow area, sq. ft.
- 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:

$$Q = VA = \frac{1.486}{n} AR^{\frac{2}{3}} S^{\frac{1}{2}} \qquad (9.16)$$

Where:

- Q = Rate of flow, cfs
- A = Flow area, sq. ft.

For storm drains flowing full, the above equations become:

$$V = \frac{0.590}{n} D^{\frac{2}{3}} S^{\frac{1}{2}} \qquad Q = \frac{0.463}{n} D^{\frac{8}{3}} S^{\frac{1}{2}} \qquad (9.17)$$

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_{full}) and discharge (Q/Q_{full}). The typical design process will use either a computer program or “Feild’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 feet per second 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 0.1 percent is considered the minimum slope for constructibility.

$$S = \frac{0.453(nV)^2}{R^{\frac{4}{3}}} \tag{9.18}$$

9.4.8.7 Maximum Grades

Slopes that incur uniform flow velocities in excess of 10 feet per second should be avoided because of the potential for abrasion. Slopes in excess of 16 percent are not preferred because of the need for anchor blocks. In steeper terrain, large elevation differences can be accommodated using drop structures. See DDM #2 in Chapter 15, Drainage Design Instructions, for details.

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 watershed, the peaks will be out of phase. To aid in the evaluation of joint probabilities, refer to Table 9-5. Joint Probability Analyses.

Table 9-5. Joint Probability Analyses

Watershed Area Ratio	Frequencies For Coincidental Occurrence			
	10-Year Design		100-Year Design	
	Main Stream	Tributary	Main Stream	Tributary
10 000 TO 1	1 10	10 1	2 100	100 2
1 000 TO 1	2 10	10 2	10 100	100 10
100 TO 1	5 10	10 5	25 100	100 25
10 TO 1	10 10	10 10	50 100	100 50
1 TO 1	10 10	10 10	100 100	100 100

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.

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.

$$HGL_{us} = HGL_{ds} + h_f + h_m \qquad (9.19)$$

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 hby 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 9-8.

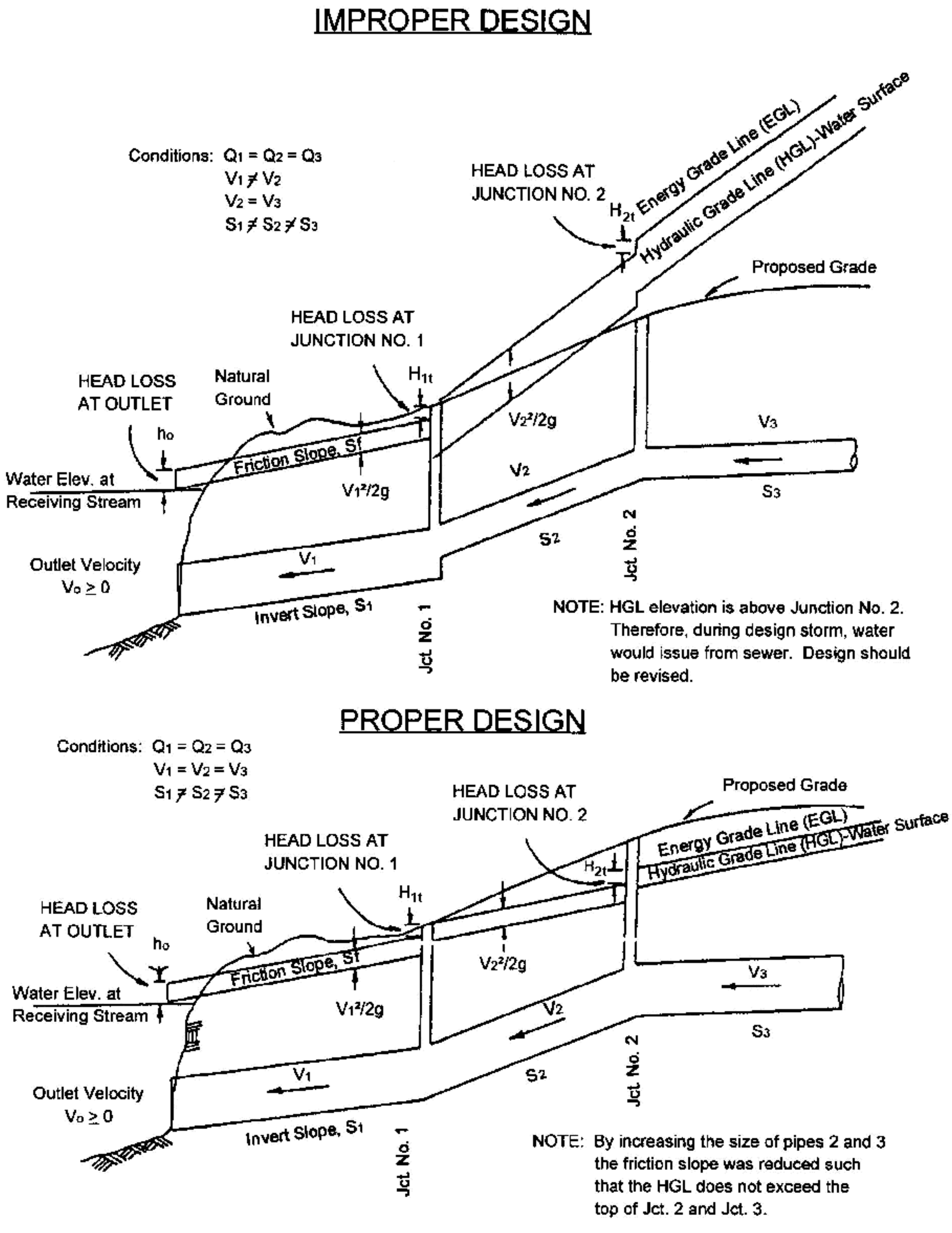


Figure 9-8. Use of Energy Losses in Analyzing a Storm Drain System

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:

$$S_{fo} = 0.453\frac{Q^2 n^2}{A^2 R^{\frac{4}{3}}} \qquad (9.20)$$

Then the head losses due to friction may be determined by the formula:

$$H_f = S_{fo}L \qquad (9.21)$$

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.

$$H_t = H_i + H_o + H_\Delta \qquad (9.22)$$

Where:

- H_t = Total junction losses, ft
- H_i, H_o, H_Δ = 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.

$$H_i = K_e\frac{V_i^2}{2g} \qquad (9.23)$$

Where:

- H_i = Entrance head loss, ft
- V_i = Velocity in the inlet conduit, fps
- g = Gravitational acceleration, 32.2 ft/s^2
- K_e = Entrance loss coefficient (VDOT K_e = 0.35). Where more than one inlet pipe is present, use the velocity of the one with the greatest momentum (Q*V).

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.

$$H_o = K_o \frac{V_o^2}{2g} \qquad (9.24)$$

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 9-9 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^o to the incoming conduit. The sharper the bend (approaching 90^o) the more severe the energy loss becomes. Conduits should not be designed to have bend angles greater than 90^o.

$$H_\Delta = K \frac{V_i^2}{2g} \qquad (9.25)$$

Where:

H_Δ = Headloss at a bend, ft
K = Bend loss coefficient
V_i = Velocity in the inlet conduit, fps

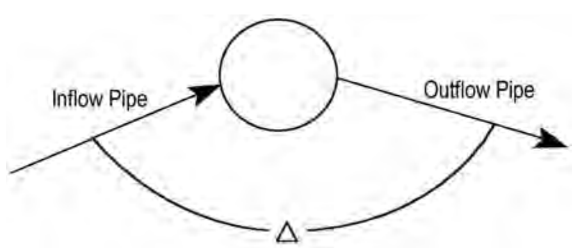


Figure 9-9. Angle Between Inflow and Outflow Pipes

VDOT recommended values of K for change in direction of flow in laterals can be found on design form LD-347, Appendix 11B-3. Figure 9-10 shows a graphical representation of the bend loss coefficient (K) to change in direction of flow lateral.

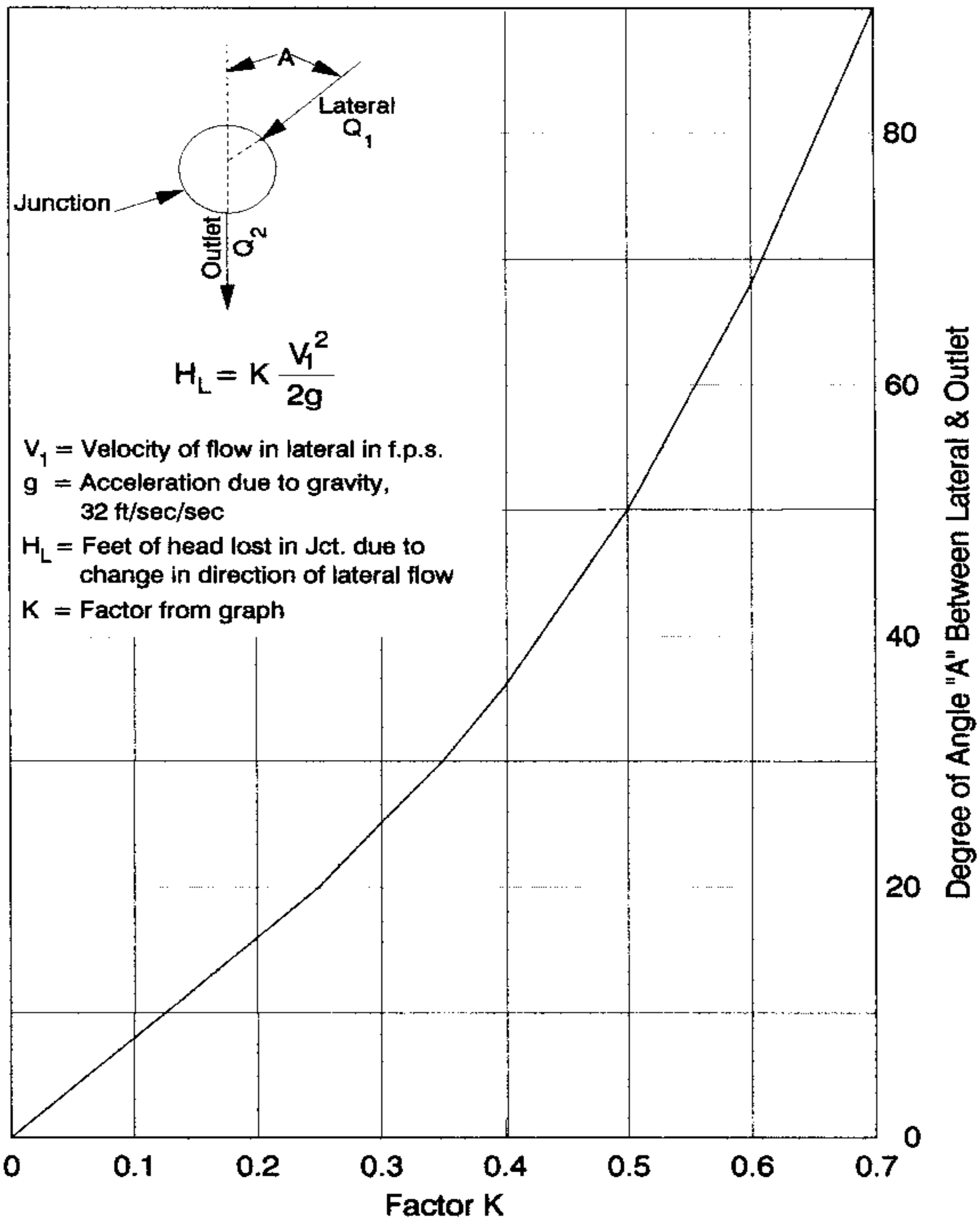


Figure 9-10. Losses in Junction Due to Change in Direction of Flow Lateral

9.4.9.3.3 Plunging Losses

Plunging losses are applied if the surface inlet inflow is 20 percent 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 percent or more of the total flow through the junction. Plunging flow losses increase the total junction loss by 30 percent as defined by Equation 9.26.

$$H_t = 1.30\left(H_o + H_i + H_\Delta\right) \qquad (9.26)$$

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 percent as defined by Equation 9.27 if there are no plunging losses or Equation 9.28 if there are plunging losses.

$$H_t = 0.50\left(H_o + H_i + H_\Delta\right) \quad \text{(Where no plunging losses occur)} \qquad (9.27)$$

or

$$H_t = (0.50)(1.30)(H_o + H_i + H_\Delta) \quad \text{(Where plunging losses occur)} \qquad (9.28)$$

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.

$$H = H_f + H_t \qquad (9.29)$$

Where:

H = Total headloss, ft

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 FHWA HEC-12 and/or HY25 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).

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.

$$Q = \frac{0.56}{n} S_x^{\frac{5}{3}} S^{\frac{1}{2}} T^{\frac{8}{3}}$$

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 cfs
Longitudinal slope, S = 0.0035 ft/ft
Cross slope, S_x = 0.0208 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 ft
Longitudinal slope, S = 0.0035 ft/ft
Cross slope, S_x = 0.0208 ft/ft
Manning's roughness coefficient, n = 0.015

Step 2: Compute gutter flow (Q), using Equation 9.1.

$$Q = \frac{0.56}{n} S_x^{\frac{5}{3}} S^{\frac{1}{2}} T^{\frac{8}{3}}$$

$$= \frac{0.56}{0.015} \times 0.0208^{\frac{5}{3}} \times 0.0035^{\frac{1}{2}} \times 14^{\frac{8}{3}}$$

$$= 3.96 \text{ cfs (Say 4.0 cfs)}$$

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.

<u>Condition 1</u>: 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).*

<u>Condition 2</u>: 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 <u>Condition 1</u> 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

<u>Condition 1</u>: 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 ft.

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 feet

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 percent.*

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 feet 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.*

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), $\left(\frac{W}{T}\right)$ *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 inch per foot (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 ft slot were used.

Step 1: Repeat Steps 5 through15 from the inlet spacing procedure presented in Section 9.5.3.

Given: Q = 2 cfs, n = 0.015, S = 0.01 ft/ft, S_x = 0.0208 ft/ft, W = 2 ft, S_w = 0.0833 ft/ft, local inlet depression = 2 in.

Use Appendix 9C-3 to find Spread (T) = to 7.6 ft.

Step 2: *Determine the ratio of flow in the depressed section (W) to the Spread (T),* $\left(\frac{W}{T}\right)$.

$$\frac{W}{T}=\frac{2}{7.6}=0.26$$

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.*

$a = (S_w - S_x)12W$ + Local Depression
$a = (0.0833\text{-}0.0208)(12)(2)+2 = 3.5$ in

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.*

$S_e = S_x + S'_w\,(E_o)$
$S_e = 0.0208 + (0.146)(0.69) = 0.121$ 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.

$$L_T = \frac{0.6Q^{0.42}S^{0.3}}{(nS_e)^{0.6}}$$

$$L_T = \frac{0.6(0.6)^{0.42}(0.01)^{0.3}}{[0.015(0.121)]^{0.6}} = 8.9 \text{ ft}$$

The minimum length for 100% interception would be L_T = 8.9 ft.

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 ft would be appropriate.

Using an inlet length of 6 ft would require proceeding to Step 9.

Step 9: Compute $\left(\frac{L}{L_T}\right)$ *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.

$Q_i = EQ_T$
$Q_i = 0.87 \times 2 = 1.74$ 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.

$Q_b = Q_T – Q_i$
$Q_b = 2–1.74 = 0.26$ cfs

Step 13: The carryover flow (Q_b) from the first inlet is entered in Col. 11 for the next downstream inlet.

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 inch 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.

$$L_R = \frac{Q}{C_w d^{1.5}} - 1.8W \tag{9.29}$$

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.

$$L_R = \frac{Q}{C_o d\sqrt{2gh}} \tag{9.30}$$

Where:

- 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/s^2
- h = 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 ft, S_x = 0.0208 ft/ft, Inlet depression = 2 in, Standard length increment =2 ft, W = 2 ft, Curb height = 6 in, slot height = 5 in.

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 inch below the top of curb.*

The depth to 1 inch below the top of curb = 6 - 1 = 5 in (0.42 ft).

The depth of allowable ponding = $T(S_x)$

$$T(S_x) = 8(0.0208) = 0.17 \text{ ft}$$

Depth of ponding is less than 1 inch below the top of curb (0.34<0.42)

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 (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.

$$L_R = \frac{Q}{C_w d^{1.5}} - 1.8W$$

$$= \frac{3}{2.3(0.34)^{1.5}} - 1.8(2)$$

$$= 2.97 \text{ ft}$$

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.*

$$L_R = \frac{Q}{C_o d\sqrt{2gh}}$$

Step 5: Select a standard inlet length (L) that is greater than the required length (L_R).

Using a factor of safety of 2, the required length (L_R) is 5.95 ft. Use an actual inlet length (L) of 6 ft.

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.

9.5.4 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 ft.

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-12, determine:

Grate open area = 6 sq. ft.
Grate perimeter = 12.8 ft.

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 ft

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.

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

$$= 3.0(12.8)(0.5)(2)^{1.5}$$

$$= 54 \text{ cfs}$$

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.

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.*

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 curves.*

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.*

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.

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:

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.

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 9.26, pipe capacity charts in Chapter 8, or from the "Feild'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_o).

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 percent 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 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_{Δ}). 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_o), Col. 13 (H_i), and Col. 15(H_{Δ}).

Step 17: If the junction incorporates surface inflow, such as from drop inlets, and this flow accounts for 20 percent 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 percent or more of the total flow through the junction, increase H_t by 30 percent. Enter the adjusted H_t in Col. 17.

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 percent 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 are based on the layout shown in Figure 9-11. Use a 10-year design storm and use concrete pipe (n = 0.013).

Design Data:

Inlet #	1	2	3	4
CA	0.9	0.5	1.25	1.98
t c (minutes)	15	12	17	16
Top Elev.	103.25’	101.25’	101.75’	98.75’

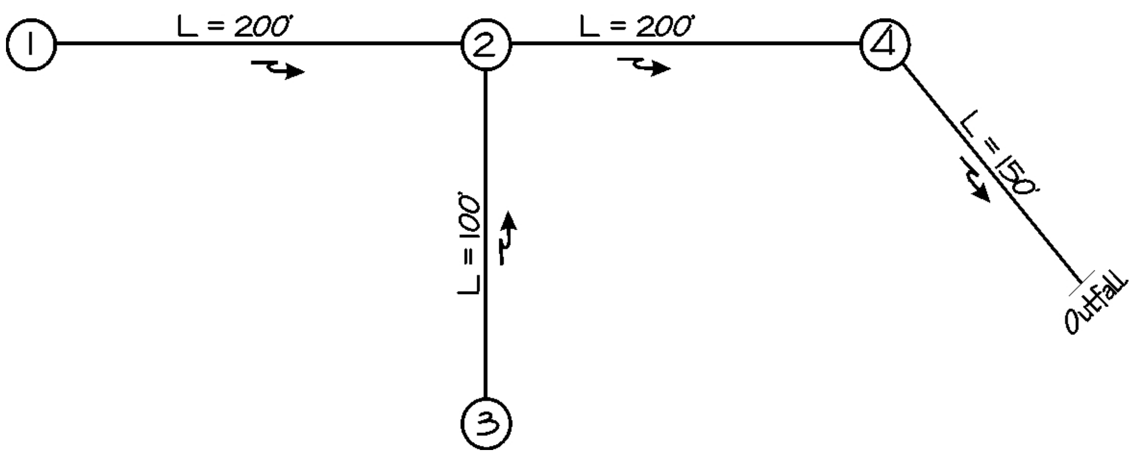


Figure 9-11. Storm Drain Layout Sample Problem

LD-229
July 2000

STORM SEWER DESIGN COMPUTATIONS

ROUTE:
PROJ: Storm Sewer
COUNTY:
DISTRICT:
DESCRIPTION: WORKSHEET 11
SHEET OF

FROM POINT	TO POINT	AREA DRAIN "A"	RUN-OFF COEF.	CA		INLET TIME	RAIN FALL	RUN-OFF Q	INVERT ELEVATIONS		LENGTH	SLOPE	DIA.	CAPA-CITY	VEL.	FLOW TIME MINUTES		REMARKS
		ACRES	C	INCRE-MENT	ACCUM-ULATED	MIN-UTES	IN./HR.	C.F.S.	UPPER END	LOWER END	FT.	FT./FT.	IN.	C.F.S.	F.P.S.	INCREMENT	ACCUMULATED	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)		(18)
1	2			0.9	0.9	15	5.1	4.6	99.80	97.80	200	0.01	15	6.7	5.7	0.6	15.6	
3	2			1.25	1.25	17	4.85	6.1	98.05	97.55	100	0.005	18	7.7	4.6	0.4	17.4	
2	4			0.50	2.65	17.4	4.8	12.7	97.30	94.80	200	0.0125	21	18	8.0	0.4	17.8	
4	5			1.98	4.63	17.8	4.6	21.3	94.50	91.80	150	0.018	24	31	10.5	0.2	18.0	

Figure 9-12. Storm Drain Design Form LD 229, Sample Problem

LD-347 Rev. 7/00
HYDRAULIC GRADE LINE

PROJECT: Worksheet 11
SHEET: ________ of ________

							JUNCTION LOSS													
INLET NUMBER (1)	Outlet Water Surface Elev. (2)	D_o (3)	Q_o (4)	L_o (5)	S_{fo} % (6)	H_f (7)	V_o (8)	H_o (9)	Q_i (10)	V_i (11)	Q_iV_i (12)	$\frac{V_i^2}{2g}$	H_i (13)	Angle (14)	H_Δ (15)	H_t (16)	1.3 H_t (17)	0.5 H_t (18)	Final H (19)	Inlet Water Surface Elev. (20)	Rim Elev. (21)
	(0.8d)																				
4	93.40	24	21.8	150	0.85	1.3	10.5	0.43	12.7	8.0		0.99	0.35	45	0.42	1.20	1.56	0.78	2.08	95.48	98.75
	(0.8d)																				
2	96.20	21	12.7	200	0.6	1.2	8.0	0.25	6.1	4.6	28.1	0.33	0.12	90	0.23	0.60	0.78	0.39	1.59	97.79	101.25
									4.6	5.7	26.2										
	(0.8d)																				
3	98.75	18	6.1	100	0.32	0.32	4.6	0.10								0.10	0.13	–	0.45	99.20	101.75
	(0.8d)																				
1	98.80	15	4.6	200	0.5	1.0	5.7	0.15								0.15	0.20		1.20	100.00	103.25

$H_i = 0.35\frac{V_i^2}{2g}$ $H_o = 0.25\frac{V_o^2}{2g}$ $H_\Delta = K\frac{V_i^2}{2g}$

FINAL H = $H_f + H_t$

$H_t = H_o + H_i + H_\Delta$

90° K = 0.70	50° K = 0.47	20° K = 0.16
80° K = 0.66	40° K = 0.38	15° K = 0.10
70° K = 0.61	30° K = 0.28	
60° K = 0.55	25° K = 0.22	

See LD-72 (D) 67

Figure 9-13. Hydraulic Grade Line Design Form LD 347, Sample Problem

9.6 References

American Association of State Highway and Transportation Officials. Volume 9, Highway Drainage Guidelines, Storm Drainage System, 1992

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

Pavement and Geometric Design Criteria for Minimizing Hydroplaning. FHWA Report No. RD-79-31, December 1979

Dah-Chen Wood. Public Roads, Vol. 52, No. 2, Bridge Drainage System Needs Criteria. U. S. Department of Transportation, September 1988

U. S. Department of Transportation, Federal Highway Administration, 1984. Drainage of Highway Pavements, Hydraulic Engineering Circular No. 12

U. S. Department of Transportation, Federal Highway Administration, 1979. Design of Urban Highway Drainage – The State of the Art. FHWA-TS-79-225

Appendix 9A-1 Definitions and Abbreviations

Definitions:

Check Storm	The use of a less frequent event, such as a 50-year storm, to assess hazards at critical locations where water can pond to appreciable depths is commonly referred to as a check storm or check event.
Hydraulic Grade Line	The elevation to which the water can be expected to rise within a storm drain (pressure head +elevation head)
Spread	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.
Storm Drain	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.
Velocity Head	A quantity of energy head proportional to kinetic energy of flowing water.

Abbreviations:

FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
NRCS	National Resources Conservation Service
VDOT	Virginia Department of Transportation

Appendix 9A-2 Symbols

Symbol	Definition	Units
a	Depth of depression	ft
A	Drainage area	acres
A	Cross-sectional flow area	ft^2
A	Clear opening area of curb inlet or grate	ft^2
b	Manhole diameter or width	ft
C	Runoff coefficient	-
C_W	Weir coefficient	-
d	Depth of gutter flow at the curb line	ft
d_i	Depth at lip of curb opening	ft
D	Diameter of pipe	ft
E	Curb opening efficiency	-
E_o	Ratio of depression flow to total gutter flow	-
g	Acceleration due to gravity	ft/s^2
h	Height of curb opening inlet	ft
$h_?$	Bend head loss	ft
h_e	Entrance head loss	ft
h_f	Friction head loss	ft
h_m	Minor head loss	ft
h_o	Exit head loss	ft
H	Head Loss	ft
HGL_{us}	Elevation of the hydraulic grade line at upstream node	ft
HGL_{ds}	Elevation of the hydraulic grade line at downstream node	ft
i	Rainfall intensity	in/hr
K	Bend loss coefficient	-
K	Entrance loss coefficient	-
K	Exit loss coefficient	-
K	Conveyance of cross section	cfs
K_o	Initial head loss coefficient	-
K_o	Conveyance of the gutter section beyond depression	cfs
K_w	Conveyance of the depressed gutter section	cfs
L	Length of grate inlet	ft
L	Length of curb opening	ft
L	Pipe length	ft
L_T	Curb opening length for 100% interception	ft
L_R	Require length of inlet	ft
n	Manning's roughness coefficient	-
P	Perimeter of grate opening	ft
P_w	Wetted perimeter	ft
Q	Total flow to inlet or flow in gutter	cfs
Q_b	Bypass flow	cfs
Q_i	Intercepted flow	cfs
Q_o	Outlet flow	cfs
Q_s	Gutter capacity above the depressed section	cfs
Q_T	Total flow	cfs
Q_t	Maximum allowable flow	cfs

Appendix 9A-2 Symbols

Q_w	Flow in width W	cfs
R	Hydraulic radius	ft
R_f	Ratio of frontal flow intercepted to total flow	-
R_s	Ratio of side flow intercepted to total flow	-
S	Slope of the energy grade line	ft/ft
S	Longitudinal slope of pavement or gutter slope	ft/ft
S_x	Cross Slope	ft/ft
S_e	Equivalent cross slope	ft/ft
S_f	Friction slope	ft/ft
S_w	Depression section slope or gutter cross slope	ft/ft
S_w	Gutter cross slope including local depression	ft/ft
T	Spread	ft
t_c	Time of concentration	min
T_s	Spread above depressed section	ft
V	Mean velocity, velocity of flow in gutter	fps
V_{ds}	Downstream velocity	fps
V_o	Gutter velocity where splash-over first occurs	fps
V_{us}	Upstream velocity	fps
W	Drainage area width	ft
W	Width of depression	ft
W	Width of gutter pan	ft
W	Width of grate	ft
y	Depth of flow in approach gutter	ft
Z	T/d, reciprocal of the cross slope	-
θ	Angle with respect to centerline of outlet pipe	degrees

Appendix 9B-1 LD-204 Stormwater Inlet Computations

LD-204 SHEET ________

Rev. 6-85 RTE ________ PROJ ________________ DATE ____________ OF ________

																													Sag Inlets Only				
INLET																																	
NUMBER	TYPE	LENGTH (FT)	STATION	DRAINAGE AREA (AC)	C	CA	sum CA	I (IN/HR)	Q INCR (CFS)	Q_b, CARRYOVER (CFS)	Q_T, GUTTER FLOW (CFS)	S, GUTTER SLOPE (FT/FT)	S_X, CROSS SLOPE (FT/FT)	T, SPREAD (FT)	W (FT)	W/T	S_W (FT/FT)	S_W /S_X	E_O (App. 9C-8)	$a = 12W(S_W - S_X)$ + Local Depression	$S'_w = a/(12W)$	$S_e = S_X + S'_W(E_O)$, (FT/FT)	COMPUTED LENGTH, L_T, (FT) (App. 9C-17)	L, SPECIFIED LENGTH (FT)	L/L_T	E (App. 9C-18)	Q_i, INTERCEPTED (CFS)	Q_b, CARRYOVER (CFS)	d (FT)	h (FT)	d/h	T, SPREAD @ SAG (FT)	**REMARKS**
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	

LD-229
July 2000

STORM SEWER DESIGN COMPUTATIONS

ROUTE: ____________ PROJ: ____________
COUNTY: ____________ DISTRICT: ____________
DESCRIPTION: ____________
____________ SHEET ______ OF ______

FROM POINT	TO POINT	AREA DRAIN "A"	RUN-OFF COEF.	CA		INLET TIME	RAIN FALL	RUN-OFF Q	INVERT ELEVATIONS		LENGTH	SLOPE	DIA.	CAPA-CITY	VEL.	FLOW TIME MINUTES	REMARKS
		ACRES	C	INCRE-MENT	ACCUM-ULATED	MIN-UTES	IN./HR.	C.F.S.	UPPER END	LOWER END	FT.	FT./FT.	IN.	C.F.S.	F.P.S.	INCREMENT / ACCUMULATED	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)

Appendix 9B-3 LD-347 Hydraulic Grade Line Computations

LD-347 Rev. 7/00
HYDRAULIC GRADE LINE

PROJECT: ____________________

SHEET: __________ of __________

							JUNCTION LOSS													
INLET NUMBER (1)	Outlet Water Surface Elev. (2)	D_o (3)	Q_o (4)	L_o (5)	S_{fo} % (6)	H_f (7)	V_o (8)	H_o (9)	Q_i (10)	V_i (11)	Q_iV_i (12)	$\frac{V_i^2}{2g}$	H_i (13)	Angle (14)	H_Δ (15)	H_t (16)	1.3 H_t (17)	0.5 H_t (18)	Final H (19)	Inlet Water Surface Elev. (20)	Rim Elev. (21)

$$H_i = 0.35\frac{V_i^2}{2g} \quad H_o = 0.25\frac{V_o^{\,2}}{2g} \quad H_\Delta = K\frac{V_i^{\,2}}{2g}$$

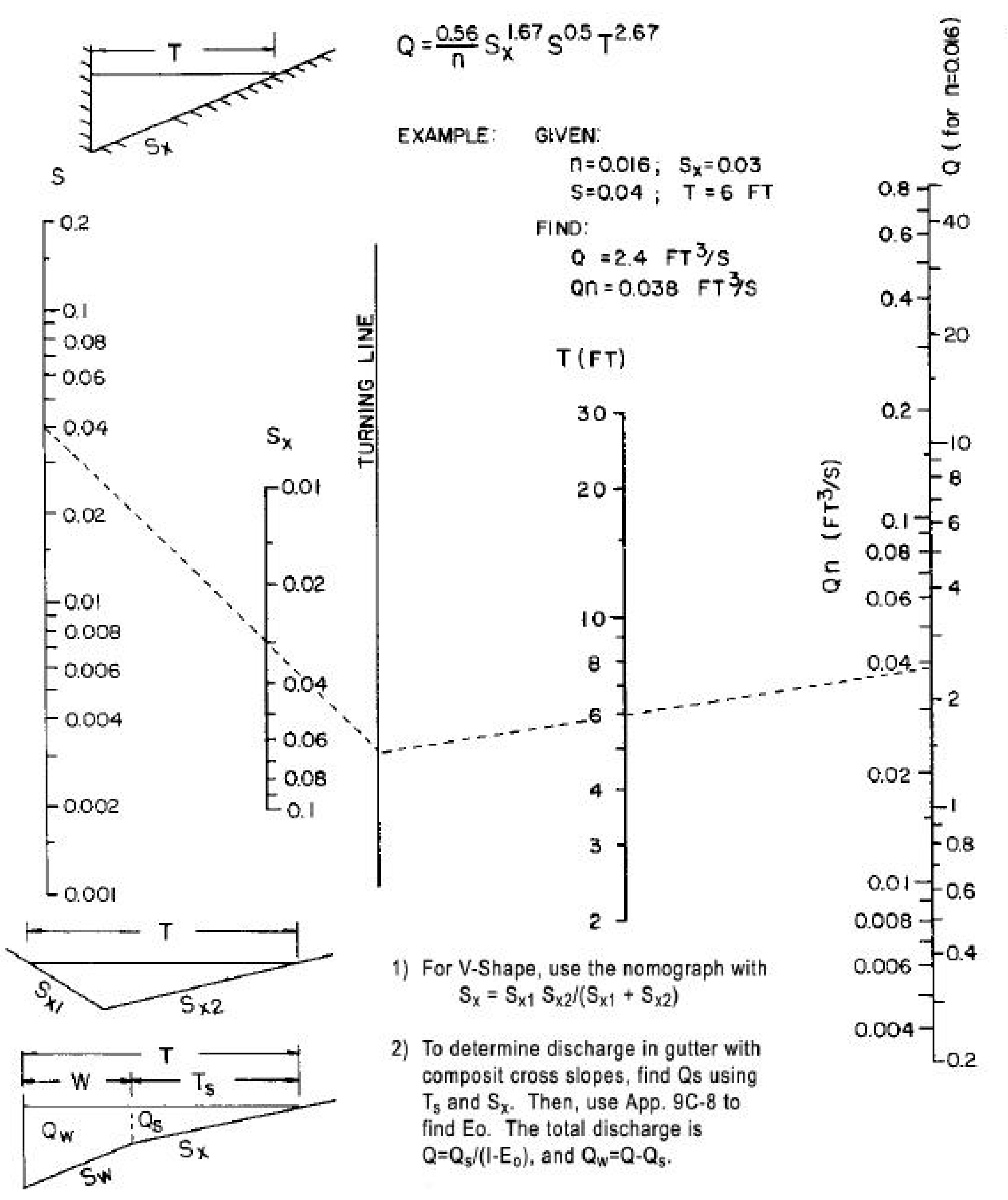
FINAL H = $H_f + H_t$

$H_t = H_o + H_i + H_\Delta$

90^o K = 0.70	50^o K = 0.47	20^o K = 0.16
80^o K = 0.66	40^o K = 0.38	15^o K = 0.10
70^o K = 0.61	30^o K = 0.28	
60^o K = 0.55	25^o K = 0.22	

See LD-72 D)67

Appendix 9C-1 Flow in Triangular Gutter Sections

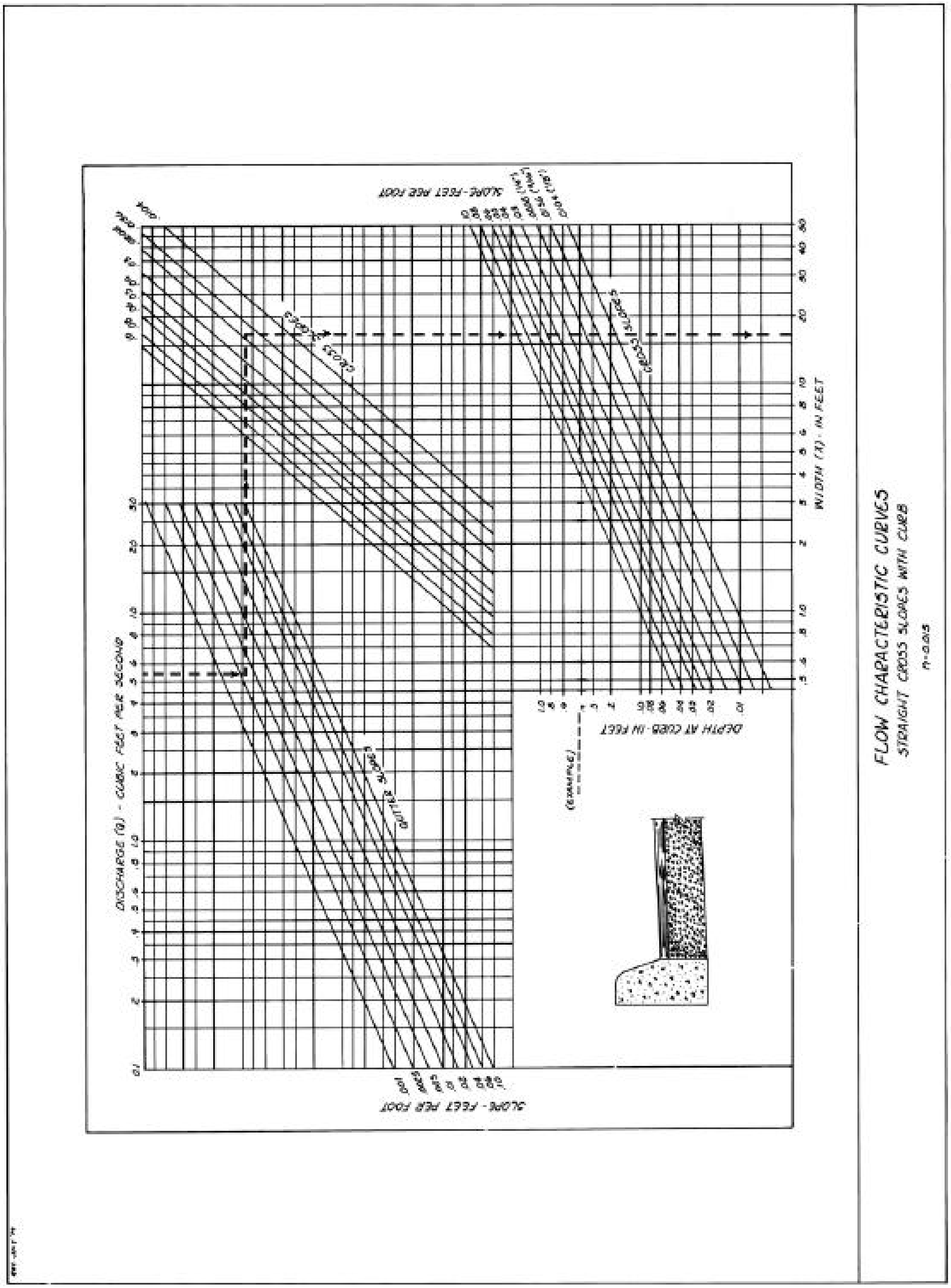


1) For V-Shape, use the nomograph with $S_x = S_{x1} S_{x2}/(S_{x1} + S_{x2})$

2) To determine discharge in gutter with composit cross slopes, find Qs using T_s and S_x. Then, use App. 9C-8 to find Eo. The total discharge is $Q=Q_s/(1-E_o)$, and $Q_w=Q-Q_s$.

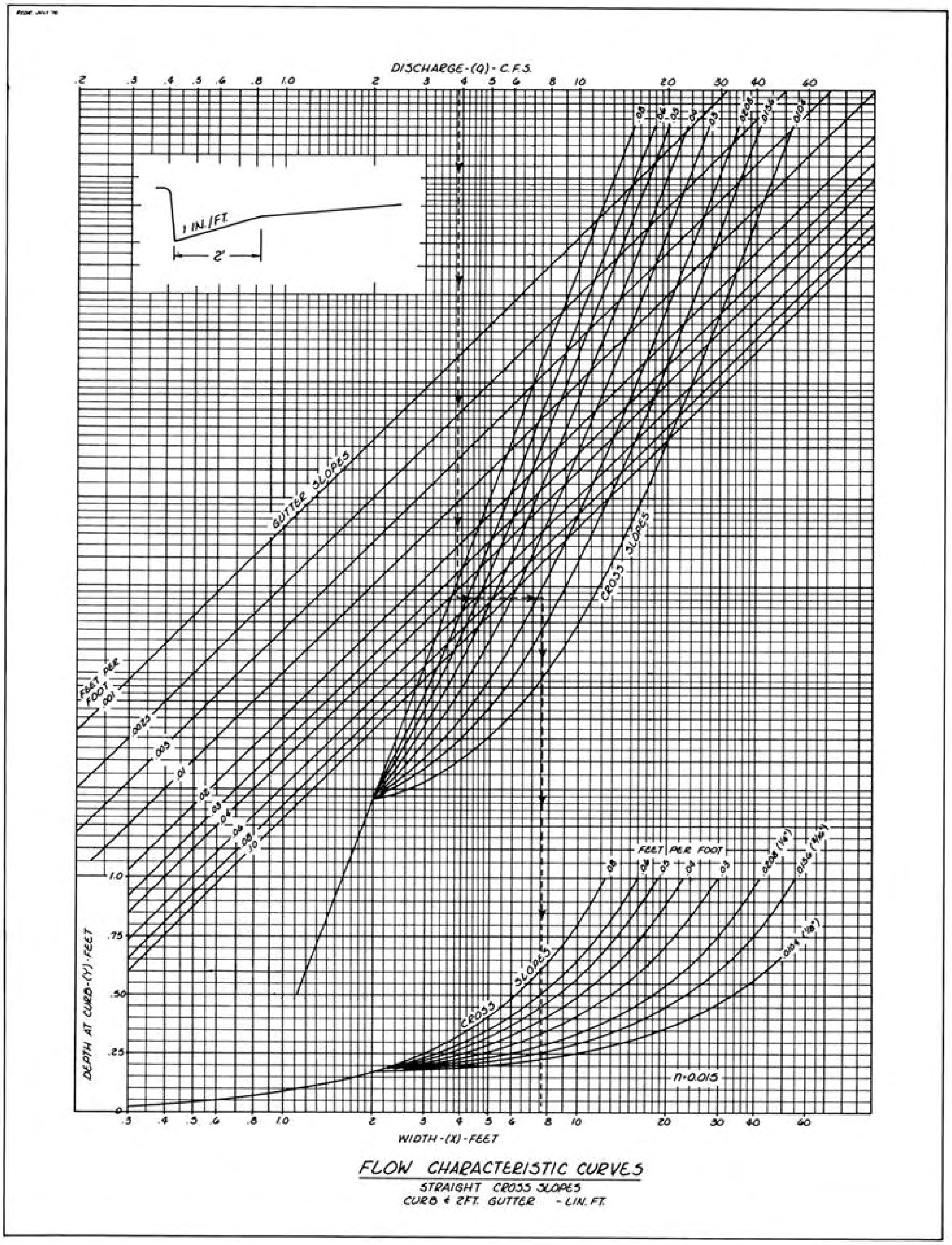
Source: HEC No. 12, FHWA

Appendix 9C-2 Flow Characteristic Curves (Straight Cross Slope with Curb)



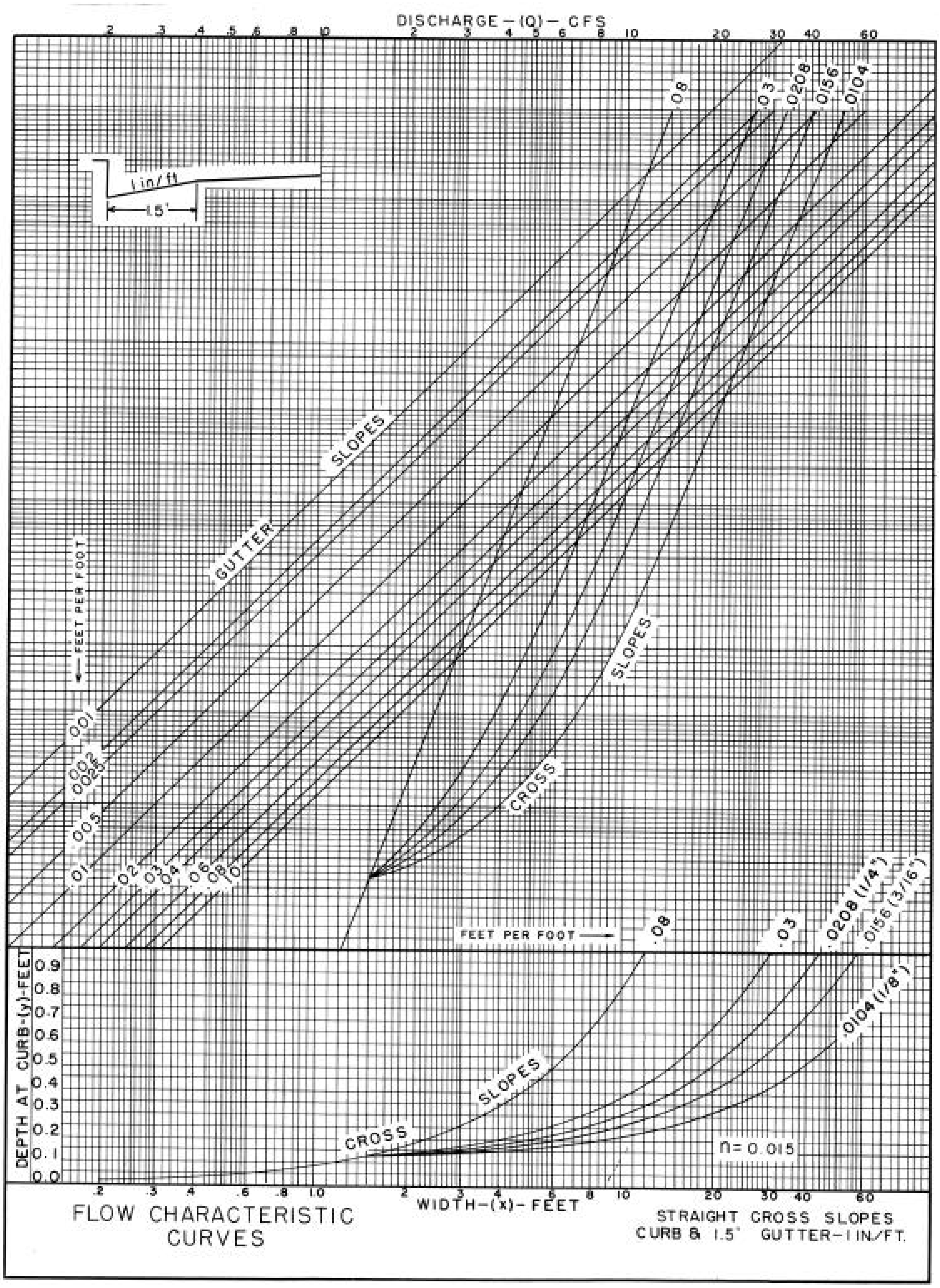
Source: VDOT
Comment: REV 6/81
REV 6/85

Appendix 9C-3 Flow Characteristic Curves (24" Gutter) – VDOT Standard



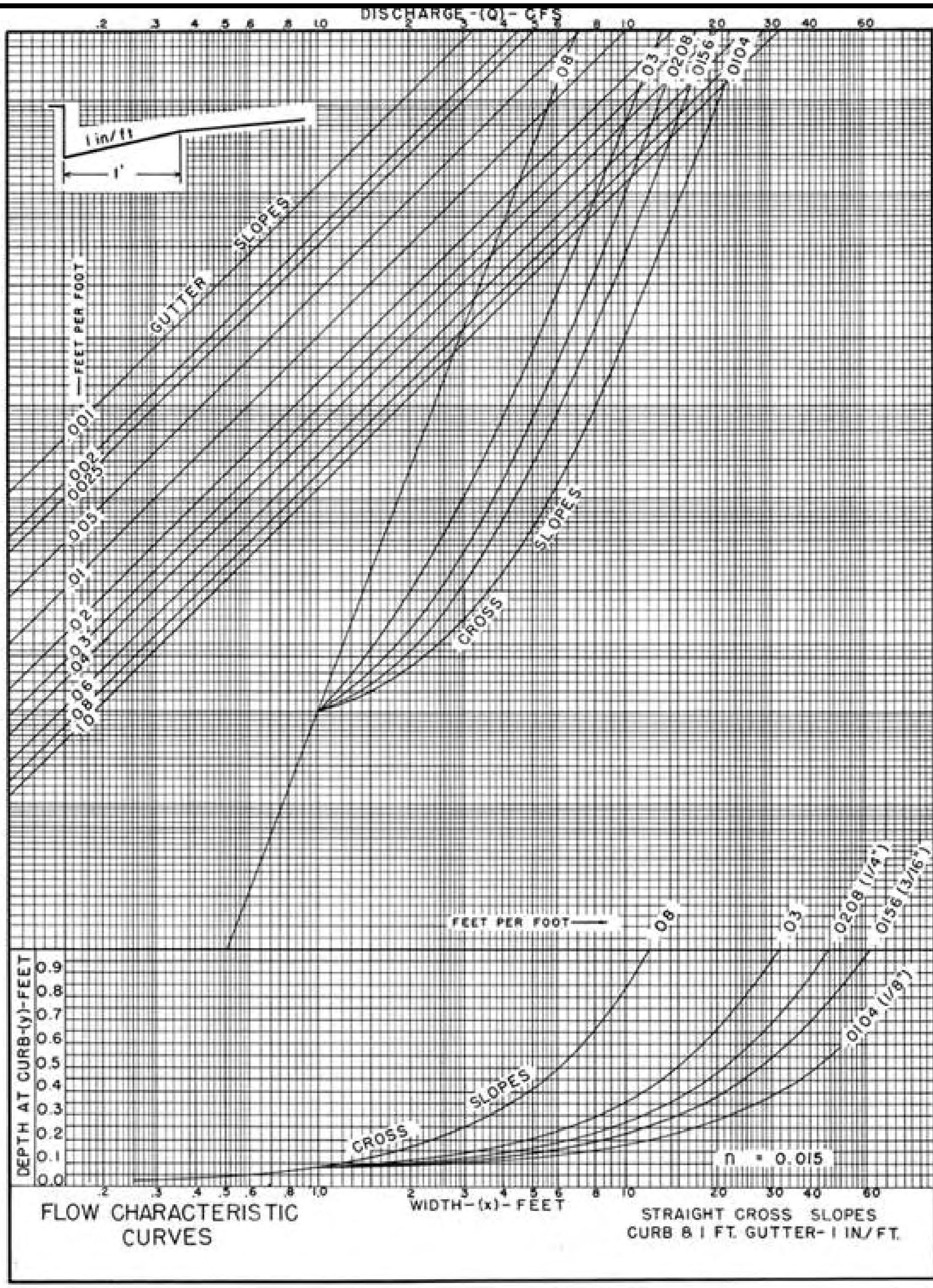
Source: VDOT

Appendix 9C-4 Flow Characteristic Curves (Straight Cross Slope, 18" Gutter)



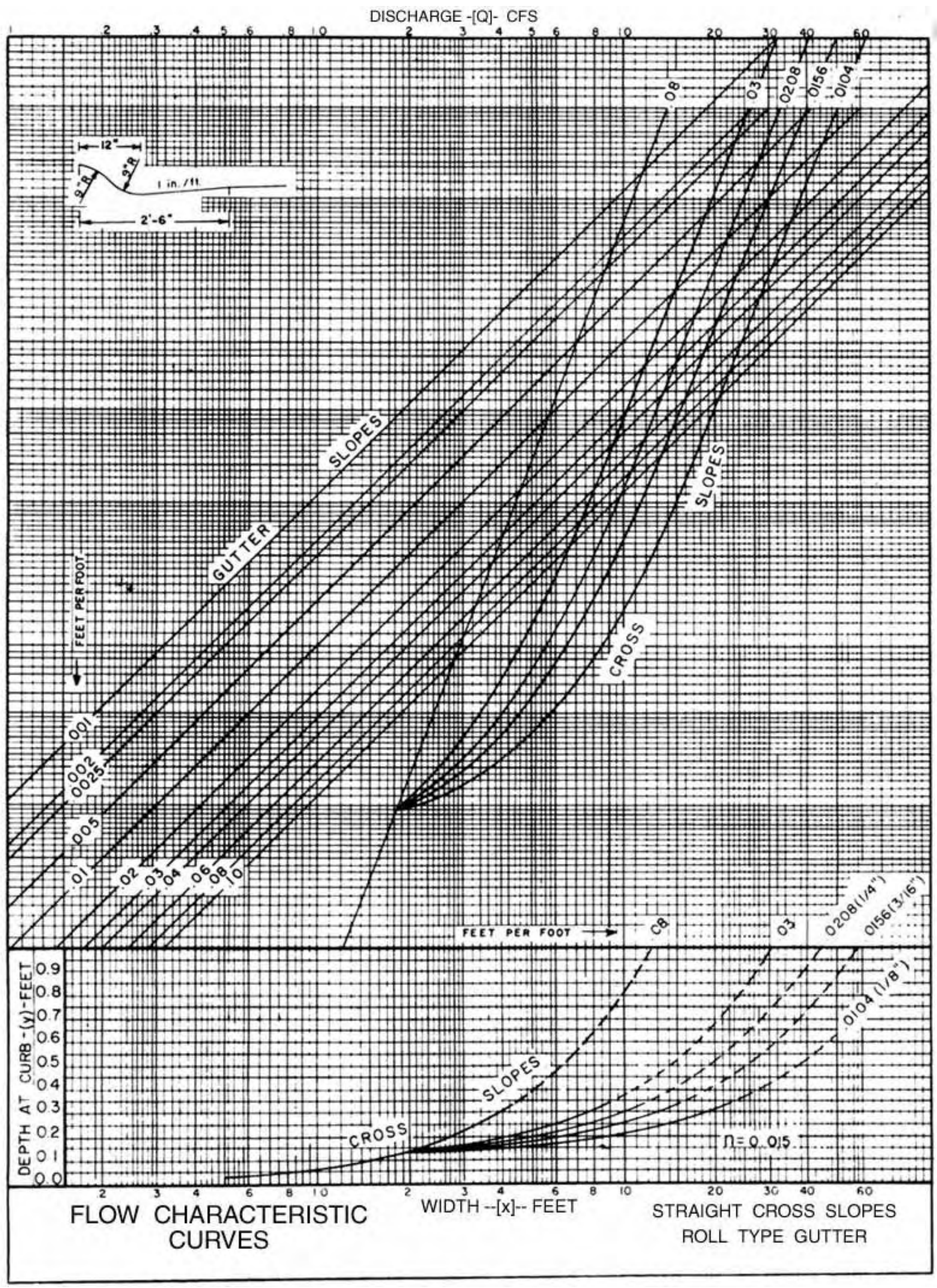
Source: VDOT
Comment: REV 6/85

Appendix 9C-5 Flow Characteristic Curves (Straight Cross Slope 12" Gutter)



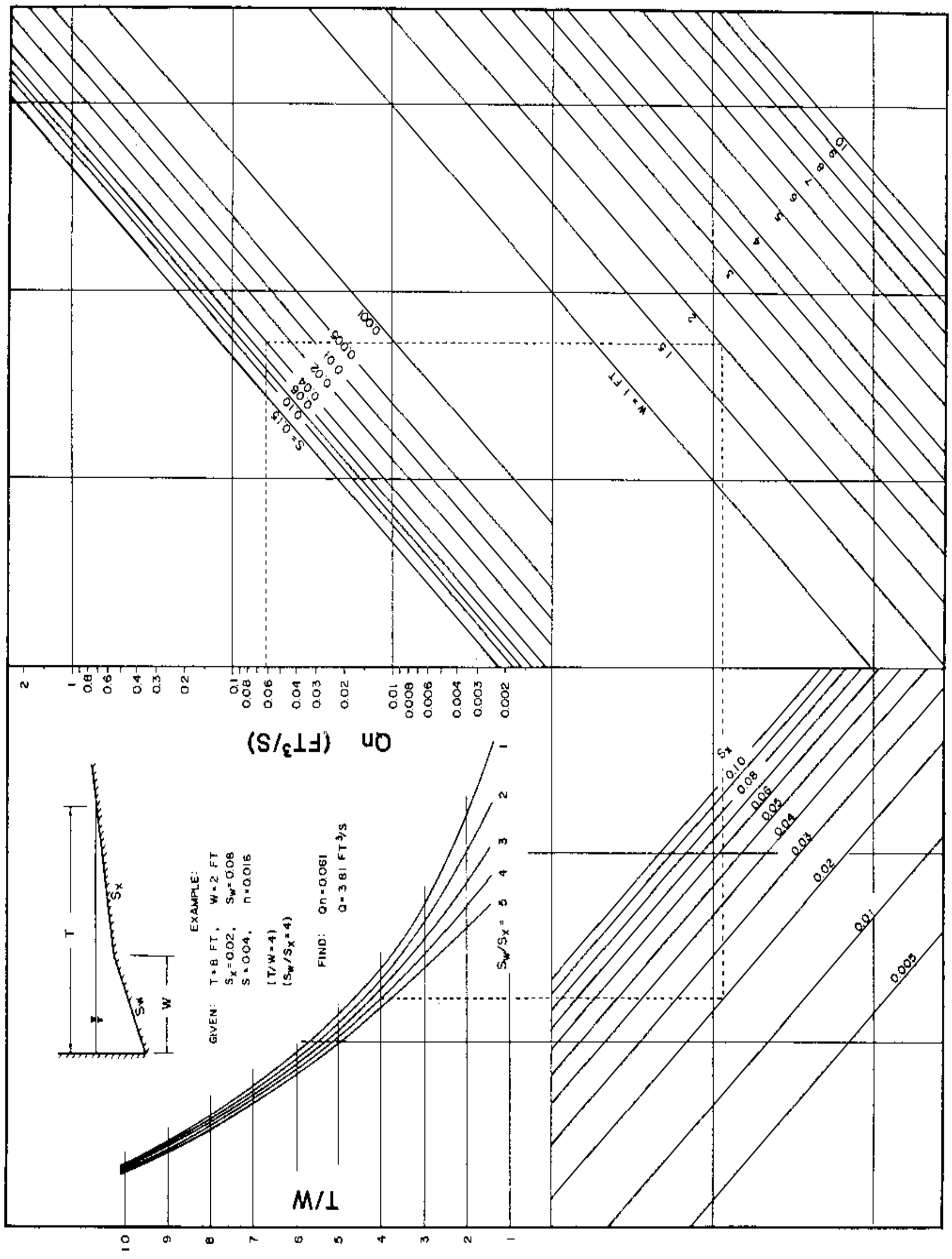
Source: VDOT
Comment: REV 6/85

Appendix 9C-6 Flow Characteristic Curve (Roll Type Gutter)



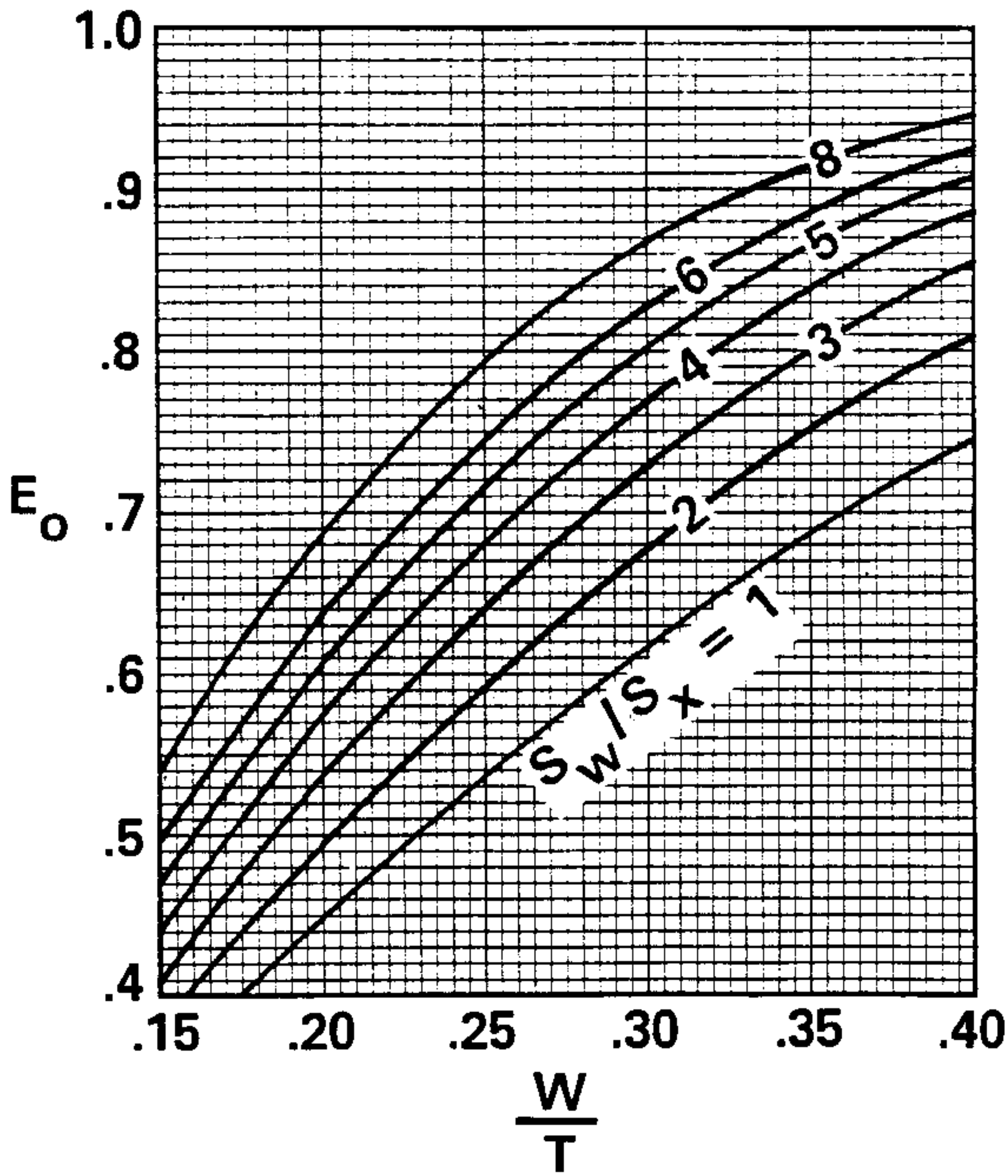
Source: VDOT
Comment: REV 6/85

Appendix 9C-7 Flow in Composite Gutter Sections



Source: HEC No. 12, FHWA

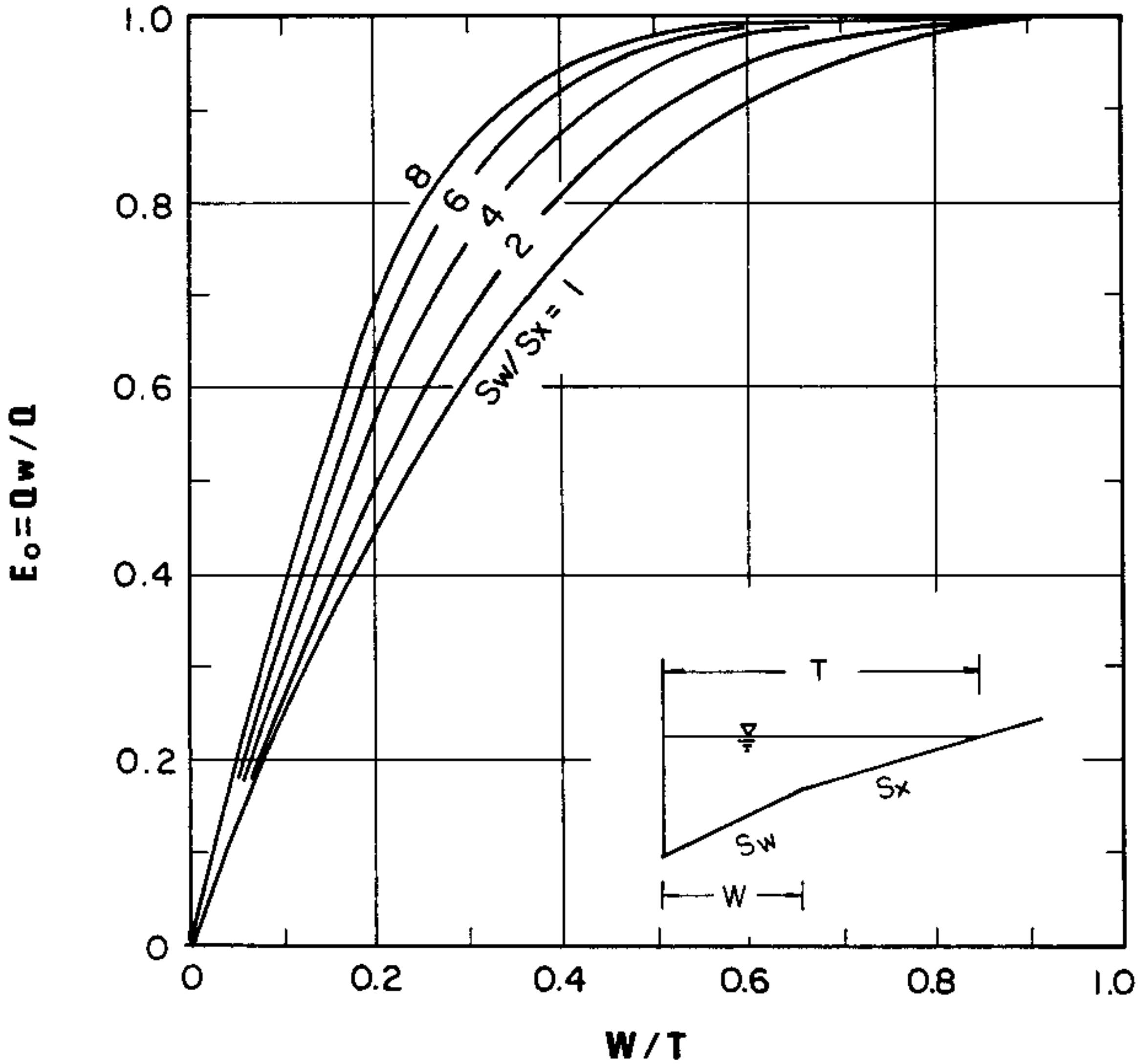
Appendix 9C-8 Ratio of Frontal Flow to Total Gutter Flow



For values of W/T greater than 0.4, use the chart on page 2 of this appendix.

Source: HEC No. 12, FHWA

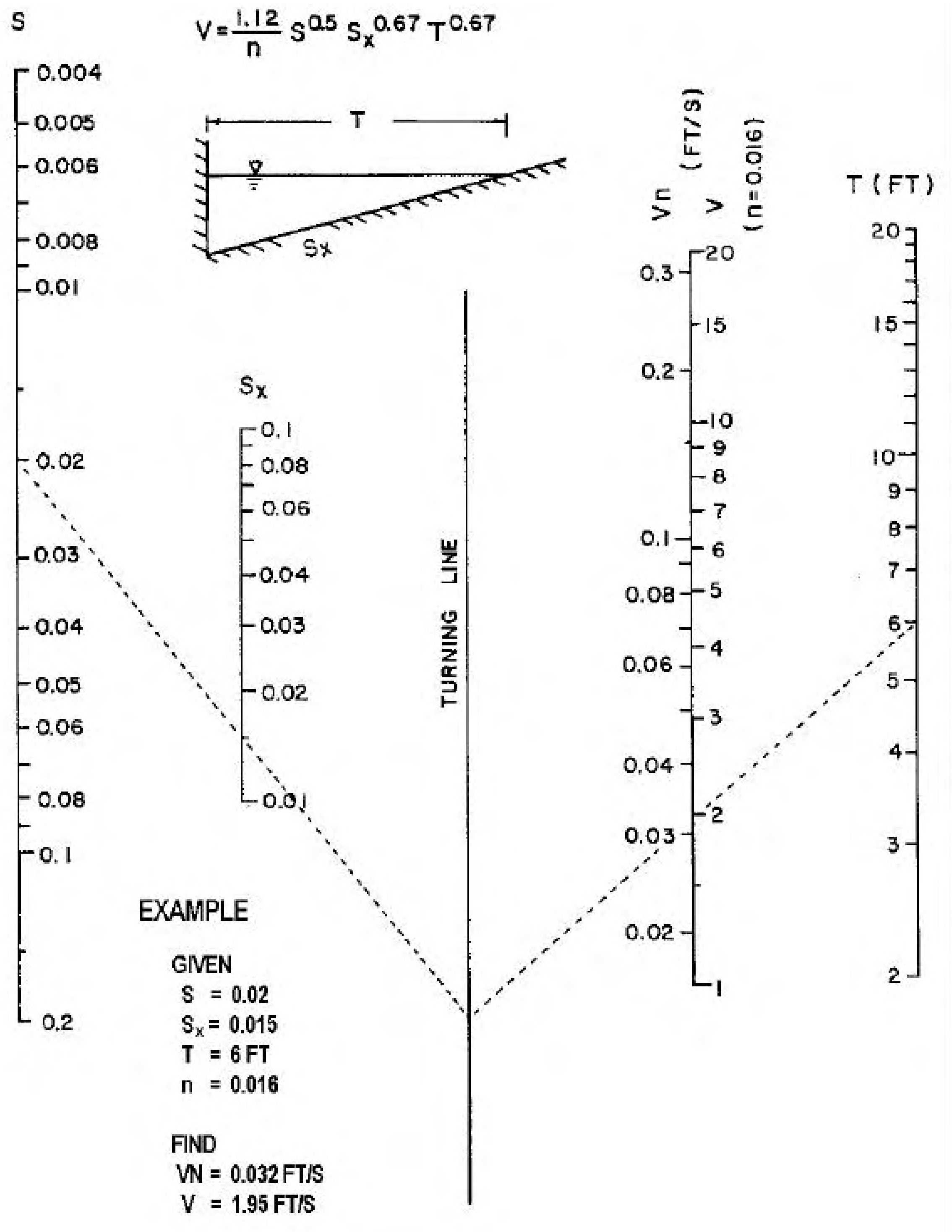
Appendix 9C-8 Ratio of Frontal Flow to Total Gutter Flow



If W/T is greater than 1.0, use W/T equal to 1.0.

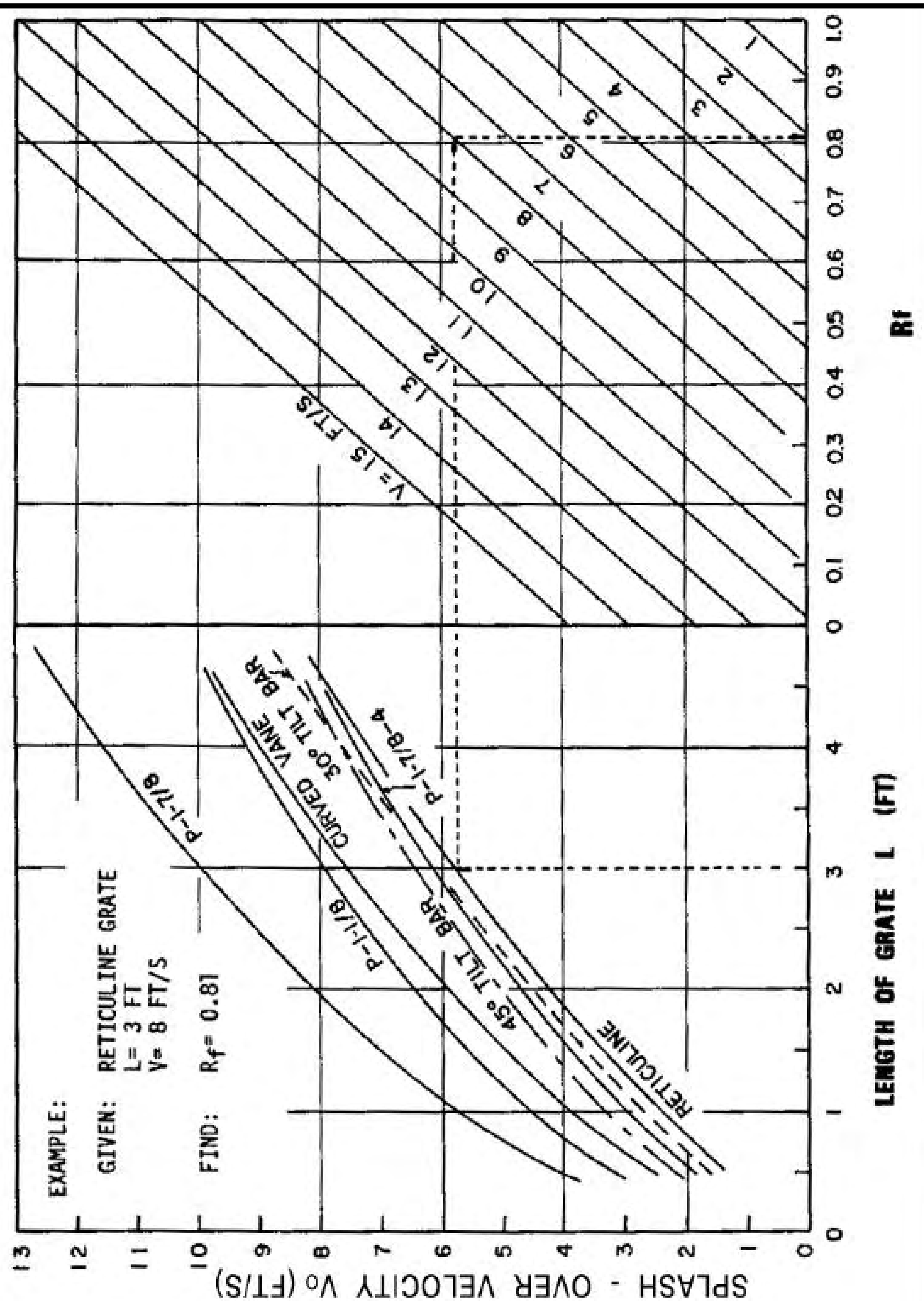
Source: HEC No. 12, FHWA

Appendix 9C-9 Velocity in Triangular Gutter Sections



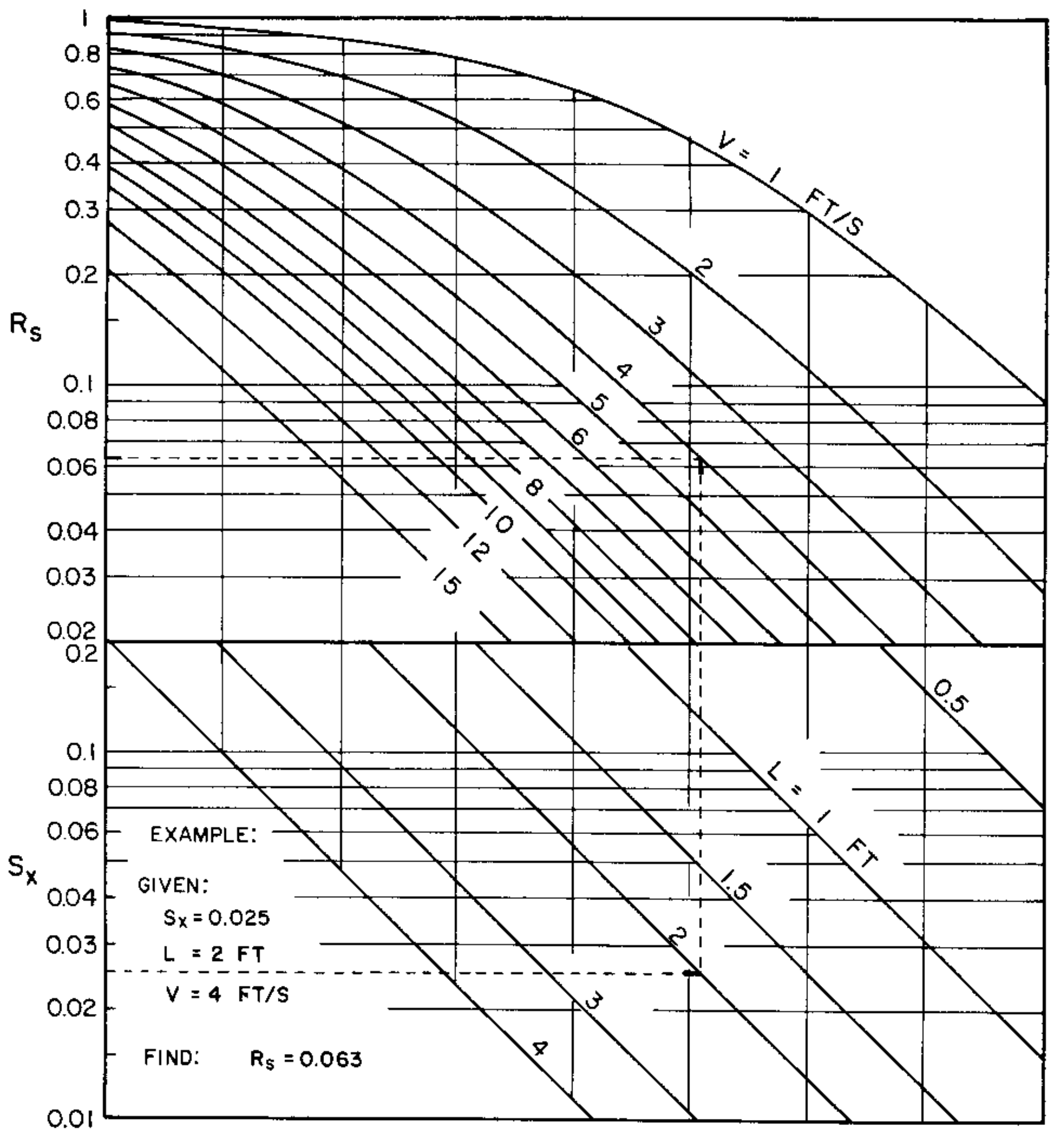
Source: HEC No. 12, FHWA

Appendix 9C-10 **Grate Inlet Frontal Flow Interception Efficiency**



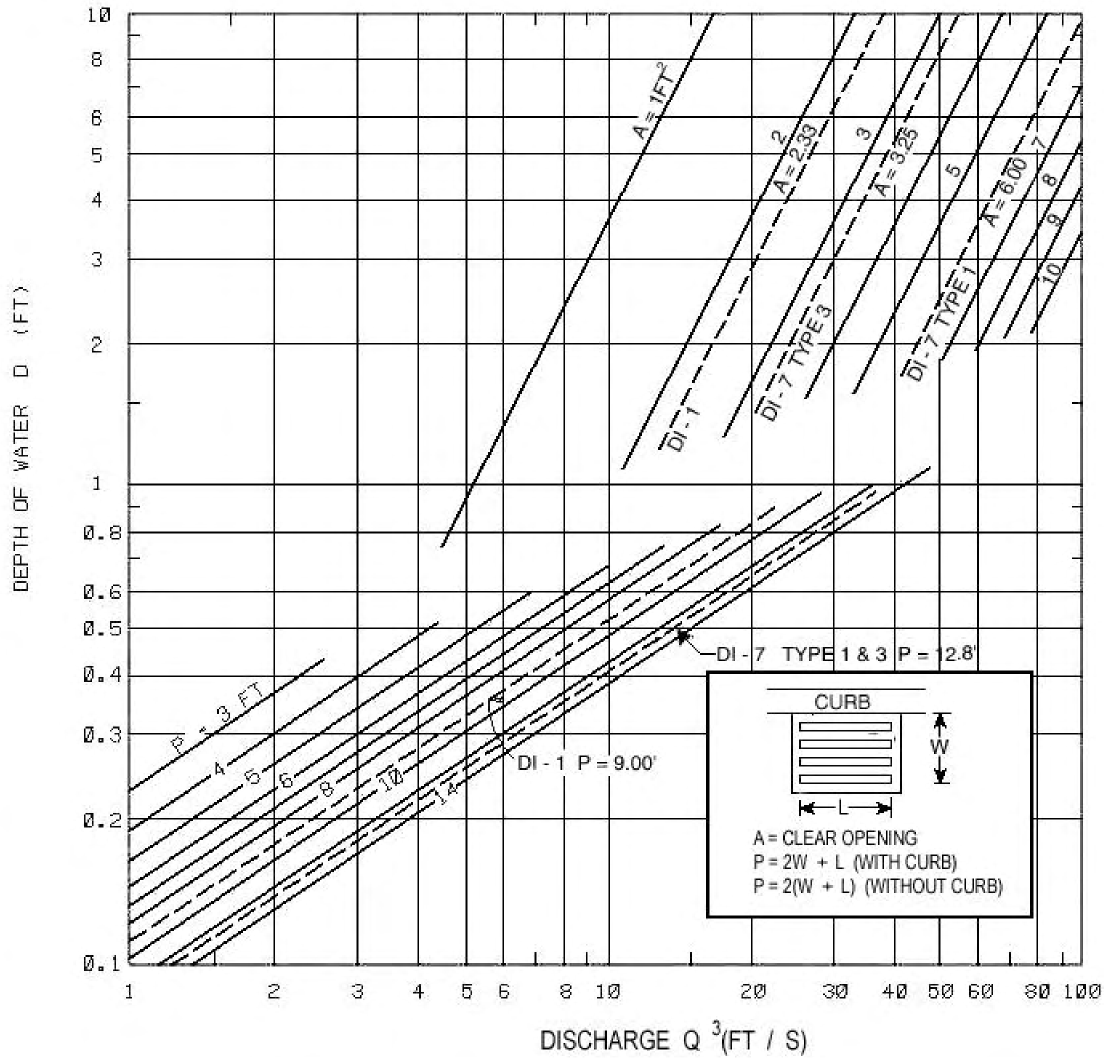
Source: HEC No. 12, FHWA

Appendix 9C-11 **Grate Inlet Side Flow Interception Efficiency**



Source: HEC No. 12, FHWA

Appendix 9C-12 Grate Inlet Capacity in Sump Conditions (VDOT Version)



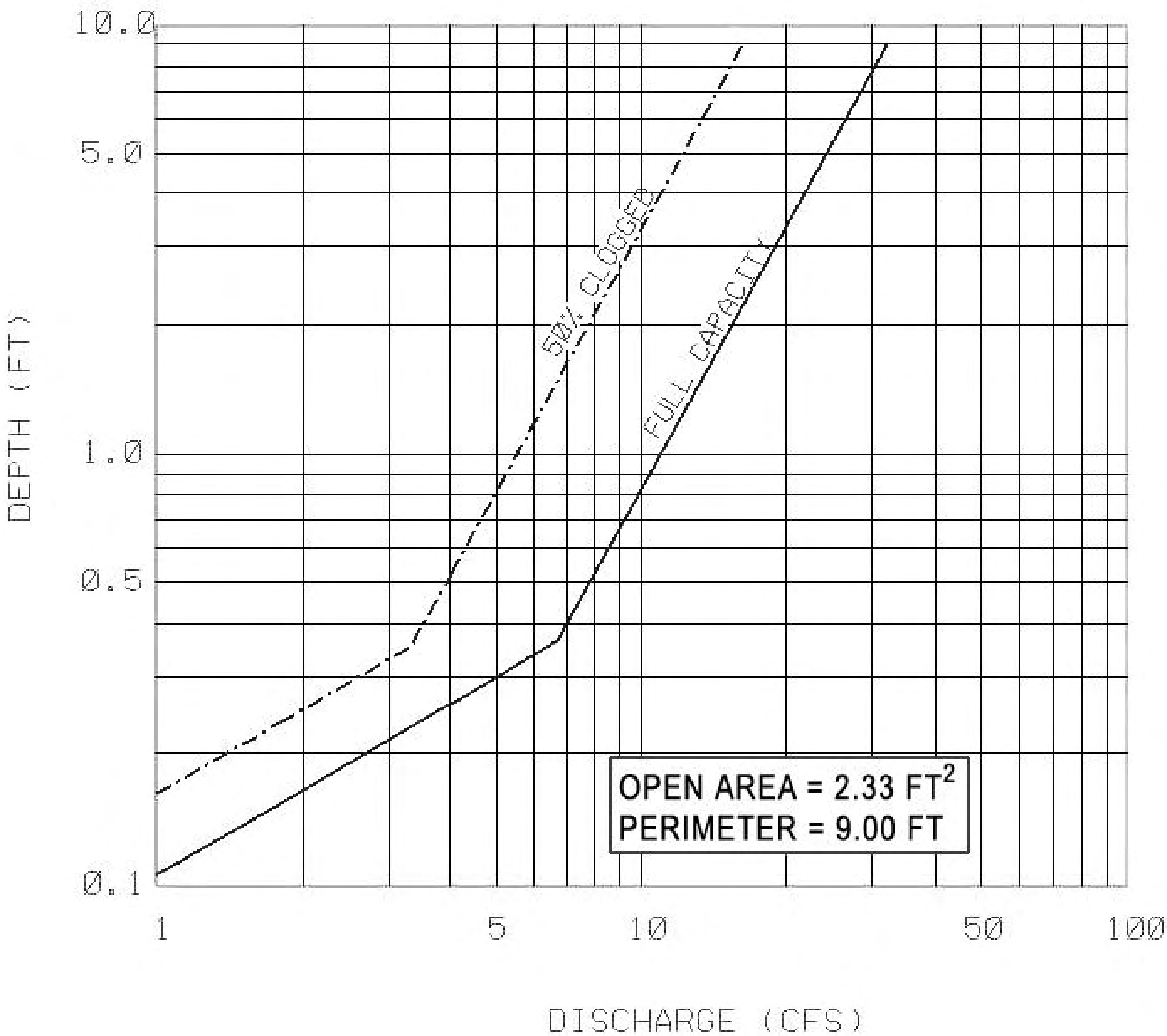
DISCHARGE Q 3 (FT / S)

VDOT STANDARD GRATE DROP INLETS

INLET TYPE	GRATE TYPE	AREA SQ-FT	PERIMETER FT	INLET TYPE	GRATE TYPE	AREA SQ-FT	PERIMETER FT
DI-1		2.33	9.00	DI-12 L=8'	TYPE I	9.00	20.50
					TYPE II	6.67	20.50
DI-7 & DI-5	TYPE !	6.00	12.80				
	TYPE III	3.25	12.80	DI-12 L=10'	TYPE I	11.24	24.50
					TYPE II	8.34	24.50
DI-9		0.36	3.66				
				DI-12 L=12'	TYPE I	13.50	28.50
DI-12 L=4'	TYPE I	4.50	12.50		TYPE II	10.00	28.50
	TYPE II	3.33	12.50				
				DI-12 L=14'	TYPE I	15.75	32.50
DI-12 L=6'	TYPE I	6.75	16.50		TYPE II	11.68	32.50
	TYPE II	5.00	16.50				

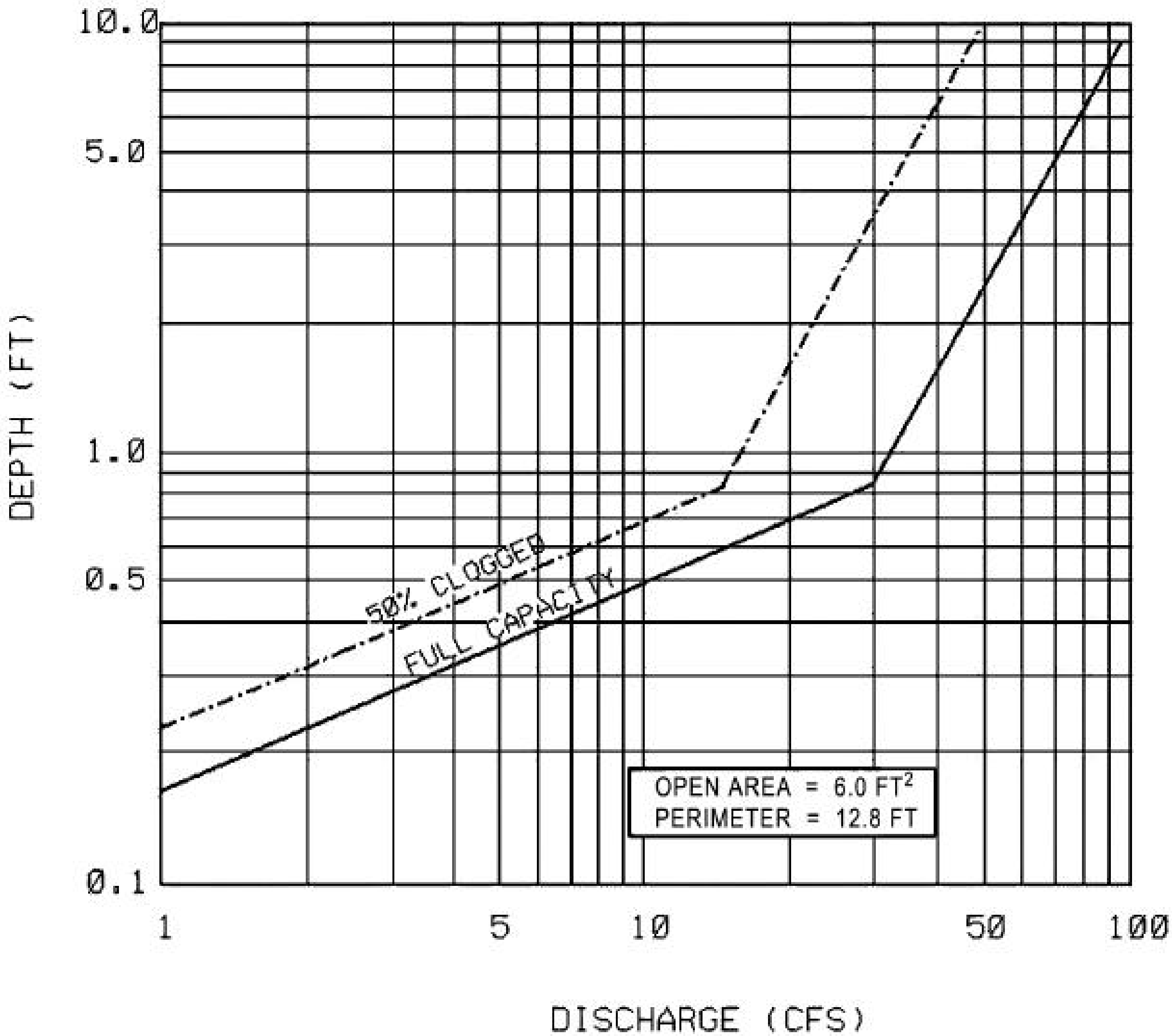
Source: VDOT

Appendix 9C-13 Performance Curve DI-1 in a Sump



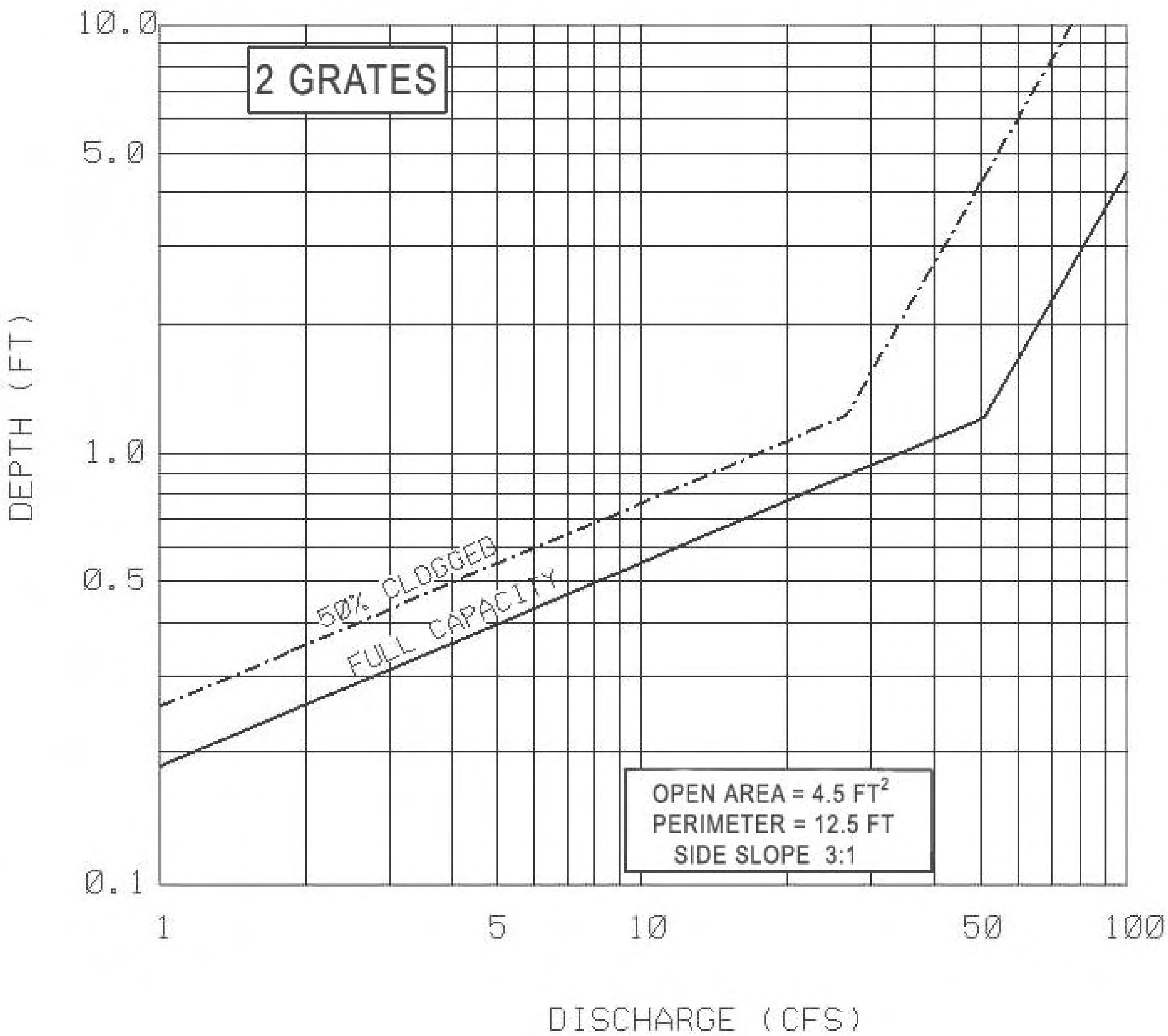
Source: VDOT

Appendix 9C-14 Performance Curve DI-7 in a Sump



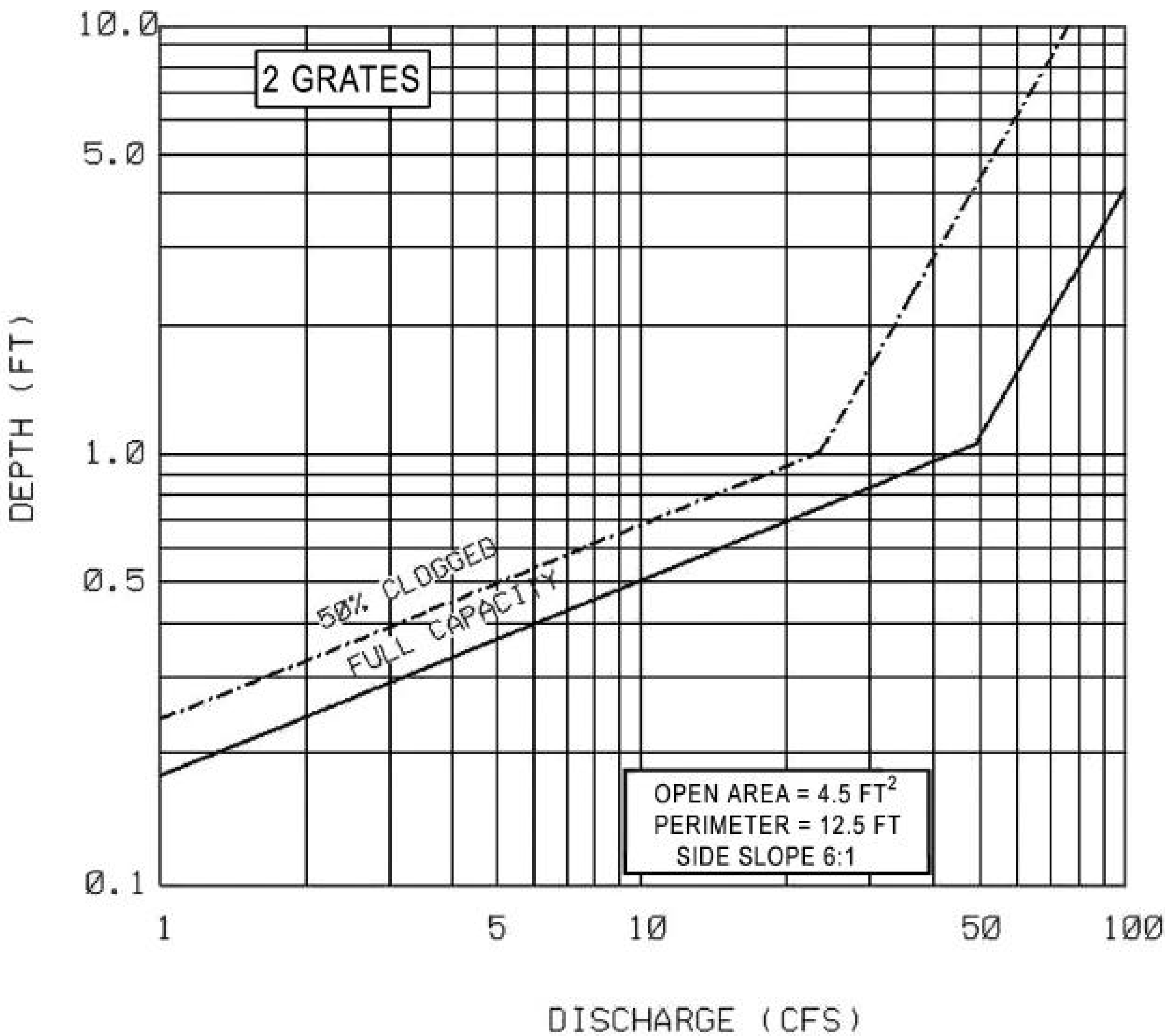
Source: VDOT

Appendix 9C-15 Performance Curve DI-12 in a Sump (Side Slope 3:1)



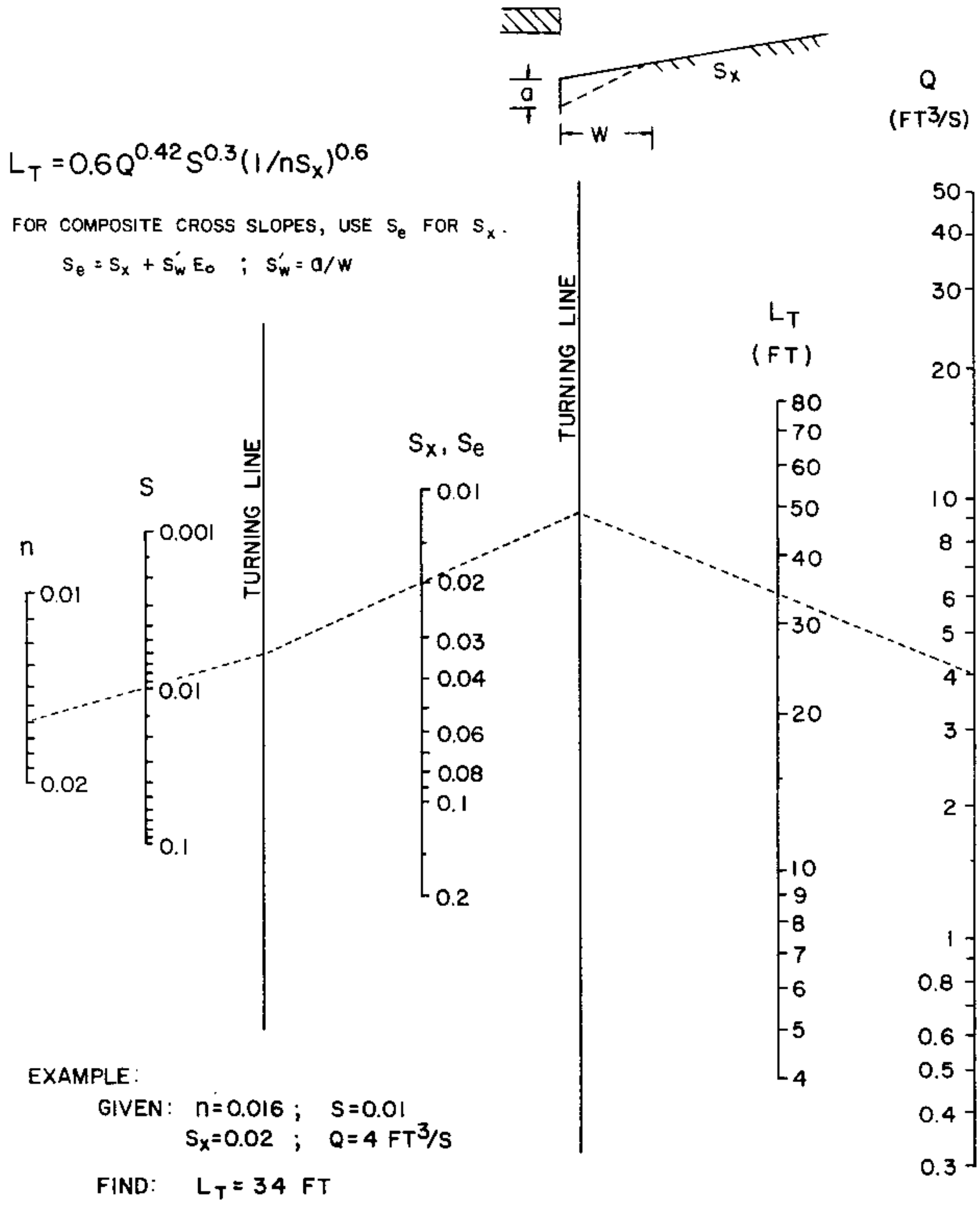
Source: VDOT

Appendix 9C-16 Performance Curve DI-12 in a Sump (Side Slope 6:1)



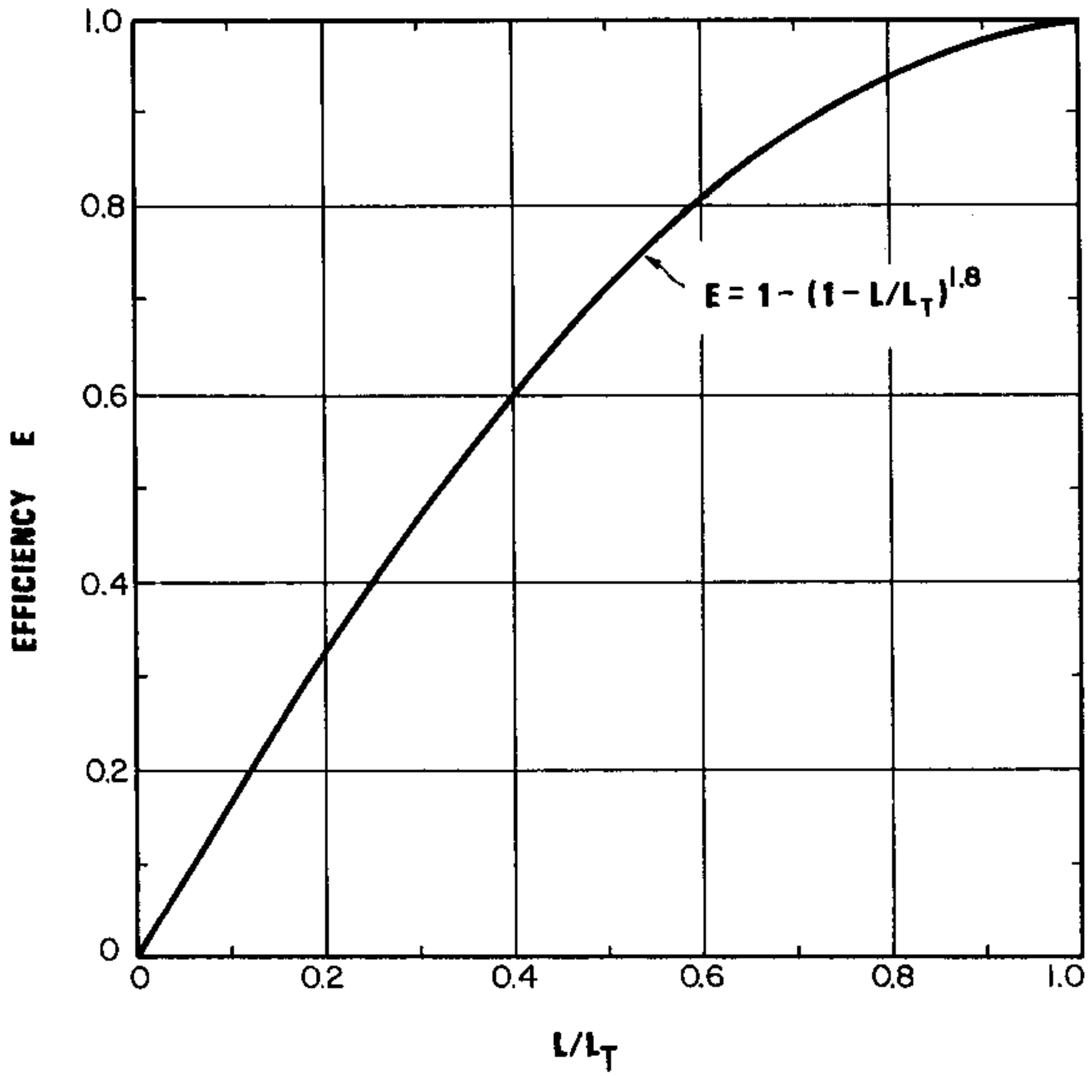
Source: VDOT

Appendix 9C-17 Curb-Opening and Slotted Drain Inlet Length for Total Interception



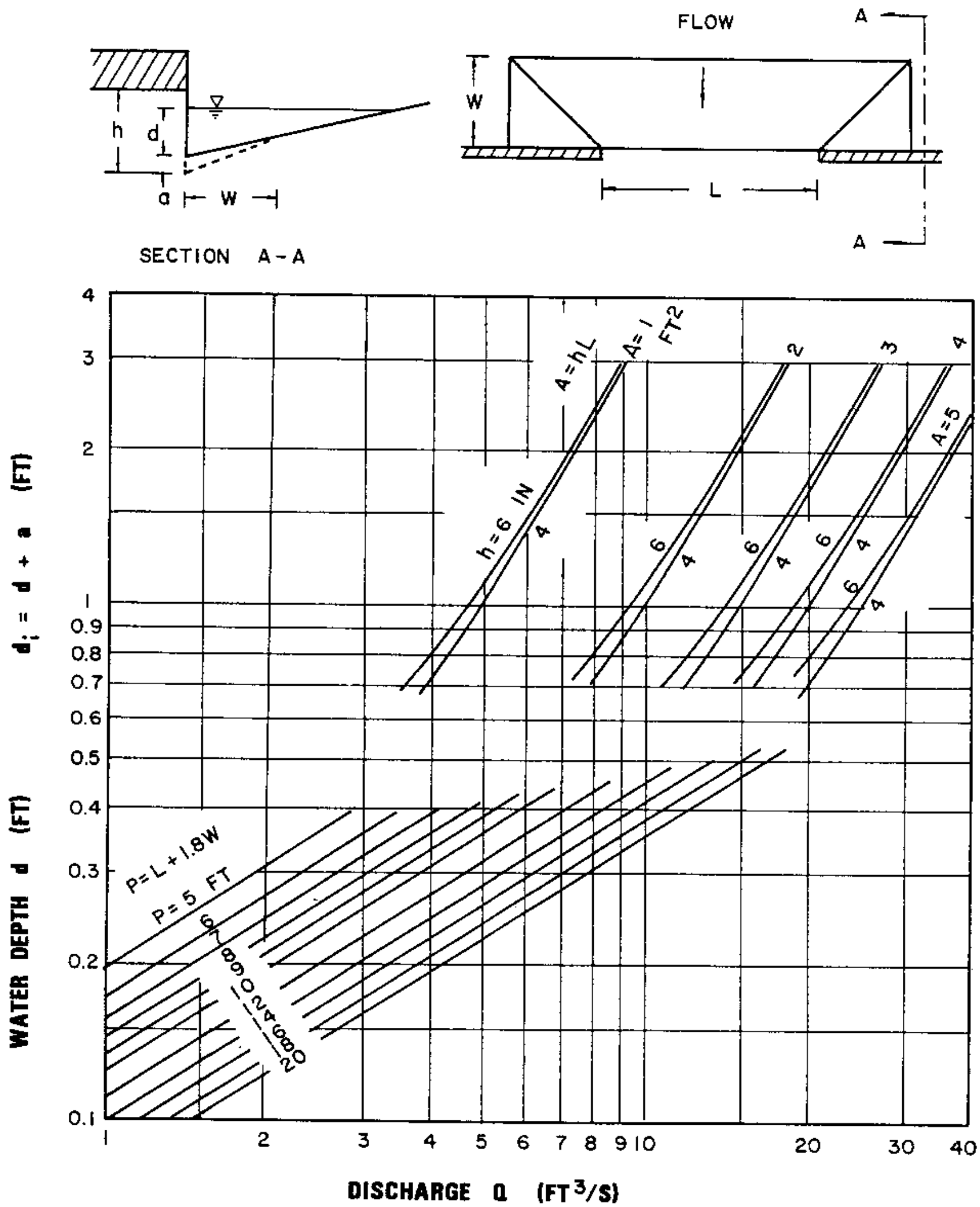
Source: HEC No. 12, FHWA

Appendix 9C-18 Curb-Opening and Slotted Drain Inlet Interception Efficiency



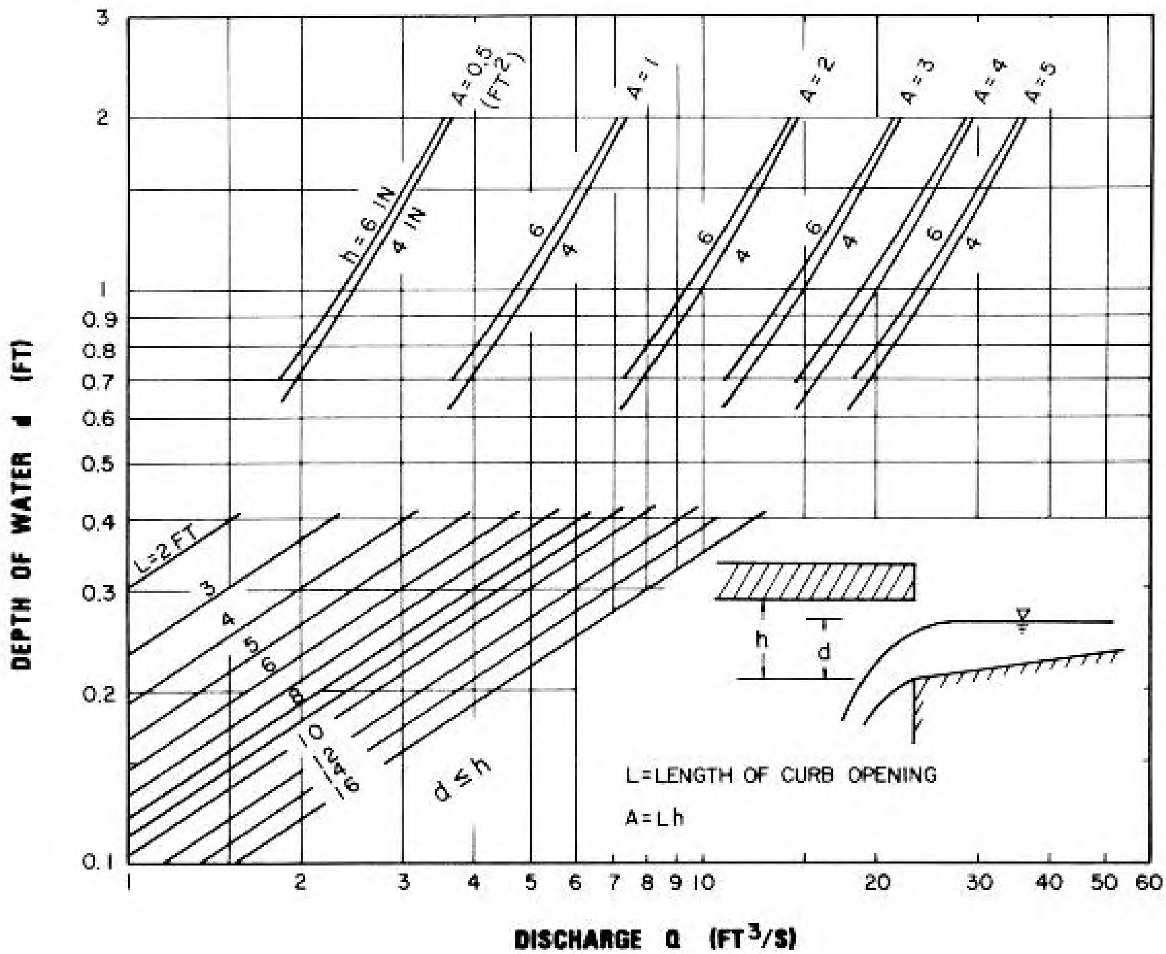
Source: HEC No. 12, FHWA

Appendix 9C-19 Depressed Curb-Opening Inlet Capacity in Sump Locations



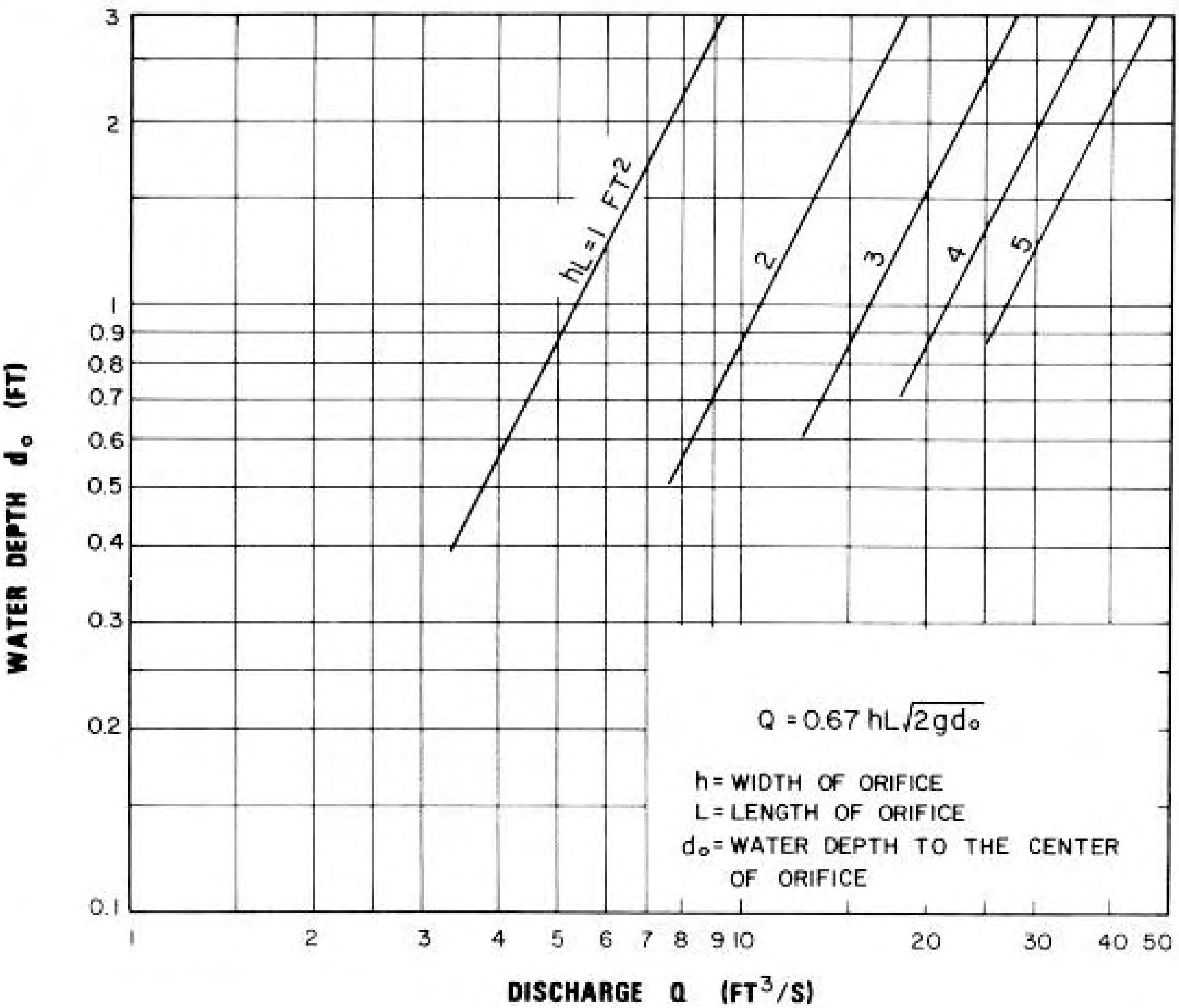
Source: HEC No. 12, FHWA

Appendix 9C-20 Curb-Opening Inlet Capacity in Sump Locations



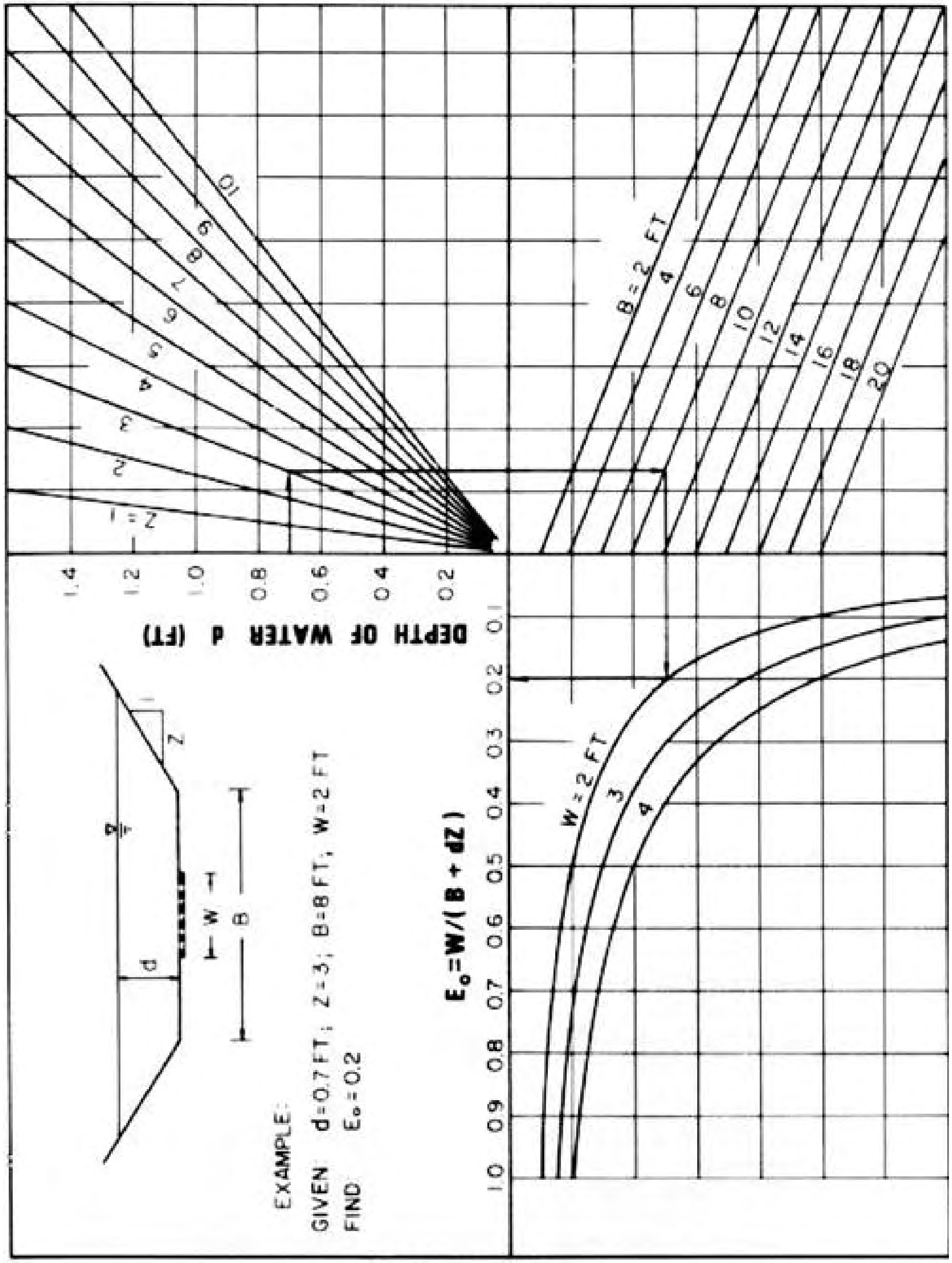
Source: HEC No. 12, FHWA

Appendix 9C-21 Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats



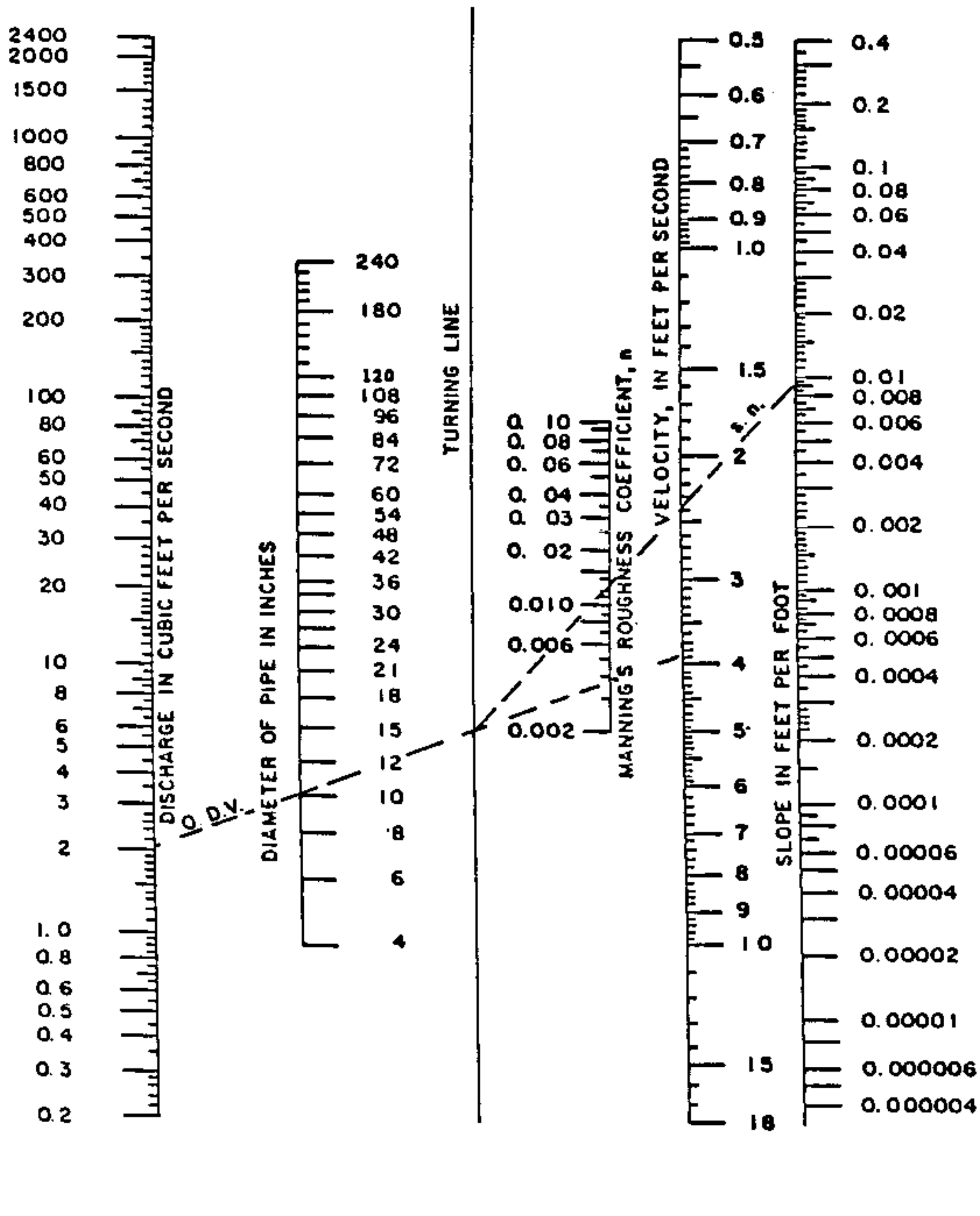
Source: HEC No. 12, FHWA

Appendix 9C-22 Ratio of Frontal Flow to Total Flow in a Trapezoidal Channel



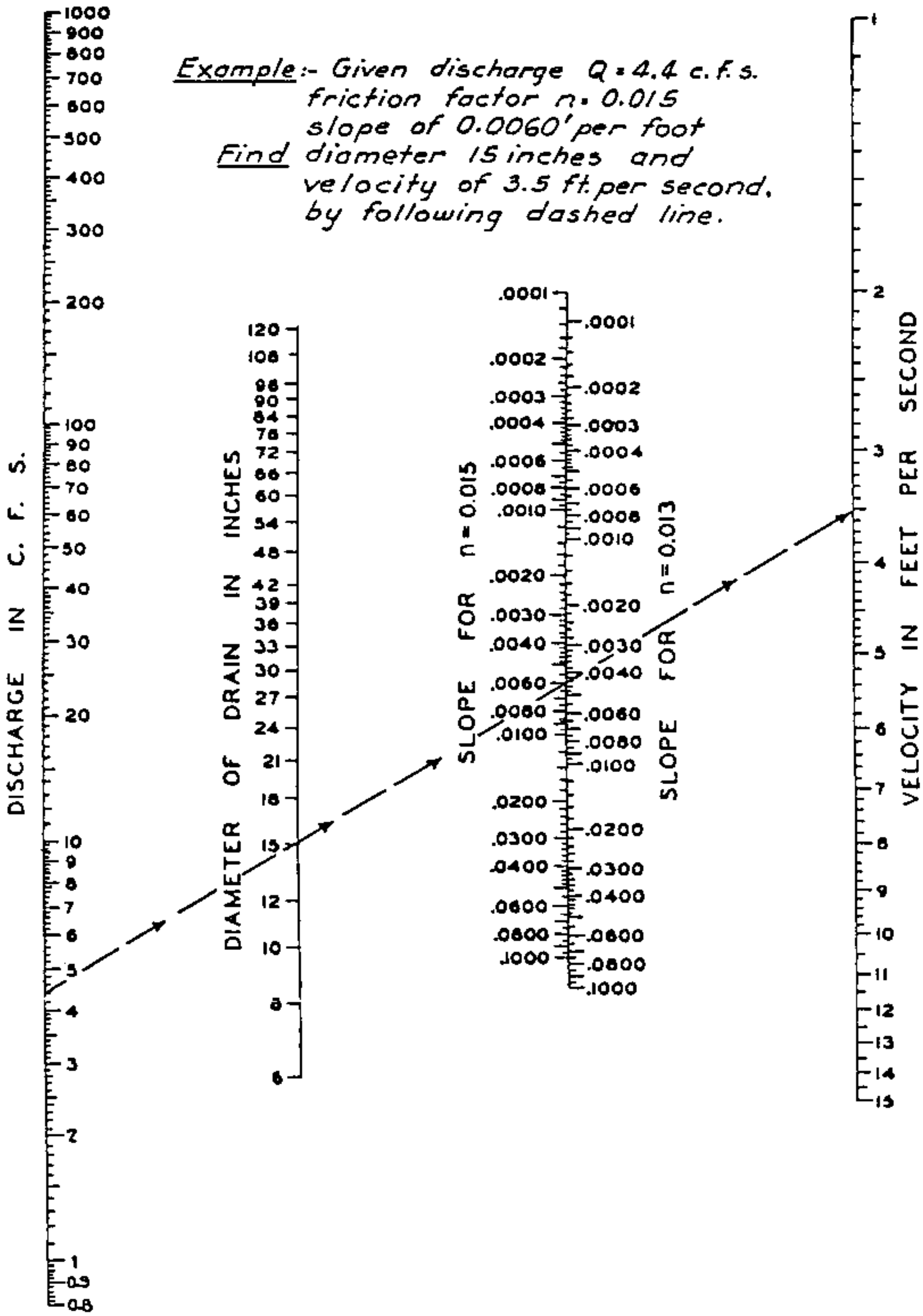
Source: HEC-12

Appendix 9C-23 **Manning's Solution for Flow in Storm Drains**



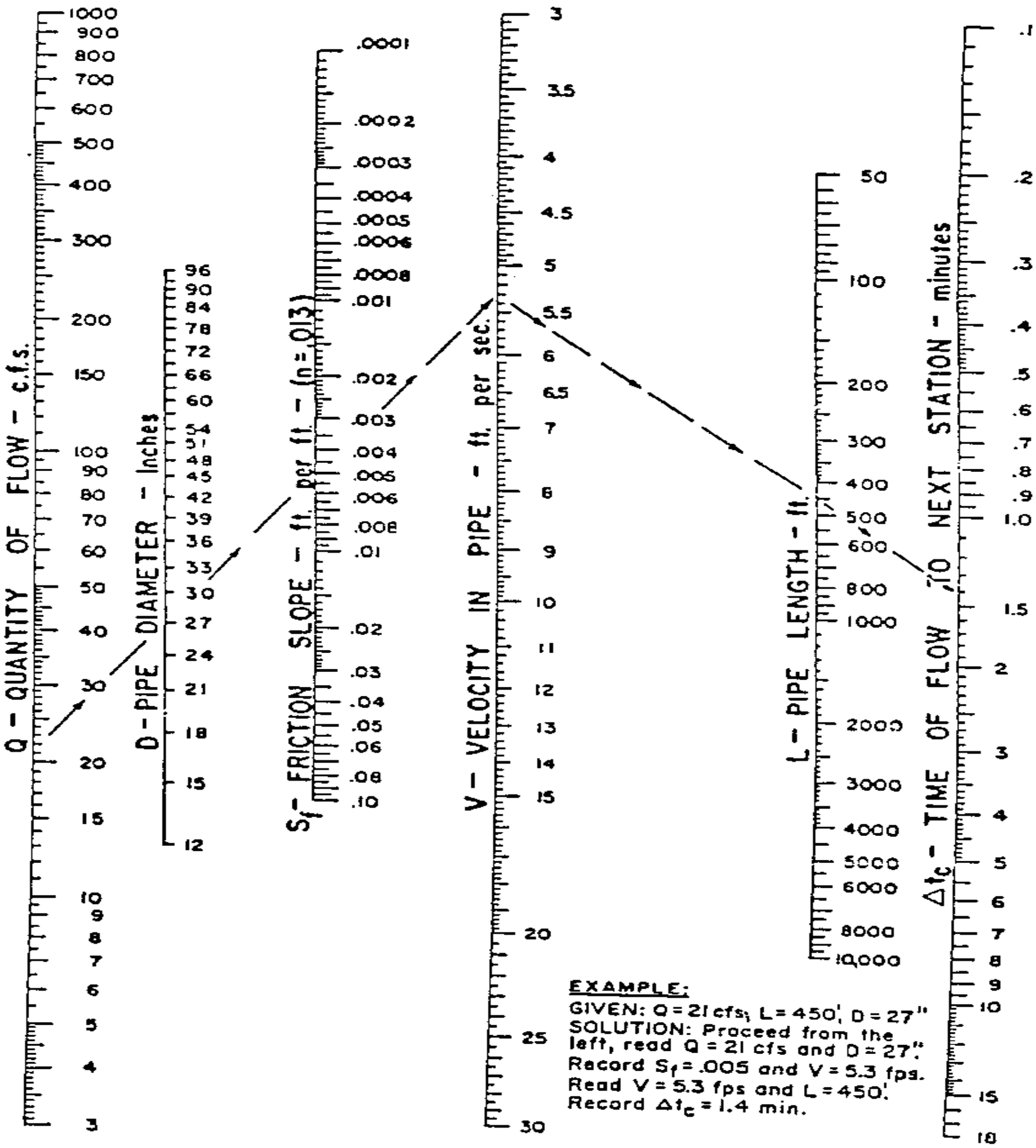
Source: HEC No. 22, FHWA

Appendix 9C-24 Nomograph for Computing Required Size of Circular Drain for Full Flow (n=0.013 or 0.015)



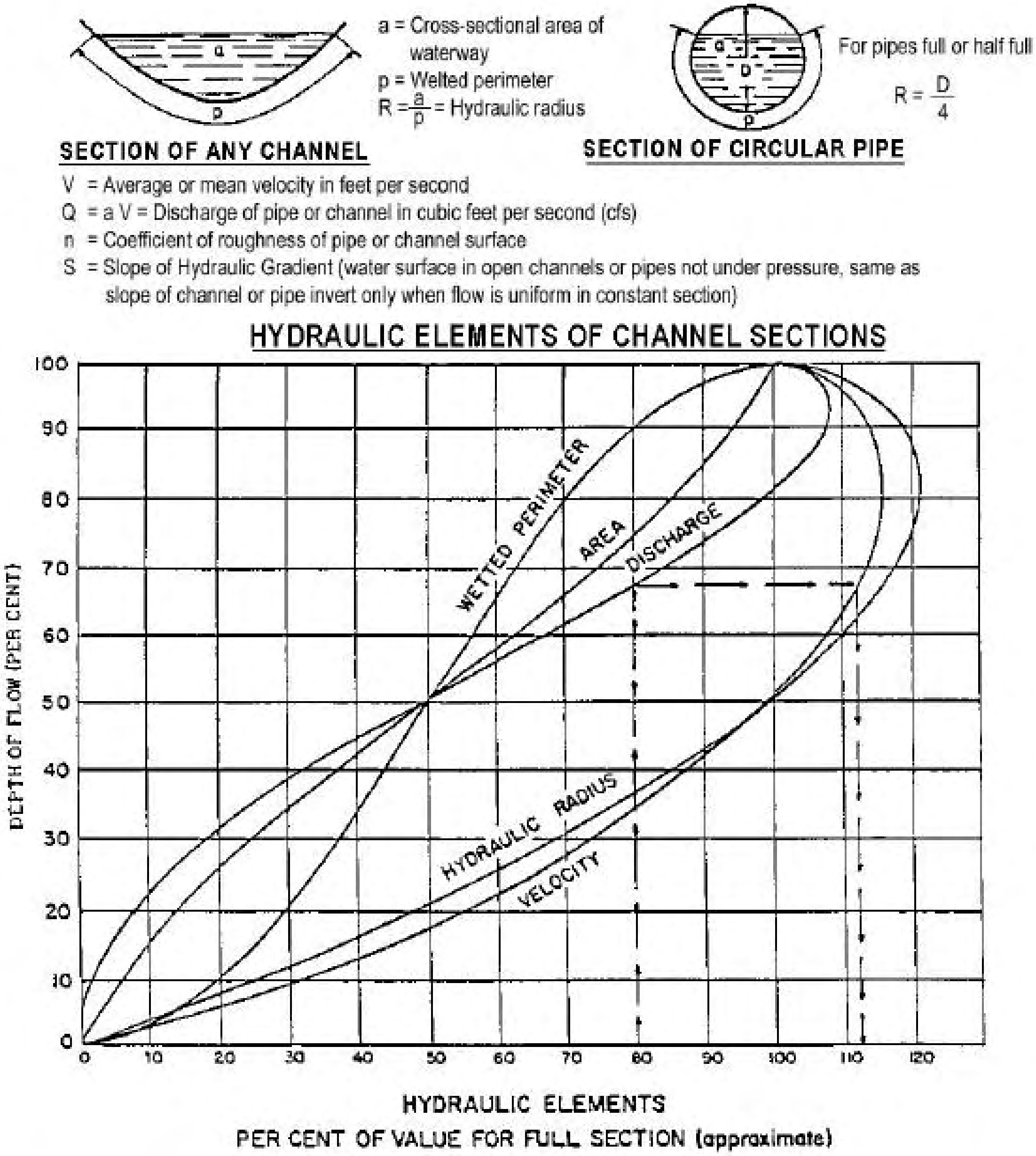
Source:

Appendix 9C-25 Concrete Pipe Flow Nomograph



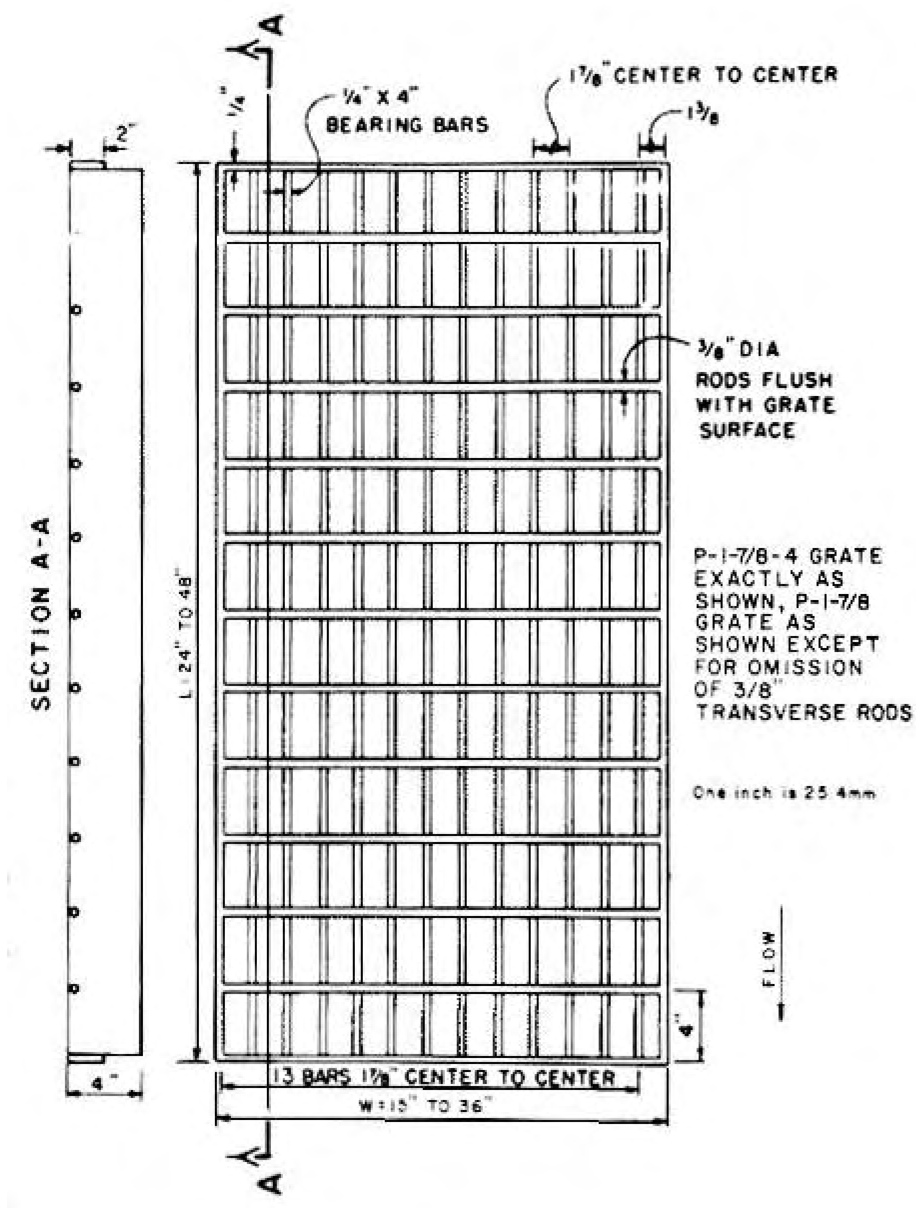
Source:

Appendix 9C-26 Values of Hydraulic Elements of Circular Section for Various Depths of Flow



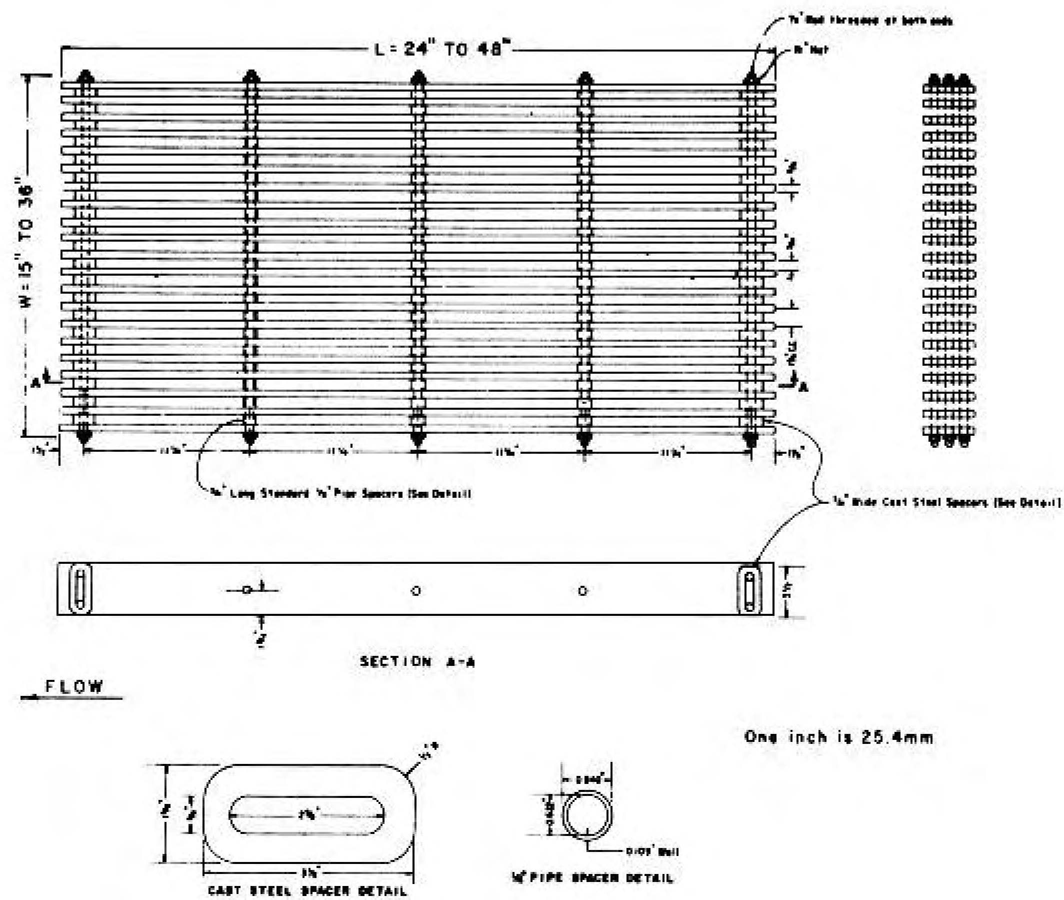
Source: HEC No. 22

Appendix 9D-1 P-1-7/8 and P-1-7/8 4 Grates – FHWA Classification



Source: HEC-12

Appendix 9D-2 P-1-1/8 Grate – FHWA Classification



Source: HEC-12

Chapter 10 - Erosion and Sediment Control

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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 effectively control soil erosion and prevent sediment from leaving the construction site in accordance with the Virginia Erosion and Sediment Control Handbook and the Virginia Erosion and Sediment Control Regulations. This Handbook can be ordered from the Virginia Department of Conservation and Recreation, Division of Soil and Water Conservation at (804) 786-2064 or from the website, <http://www.dcr.state.va.us/sw/e&s.htm>.

The Virginia Erosion and Sediment Control Regulations (VESCR) (4VAC50-30-40) can be accessed from the website, <www.dcr.state.va.us/sw/docs/eslawrgs.pdf>.

10.1.2 Principal Factors Influencing Erosion

10.1.2.1 Soil Characteristics

The properties of soil which influence erosion by rainfall and runoff are ones which affect the infiltration capacity of a soil and those which affect the resistance of a soil to detachment and being carried away by falling or flowing water. 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.

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 evapo-transpiration

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.

10.2 Design Policy

The 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 and the 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. There are also federal control requirements exerted by numerous agencies such as the Corps of Engineers (COE), Department of Conservation and Recreation (DCR), Environmental Protection Agency (EPA), Fish and Wildlife Service (FWS), etcetera, through their administration of various permitting requirements (Section 404, Section 402 of the Federal Water Pollution Control Act (FWPCA), and Section 9 and 10 of the River and Harbor Act).

10.2.2 State Policy

The Department of Conservation and Recreation annually reviews and approves VDOT’s Erosion and Sediment Control Plan. This Annual Plan 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.

Any maintenance or construction activity that disturbs more than 10,000 square feet (929 m^2) in areas west of Route 95, or 2,500 square feet (232 m^2) in areas within the Chesapeake Bay Watershed, must have a specific erosion and sediment control plan developed and implemented in accordance with VDOT’s Erosion and Sediment Control Annual Plan. The requirements of the Virginia Erosion and Sediment Control Regulations (VESCR), <http://www.dcr.state.va.us/sw/docs/eslawrgs.pdf>, and the VDOT

Erosion and Sediment Control Annual Plan will be incorporated into every design and will be enforced on all VDOT operations.

Refer to the latest Location and Design Instructional and Informational Memorandum I&IM LD (D) 11 for additional policy and design guidelines.

10.3 Documentation

10.3.1 Design Documentation

Drainage designers should use the guidelines and checklists such as those provided in the Virginia Erosion and Sediment Control Handbook (VESCH) 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 should be supported by engineering calculations which should be included as a part of the project's drainage report. Instruction for designing erosion and sediment control measures can be found in the VESCH.

10.4 References

AASHTO, Highway Drainage Guidelines, Volume III, Erosion and Sediment Control in Highway Construction.

Division of Soil and Water Conservation, Virginia Department of Conservation and Recreation, 1980. Virginia Erosion & Sediment Control Handbook, Second Edition.

Appendix 10A-1 Definitions and Abbreviations

Abbreviations:

DCR	Department of Conservation and Recreation
EPA	Environmental Protection Agency
FEMA	Federal Emergency Management Agency
FWPCA	Federal Water Pollution Control Act
FWS	Fish and Wildlife Service
USCOE/USACE	United States Corps of Engineers
VESCH	Virginia Erosion and Sediment Control Handbook
VESCR	Virginia Erosion and Sediment Control Regulations
VDOT	Virginia Department of Transportation

Chapter 11 – Stormwater Management

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CHAPTER 11 - STORMWATER MANAGEMENT

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Chapter 11 - Stormwater Management

11.1 Introduction

Development of watersheds generally causes an increase in the peak rate of stormwater runoff. This increase is often the cause of flood damage, erosion, and siltation control problems. 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 definite, or inadequate, outfall, the total volume of runoff is critical and storage facilities can be used to store the increases in volume and control discharge rates.

11.1.1 Objective

The goal of stormwater management 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 peak discharge to match the non-detrimental discharge capacity of the downstream drainage system.

Stormwater Quality Control

Stormwater quality control pertains to reducing the amount of constituents generated by land development projects.

Stormwater Quantity Control

Stormwater quantity control, or flooding and erosion control, pertains to reducing the water quantity post-development runoff characteristics, as nearly as practicable, equal to or better than the pre-development runoff characteristics.

11.2 Design Policy

11.2.1 General

Acts of the General Assembly have resulted in the issuance of Stormwater Management Regulations (SWMR) and Erosion and Sediment Control Regulations (VESCR). These regulations can be obtained at the Department of Conservation and Recreation (DCR) Soil and Water Division website, <www.dcr.state.va.us/sw/>. The general application to highway drainage design associated with these regulations is addressed here and also in VDOT Drainage Design Memorandum, DDM #2, "Management of Stormwater", Appendix 11F-1. Water quantity control is governed by the Virginia Erosion and Sediment Control Regulations Minimum Standard 19 (MS-19), which requires an adequate receiving channel for stormwater outflows from all projects with more than 10,000 square feet of land disturbance and can be obtained at <www.dcr.state.va.us/sw/>.

- Land development projects, including both linear development projects, such as roadways and site development projects, such as parking lots, buildings and weigh stations shall comply with SWMR and VESCR. **[4 VAC 3-20-60 (L)]**
- State projects within the Chesapeake Bay Preservation Area complying with SWMR and VESCR must comply with the Chesapeake Bay Preservation Act (CBPA).
- Stormwater management plans prepared for state projects shall comply with the criteria specified in SWMR and to the maximum extent practicable, any local stormwater management requirements adopted pursuant to the SWMR.
- For land development projects, the post-development stormwater runoff from the impervious cover should be treated with the most appropriate best management practice (BMP). **[4 VAC 3-20-71 (C)]**
- Outflows from the stormwater management facilities should be discharged into adequate receiving channels. **[4 VAC 3-20-60 (G)]**
- Downstream properties and waterways should be protected from erosion and damages from localized flooding due to increases in volume, velocity, and peak flow rate of stormwater runoff. **[4 VAC 3-20-81 (A) and 4 VAC 3-20-85 (A)]**
- Stormwater discharges in environmentally sensitive areas may be subject to additional stormwater requirements.
- Impounding structures (dams) that are not covered by the Virginia Dam Safety Regulations should be checked for structural integrity and floodplain impacts for the 100-year storm event. **[4 VAC 3-20-60 (E)]**
- Construction of stormwater management facilities within FEMA designated 100-year floodplains should be avoided to the extent possible. When this is unavoidable, the construction of stormwater management facilities in floodplains should be in compliance with all applicable regulations under the National Flood Insurance Program (NFIP). **[4 VAC 3-20-71 (J)]**

11.3 Design Criteria

11.3.1 General

The design criteria for stormwater management facility design addresses the following:

- Water quality volume
- Water quantity volume
- Allowable peak discharges
- Grading and depth requirements for excavated basins and embankments
- Sediment forebays
- Physical requirements
- Environmental impacts
- Integration with roadway
- Maintenance requirements

11.3.2 Quality

Stormwater management 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 one half-inch of runoff multiplied by the total impervious area of the land development project.

BMP requirements for quality control are “technology-based” and the type of BMP is determined by the percent of **new** impervious area and the area within the right-of-way and easements at each project outfall. As these methods are studied and monitored, the design criteria for determining the WQV may be refined to achieve a greater overall level of treatment. Table 11-1 shows various BMPs as a function of the percentage of new impervious cover.

Table 11-1. BMP Selection Table

Water Quality BMP	Treatment Volume	Target Phosphorus Removal Efficiency	Percent Impervious Cover**
Vegetated filter strip Grassed swale		10% 15%	16-21%
Constructed wetlands Extended detention Retention basin I	2xWQV 2xWQV 3xWQV	30% 35% 40%	22-37%
Bioretention basin Bioretention filter Extended detention-enhanced Retention basin II Infiltration	 4xWQV 1xWQV	50% 50% 50% 50% 50%	38-66%
Sand filter Infiltration Retention basin III with aquatic bench	 2xWQV 4xWQV	65% 65% 65%	67-100%

* Innovative or alternate BMPs not included in this table may be allowed at the discretion of DCR and VDOT.

** Percent Impervious Cover: The ratio of the **new** impervious area and the area within the right-of-way and easements per project outfall.

11.3.3 Quantity

The Virginia Erosion and Sediment Control Regulations Minimum Standard 19 (MS-19) and Virginia Stormwater Management Regulations shall govern water quantity control. The following criteria apply:

- Pre-development conditions should be that which exist at the time the road plans are approved for right-of-way acquisition.
- All land cover should be assumed to be in good condition regardless of actual existing conditions at the time design begins.
- An adequate receiving channel is required for stormwater outflows from all projects with more than 10,000 square feet of land disturbance.
- Natural channels should be analyzed by use of a 2-year storm to verify that stormwater will not overtop channel banks or cause erosion of the channel bed and banks.
- All previously constructed man-made channels should be analyzed by use of a 10-year storm to verify that stormwater will not overtop its banks and by use of a 2-year storm to demonstrate that the stormwater will not cause erosion of the channel bed or banks.
- Pipes and storm drain systems should be analyzed by use of a 10-year storm to verify that the stormwater will be contained within the pipe or storm drain system.

- The receiving channel at a pipe or storm drain outlet should be analyzed by use of a 2-year storm for natural channels or the 10-year storm for man made channels to verify that stormwater will not overtop the banks.
- Water quantity control for the 1-year storm (in lieu of the 2-year storm required by Minimum Standard 19) may be needed if there is existing or anticipated erosion downstream.
- Existing swales being utilized as natural outfall conveyances for pre-development runoff will be considered as channels. If the swale satisfactorily meets the criteria contained in MS-19 of the VESCR for post-development runoff, it will be considered an adequate receiving channel.
- 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 of the project, the receiving channel may be considered adequate, with respect to channel stability requirements under the VESCR, without further computation.

11.3.4 Exemptions

Linear development (highway) projects are <u>exempt</u> from the stormwater management regulations provided that:

- Less than one acre of new impervious area will be added per outfall and
- There will be an insignificant increase in peak flow rates and
- There are no existing or anticipated flooding or erosion problems downstream

The designer must consider that linear development projects are not exempt from VESCR and must meet MS-19 criteria.

For locations where water quality control is required and there is an adequate receiving channel, the BMP may not need to be designed for quantity control. In these situations, the dam and the emergency spillway should be designed to safely pass the 100-year storm.

Where two or more outfalls flow directly into an adjacent waterway or where two or more outfalls converge into one waterway a short distance downstream of the project, the combined additional impervious area of all affected outfalls should be considered when determining whether treatment is required.

11.3.5 Compensatory Treatment

Compensatory treatment for water quality requirements (overtreating 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 SWM facilities at the treated outfall are designed to account for the water quality volumes for those areas where SWM facilities 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 SWM facilities 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.

11.3.6 Embankment (Dam)

The following details are to be incorporated into the design of dams for VDOT stormwater management (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 that about 2 feet for water quality and about 4 feet for the 10-year storm (Q_{10}) quantity control.

- Foundation data for the dam is to be secured from the Materials Division in order to determine if the native material will support the dam and not allow ponded water to seep under the dam.

- The foundation material under the dam and the material used for the embankment of the dam should be 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 feet or the recommendation of 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. Pipe Specifications: 232 (AASHTO M170), Gasket Specification: 212 (ASTM C443)

- 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. A diagram of the cradle can be found in DDM#2, Management of Stormwater, Appendix 11F-1.

- If the height of the dam is greater than 15 feet, the design of the dam is to include a homogenous embankment with seepage controls or zoned embankment or similar design and is to be approved by the Materials Division.

The minimum top width should be 10 feet. 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, especially if the embankment height is 6 feet or greater. A typical cross-section of a SWM basin dam is shown in Figure 11-1.

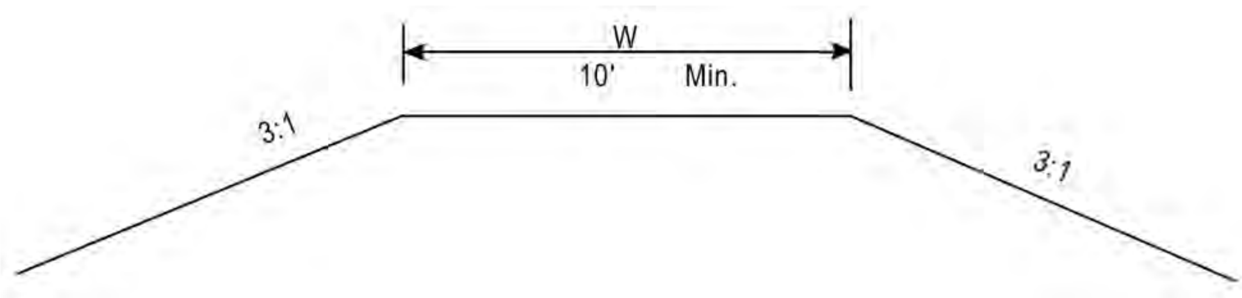

Figure 11-1. Typical SWM Basin Dam

11.3.7 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 cleanout
- The bottom slope of dry detention basins should be no more than 2 percent and no less than 0.5 percent
- Where safety is a concern, and fencing is not practical, use 4:1 side slope
- The depth of water in the basin to the primary overflow (crest of riser, or orifice or weir) should be no more than 3 feet if possible, in order to reduce the hazard potential. If the depth needs to be more than 3 feet, 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

Table 11-2 summarizes the design criteria for dry and wet basin designs:

Table 11-2. Summary of Design Criteria for Dry and Wet Basins

Design Requirement	Dry Basin Design	Wet Basin Design
Quality control	Detain WQV for 30-hour minimum	Permanent pool volume is a function of the BMP selected (Table 11-1)
Quantity control	Control 2- and 10-year peak flows and maintain a non-erosive outfall velocity	Control 2- and 10-year peak flows and maintain a non-erosive outfall velocity
Shape	3:1 length-to-width ratio; wedge shaped (wider at the outlet)	3:1 length-to-width ratio; wedge shaped (wider at the outlet); permanent pool depth to 3 feet max, if possible
Safety		Fence around basin if depth is greater than 3 feet; shallow safety ledge around basin. See following notes on fencing. (Section 11.3.8)
Other Considerations	3:1 side slopes for easy maintenance access; 0.5-2% bottom slope to prevent ponding; sediment forebay to reduce maintenance requirements	3:1 side slopes for easy maintenance access; sediment forebay to reduce maintenance requirements; provide valve to drain pond for maintenance

Source: VDOT Manual of Practice for Planning Stormwater Management, March 1992.

11.3.8 Fencing

All stormwater management basins should be reviewed for the needs of fencing, barricades and no trespassing signs in accordance with the VDOT guideline for fencing of stormwater management basins.

Fencing of stormwater management basins is normally not required and should not be used for most basins due to:

- Insignificant Hazard - Ponding of water in a dry basin should only occur with very heavy storms and be noticeable for a few hours for smaller storm events. Greater magnitude storms store more runoff for longer periods of time. The ponded depth will normally be no more than about 3 feet. Ponds and lakes are almost never fenced, even though they may be located in subdivisions and have deeper, permanent pools.
- Limits Maintenance - Fencing will limit maintenance operations and could deter the frequency of maintenance. Maintenance operations can damage fencing, particularly if equipment becomes stuck.
- Attractive Nuisance – Fencing a basin may create an attraction and is not a formidable barrier.

Fencing of stormwater management basins <u>may occasionally be needed</u> and should be used when:

- The basin is deep with a ponded depth greater than about 3 feet and/or has steep side slopes with two or more side slopes steeper than 3:1
- The basin is in close proximity to schools, playgrounds or similar areas where children may be expected to frequent
- Recommended by the Field Inspection Report, the Resident Engineer or the City/County (where City/County will take over maintenance responsibility)
- A chain or gate may be needed on some basins to prohibit vehicular access if there is concern with dumping or other undesirable access

“No Trespassing” signs should be considered for use on all basins, whether fenced or unfenced, and should be recommended as needed on the Field Inspection Report.

11.3.9 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 should 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 11-2 shows a typical sediment forebay.

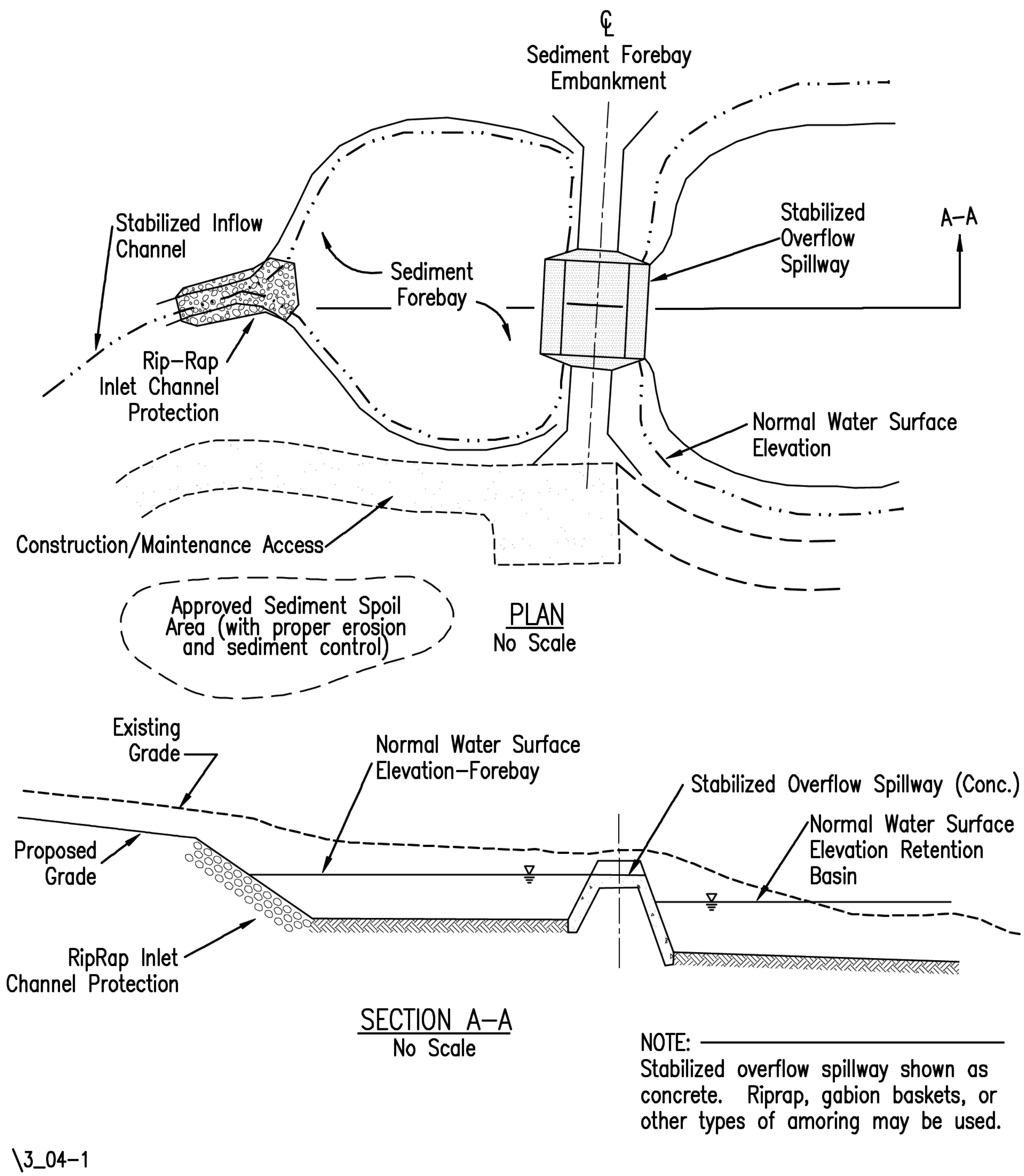

Figure 11-2. Typical Sediment Forebay Plan and Section

A sediment forebay should be used on extended detention, extended detention-enhanced, and retention basins. The volume should be between 0.1 inch and 0.25 inch times the new impervious area collected or 10 percent of the required detention volume.

11.3.10 Maintenance

An important step in the design process is identifying whether special provisions are warranted to properly construct or maintain proposed storage facilities. To assure acceptable performance and function, storage facilities that require frequent maintenance are discouraged.

Proper design should focus on the elimination or reduction of maintenance requirements by addressing the following potential problems:

- Both weed growth and grass maintenance 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. Very small pipes tend to block quite easily and should be avoided.
- 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.
- Access for inspection and maintenance personnel should be provided at each SWM facility. A turnaround should be provided on vehicular entrances when needed based upon accessibility and traffic volume. Appropriate surface material should be provided for each vehicular entrance.
- VDOT maintenance procedures include inspecting each stormwater management facility on a semiannual basis, and inspecting each stormwater management facility 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.

11.4 Design Concepts

11.4.1 Quality

Stormwater runoff can have a significant impact on the aquatic ecosystem. Various soluble and particulate pollutants are found in stormwater runoff. Studies have shown that the source of these pollutants is atmospheric deposition, urban and agricultural lands, and natural spaces. The impervious surfaces, such as parking lots, rooftops and roads, which are associated with land development, serve to accumulate and transport these pollutants to receiving stream channels.

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 stormwater management 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 11-3 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 11-3. Functional Goals of Stormwater BMPs

Stormwater BMP	Quality	Stream Channel Erosion	Quantity/ Flooding
Vegetated filter strip	+++		
Grasses Swale (w/check dams)	+++	+	
Constructed wetlands	+++	+	
Extended detention	++	+++	+
Extended detention enhanced	+++	++	+
Bioretention	+++		
Retention basin	+++	++	+++
Sand filter	+++		
Infiltration	+++		
Infiltration Basin	++	+	+
Detention		++	+++
Manufactured BMPs (Water Quality Structures)	+++		

Legend: +++ Primary functional goal
++ Potential secondary functional goal
+ Potential secondary functional goal with design modifications or additional storage

Source: Virginia Stormwater Management Handbook, Vol. 1, 1st Ed.

11.4.2 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 stormwater management facilities.

11.4.3 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 is shown in Appendix 11G-1. A typical retention basin plan is shown in Appendix 11G-3.

11.4.4 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 11-3 shows removal rate versus detention time for selected pollutants.

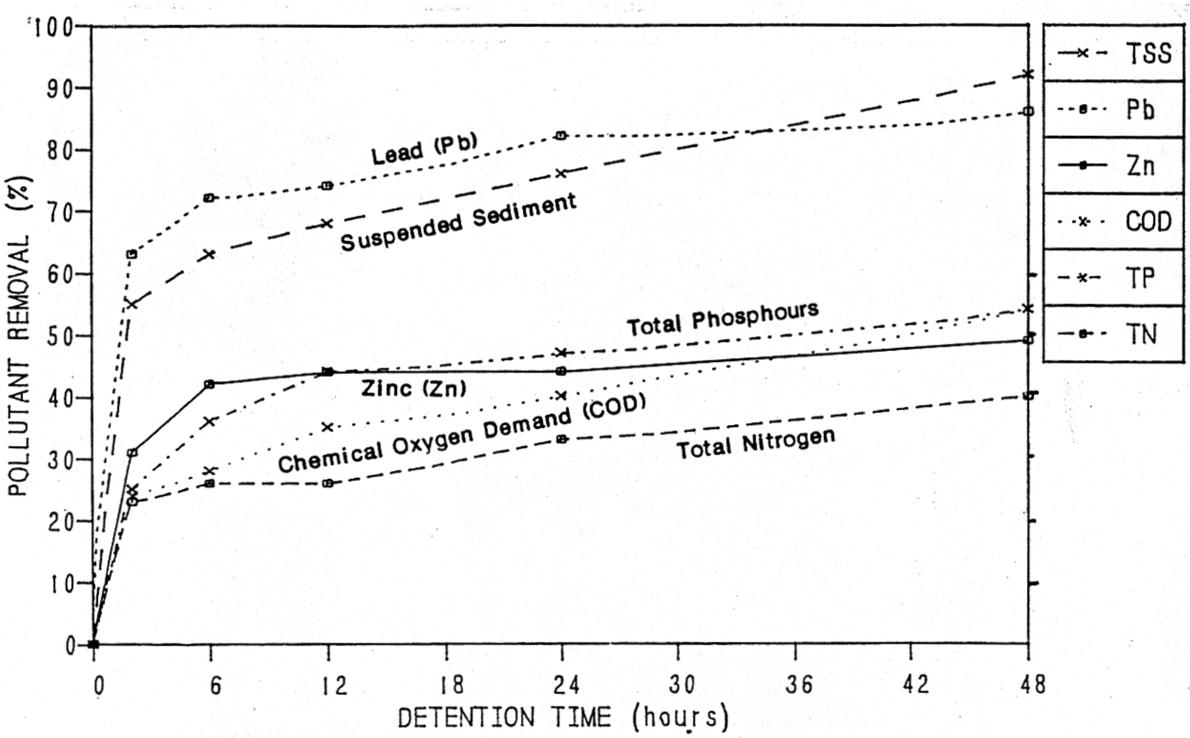

Figure 11-3. Removal Rates vs. Detention Time

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 stormwater management facility. 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.

11.4.5 Release Rates

Control structure release rates are usually designed to approximate pre-developed peak runoff rates for the 2- and 10-year design storms with an emergency spillway capable of handling the 100-year peak discharge. Design calculations are required to demonstrate that the post-development release rates for the 2- and 10-year design storms are equal to or less than the pre-development release rates. If it can be shown that the 2- and 10-year 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-year storms. This can be accomplished through the use of orifices and weirs and is discussed in Section 11.4.7.

11.4.5.1 Channel Erosion Control – Q_1 Control

Water quantity control for the 1-year design storm (in lieu of the 2-year 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-year design storm.

11.4.6 Hydrology

Hydrology should be performed using the appropriate hydrograph procedures presented in Chapter 6, Hydrology.

11.4.7 Outlet Hydraulics

11.4.7.1 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 DDM #2, Management of Stormwater, Appendix 11F-1. For square-edged entrance conditions, the orifice equation is expressed as:

$$Q = CA\sqrt{2gh} \qquad (11.1)$$

Where:

- Q = Discharge, cfs
- C = Orifice entrance coefficient (generally 0.6)
- A = Cross-sectional area of orifice, sq. ft.
- g = Acceleration due to gravity, 32.2 ft/s^2
- h = Head on orifice, ft

11.4.7.2 Weirs

The most common type of weir associated with stormwater management is the broad-crested weir as is defined by Equation 11.2:

$$Q = CLH^{\frac{3}{2}} \tag{11.2}$$

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.

11.4.7.3 Types of Outlet Structures

11.4.7.3.1 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-year design storm without overtopping the facility. The designer should consider partial clogging (50%) of the principal spillway during the 100-year design storm to ensure the facility would not be overtopped. For large storage facilities, 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-year 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.

In a stormwater management 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:

11.4.7.3.2 SWM-1 (VDOT Standard)
The VDOT standard riser outlet structure is identified as a stormwater management 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 DDM #2, Management of Stormwater, Appendix 11F-1.

11.4.7.3.3 Weir Wall
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 feet. 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 stormwater management facilities 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.

11.4.8 Routing

11.4.8.1 Data Required

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 storage facility
- 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

11.5 Design Procedures and Sample Problems

11.5.1 Documentation Requirements

The following documentation will be required for stormwater management facility 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 Q_2 velocity and Q_{10} 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
- Project stormwater management accountability.
  The designer will complete the SWM and TSB Summary Sheet as provided in Appendix 11D-1
- SWM Facility 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

11.5.2 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 right-of-way 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 11-1 *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 on the BMP determined from Step 5. This provides the required treatment volume.*

11.5.2.1 Water Quality Volume Computation and BMP Selection Sample Problem

Assume the basin is to be an extended detention basin based upon 35 percent new impervious area within the right-of-way.

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 right-of-way 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 new impervious area and convert the units to cubic feet.*

WQV = ½ inch x New Impervious Area

$$WQV = \frac{1}{2}\text{ in.}\left(\frac{1\text{ ft.}}{12\text{ in.}}\right)2.4\text{ ac.}\left(\frac{43560\text{ sq. ft.}}{1\text{ ac.}}\right) = 4356\text{ cu. ft. (Say 4360 cu. ft.)}$$

Step 5: *Refer to Table 11-1 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.

11.5.3 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:

1. Using the average hydraulic head associated with the water quality volume (WQV) and the required drawdown time. This is the VDOT preferred option.

2. 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.

Table 11-4. WQV Orifice Sizes

Diameter	
Inches	**Square Feet**
½	0.0013
¾	0.003
1	0.005
1 ½	0.012
2	0.022
2 ½	0.0034
3	0.049
3½	0.067
4	0.087
4 ½	0.110
5	0.136
5 ½	0.165
6	0.196

After calculating the needed orifice size the designer should select the nearest nominal size opening from Table 11-4.

11.5.3.1 Average Hydraulic Head Method (DCR 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.

11.5.3.1.1 Average Hydraulic Head Sample Problem

Find the orifice size for the required treatment volume using the average hydraulic head method.

h_{max} = 1.1 ft.

Volume = 8,720 cu. ft. (From Sample Problem 11.5.2.1)

$$h_{avg} = \frac{1.1}{2} = 0.55 \text{ ft.}$$

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 = 1.1 ft.

Calculate the discharge through the orifice based on the required treatment volume.

$$Q_{avg} = \frac{\text{Treatment Volume}}{\text{Time}} = \frac{8720\ \text{cu.ft.}}{30\,\text{hr}\ (3600\,\frac{\text{sec}}{\text{hr}})} = 0.081\,\text{cfs}$$

Calculate the orifice area by rearranging Equation 11.1.

$$A = \frac{Q}{C\sqrt{2gh_{avg}}} = \frac{0.081}{0.6\sqrt{2(32.2)(0.55)}} = 0.0223\ \text{sq.ft.}$$

From Table 11-4, select a 2-inch orifice with A = 0.022 sq. ft.

11.5.3.2 Maximum Hydraulic Head Method (DCR 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.

11.5.3.2.1 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} x 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 ft.
$Q_{max} = 2Q_{avg} = 2(0.081) = 0.16\ \text{cfs}$

Calculate the orifice area by rearranging Equation 11.1.

$$A = \frac{Q}{C\sqrt{2gh_{avg}}} = \frac{0.16}{0.6\sqrt{2(32.2)(1.1)}} = 0.032\ \text{sf.ft.}$$

From Table 11-4, select a 2½-inch orifice with A = 0.034 sq. ft.

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 cubic feet per second. The problem could also be due to the need for small orifice sizes such as 2 inches.

11.5.3.2.2 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 TR-55 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 (Appendix 11C-1) for the impervious area, Rainfall (RF) = 1.2 inches to produce RUNOFF (RO) = 1 inch (Appendix 11C-2) 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.

11.5.3.2.3 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-year 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 cubic feet per second 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.

11.5.3.3 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-year 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.

11.5.3.3.1 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 in.
CN = 75
1-year rainfall depth of runoff = 0.8 in.

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})*

$$V_{ce} = 25\text{ ac.}(0.8\text{ in.})\left(\frac{1\text{ ft.}}{12\text{ in.}}\right) = 1.67\text{ ac.ft.}$$

To account for the routing effect, reduce the channel erosion control volume by 60%:

$$V_{ce} = 0.60(1.67) = 1.0\text{ ac.ft. or } 43{,}560\text{ cu.ft.}$$

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 feet.

Step 5: *Determine the average discharge (Q_{avg}) resulting from the 24-hour drawdown requirement.*

$$Q_{avg} = \frac{43{,}560\text{ cu ft}}{(24\text{ hr})\,(3{,}600\ \frac{\text{sec}}{\text{hr}})} = 0.50\text{ cfs}$$

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:

$$A = \frac{\pi d^2}{4}$$

$$d = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4(0.104)}{\pi}} = 0.364 \text{ sq ft}$$

d = 4.4-in (Say 4.5-in)

The designer can also use Table 11-4 to determine a 4½-inch diameter extended detention orifice for channel erosion control.

11.5.4 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.

11.5.4.1 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 right-of-way. Select the type of BMP needed from Table 11-1. 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.

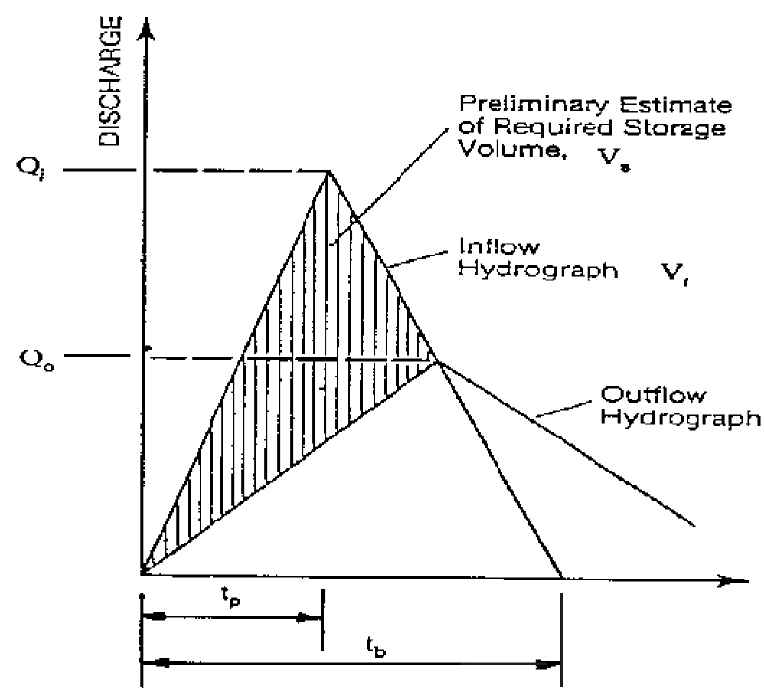

Figure 11-4. Simplified Triangular Hydrograph Method

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_s = \frac{1}{2}T_b(Q_i - Q_o) \tag{11.3}$$

Where:

V_s = Storage volume estimate, cu. ft.
Q_i = Peak inflow rate, cfs
Q_o = Peak outflow rate, cfs
T_b = Duration of basin inflow, sec.

11.5.4.2 DCR 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:

$$V = \left[Q_i T_d + \frac{Q_i t_c}{4} - \frac{q_o T_d}{2} - \frac{3 q_o t_c}{4}\right]60 \tag{11.4}$$

Where:

V = Required storage volume, cu. ft.
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:

$$T_d = \sqrt{\frac{2CAa(b - \frac{t_c}{4})}{q_o}} - b \tag{11.5}$$

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 (Refer to Chapter 6, Appendices 6B-2 through 6B-18)
t_c = Time of concentration, min.
q_o = Allowable peak outflow, cfs

11.5.4.3 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 11-5 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.

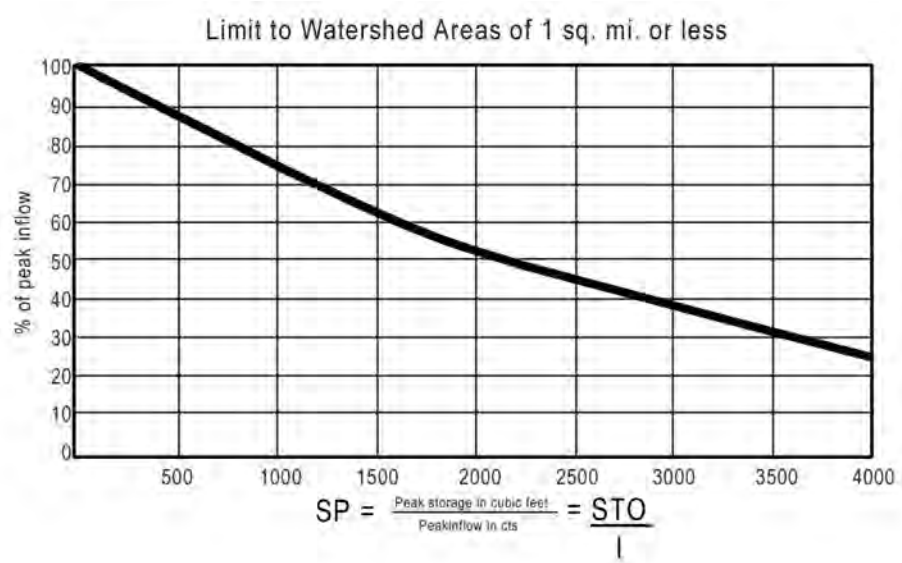

Figure 11-5. Pagan Method Curve

Step 1: Determine pre- and post-development peak discharges.

Step 2: Determine the Storage Parameter (SP).

SP is determined from Figure 11-5 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):

$$STO = SP(I)$$

11.5.4.4 Sample Problems – Using 3 Methods to Estimate Volume of Storage for Quantity Control

Condition	Rational Method			
	D.A	**C**	**T_c**	**Q_{10}**
Pre-developed	25ac.	0.38	52 min.	24 cfs
Post-developed	25 ac	0.59	21 min.	65 cfs

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, cu. ft.
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: DCR Critical Storm Duration Method

Based on the methodology in 11.5.4.2, determine the 10-year critical storm duration T_{d10} as follows*:*

A = 189.2
b = 22.1
C = 0.59 (Post-development)
A = 25 acres
t_c = 21 min (Post-development)
q_{o10} = 24 cfs (Allowable outflow based on pre-development)

$$T_d = \sqrt{\frac{2CAa(b-\frac{t_c}{4})}{q_o}} - b$$

$$T_{d_{10}} = \sqrt{\frac{2(0.59)(25.0)(189.2)\left(22.1-\frac{21}{4}\right)}{24}} - 22.1$$

$$T_{d_{10}} = 40.5 \text{ min}$$

Solve for the 10-year critical storm duration intensity (I_{10})

$$I_{10} = \frac{189.2}{22.1+40.5} = 3.02 \text{ in/hr}$$

Determine the 10-year peak inflow (Q_{10}) using the Rational Equation and the critical storm duration intensity (I_{10})

$$Q = C_f CiA$$
$$Q_{10} = 1.0(0.59)(3.02)(25) = 44.5 \text{ cfs}$$

Determine the required 10-year storage volume (V_{10}) for the 10-year critical storm duration (T_{d10})

$$V = \left[Q_i T_d + \frac{Q_i t_c}{4} - \frac{q_o T_d}{2} - \frac{3q_o t_c}{4}\right]60$$

$$V_{10} = \left[(44.5)(40.5) + \frac{(44.5)(21)}{4} - \frac{(24)(40.5)}{2} - \frac{3(24)(21)}{4}\right]60$$

$$= 70{,}313 \text{ cu. ft. (Say 70,300 cu. ft.)}$$

Method 3: Pagan Method

Based on the methodology in 11.5.4.3, solve for the storage volume as follows:

$$\frac{Q_o}{Q_i} = \frac{24}{65} = 0.37 \ (37\%)$$

SP = 3100 seconds.

$$STO = SP(I)$$
$$= 3100(65)$$
$$= 201{,}500 \text{ cu. ft.}$$

11.5.5 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 feet. 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 cu. ft.

For a 4-ft depth, $\frac{51,660}{4} = 12,915 \text{ sq.ft.}$

About 80'x160'

From Method 2: DCR Critical Storm Duration Method

V_{10} = 70,308 cu. ft.

For 4' depth, $\frac{70,300}{4} = 17,575 \text{ sq.ft.}$

About 90'x195'

From Method 3: Pagan Method

V_{10} = 201,500 cu. ft.

For a 4' deep, $\frac{201,500}{4} = 50,375 \text{ sq.ft.}$

About 150'x335'

Summary: Preliminary trial size basin would be recommended about 100'x200'

11.5.6 Final Basin Sizing-Reservoir Routing

11.5.6.1 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 11-6 and Figure 11-7 respectively.

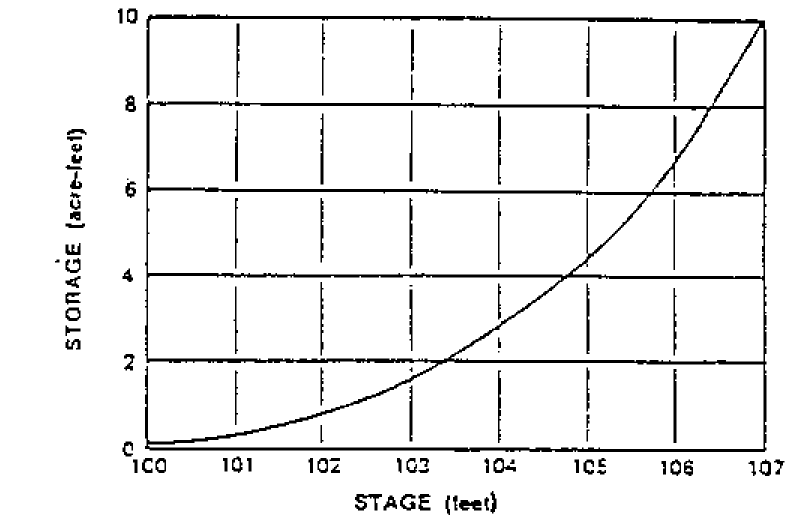

Figure 11-6. Stage-Storage Curve

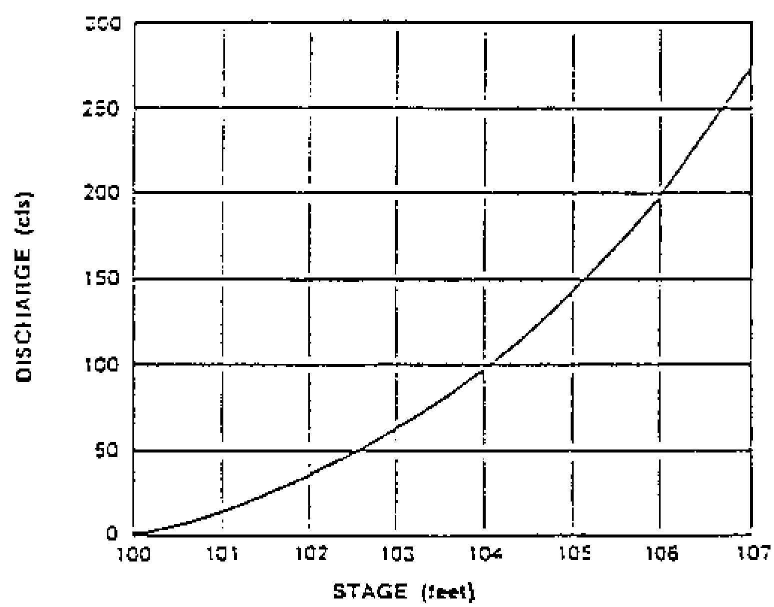

Figure 11-7. Stage-Discharge Curve

Step 2: *Select a routing time period (Δt) to provide at least five points on the rising limb of the inflow hydrograph. Use t_p divided by 5 to 10 for Δt.*

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 11-5.*

Table 11-5. Storage Characteristics

(1)	(2)	(3)	(4)	(5)	(6)
Stage (H) (ft.)	Storage[1] (S) (ac-ft)	Discharge[2] (Q) (cfs)	Discharge[2] (Q) (ac-ft/hr)	$S - \frac{O}{2}\Delta T$ (ac-ft)	$S + \frac{O}{2}\Delta T$ (ac-ft)
100	0.05	0	0	0.05	0.05
101	0.05	15	1.24	0.20	0.40
102	0.05	35	2.89	0.56	1.04
103	1.6	63	5.21	1.17	2.03
104	2.8	95	7.85	2.15	3.45
105	4.4	143	11.82	3.41	5.39
106	6.6	200	16.53	5.22	7.98

[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_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, Figure 11-8.*

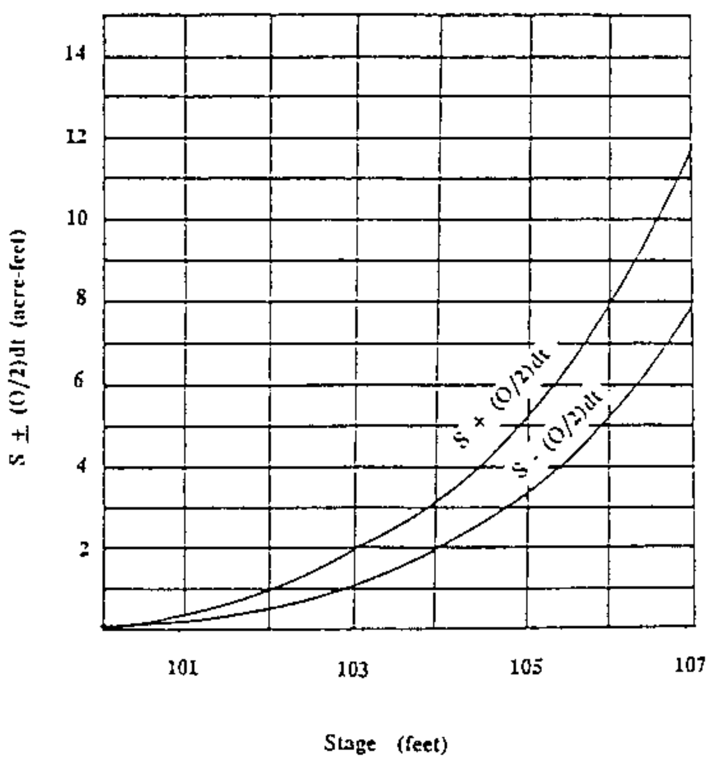

Figure 11-8. Storage Characteristics Curve

Step 5 Determine the value of $S_2+\frac{O_2}{2}\Delta T$ *from the following equation:*

$$S_2+\frac{O_2}{2}\Delta T=S_1-\frac{O_1}{2}\Delta T+\frac{I_1+I_2}{2\Delta T} \quad (11.6)$$

Where:

S_2 = Storage volume at time 2, cu. ft.
O_2 = Outflow rate at time 2, cfs.
ΔT = Routing time period, sec
S_1 = Storage volume at time 1, cu. ft.
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.*

11.5.6.2 Storage – Indication Method Routing Sample Problem

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 runoff from both the 2- and 10-year design storms and an analysis done using the 100-year 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-year design storms are as follows:

- Pre-developed 2-year peak discharge = 150 cfs
- Pre-developed 10-year peak discharge = 200 cfs
- Post-development 2-year peak discharge = 190 cfs
- Post-development 10-year 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-year 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 11-6 below. Inflow durations from the post-development hydrographs are about 1.2 and 1.25 hours, respectively, for runoff from the 2- and 10-year storms.

Table 11-6. Runoff Hydrographs

	Pre-Development Runoff		**Post-Development Runoff**	
(1)	**(2)**	**(3)**	**(4)**	**(5)**
Time (hrs)	2-year (cfs)	10-year (cfs)	2-year (cfs)	10-year (cfs)
0	0	0	0	0
0.1	18	24	38	50
0.2	61	81	125	178
0.3	127	170	190 >150	250 >200
0.4	150	200	125	165
0.5	112	150	70	90
0.6	71	95	39	50
0.7	45	61	22	29
0.8	30	40	12	16
0.9	21	28	7	9
1.0	13	18	4	5
1.1	10	15	2	3
1.2	8	13	0	1

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-year storms, the required storage volumes, V_S, are computed using Equation 11.3:

$$V_s = \frac{1}{2}T_b(Q_i - Q_o)$$

$$V_{s_2} = \frac{\frac{1}{2}(1.2)(3600)(190\text{-}150)}{43{,}560} = 1.98\ \text{ac.ft.}$$

$$V_{s_{10}} = \frac{\frac{1}{2}(1.2)(3600)(250\text{-}200)}{43{,}560} = 2.48\ \text{ac.ft.}$$

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-year design storms are presented below in Table 11-7. The storage-discharge relationship was developed and required that the preliminary storage volume estimates of runoff for both the 2-and 10-year 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 feet, and no tailwater submergence. The capacity of storage relief structures was assumed to be negligible.

Step 2: Select a routing time period (Δt) to provide at least five points on the rising limb of the inflow hydrograph. Use t_p divided by 5 to 10 for Δt.

$$\Delta T = \frac{t_p}{5} = \frac{0.5}{5} = 0.10 \text{ hr}$$

Step 3: Use the storage-discharge data from Step 1 to develop storage characteristics curves (Stage-Discharge-Storage) that provide values of $S \pm \frac{O}{2}\Delta T$ versus stage.

Table 11-7. Stage-Discharge-Storage Data

(1)	(2)	(3)	(4)	(5)
Stage (H) (ft)	Discharge (Q) (cfs)	Storage (S) (ac-ft)	$S - \frac{O}{2}\Delta T$ (ac-ft)	$S + \frac{O}{2}\Delta T$ (ac-ft)
0.0	0	0.00	0.00	0.00
0.9	10	0.26	0.30	0.22
1.4	20	0.42	0.50	0.33
1.8	30	0.56	0.68	0.43
2.2	40	0.69	0.85	0.52
2.5	50	0.81	1.02	0.60
2.9	60	0.93	1.18	0.68
3.2	70	1.05	1.34	0.76
3.5	80	1.17	1.50	0.84
3.7	90	1.28	1.66	0.92
4.0	100	1.40	1.81	0.99
4.5	120	1.63	2.13	1.14
4.8	130	1.75	2.29	1.21
5.0	140	1.87	2.44	1.29
5.3	150	1.98	2.60	1.36
5.5	160	2.10	2.76	1.44
5.7	170	2.22	2.92	1.52
6.0	180	2.34	3.08	1.60

Storage routing was conducted for runoff from both the 2- and 10-year 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-year 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:*

$$S_2 + \frac{O_2}{2}\Delta T = S_1 - \frac{O_1}{2}\Delta T + \frac{I_1 + I_2}{2\Delta T} \quad (11.7)$$

Summarized in Table 11-8 and Table 11-9 for the 2-year and 10-year 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 Table 11-8 and Table 11-9 for the 2-year and 10-year 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 Table 11-8 and Table 11-9 for the 2-year and 10-year 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 Table 11-8 and Table 11-9 for the 2-year and 10-year design storms.

Table 11-8. Storage Routing for the 2-Year Storm

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (T) (hrs)	Inflow (I) (cfs)	$\frac{I_1 + I_2}{2}\Delta T$ (ac-ft)	Stage (H_1) (ft)	$S_1 - \frac{O_1}{2}\Delta T$ **(6)-(8)** (ac-ft)	$S_2 + \frac{O_2}{2}\Delta T$ **(3)+(5)** (ac-ft)	Stage (H) (ft)	Outflow (O) (cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	38	0.16	0.00	0.00	0.16	0.43	3
0.2	125	0.67	0.43	0.10	0.77	2.03	36
0.3	190	1.30	2.03	0.50	1.80	4.00	99
0.4	125	1.30	4.00	0.99	2.29	4.80	130<150 OK
0.5	70	0.81	4.80	1.21	2.02	4.40	114
0.6	39	0.45	4.40	1.12	1.57	3.60	85
0.7	22	0.25	3.60	0.87	1.12	2.70	55
0.8	12	0.14	2.70	0.65	0.79	2.02	37
0.9	7	0.08	2.08	0.50	0.58	1.70	27
1.0	4	0.05	1.70	0.42	0.47	1.03	18
1.1	2	0.02	1.30	0.32	0.34	1.00	12
1.2	0	0.01	1.00	0.25	0.26	0.70	7
1.3	0	0.00	0.70	0.15	0.15	0.40	3

Table 11-9. Storage Routing for the 10-Year Storm

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (T) (hrs)	Inflow (I) (cfs)	$\frac{I_1 + I_2}{2}\Delta T$ (ac-ft)	Stage (H_1) (ft)	$S_1 - \frac{O_1}{2}\Delta T$ **(6)-(8)** (ac-ft)	$S_2 + \frac{O_2}{2}\Delta T$ **(3)+(5)** (ac-ft)	Stage (H) (ft)	Outflow (O) (cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	50	0.21	0.21	0.00	0.21	0.40	3
0.2	178	0.94	0.40	0.08	1.02	2.50	49
0.3	250	1.77	2.50	0.60	2.37	4.90	134
0.4	165	1.71	4.90	1.26	2.97	2.97	173<200 OK
0.5	90	1.05	5.80	1.30	2.35	4.00	137
0.6	50	0.58	4.95	1.25	1.83	4.10	103
0.7	29	0.33	4.10	1.00	1.33	3.10	68
0.8	16	0.19	3.10	0.75	0.94	2.40	46
0.9	9	0.10	2.40	0.59	0.69	1.90	32
1.0	5	0.06	1.90	0.44	0.50	1.40	21
1.1	3	0.03	1.40	0.33	0.36	1.20	16
1.2	1	0.02	1.20	0.28	0.30	0.90	11
1.3	0	0.00	0.90	0.22	0.22	0.60	6

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-year 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 for the 10-year design storms.

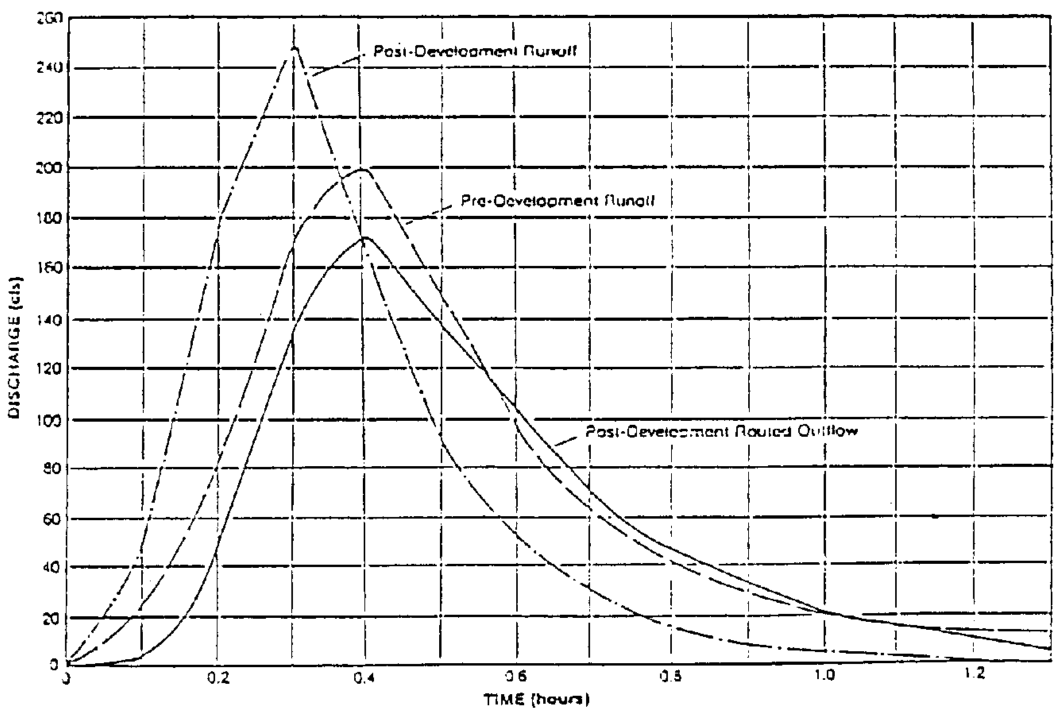

Figure 11-9. Runoff Hydrographs

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 percent. As shown in Figure 11-9, the sample problem results are well below 20 percent; downstream effects can thus be considered negligible and downstream flood routing or Q_1 control omitted.

11.5.6.3 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 11-1 – 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-year 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 11-1 the required treatment volume for an extended detention basin is 2 x WQV. The WQV being equal to ½ inches x New Impervious area. 2 x ½ inch = 1 in or 0.083 ft
- New pavement within the drainage area for this outfall = 2.98 ac or 129,809 sq. ft.

$$\begin{aligned}\text{Treatment Volume} &= 2\text{xWQV}\\ &= 2\left[\frac{0.5(2.98)(43560)}{12}\right]\\ &= 10{,}817\ \text{cu.ft.}\end{aligned}$$

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 cu. ft. for wet storage

  67 cu. yd. x 13 ac = 23,517 cu. ft. for dry storage

  The total volume required for temporary sediment storage, wet plus dry = 47,034 cu. ft. This is much larger than the 10,817 cu. ft. 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 inches per acre of new impervious area or 10 percent 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 cu. ft.

  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

1. An extended detention basin is required for this site.
2. QUANTITY CONTROL FOR Q_2 PEAK IS REQUIRED. The required volume will be estimated in the design process.
3. Alternative Q_1 control is not needed.
4. The required WQV is 10,817 cu. ft.
5. The temporary sediment volume (if needed) is 47,034.
6. The estimated forebay volume is 1,082 to 2,704 cu. ft.

Determining the Water Quality Volume

Calculate required WQV (for extended detention) = 10,817 cu. ft.

From Preliminary Elevation/Storage Table:

The WQV required is met @ Elev. 423.25

Depth = 1.95 ft

Actual Volume = 11,051 cu. ft. @ Elev. 423.25

WQV Computations – Determining the Orifice Size Required
Using DCR Method #2 Average Hydraulic Head (VDOT Preferred)

- *Assume depth, h = 1.95 ft (Say 2.0 ft)*

$$h_{avg} = \frac{2.0}{2} = 1.0 \text{ ft.}$$

- *Compute the Q_{avg} for the WQV using the required 30-hour drawdown time:*

$$Q_{avg} = \frac{\text{Treatment Volume}}{\text{Time}} = \frac{11{,}051 \text{ cu.ft.}}{30 \text{ hr } (3600 \frac{\text{sec}}{\text{hr}})} = 0.102 \text{ cfs}$$

- *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 11-4, use a 2-inch orifice with an area = 0.022 sq. ft.

Q_1 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 Q_1 instead of the Q_2 as required by MS-19. Control of the Q_1 requires containing the entire volume of the Q_1 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 Q_1 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 Q_1 Control Volume:
Use (DCR) Method #2 – Average Hydraulic Head (Recommended Method)

Find the Q_1 Control volume.

Given from design computations:

DA = 12.98 ac
C = 0.67
T_c = 16 min
Q_2 = 29.6 cfs.

- *Use TR-55 to find the volume for Q_1:*
- *Convert the runoff coefficient, C = 0.67 from the Rational Method to CN = 80. Refer to Appendix 11C-1.*
- *Find the 1-year frequency 24-hour rainfall (RF) for the appropriate county from Appendix 11C-3.*

  RF = 2.8 inches.

- *Find the runoff depth for CN= 80 and RF = 2.8 inches from Appendix 11C-2 or use TR-55.*

  Runoff (RO) = 1.1 inches

- *Compute the Q_1 Control volume:*

$$V_{ce} = 12.98\text{ ac.}(1.1\text{ in.})\left(\frac{1\text{ ft.}}{12\text{ in.}}\right)\left(\frac{43{,}560\text{ sq.ft.}}{1.0\text{ ac.}}\right) = 51{,}829\text{ cu.ft.}$$

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 Q_1 Volume

1. Use the Rational Method triangular hydrograph (HYG) to estimate the volume needed:

   - From 24 hour rainfall (RF) table (Appendix 11C-3):

     RF_1 = 2.8 inches
     RF_2 = 3.5 inches

   - $\frac{RF_1}{RF_2} = \frac{2.8}{3.5} = 0.80\ (80\%)$

     Thus Q_1 = 80% of Q_2

     Q_2 = 29.6 cfs

     $$\begin{aligned} Q_1 &= 0.80Q_2 \\ &= 0.80(29.6) \\ &= 23.7 \text{ cfs} \end{aligned}$$

   - Compute the volume from a triangular HYG:

     Using t_c = 16 min., $T_b = 2t_C$ = 32 min.

     $$\begin{aligned} V_1 &= 0.5(Q_1)(T_b) \\ &= 0.5(23.7 \text{ cfs})(32 \text{ min.})(60 \tfrac{\text{sec}}{\text{min}}) \\ &= 22{,}752 \text{ cu.ft.} \end{aligned}$$

   - Compute the volume from a trapezoidal HYG:

     Using t_c = 16 min. and determining the critical storm duration, T_d = 22 min.

     $T_b = t_c + T_d$ = 38 min.

     $$\begin{aligned} V_1 &= 0.5(Q_1)\left(\frac{T_d - t_c}{T_b}\right) \\ &= 0.5(23.7 \text{ cfs})\left(\frac{22 \text{ min.} - 16 \text{ min.}}{38 \text{ min.}}\right)\left(60 \tfrac{\text{sec}}{\text{min}}\right) \\ &= 31{,}284 \text{ cu.ft.} \end{aligned}$$

     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 cfs with t_c = 16 minutes and the duration = 22 minutes, the volume of the HYG = 31,284 cubic feet which is very close to the volume of 31,097 cubic feet 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:

$$Q_{avg} = \frac{V_{ce}}{Time} = \frac{31{,}097 \text{ cu. ft.}}{24 \text{ hr.}(3600 \frac{sec}{hr})} = 0.360 \text{ cfs}$$

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 11-4, use a 3 ½-inch orifice with an area = 0.067 sq. ft.

11.6 References

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Appendix 11A-1 Definitions and Abbreviations

Definitions:

Brim Drawdown Time	The time required for the entire calculated volume to drain from the basin.
Detention Basin	A stormwater management facility which temporarily impounds runoff and discharges it through a hydraulic outlet structure to a downstream conveyance system. Since an extended detention basin impounds runoff only temporarily, it is normally dry during non-rainfall periods.
Extended Detention Basin	A stormwater management facility which temporarily impounds runoff and discharges it through a hydraulic outlet structure <u>over a specified period of time</u> to a downstream conveyance system for the purpose of water quality enhancement or stream channel erosion control. Since an extended detention basin impounds runoff only temporarily, it is normally dry during non-rainfall periods.
Land Development Project	A manmade change to the land surface that potentially changes its runoff characteristics as a permanent condition. The permanent condition should consider the effects of mature vegetative cover and should not be concerned with temporary changes due to construction activities. Temporary changes are addressed by the VESCR.
Offsite	Refers to drainage area that is not part of the project construction. At times, drainage area beyond the construction limits contributes to project discharges. More often, the term is used when developing a stormwater management plan, and offsite impervious area is used to compensate for untreated project impervious area.

Appendix 11A-1 Definitions and Abbreviations

Q_1 Control	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.

Abbreviations:

AASHTO	American Association of State Highway Transportation Officials
BMP	Best Management Practice
DCR	Department of Conservation and Recreation
HYG	Hydrograph
LDP	Land Development Project
MS	Minimum Standard
NRCS	National Resource Conservation Service, formerly Soil Conservation Service (SCS)
SCS	Soil Conservation Service
SWMR	Stormwater Management Regulations
VDOT	Virginia Department of Transportation
VESCR	Virginia Erosion and Sediment Control Regulations
WQV	Water Quality Volume

Appendix 11A-2 Symbols

Symbol	Definition	Units
a	Rainfall regression constant	-
A	Cross-sectional or surface area	ft^2
A	Drainage area	ac
b	Rainfall regression constant	-
C	Runoff coefficient	-
C	Broad-crested weir coefficient or orifice coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
d	Orifice diameter	ft
g	Acceleration due to gravity	ft/s^2
H	Depth of water	ft
h	Head	ft
h_{avg}	Average head	ft
h_{max}	Maximum head	ft
I	Inflow rate	cfs
I_1	Inflow rate at time 1	cfs
I_2	Inflow rate at time 2	cfs
L	Broad-crested weir length	ft
O_1	Outflow rate at time 1	cfs
O_2	Outflow rate at time 2	cfs
Q_{avg}	Average flow rate	cfs
Q_i	Peak inflow rate	cfs
q_o	Allowable outflow rate	cfs
Q_o	Peak outflow rate	cfs
Q	Discharge or flow rate	cfs
S	Storage volume	ft^3 , ac.ft
S_1	Storage volume at time 1	ft^3
S_2	Storage volume at time 2	ft^3
SP	Storage parameter (Pagan Method)	-
STO	Maximum storage volume (Pagan Method)	-
?t	Routing time period (timestep)	sec
t_b	Time base on hydrograph	hrs or min
t_c	Time of concentration	min
T_d	Critical storm duration	min
T_i	Duration of basin inflow	hrs or min
t_p	Time to peak	hrs or min
V_{ce}	Channel erosion control volume	ft^3 , ac.ft
V	Storage volume	ft^3 , ac.ft
V_s	Storage volume estimate	ft^3 , ac.ft
WQV	Water quality volume	ft^3

Appendix 11B-1 SWM Design Checklist

1. **TYPE OF BMP-QUALITY CONTROL** – Determine the type of BMP to be used from Table 11-1. Find the new percent impervious area within the project area (right of way and permanent easements) per outfall.

2. **QUANTITY CONTROL** – Check for an adequate receiving channel in accordance with MS-19 of the erosion and sediment control regulations. If the receiving channel is not adequate, the BMP must provide attenuation of the post-development peak discharge to pre-development discharge levels.

   - Natural Channels: Q_2 for discharge and velocity
   - Man-made Channels: Q_2 for velocity and Q_{10} for discharge
   - Storm Drainage Systems: Q_{10} for capacity

3. **QUANTITY CONTROL (ALTERNATIVE)** – Control of the runoff from the 1-year frequency storm, in lieu of the 2- and 10-year frequency storms, may be required if:

   - A field survey of the receiving channel indicates that significant erosion is occurring under existing conditions
   - It is anticipated that erosion may occur in the receiving channel due to increased frequency of bankful flow conditions as a result of standard peak flow attenuation

   If attenuation of the 1-year frequency storm is required, the volume requirements are based upon containing the entire volume of runoff from the 1-year frequency event for a period of 24-hours.

4. **QUALITY CONTROL** – Determine the required water quality volume (WQV) using Table 11-1 and compute the volume requirements.

5. **TEMPORARY SEDIMENT STORAGE** – If the BMP is to be used as a temporary sediment basin during construction, calculate the volume requirements:

   - Wet Storage – 67 cu. yds. per acre of the total contributing drainage area plus
   - Dry Storage – 67 cu. yds. per acre of the total contributing drainage area

Appendix 11B-1 SWM Design Checklist

6. **FOREBAY** – If the BMP is to have a sediment/debris forebay, calculate the volume requirements. Forebays are recommended for most types of basins.

7. **OTHER DESIGN CONSIDERATIONS**
(Refer to DDM #2, Stormwater Management, in Appendix 11F-1)

- Use “Design Guidelines for SWM Basins”
- Use “Details for Design of Dams”
- Use “Perimeter Control Guidelines”
- Design of the emergency spillway for conveying Q_{100}
- Request foundation information for basin and dam
- Request aquatic planting plan from the Environmental Division (when required)
- Provide maintenance access with turnaround (include chain barricade when required)
- Provide sufficient right-of-way and easement for construction and maintenance
- Provide information for Stormwater Management Data Base (complete the “SWM Facility – Tabulation Sheet” provided in Appendix 11E-1)

Appendix 11C-1 Equivalent Runoff Curve Number (RCN) for Rational 'C'

RATIONAL METHOD "C" VALUES		N.R.C.S. "TR-55" METHOD "CN" VALUES					
LAND COVER	RUNOFF COEFICIENT "C"	COVER TYPE & HYDROLOGIC CONDITION	Avg.% Imp.	Curve Numbers for Hydrologic Soil Group*			
				A	B	C	D
Business, industrial and commercial	0.80 to 0.90	Commercial and business	85	89	92	94	95
		Industrial	72	81	88	91	93
Residential	0.40 to 0.50	Residential area by lot size:					
		1/8 acre or less (town houses)	65	77	85	90	92
lots 10,000 sq. ft.	0.40 to 0.45	¼ acre	38	61	75	83	87
- lots 12,000 sq. ft.		1/3 acre	30	57	72	81	86
- lots 17,000 sq. ft.	0.35 to 0.45	½ acre	25	54	70	80	85
- lots ½ ac. or more		1 acre	20	51	68	79	84
	0.30 to 0.40	2 acres	12	46	65	77	82
		Farmsteads – buildings, lanes, driveways, and surrounding lots	n.a.	59	74	82	86
Parks, cemeteries and unimproved areas	0.20 to 0.35	Open space (lawns, parks, golf courses Cemeteries, etc.) grass cover > 75%	n.a.	39	61	74	80
Lawns	0.20 to 0.40						
Paved and roof areas	0.9	Streets & roads:					
		Paved parking lots, roofs, driveways, etc.	n.a.	98	98	98	98
		Paved: open ditches (excluding R/W)	n.a.	83	89	92	93
		Gravel (including R/W)	n.a.	76	85	89	91
		Dirt (including R/W)	n.a.	72	82	87	89
Cultivated areas	0.50 to 0.70	Cultivated areas (combination of straight & Row crops)	n.a.	71	80	87	90
Pasture	0.35 to 0.45	Pasture, grassland, or range	n.a.	39	61	74	80
		Meadow – continuous grass	n.a.	30	58	71	78
		Brush-brush-weed-grass mixture with brush the major element	n.a.	30	48	65	73
Forest	0.20 to 0.30	Woods	n.a.	30	55	70	77
		Woods/grass combination	n.a.	32	58	72	79

If the accurate soil information is not available use Soil Group B.

Appendix 11C-2 Runoff Depth for Runoff Curve Number (RCN)

Runoff depth for selected NRCS TR-55 CN's and rainfall amounts*

Rainfall (inches)	Runoff depth (in inches) for Curve Number (CN) of -												
	40	45	50	55	60	65	70	75	80	85	90	95	98
1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.08	0.17	0.32	0.56	0.79
1.2	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.07	0.15	0.27	0.46	0.74	0.99
1.4	0.00	0.00	0.00	0.00	0.00	0.02	0.06	0.13	0.24	0.39	0.61	0.92	1.18
1.6	0.00	0.00	0.00	0.00	0.01	0.05	0.11	0.20	0.34	0.52	0.76	1.11	1.38
1.8	0.00	0.00	0.00	0.00	0.03	0.09	0.17	0.29	0.44	0.65	0.93	1.29	1.58
2.0	0.00	0.00	0.00	0.02	0.06	0.14	0.24	0.38	0.56	0.80	1.09	1.48	1.77
2.5	0.00	0.00	0.02	0.08	0.17	0.30	0.46	0.65	0.89	1.18	1.53	1.96	2.27
3.0	0.00	0.02	0.09	0.19	0.33	0.51	0.71	0.96	1.25	1.59	1.98	2.45	2.77
3.5	0.02	0.08	0.20	0.35	0.53	0.75	1.01	1.30	1.64	2.02	2.45	2.94	3.27
4.0	0.06	0.18	0.33	0.53	0.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
4.5	0.14	0.30	0.50	0.74	1.02	1.33	1.67	2.05	2.46	2.91	3.40	3.92	4.26
5.0	0.24	0.44	0.69	0.98	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	0.50	0.80	1.14	1.52	1.92	2.35	2.81	3.28	3.78	4.30	4.85	5.41	5.76
7.0	0.84	1.24	1.68	2.12	2.60	3.10	3.62	4.15	4.69	5.25	5.82	6.41	6.76
8.0	1.25	1.74	2.25	2.78	3.33	3.89	4.46	5.04	5.63	6.21	6.81	7.40	7.76
9.0	1.71	2.29	2.88	3.49	4.10	4.72	5.33	5.95	6.57	7.18	7.79	8.40	8.76
10.0	2.23	2.89	3.56	4.23	4.90	5.56	6.22	6.88	7.52	8.16	8.78	9.40	9.76
11.0	2.78	3.52	4.26	5.00	5.72	6.43	7.13	7.81	8.48	9.13	9.77	10.39	10.76
12.0	3.38	4.19	5.00	5.79	6.56	7.32	8.05	8.76	9.45	10.11	10.76	11.39	11.76
13.0	4.00	4.89	5.76	6.61	7.42	8.21	8.98	9.71	10.42	11.10	11.76	12.39	12.76
14.0	4.65	5.62	6.55	7.44	8.30	9.12	9.91	10.67	11.39	12.08	12.75	13.39	13.76
15.0	5.33	6.36	7.35	8.29	9.19	10.04	10.85	11.63	12.37	13.07	13.74	14.39	14.76

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

Source: SCS TR-55

Appendix 11C-3 24-hr Rainfall Depths

	YEAR						
COUNTY	1	2	5	10	25	50	100
Accomack	3.0	3.7	4.9	6.0	6.8	7.5	8.5
Albemarle	3.3	4.0	5.0	6.0	7.0	8.0	8.5
Alleghany	2.5	3.0	4.0	5.0	5.5	6.0	7.0
Amelia	3.0	3.5	4.5	5.5	6.0	7.0	7.5
Amherst	3.3	4.0	5.0	6.0	7.0	8.0	8.5
Appomattox	3.0	4.0	4.7	5.8	6.2	7.0	8.0
Augusta	3.0	4.0	4.5	5.5	6.5	7.2	8.0
Bath	2.5	3.0	4.0	5.0	5.5	6.0	7.0
Bedford	3.3	4.0	5.0	5.8	6.8	7.5	8.2
Bland	2.4	2.9	3.9	4.6	5.0	5.8	6.0
Botetourt	3.0	3.5	4.5	5.0	6.0	7.0	7.8
Brunswick	3.0	3.5	4.6	5.6	6.2	7.0	8.0
Buchanan	2.4	2.9	3.7	4.3	4.8	5.5	6.2
Buckingham	3.0	3.5	4.7	5.8	6.3	7.0	8.0
Campbell	3.0	3.7	4.7	5.8	6.3	7.0	7.9
Caroline	2.7	3.5	4.5	5.5	6.0	6.8	7.7
Carroll	2.8	3.2	4.0	4.9	5.2	6.0	6.8
Charles City	3.0	3.5	4.5	5.5	6.2	7.0	7.9
Charlotte	3.0	3.5	4.5	5.5	6.0	7.0	7.7
Chesapeake	3.2	3.8	5.1	6.0	7.0	8.0	8.9
Chesterfield	3.0	3.9	4.5	5.5	6.0	7.0	7.6
Clarke	2.7	3.1	4.5	5.0	6.0	7.0	7.6
Craig	2.5	3.0	4.0	4.7	5.5	6.0	6.5
Cumberland	3.0	3.5	4.7	5.8	6.3	7.0	8.0
Culpeper	3.0	3.6	4.7	5.5	6.5	7.5	8.0
Dickenson	2.4	2.9	3.7	4.3	4.8	5.5	6.2

Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 11C-3 24-hr Rainfall Depths

COUNTY	YEAR 1	2	5	10	25	50	100
Dinwiddie	2.9	3.5	4.6	5.6	6.2	7.0	8.0
Essex	3.0	3.2	4.5	5.5	6.0	6.9	7.8
Fairfax	2.7	3.2	4.5	5.2	6.0	7.0	7.7
Fauquier	2.9	3.5	4.5	5.4	6.5	7.2	7.7
Floyd	3.0	3.3	4.3	5.0	5.5	6.2	7.0
Fluvanna	3.0	3.5	4.7	5.7	6.5	7.0	8.0
Franklin	3.3	3.7	4.7	5.7	6.0	7.0	8.0
Frederick	2.5	3.0	4.0	4.9	5.8	6.5	6.0
Giles	2.4	2.9	3.9	4.7	5.0	5.9	6.0
Gloucester	3.0	3.5	4.7	5.9	6.8	7.4	8.0
Goochland	3.0	3.5	4.7	5.7	6.5	7.0	8.0
Grayson	2.8	3.2	4.0	4.9	5.2	6.0	6.8
Greene	3.3	4.0	5.0	6.0	7.0	8.0	9.0
Greensville	3.0	3.5	4.7	5.6	6.5	7.2	8.0
Halifax	3.0	3.5	4.5	5.5	6.0	7.0	7.5
Hanover	2.8	3.3	4.5	5.5	6.0	6.9	7.6
Henrico	2.8	3.3	4.5	5.5	6.0	7.0	7.8
Henry	3.0	3.5	4.6	5.2	6.0	6.5	7.5
Highland	2.8	3.0	4.0	4.9	5.5	6.0	6.8
Isle of Wight	2.9	3.7	5.0	5.8	6.6	7.5	8.4
James City	2.8	3.5	4.7	5.8	6.4	7.2	8.0
King and Queen	2.8	3.4	4.5	5.7	6.2	7.0	7.9
King George	2.8	3.2	4.5	5.5	6.0	7.0	7.5
King William	2.8	3.4	4.5	5.7	6.2	7.0	7.9
Lancaster	2.8	3.5	4.7	5.7	6.5	7.2	8.0
Lee	2.7	3.0	3.7	4.5	5.0	5.6	6.0

Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 11C-3 24-hr Rainfall Depths

COUNTY	YEAR 1	2	5	10	25	50	100
Loudoun	3.0	3.3	4.5	5.2	6.0	6.9	7.5
Louisa	2.9	3.5	4.7	5.5	6.0	7.0	8.0
Lunenburg	2.9	3.5	4.5	5.5	6.0	7.0	7.5
Madison	3.3	4.0	5.0	6.0	7.0	8.0	9.0
Mathews	3.0	3.6	4.8	5.8	6.6	7.2	8.1
Mecklenburg	2.9	3.5	4.5	5.5	6.0	7.0	7.8
Middlesex	3.0	3.5	4.7	5.7	6.5	7.0	8.0
Montgomery	2.5	3.0	4.0	5.0	5.5	6.0	7.0
Nelson	3.3	4.0	5.0	6.0	7.0	8.0	8.5
New Kent	2.8	3.5	4.5	5.6	6.2	7.0	7.9
Northampton	3.1	3.7	5.0	6.0	6.8	7.6	8.6
Northumberland	2.8	3.5	4.7	5.7	6.5	7.2	8.0
Nottoway	3.0	3.5	4.5	5.5	6.0	7.0	7.9
Orange	3.2	3.5	4.7	5.5	6.5	7.5	8.0
Page	2.5	3.2	4.7	5.5	7.0	7.5	8.5
Patrick	2.8	3.5	4.5	5.0	5.8	6.2	7.3
Pittsylvania	2.8	3.5	4.5	5.2	6.2	6.7	7.5
Powhatan	3.0	3.5	4.5	5.5	6.0	7.0	7.5
Prince Edward	3.0	3.5	4.5	5.5	6.0	7.0	7.8
Prince George	3.0	3.5	4.7	5.7	6.2	7.0	8.0
Prince William	3.0	3.5	4.S	5.3	6.0	7.0	7.8
Pulaski	2.5	3.0	4.0	4.8	5.0	6.0	6.5
Rappahannock	3.0	4.0	4.7	5.7	7.0	8.0	8.5
Richmond	3.0	3.5	4.5	5.7	6.2	7.0	7.9
Roanoke	3.0	3.5	4.5	5.0	6.0	6.7	7.5

Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 11C-3 24-hr Rainfall Depths

	YEAR						
COUNTY	1	2	5	10	25	50	100
Rockbridge	3.0	3.5	4.5	5.5	6.2	7.0	8.0
Rockingham	3.0	3.5	4.5	5.0	6.0	7.0	8.0
Russell	2.5	3.0	3.8	4.4	5.0	5.5	6.0
Scott	2.6	3.0	3.7	4.5	5.0	5.5	6.0
Shenandoah	2.5	3.0	4.0	5.0	6.0	6.5	7.0
Smyth	2.6	2.9	3.8	4.5	5.0	5.6	6.0
Southampton	2.8	3.4	4.8	5.7	6.5	7.2	8.0
Spotsylvania	3.1	3.5	4.5	5.5	6.0	7.0	7.5
Stafford	2.9	3.5	4.5	5.5	6.0	7.0	7.5
Suffolk	3.2	3.7	5.0	6.0	6.7	7.7	8.5
Surry	2.8	3.4	4.8	5.7	6.5	7.2	8.0
Sussex	2.8	3.4	4.8	5.7	6.5	7.2	8.0
Tazewell	2.5	2.9	3.8	4.4	5.0	5.5	6.0
Virginia Beach	3.0	3.8	5.0	6.0	7.0	8.0	9.0
Warren	2.8	3.5	4.5	5.1	6.5	7.0	8.0
Washington	2.6	3.0	3.8	4.5	5.0	5.6	6.0
Westmoreland	2.8	3.5	4.5	5.6	6.1	7.0	7.9
Wise	2.5	2.9	3.8	4.5	5.0	5.5	6.0
Wythe	2.6	2.9	3.8	4.6	5.0	5.8	6.0
York	3.0	3.7	4.8	6.0	6.6	7.4	8.2

Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999

Appendix 11D-1 Stormwater Management and Temporary Sediment Basin Summary

OUTFALL LOCATION STA.	Total Drainage Area (AC)	Road Area (AC)	Area of Impervious (AC)	Peak Discharge (CFS)	% Diff. (2-yr)	Peak Discharge (CFS)	% Diff. (10-yr)	Vel. (FPS)	Channel Depth (FT)	Normal Depth (FT)	Channel Adequacy	D.A Draining Across Dist. Area (AC)	SWM Req'd	TSB Req'd	Diverted Outfall 100-yr Adequacy	NOTES

Appendix 11E-1 SWM Facility Tabulation Sheet

ORIGINAL SUBMISSION ______ REVISED SUBMISSION ______ (Check one)
(1) DISTRICT NO: ______ (2)RTE NO: ______ (3)COUNTY/CITY ______
(4)PROJECT NUMBER: ______ (5)AD DATE: ______
(6)LOC./STA.: ______ (7)TYPE BASIN ______
(8)STORAGE VOL.: ______ (in CU. FT. or AC. FT.)
(9)WATERSHED NAME: ______
(10)REM/MONITOR:

ORIGINAL SUBMISSION ______ REVISED SUBMISSION ______ (Check one)
(1) DISTRICT NO: ______ (2)RTE NO: ______ (3)COUNTY/CITY ______
(4)PROJECT NUMBER: ______ (5)AD DATE: ______
(6)LOC./STA.: ______ (7)TYPE BASIN ______
(8)STORAGE VOL.: ______ (in CU. FT. or AC. FT.)
(9)WATERSHED NAME: ______
(10)REM/MONITOR:

ORIGINAL SUBMISSION ______ REVISED SUBMISSION ______ (Check one)
(1) DISTRICT NO: ______ (2)RTE NO: ______ (3)COUNTY/CITY ______
(4)PROJECT NUMBER: ______ (5)AD DATE: ______
(6)LOC./STA.: ______ (7)TYPE BASIN ______
(8)STORAGE VOL.: ______ (in CU. FT. or AC. FT.)
(9)WATERSHED NAME: ______
(10)REM/MONITOR:

ORIGINAL SUBMISSION ______ REVISED SUBMISSION ______ (Check one)
(1) DISTRICT NO: ______ (2)RTE NO: ______ (3)COUNTY/CITY ______
(4)PROJECT NUMBER: ______ (5)AD DATE: ______
(6)LOC./STA.: ______ (7)TYPE BASIN ______
(8)STORAGE VOL.: ______ (in CU. FT. or AC. FT.)
(9)WATERSHED NAME: ______
(10)REM/MONITOR:

ORIGINAL SUBMISSION ______ REVISED SUBMISSION ______ (Check one)
(1) DISTRICT NO: ______ (2)RTE NO: ______ (3)COUNTY/CITY ______
(4)PROJECT NUMBER: ______ (5)AD DATE: ______
(6)LOC./STA.: ______ (7)TYPE BASIN ______
(8)STORAGE VOL.: ______ (in CU. FT. or AC. FT.)
(9)WATERSHED NAME: ______
(10)REM/MONITOR:

Appendix 11F-1 DDM #2 Management of Stormwater

VIRGINIA DEPARTMENT OF TRANSPORTATION

LOCATION AND DESIGN DIVISION

DRAINAGE DESIGN MEMORANDUM

GENERAL SUBJECT: MANAGEMENT OF STORMWATER	NUMBER: DDM 2
SPECIFIC SUBJECT: Engineering and Plan Preparation	DATE: November 17, 1999
	SUPERSEDES: LD-99 (D) 195.3

BACKGROUND

- Acts of the General Assembly have resulted in the issuance of Stormwater Management (SWM) and Erosion and Sediment Control (E&S) regulations. The general application to highway operations associated with these regulations is addressed in this memorandum. Instructions for the incorporation of the E&S details in plan assemblies are contained in the current version of I&IM LD-(D) 11.

- Additional details and examples of the engineering application of the State SWM regulations in the design of VDOT projects can be obtained from the VDOT Hydraulics Section in any of the various district offices or the central office in Richmond.

- Further details and information regarding either the State SWM Regulations or the E&S Regulations can be obtained from: Virginia Department of Conservation and Recreation (DCR), 203 Governor Street, Richmond, VA 23219 or via the Internet: www.state.va.us/~dcr. Details may also be obtained from the SWM Handbook and the E&S Handbook published by DCR and available for reference in all VDOT Hydraulics Sections.

Appendix 11F-1 DDM #2 Management of Stormwater

OBJECTIVE

SWM

- To inhibit the deterioration of the aquatic environment by instituting a stormwater management 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.

E&S

- To effectively control soil erosion, sediment deposition, and post development runoff to minimize soil erosion and to prevent any sediment from escaping the project limits.

CRITERIA

General

- The runoff control provisions of both regulations are complementary and will be addressed under a single set of criteria. The information and instructions contained in this memorandum supersede all previous departmental documents. Where there are conflicts with previous instructions, this memorandum shall take precedence.

- The Erosion and Sediment Control Regulations apply to all activities that disturb 929 square meters (10,000 square feet) or more of land area.

- The Stormwater Management Regulations are applicable to all state projects.

EXEMPTIONS

- Linear development (highway) projects are exempt from the STORMWATER MANAGEMENT REGULATIONS provided that:

  1. Less than one acre of new impervious area will be added per outfall and

  2. There will be insignificant increase in peak flow rates and

  3. There are no existing or anticipated flooding or erosion problems downstream.

- "State Projects" are those land development activities wherein VDOT has funded any portion of the design, right of way acquisition, or construction (including public/private partnerships used for constructing state highways). Projects which are designed and

Appendix 11F-1 DDM #2 Management of Stormwater

constructed by other parties and that are accepted into the state system for maintenance after completion of construction (including subdivision streets) are not considered state projects and must conform to appropriate local regulations. Land development activities occurring within existing VDOT right of way, which are allowed by permit and which are designed, constructed, and funded by other parties, are not considered state projects and must conform to appropriate local regulations.

- "Land Development Project" is defined as a manmade change to the land surface that potentially changes its runoff characteristics as a permanent condition. The permanent condition should consider the effects of mature vegetative cover and should not be concerned with temporary changes due to construction activities. The temporary changes are addressed by the E & S regulations.

Water Quantity Control

- Water quantity control shall be governed by the Virginia Erosion and Sediment Control Regulations Minimum Standard 19 which requires an adequate receiving channel for stormwater outflows from all projects with more than 929 square meters (10,000 square feet) of land disturbance.

- Receiving channels, pipes and storm sewers shall be reviewed for adequacy based upon the following criteria:

  Natural channels shall be analyzed by the use of a 2-year storm to verify that stormwater will not overtop channel banks or cause erosion of the channel bed and banks. All previously constructed manmade channels shall be analyzed by the use of a 10-year frequency storm to verify that the stormwater will not overtop the banks and analyzed by the use of a 2-year storm to verify that the stormwater will not cause erosion of the bed or banks. Pipes and storm sewer systems shall be analyzed by the use of a 10-year frequency storm to verify that the stormwater will be contained within the pipe or storm sewer system. The receiving channel at the outlet of the pipe or storm sewer shall be analyzed for adequacy of the 2-year storm for natural channels or the 10-year storm for man made channels.

- Water quantity control for the 1-year storm (in lieu of the 2 year storm required by Minimum Standard 19) may be needed if there is existing or anticipated erosion downstream. 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. The computations are similar to those used for detaining the

- Water Quality Volume (WQV) and releasing over a 30-hour period. See the SWM Handbook by DCR: pages 1-23 and 5-38 thru 5-41. 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. The control of the 1-year storm may require a basin size that is 1.5 to 2 times larger than a basin used to control the increase in Q from a 2-year or a 10-year storm.

Appendix 11F-1 DDM #2 Management of Stormwater

- Pre-development conditions shall be that which exist at the time the road plans are approved for right of way acquisition. All land cover shall be assumed to be in good condition regardless of actual conditions existing at the time the computations are made.

- Impounding structures (dams) that are not covered by the Virginia Dam Safety Regulations shall be checked for structural integrity and floodplain impacts for the 100-year storm event.

- Outflows from the stormwater management facilities shall be discharged into an adequate receiving channel.

- Existing swales being utilized as natural outfall conveyances for pre-development run-off will be considered as channels and, if the swale satisfactorily meets the criteria contained in Minimum Standard 19 of the Virginia Erosion and Sediment Control Regulations for post-development run-off, it will be considered as an adequate receiving channel.

- Surface runoff from drainage areas of three acres or more that pass through a disturbed area must be controlled by a sediment basin. The sediment basin shall be designed and constructed to accommodate the anticipated sediment loading from the land disturbing activity and adjacent property within the watershed that has a high erosion potential. The design of the outfall device or system shall take into account the total drainage area flowing through the basin.

- Construction of stormwater management facilities should be avoided in floodplains. When this is unavoidable, a special examination to determine the adequacy of the proposed stormwater management facilities during the 10-year flood shall be required. The purpose of this analysis is to ensure that the stormwater management facility will operate effectively. The stormwater management facility shall also be examined for structural stability during the passage of the 100-year flood event on the floodplain and shall be examined for any possible impacts caused by the basin on the 100-year flood characteristics of the floodplain. The construction of stormwater management facilities shall be in compliance with all applicable regulations under the National Flood Insurance Program.

- 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 of the project, the receiving channel may be considered adequate, with respect to channel stability requirements under the Erosion & Sediment Regulations, without further computation.

Water Quality Control

- SWM design for water quality control is to be in accordance with the latest revisions to the Virginia SWM Regulations. The following comments represent the significant points of the current regulations and the page numbers given are referenced to the SWM Handbook from DCR.

Appendix 11F-1 DDM #2 Management of Stormwater

1. BMP requirements for quality control are "Technology Based" and the type of BMP is determined by the percent of **new** impervious area within the site (or Right of Way) per outfall (SEE TABLE 1 – BMP SELECTION TABLE) and also the drainage area size (in accordance with the general design criteria as outlined in the SWM Handbook).

2. BMP requirements for flooding or quantity control are set by the Erosion and Sediment Control Regulations Minimum Standard 19 for adequate receiving channels.

3. Extended Detention Basins and Extended Detention Basins Enhanced require a Water Quality Volume (WQV) of 2 x the standard WQV or 1" of Runoff from the new impervious area.

4. Extended Detention Basins and Extended Detention Basins Enhanced require a 30-hour drawdown time for the required WQV. The 3" minimum size water quality orifice has been eliminated. If the required orifice size is found to be significantly less than 3" an alternative water quality BMP should be investigated such as a linear facility which treats the first flush and allows large storms to overflow. The calculation procedure for drawdown time and orifice sizing is shown on PAGES 5-33 THRU 5-38 and also in example problems available from VDOT.

5. Sediment Forebays should be used on Extended Detention Basins and Extended Detention Basins Enhanced with the volume set as 0.1" – 0.25" x the new impervious area or 10% of the required detention volume. SEE DETAILS PAGE 3.04-6. The stabilized overflow spillway may be constructed of riprap or concrete.

6. Suggested details for the Extended Detention Basin are shown on PAGES 3.07-04 AND 3.07-5. The riprap lined low flow channel thru the basin is not recommended by VDOT due to maintenance concerns.

7. Suggested details for the Extended Detention Basin Enhanced are shown on PAGES 3.07-6 AND 3.07-7. The geometric design will probably need to be more symmetrical than shown in order to construct the basin to the dimensions needed.

- Non-structural practices including, but not limited to, control of land use development, 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 the water quality requirements. Approval of such non-structural measures will be secured in advance from the Department of Conservation and Recreation (DCR).

MULTI-USE BASINS

Quantity Control – Quality Control – Temporary Sediment Storage

SWM basins will normally be used for both quantity control and for quality control. Under the revised regulations some basins may occasionally only be needed for quality control. Most SWM basins are needed to serve as temporary sediment basins and the design and computations will need to address the dual function. The design that is needed for a permanent SWM basin may need to provide additional temporary sediment storage volume that is in excess of the required WQV. The two different volumes (WQV and temporary sediment storage volume) should not be added together and the larger of the two should govern the design. The additional volume needed for sediment storage may be provided by excavating the bottom of the basin lower than required for WQV. The permanent outlet structure (riser or wall) can be temporarily altered to serve as the control structure for the sediment basin. (See the enclosed design detail drawings and the DCR SWM Handbook). When the project is nearing completion and the basin is no longer needed for sediment control, the basin can be easily converted to a permanent SWM basin.

IMPLEMENTATION

Plan Preparation

- Standard and minimum plan projects shall show stormwater management and erosion control measures on the plans as directed in IIM LD- (D) 11 and the Road Design Manual.

- No-plan projects must have the erosion and sediment control measures included in the construction narrative addressing their placement. This narrative may be supplemented by appropriate "simple" sketches. Stormwater management facilities may be addressed in a similar fashion provided sufficient detail is included to ensure their proper construction. When this is not practicable, additional sketches shall be included in the no-plan assembly to define the construction of these items.

- Any other type of project activity that does not have a plan, such as some SAAP Projects, shall conform to the no-plan requirements.

- Maintenance activities which disturb more than 929 square meters (10,000 square feet) must have a plan developed by the appropriate personnel that addresses the erosion and sediment control requirements for that activity. Maintenance activities which involve a

- "Land Development Project" of one acre or more of land must have a stormwater management plan. The plan shall conform to the requirements for a no-plan project.

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- Normal ditch cleaning and pulling of shoulders are not considered land disturbing activities for the purposes of erosion and sediment control if less than 929 square meters (10,000 square feet) of drainage area is disturbed feeding any one pipe or ditch outlet.

Foundation Data

- Foundation data (a soil boring) for the base of the dam should be requested for all Stormwater Management 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:

  1. Excavation from the basin may be used to construct the dam, OR
  2. Rock may be encountered in the area of excavation, OR
  3. A high water table is suspected which 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 may be needed. The number and locations of the borings are to be determined by the drainage designer.

- The foundation data should be requested by the drainage designer when the request is initiated for culvert foundation data. See I&IM LD-(D) 121.

Right of Way

- Permanent stormwater management facilities may be placed in fee right of way or in permanent easements. It is recommended that all permanent stormwater management facilities (dams, ponds, risers, etc.) be placed within fee right of way initially. Ditches and similar features may initially be placed in permanent easements. The final decision on right of way versus permanent easement can be made at the field inspection or as a result of the design public hearing. The Department will generally be amenable to the desires of affected landowners in this matter. The multiple use of property for stormwater management and such features as utilities is permissible. The decision on the advisability of such actions must be made on an individual site basis.

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Table 1*

BMP SELECTION TABLE

Water Quality BMP	Target Phosphorus Removal Efficiency	Percent Impervious Cover**
Vegetated filter strip Grassed swale	10% 15%	16-21%
Constructed wetlands Extended detention (2xWQ Vol) Retention basin I (3xWQ Vol.)	30% 35% 40%	22-37%
Bioretention basin Bioretention filter Extended detention-enhanced Retention basin II (4xWQ Vol) Infiltration (1xWQ Vol)	50% 50% 50% 50% 50%	38-66%
Sand filter Infiltration (2xWQ Vol) Retention basin III (4xWQ Vol with aquatic bench)	65% 65% 65%	67-100%

* Innovative or alternate BMPs not included in this table may be allowed at the discretion of DCR.

** Percent Impervious Cover: New impervious area within the site or Right of Way per outfall.

DETAILS FOR DESIGN OF DAMS

VDOT STORMWATER MANAGEMENT BASINS

The following details are to be incorporated into the design of VDOT Stormwater Management (SWM) Basins in order to be in compliance with the State SWM Regulations Revisions of 1998 and the SWM HANDBOOK of the Virginia Department of Conservation and Recreation (DCR). The revisions are also due to concerns with seepage thru the dam and along the culvert due to the ponding of water in the basins being of longer duration than previous designs that used a minimum 3” water quality orifice.

1. Foundation data for the dam is to be secured from the Materials Division in order to determine if the native material will support the dam and not allow ponded water to seep under the dam.

2. The foundation material under the dam and the material used for the embankment of the dam should be type A-4 or finer* 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

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4' or the recommendation of 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 or an HDPE or LDPE liner) and be approved by the Materials Division.

3. The pipe culvert under or thru the dam is to be concrete sewer pipe with rubber gaskets. Pipe Specifications: 232 (AASHTO M170), Gasket Specification: 212 (ASTM C443)

4. A concrete cradle is to be used under the pipe to prevent seepage thru the dam. The concrete cradle is to extend from the riser or inlet end of the pipe to the outlet of the pipe. See attached Special Design Drawing No. 2209.

5. If the height of the dam is greater than 15' the design of the dam is to include a Homogenous Embankment with seepage controls or Zoned Embankment or similar design and is to be approved by the Materials Division.

*In accordance with the AASHTO Classification System (M145)

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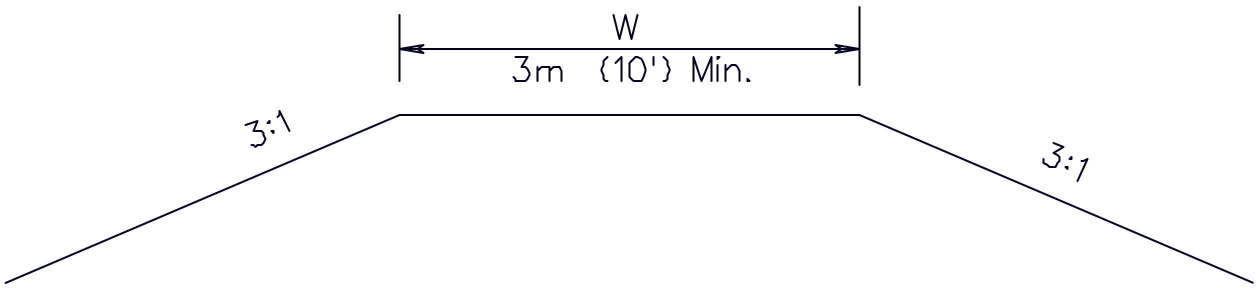
DESIGN GUIDELINES
FOR
STORMWATER (SWM) BASINS



DAM or EMBANKMENT

The top width (W) of the SWM basin dam should be 3m (10') minimum to facilitate both construction and maintenance.

The side slopes should be no steeper than 3:1 to permit mowing and cleanout. The bottom slope of the basin should be no more than 2% or no less than 0.5%.

The depth of the basin to the primary overflow (crest of riser, or orifice or weir) should be no more than 1m (3 ft.) if possible, in order to reduce the hazard potential. If the depth needs to be more than about 1m (3 ft.) fencing should be considered.

All SWM basins should be reviewed for the needs of fencing, barricades and no trespassing signs in accordance with the VDOT guideline for Fencing of SWM Basins.

The length to width ratio should be about 3:1 (wider at the outlet end). If the ratio is less than about 2:1, and if there is concern that the velocity of flow through the basin is high, consideration should be given to using baffles within the basin to reduce velocity. Baffles should be constructed of "pervious" type material such as snow fence rather than earth berms which do not reduce the velocity.

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Perimeter Controls

- Fencing of SWM Basins

  - Fencing of stormwater management basins is normally not required and should not be used for most basins due to:

    - Insignificant Hazard

      Ponding of water in the basin should only occur with very heavy storms and be noticeable for a few hours. The ponded depth will normally be no more than about 1 meter (3 feet). Ponds and lakes are almost never fenced, even though they may be located in subdivisions and have deep, permanent ponding.

    - Limits Maintenance

      Fencing will limit maintenance operations and could deter the frequency of maintenance. Maintenance operations can damage fencing particularly if equipment becomes stuck.

  - Fencing of SWM basins may occasionally be needed and should be used when:

    - Basin is deep with ponded depth greater than about 1 meter (3 feet) and/or has steep side slopes with 2 or more side slopes steeper than 3:1;

      or

    - Basin is in close proximity to schools, playgrounds or similar areas where children may be expected to frequent;

      or

    - Recommended by the Field Inspection Report, the Resident Engineer or the City/County (where City/County will take over maintenance responsibility.)

- Barricades

  - A chain or gate may be needed on some basins to prohibit vehicular access if there is concern with dumping or other undesirable access.

- Signs

  - “No Trespassing signs” shall be considered for use on all basins, whether fenced or unfenced and should be recommended as needed on the Field Inspection Report.

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Regional Facilities

- There are many cases where it is more feasible to develop one large stormwater management facility to control a watershed area rather than a number of small individual facilities controlling small drainage basins. The concept of regional stormwater management facilities is endorsed by VDOT provided that certain requirements are met.

- Development and use of regional stormwater management facilities must be a joint undertaking by VDOT and the local governing body. The site must be part of a master stormwater management plan developed by the local governing body and any agreements related to these facilities must be consummated between VDOT and the local governing body. VDOT will not enter into an agreement with private individuals or corporations.

- Where the roadway embankment serves as an impounding structure, the right of way line will normally be set at the inlet face of the drainage structure. The local government will be responsible for the maintenance and liabilities outside of the right of way and VDOT will accept the same responsibilities inside the right of way.

- Hydraulic design of regional management facilities must address any mitigation needed to offset increases in runoff from the roadway. Stormwater management facilities located upstream of the roadway shall provide sufficient mitigation of peak outflow to compensate for roadway runoff which may bypass the facility.

Maintenance

- Requirements for maintenance of stormwater management facilities, the recommended schedule of inspection and maintenance, and the identification of persons responsible for the maintenance will be addressed by the Maintenance Division.

Future Reconstruction

- If a stormwater management facility is constructed to address the increase in runoff from a current project and, at some time in the future, is displaced to accommodate future construction, a new facility constructed at that time must address increases in runoff due to the future construction and the increases in runoff that were mitigated by the original stormwater management facility.

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Reporting

- VDOT is required to submit an annual report to the Department of Conservation and Recreation (DCR) that identifies the location, number and type of stormwater management facilities installed during the preceding year, their storage capacities, the affected water body, and a summary of any water quality monitoring data associated with the facility. A database has been established on the Hydraulics Section's telecommunication file system to record this type of data for all projects. It shall be the responsibility of the district drainage engineer and the hydraulic design engineers in the Central office to ensure that the required information is logged on the database for all stormwater management facilities that are designed for roadway projects. In order for the database to reflect those facilities constructed during the preceding year, it is recommended that the required information be logged at the time of the first submission of plans to the Construction Division. The reporting period will be from July 1 to June 30.

PLAN DETAILS

Stormwater Management Drainage Structure Standard SWM-1

- To be used at all applicable locations where a drop inlet type control structure is desired.

Stormwater Management Riser Pipe Standard SWM-RP

- To be used at all applicable locations where an open top manhole type control structure is desired.
- Diameters from 900 mm to 1500 mm in 150 mm increments (36" to 60" in 6" increments).
- Height of structure above outlet pipe invert should be limited to about 1 meter (3 feet) maximum.

Stormwater Management Dam

- To be used at locations where a wall type control structure is desired (includes modifications to standard endwalls). Normally used for shallow depths of ponding.
- Details to be provided for individual locations.

Details of control structures other than those above shall be submitted to the office of the State Hydraulics Engineer to facilitate future development of standard details.

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Stormwater Management Details Standard SWM-DR

- Provide at each location requiring a water quality orifice. The size opening for the water quality orifice shall be specified for each basin.

Access

- A means of access for inspection and maintenance personnel shall be provided at each SWM facility location.
- A turnaround should be provided on vehicular entrances when needed based upon accessibility and traffic volume.
- Appropriate surface material shall be provided for each vehicular entrance.

Method of Measurement – Basis of Payment

Stormwater Management Drainage Structure (SWM-1):

- Basis of payment to be linear meters (feet) measured from invert of structure to top of concrete cover.

Stormwater Management Riser Pipe:

- Basis of payment to be linear meters (feet) of the size specified measured from invert of structure to the top of the structure.

Stormwater Management Dam:

- Basis of payment to be m^3 (cubic yards) of Concrete Class A3 Miscellaneous and kilograms (pounds) of Reinforcing Steel.

Grading:

- Excavation for stormwater management basins will be measured and paid for as m^3 (cubic yards) of Stormwater Management Basin Excavation.
- If additional fill material is needed for dams or berms this will be measured and paid for as m^3 (cubic yards) of Regular Excavation, Borrow Excavation or Embankment.
- The Grading Diagram is to reflect how the m^3 (cubic yards) of Stormwater Management Excavation and m^3 (cubic yards) of Embankment or Borrow is to be distributed.

Stormwater Management Summary

- All items related to the construction of stormwater management facilities shall be summarized, by location, in a separate summary located on or near the Drainage Summary (see attached example).

- If Borrow or Embankment is needed, include in roadway totals on Grading Diagram and Summary.

PAY ITEMS

The following pay items are established:

PAY ITEM			UNIT		ITEM CODE
			Metric	Imperial	
SWM Basin Excavation			m^3	Cu. Yds.	27545
SWM Drainage Structure (SWM-1)			m	Lin. Ft.	27550
SWM Riser Pipe	900 mm	(36”)	m	Lin. Ft.	27560
SWM Riser Pipe	1050 mm	(42”)	m	Lin. Ft.	27562
SWM Riser Pipe	1200 mm	(48”)	m	Lin. Ft.	27564
SWM Riser Pipe	1350 mm	(54”)	m	Lin. Ft.	27566
SWM Riser Pipe	1500 mm	(60”)	m	Lin. Ft.	27568
SWM Dam: Conc. Cl. A3 Misc. Reinf. Steel			 m^3 kg	 Cu. Yds. Lbs.	 00525 00540

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(METRIC) STORMWATER MANAGEMENT CONTROL SUMMARY

LOCATION	STORMWATER MANAGEMENT BASIN EXCAVATION	STORMWATER MANAGEMENT DRAINAGE STRUCTURE SWM-1	STORMWATER MANAGEMENT RISER PIPE					STORMWATER MANAGEMENT DAM		DRY RIP RAP CLASS__
			900 mm	1050 mm	1200 mm	1350 mm	1500 mm	CONC. CLASS A-3 MISC.	REINF. STEEL	
	m^3	meters	meters					m^3	kg	Metric Tons
TOTAL										

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(IMPERIAL) STORMWATER MANAGEMENT CONTROL SUMMARY

LOCATION	STORMWATER MANAGEMENT BASIN EXCAVATION	STORMWATER MANAGEMENT DRAINAGE STRUCTURE SWM-1	STORMWATER MANAGEMENT RISER PIPE					STORMWATER MANAGEMENT DAM		DRY RIP RAP CLASS__
			36”	42”	48”	54”	60”	CONC. CLASS A-3 MISC.	REINF. STEEL	
	C.Y.	Lin. Ft.	Lin. Ft.					C.Y.	Lbs.	Tons.
TOTAL										

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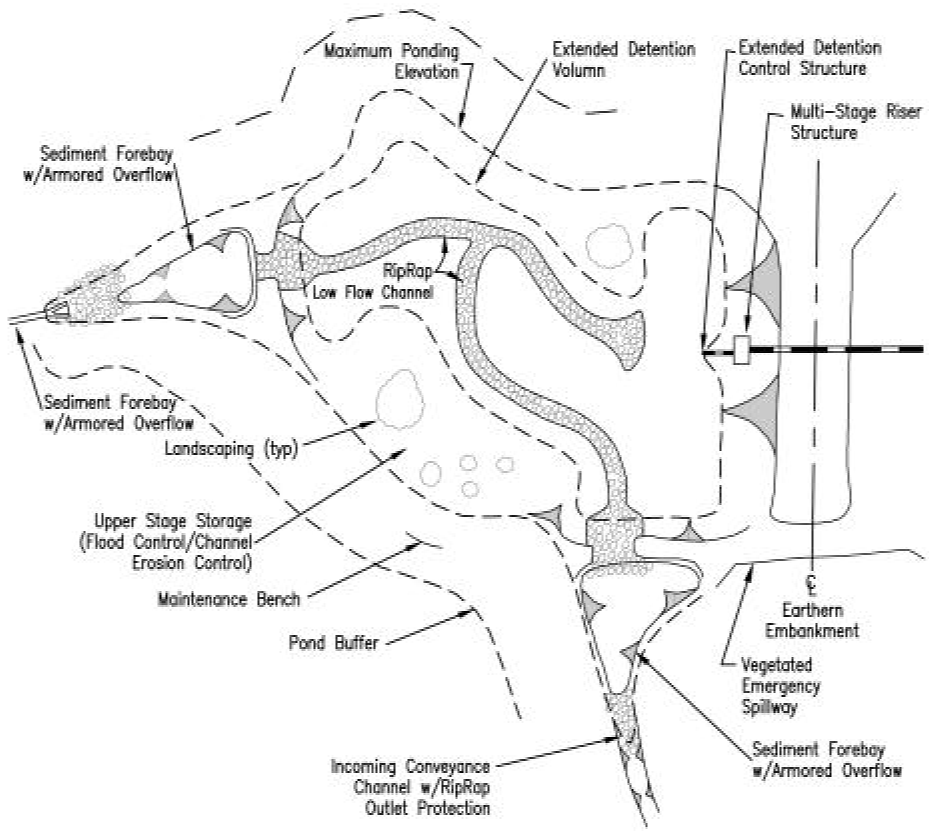
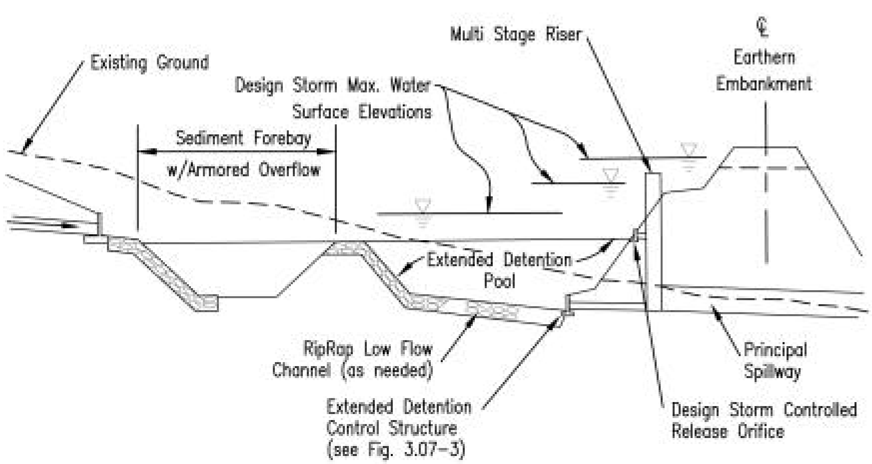
SPECIAL PROVISION SECTIONS 302 AND 303

- The current Special Provision/Copied Note for measurement and payment for stormwater management items is accessible through the Internet at http://www.vdot.state.va.us. The path is “Opportunities Network”, “Construction and Maintenance Contracting Opportunities”, “Resources”, “Road and Bridge Specifications”. Win Zip is available from this web site to enable viewing of the information. Questions pertaining to the web site may be addressed to the Construction Division (Ms. Mary Roane) at (804) 786-2124. Questions concerning the Special Provisions/Copied Notes may be addressed to Ms. Norma Gilbert at (804) 786-2356. Please note the effective advertisement date for these provisions.

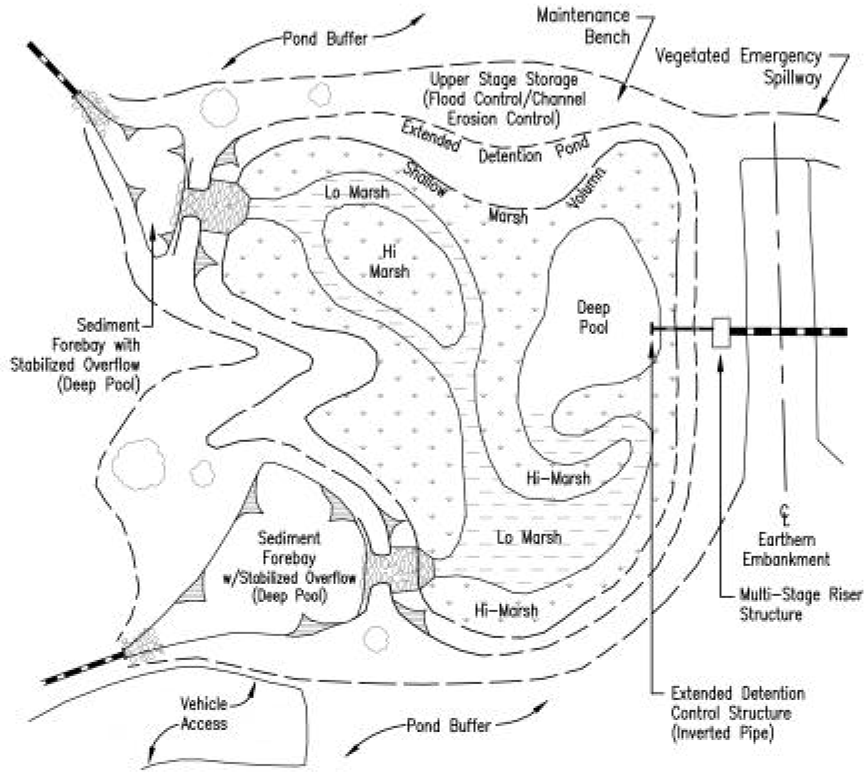
Appendix 11G-1 Extended Detention Basin – Plan and Section





Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. I, 1999.

Appendix 11G-2 Enhanced-Extended Detention Basin – Plan and Section



Depth Zone	% of Surface Areas
Deep Pool	20%
Lo Marsh	35%
Hi Marsh	45%

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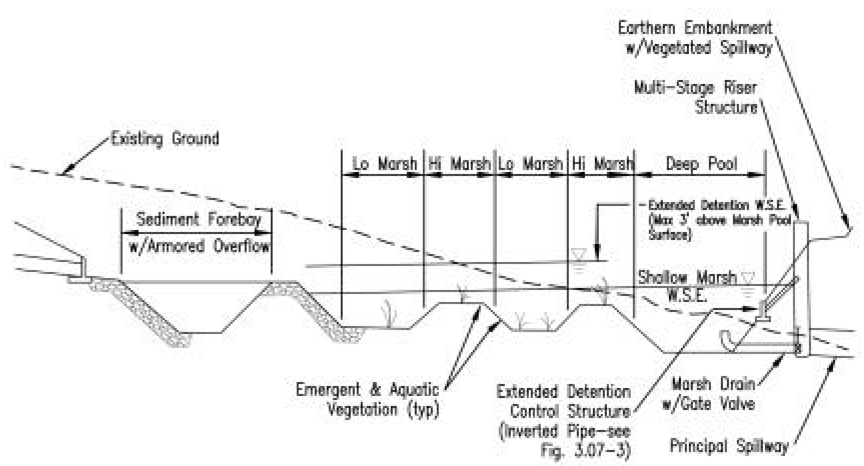
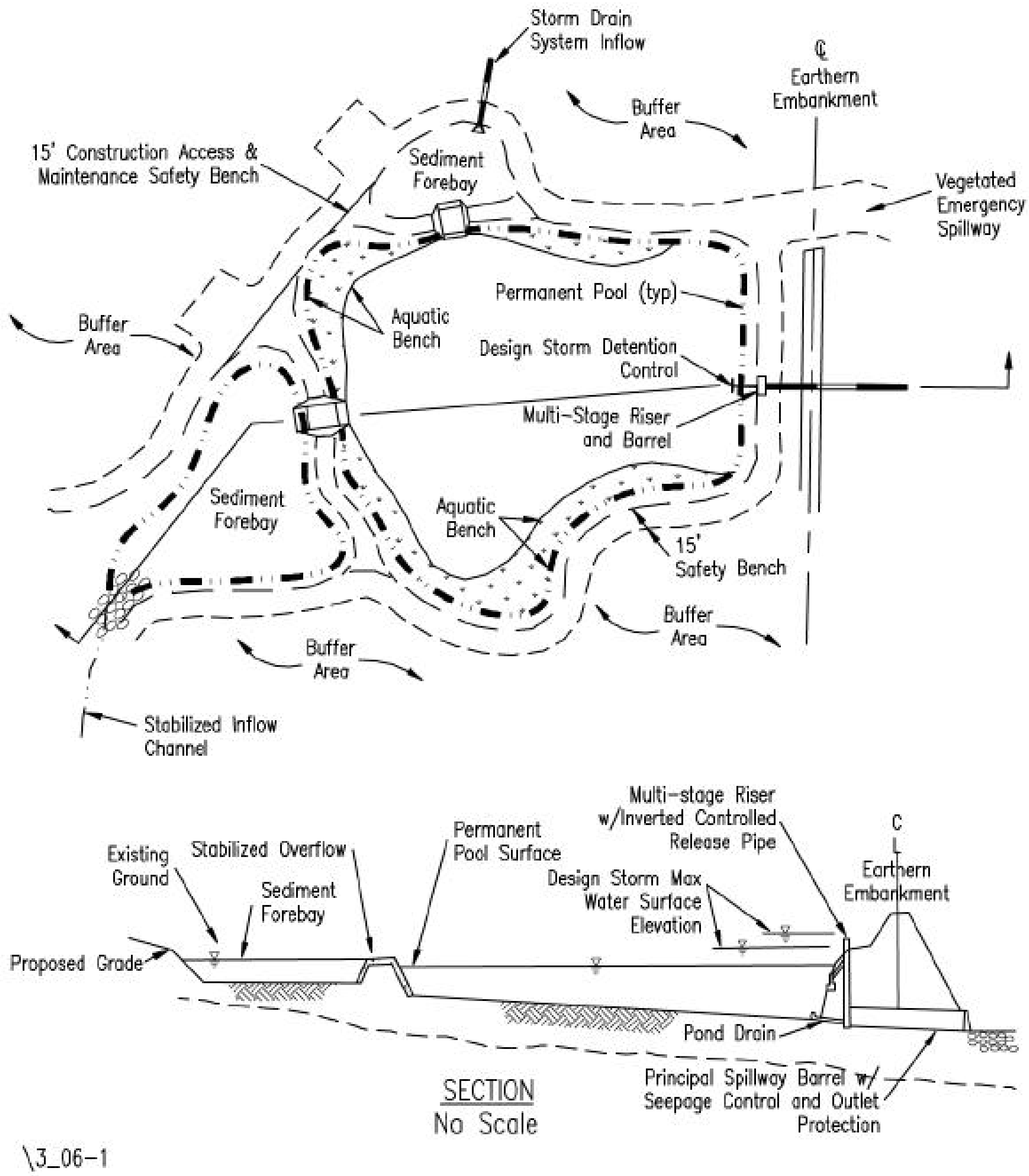


Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. I, 1999.

Appendix 11G-3 Retention Basin - Plan and Section



Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. I, 1999.

Chapter 12 – Bridge and Structure Hydraulics

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Chapter 12 - Bridge and Structure Hydraulics

12.1 Introduction

12.1.1 Definition

Bridges are defined as:

- Structures that transport traffic over waterways or other obstructions
- Part of a stream crossing system that includes the approach roadway over the floodplain, relief openings, and the bridge structure
- Structures with a centerline span of 20 feet or more. However, structures designed hydraulically as bridges, as described above, are treated as bridges in this chapter, regardless of length

12.1.2 Analysis/Design

Proper hydraulic analysis and design is as vital as the structural design. Stream crossing systems 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
- Full compliance with the requirements of existing Federal Emergency Management Agency (FEMA) or other officially delineated or regulatory floodplains

12.2 Design Policy

12.2.1 FEMA Floodplain Compliance

- The final design selection should be in full compliance with the maximum water surface elevation allowed by FEMA
- The final design should not significantly alter the flow distribution in the floodplain
- Where design considerations permit, the "crest-vertical curve profile" should be considered as the preferred highway crossing profile when allowing for embankment overtopping at a lower discharge
- Degradation or aggradation of the river should be estimated and contraction and local scour determined and appropriate positioning of the foundation, below the total scour depth if practicable, should be included as part of the final design

12.3 Design Criteria

12.3.1 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:

- Backwater will not significantly increase flood damage to property upstream of the crossing
- Velocities through the structure(s) will not damage either the highway facility or increase damages to adjacent property
- Maintain the existing flow distribution to the extent practicable
- 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

12.3.2 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 water surface profile programs such as HEC-RAS, HEC-2, or WSPRO.

12.3.2.1 Travelway

Inundation of the travelway dictates the level of traffic services provided by the facility. The travelway overtopping flood level identifies the limit of serviceability. Desired minimum levels of protection from travelway inundation for functional classifications of roadways are presented in Chapter 6, Hydrology.

12.3.2.2 Risk Evaluation

The selection of hydraulic design criteria for determining the waterway opening, road grade, scour potential, riprap, and other features should consider the potential impacts to:

- Traffic
- Adjacent property
- Environment

The consideration of the potential impacts constitutes an assessment of risk for the specific site. The least total expected cost (LTEC) alternative should be developed in accordance with Federal Highway Administration (FHWA) HEC-17 only where a need for this type of analysis is indicated by the risk assessment. This analysis provides a comparison between other alternatives developed in response to considerations such as environmental, regulatory, and political.

12.3.2.3 Design Floods

Design floods for such things as the evaluation of backwater, clearance, and overtopping, unless available from FEMA or other appropriate sources, should be established predicated on local site conditions. They should reflect consideration of traffic service, environmental impact, property damage, hazard to human life and floodplain management criteria. Design floods for roadway inundation are specified in Chapter 6, Hydrology. It should be noted, in the case of bridged waterways, that the design flood is normally whichever of the customarily documented events (i.e. the 2, 5, 10, 25, 50, 100, & 500-yr. floods) that will pass under the bridge superstructure at its lowest elevation with at least one or more feet of freeboard, provided that level of protection is acceptable to the bridge designer.

12.3.2.4 Backwater/Increases Over Existing Conditions

Designers shall conform to FEMA regulations for sites covered by the National Flood Insurance Program (NFIP). It is the Department's policy not to allow any increase in the level of the 1 percent flood for delineated floodplains established under the NFIP and for the increase to not exceed one foot during the passage of the 1 percent flood for sites not covered by NFIP. Refer to section 12.6.1 for additional details.

12.3.2.5 Clearance

Where practical a minimum clearance of one foot should be provided between the design approach water surface elevation and the low chord of the bridge for the design flood. Where this is not practicable, the bridge designer should establish the clearance based on the desired level of protection.

12.3.2.6 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 percent redistribution of flow.

12.3.2.7 Scour

Design for bridge foundation scour should consider the magnitude of the flood that generates the maximum scour depth. The design should use a geotechnical design practice factor of safety from 2 to 3. The resulting design should then be checked using a super-flood such as the 0.2 percent event and a geotechnical design practice safety factor of at least 1.0. A plot or sketch showing the scoured bed profile for both the design and super-flood events shall be prepared and included with documentation (LD-293) described in Section 12.6.5.2.

12.4 Design Concepts

12.4.1 Methodologies

A step-backwater computer model is usually 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. However, it has been found that, with careful attention to the setup requirements of each method, acceptable results can usually be achieved regardless of the step-backwater computer model being employed. Where the use of a one-dimensional step backwater computer model is indicated, the Department accepts any computer model currently approved by FEMA but prefers HEC-2 or HEC-RAS.

12.4.2 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. Both these publications can be accessed and/or downloaded from the publications section of the FHWA's Internet web site at http://www.fhwa.dot.gov\bridge\hydpub.htm.

12.4.3 Riprap

Riprap is not to be used for scour protection at piers for new bridges. Riprap may be used to protect exposed abutment slopes or as a scour countermeasure at existing bridge piers and abutments. Design guidelines for placement and sizing of riprap are presented in the FHWA's "Bridge Scour and Stream Instability Countermeasures" (HEC-23) publication. This publication can be accessed and/or downloaded from the publications section of the FHWA's Internet web site at http://www.fhwa.dot.gov\bridge\hydpub.htm.

12.5 Design Procedure

12.5.1 Hydraulic Performance of Bridges

The stream-crossing system is subject to either free-surface flow or pressure flow through one or more bridge openings with possible embankment overtopping. These hydraulic complexities should be analyzed using a one-dimensional step backwater computer model (i.e. all flow is assumed to be proceeding in a downstream direction). See Section 12.4.1 for the Department's recommendation for an appropriate "1-D" computer model. There will be situations where a two-dimensional model (i.e. flow can move in a lateral as well as downstream direction) such as FESWMS would be more appropriate. The use of a two-dimensional model in any given situation should be approved by the Department's Hydraulics Section.

It is impracticable to perform the hydraulic analysis for a bridge by manual calculations due to the interactive and complex nature of those computations. However, the procedures recommended by the FHWA are described in their publication *Hydraulics of Bridge Waterways* (Hydraulic Design Series No. 1).

12.6 Performance and Documentation of Riverine H&HA

12.6.1 Background

A detailed hydrologic and hydraulic analysis (H&HA) should be performed for all of the Department's new or replacement major drainage structures, bridged waterways and significant lateral encroachments (resulting from the placement of highway fill embankments within a floodplain). It is necessary to do this such that Department construction is in compliance with national (i.e. FHWA, FEMA, etc.), state Department of Conservation and Recreation (DCR, etc.), and municipal (locally delineated floodplains) rules and regulations. Detailed analysis, as used here, means that the hydraulic analysis shall be performed using an appropriate step-backwater computer model. See Section 12.4.1 for recommendations. In the case of Department construction in or in proximity to a FEMA floodplain, the same step-backwater computer model used to establish the FEMA floodplain should be employed to assess the impact of the Department's construction. A detailed H&HA should be performed for all bridged waterways regardless of whether or not it falls within a FEMA or other officially delineated floodplain. A detailed H&HA should also be performed for floodplain encroachments (brought about as a result of filling in conjunction with VDOT construction) when they fall within a FEMA or other officially delineated floodplain.

Major culvert installations that do not fall within a FEMA or other officially delineated floodplain may be analyzed using procedures such as presented in the FHWA's Hydraulic Design of Highway Culverts (HDS-5) publication. A "major culvert" in this sense would be defined as one conveying a stream for which the 100-year peak discharge is equal to or greater than 500 cubic feet per second (cfs).

Regardless of whether a bridged waterway, culvert, or encroachment is being evaluated, in situations where a FEMA or other officially delineated floodplain is being considered, no increase in the established natural 100-yr. flood level will be permitted either up or downstream. In situations where no FEMA or other officially delineated floodplain exits, 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.

12.6.2 Necessary Resources

The resources necessary to perform an H&HA usually include, but would not be limited to: topographic maps, aerial photographs, and 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 (i.e. HEC-2 input data, etc.). 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.

12.6.3 Hydrologic Analysis

If the site is not covered by a FEMA (or other officially delineated) floodplain, it will be necessary to determine a range of design peak discharges to use in the subsequent hydraulic analysis. This would typically be done, for ungaged sites, using empirical procedures such as the:

- "Franklin Snyder" method (See Chapter 6 – Hydrology)
- "Effects of Urban Development on Floods in Northern Virginia" (1968) by Daniel G. Anderson (USGS)
- "Methods for Estimating the Magnitude and Frequency of Peak Discharges of Rural, Unregulated Streams in Virginia" (1994) by James A. Bisese (USGS)
- Areal adjustments of design peak discharges from appropriate gaged sites

See Chapter 6, Hydrology, for detailed information on application and procedures of hydrologic methods.

For gaged sites (or streams having stream gages in the proximity of the site), a Log-Pearson Type III frequency distribution (prorated up or downstream as appropriate) would be the preferred method for determining peak discharges. Regional drainage area versus discharge curves would also be appropriate.

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 shouldn'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 peak discharges employed in making the official floodplain delineation shall be employed. Any exception to this policy must be approved by the VDOT Hydraulics Section.

In situations where the officially delineated study considers only the 10, 50, 100, and 500-year flood events, it will be acceptable to estimate the magnitude of intermediate frequency events such as the 2, 5, and 25-year flood events.

In all cases the 2, 5, 10, 25, 50, 100, and 500-year flood magnitudes will either be determined or obtained (from appropriate sources) and employed and documented in the subsequent hydraulic analysis. In addition, the Ordinary Highwater discharge, usually taken to be the drainage area (in square miles) times 1.1 in units of cubic feet per second, shall be determined and documented for the purposes of applying for certain environmental permits.

12.6.4 Hydraulic Analysis

The Department prefers a three-step procedure for performing the hydraulic analysis using an approved (or preferred) step-backwater computer model as described above.

12.6.4.1 Existing Conditions Model

If a FEMA (or other officially delineated) floodplain is involved, the first step will be to mathematically reproduce the hydraulic model using the same step-backwater computer model on which the original floodplain was predicated. This means if the original study was done using HEC-2, the VDOT study shall be done using HEC-2. If the step-backwater 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. Any exception to this criteria must be approved by the Department's hydraulics section. The first hydraulic model will be referred to as the "EXISTING CONDITIONS model.

12.6.4.2 Adjusted Existing Conditions Model

The second step would be to add, delete, and/or adjust any natural ground cross sections to the locations necessary to subsequently model any proposed construction. It should be emphasized that any changes made in the "EXISTING CONDITIONS" model should be solely for the purpose of facilitating the modeling of proposed conditions. This should not be taken as the latitude to change the official model for such things as n-values, new cross section geometry, peak discharges, etc. 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 "ADJUSTED EXISTING CONDITIONS" model. For this model, it will also be necessary to determine the magnitude of the 2, 5, and 25-year flood events (by interpolation and/or extrapolation of the known magnitudes as necessary) and include them (in this and the "PROPOSED CONDITIONS" model).

12.6.4.3 Proposed Conditions Model

The third hydraulic model will include any and all proposed construction (superimposed on the "ADJUSTED EXISTING CONDITIONS" model) and will be referred to as the "PROPOSED CONDITIONS" model. If the "PROPOSED CONDITIONS" model shows any change (from the "ADJUSTED EXISTING CONDITIONS" model), be it in water surface elevations, velocity of flow, or flow distribution, the proposed construction is to be considered as unacceptable and must be adjusted until no changes occur. It should be noted here that VDOT's policy is to permit no increase in either the 100-year natural floodplain elevation or 100-year floodway elevation, despite FEMA's policy of allowing up to one-foot increase in the natural 100-year floodplain.

12.6.4.4 Procedure When No Existing Conditions Model is Available

In instances where there is no FEMA (or other officially delineated) floodplain involvement, and a bridged waterway is involved, it will be necessary to establish the existing hydraulic conditions through a process that will be referred to as calibration. This calibration is to be accomplished by attempting to reconcile the historical high water elevation secured by or during the survey. The procedure is:

Step 1: Set up a hydraulic model reflecting existing conditions using whatever topographic and terrain data (i.e. bridge situation survey) that's available. Run the model to determine what discharge is required to generate the recorded high water elevation.

Step 2: Using either stream gaging data or hydrographic analysis using actual rainfall data, determine the peak discharge that occurred on the date recorded for the historical high water elevation.

Step 3: If the two discharges match or are close, the existing conditions hydraulic model may be considered calibrated.

If the two discharges don't match (or aren't very close), it will be necessary to either revise the hydraulic model or the hydrologic calculations or both until the two discharges match or are very close. In doing this, extreme care must be taken not to go beyond the realm of reason with either the hydraulic or hydrologic computations. It is always possible that the recorded high water elevation (and/or date) may be in error. If a legitimate calibration can not be achieved, the documentation should fully describe the process leading to the unsuccessful attempt and an explanation offered as to the inability to achieve a calibration.

The hydraulic model used for "calibration" purposes may be either a separate model or may be a part of the "EXISTING CONDITIONS" model. The "PROPOSED CONDITIONS" model should reflect all proposed construction. In this situation, VDOT's policy is to permit up to but not exceeding a one-foot increase in elevation for the 100-year flood event provided the increase doesn't impact upstream development.

12.6.5 Documentation

12.6.5.1 Detailed Hydrologic & Hydraulic Analysis (H & HA) Outline – LD-293D

The first part consists of an outline in which every item shown is to be addressed in its entirety. The outline will be permanently filed as part of the computation assembly. A blank outline is included in Appendix 12B-2. In the case of a lateral floodplain encroachment due to a highway fill embankment, the outline may be adjusted to allow for the fact that a drainage structure is not involved. In such cases it should also be modified or supplemented as necessary to include a tabulation or spreadsheet showing existing and proposed water surface elevations at various locations along the highway embankment. A separate narrative and tabulation may be prepared in lieu of using the outlined in this case.

12.6.5.2 Multipart Letter - LD-293

The second part consists of multi-part letter, officially known as the LD-293 assembly, advising various disciplines within the Department of the results of the hydrologic and hydraulic analysis. Both the outline and the LD-293 assembly are available, upon request, as a series of blank document files in "MICROSOFT WORD" word processing formats. A copy of the entire LD-293 assembly is to be retained with the permanent computation file. Blank copies of the LD-293 assembly are included in Appendices 12B-3 through 12B-5. It should be noted that form LD-293 is only used in the case of a bridge waterway to report the results of the H&HA to the bridge designer. Form LD-293B may additionally be used for a major culvert installation to report pertinent hydraulic design information to the road designer, as a cover letter forwarding form LD-293C (hydraulic commentary necessary for environmental permit applications) to the appropriate District Environmental Manager, and as notification of anticipated hydraulic impacts to the Location & Design Public Involvement Section. A form letter is not available which addresses the hydraulic impacts associated with a lateral encroachment due to a highway fill embankment. However, any necessary changes, modifications, etc. affecting the roadway alignment and/or grade must be coordinated with the road designer.

12.6.5.3 System of Units

The LD-293 assembly will be prepared reflecting exclusively those units employed in the road plans and/or bridge plans whereas, the remainder of the documentation (the actual hydrologic and hydraulic analysis, the outline, etc.) will be left to the discretion of the engineer performing the work. The reason for this is that most of our resources (topographic sheets, mapping, gage records, computational procedures, etc.) are still predicated almost exclusively on English units. It will not be necessary to prepare an LD-293 assembly for H&HA's not associated with a bridged waterway.

12.6.5.4 Level of Precision for Documentation

The following are guidelines governing the level of accuracy to show in the LD-293 assembly:

1. Elevations, etc. obtained from the survey, whether in English or metric, are to be shown exactly as obtained.

2. Elevations, distances, etc. obtained from the plans, whether in English or metric, are to be shown exactly as obtained.

3. The magnitude of peak discharges should be shown to three significant digits in English or metric units. For example, show 12,687 cfs as 12,700 cfs. Show 359.3 cms as 359 cms.

4. Show velocities to the nearest 0.5 (half) fps or 0.1 (tenth) mps.

5. Show calculated water surface elevations to the nearest 0.5 (half) ft. or 0.1 (tenth) m.

6. Show changes in calculated water surface elevations to the nearest 0.5 (half) ft. or 0.1 (tenth) m.

7. Show watershed areas to the nearest sq. mi. or sq. km.

8. There will be occasions where it will be necessary to show a higher level of precision than 0.5 ft. or 0.1 m (e.g. FEMA or other officially delineated floodplains which are

typically shown to the nearest 0.1). If there is any question whatsoever, guidance should be sought from the VDOT Hydraulics Section.

12.6.6 H&HA Submission

When the H&HA has been performed by the consultant for the Department, the following items should be submitted to the Department:

1. The completed H&HA outline, as a document file on diskette and/or as a hard copy printout;

2. The completed LD-293 assembly, as a document file on diskette and as a hard copy printout (including a hardcopy printout or sketch of the anticipated final scoured bed profile for both the design and check flood events) in the event the H&HA was for a bridged waterway.

3. A diskette containing any and all copies of the hydraulic model data on which the H&HA was predicated (i.e. HEC-2, HEC-RAS, etc. data files).

4. Hard copy printouts of all hydraulic model data calculations (i.e. output); Hard copies of full output reports should be printed out for analyses using HEC-2. For HEC-RAS, printouts of Standard Tables 1 and 2 should be submitted. The consultant should contact the Department for guidance when using other models.

5. Copies of any supplemental calculations incidental to the H&HA;

6. Copies of any supplemental documentation not covered in either the H&HA outline or the LD-293 assembly. Any materials and/or resources that have been loaned out by the Department to assist in performing the H&HA such as FEMA studies, etc.

7. If the project crosses or otherwise impacts a FEMA regulatory floodplain or floodway, an excerpt from the FEMA Community Map Panel covering the site should be included.

This information is to be submitted to the VDOT contact person who has been designated as the coordinator for drainage design.

12.7 H&HA for Major Tidal Structures and Bridges

12.7.1 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.

Detailed analysis, as used here, means that for the analysis of bridge crossings of tidal waterways, a three-level analysis approach similar to the approach outlined in HEC-20 and HEC-18 will be employed to assess the impact of the Department's construction and 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.

12.7.2 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, and sufficient roadway plans, profiles, and typical sections to cover the width of the floodplain in the vicinity of the crossing.

Other resources 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 datums.

12.7.3 Coastal Bridge and Culvert Design Techniques

The hydraulic design guidelines for coastal or tidally influenced waterway bridge openings lags behind similar designs on riverine systems. The complicated phenomenon is difficult to simulate for several reasons, but primarily because tidal simulations often require modeling dynamic (time-varying) conditions. 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 costal 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.

The FHWA, in their publication Evaluating Scour at Bridges (HEC-18), presents procedures for performing hydraulic analysis of tidal waterways. HEC-18 procedures are recommended until better or more standardized methods are developed. The FHWA intends to publish HEC-25 from results of the tidal pooled fund research project, which will provide design guidance for tidal hydraulic modeling of bridges.

12.7.4 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) or the U. S. G. S. FHWA WSPRO (HY-7).

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 over estimated 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.

12.7.5 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 Neil'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.

12.7.5.1 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.

12.7.5.2 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.

12.7.5.3 Drainage Area Characterists

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.) Note: copies of this publication are available on request from the department's Hydraulics Section (as a ".PDF" file) until such time as the FHWA releases their upcoming HEC-25 publication.

- 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.

12.7.5.4 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 vs elevation. Since the maximum tidal flow normally occurs at midtide, 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 midtide 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:

$$Q=24312\ A_s \frac{H}{T}$$

where Q = Tidal flow, in cfs

A_s = Surface area of the tidal area upstream from the bridge at the midtide elevation, in sq. mi.

H = Tidal height, between low tide and high tide, in ft.

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.

12.7.5.5 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 midtide 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 feet per second 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

12.7.6 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 feet per second or less.

Where the waterway is constricted and estimated flow velocities exceed 3 feet per second, 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 feet per second or greater, routing of the storm tide through the structure should definitely 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. As noted above, this publication will be available on request from the department's Hydraulics Section until such time as the FHWA's HEC-25 publication becomes available.

VDOT prefers a three-step procedure for performing the hydraulic analysis as described in the instructions for an H&HA of a riverine site (see Section 12.6.4).

12.7.7 Documentation

The documentation for a tidal H&HA will be the same as required for a riverine H&HA (see Section 12.6.5).

12.7.8 H&HA Submission

When the H&HA has been performed for a tidal site by the consultant for the Department, the level of documentation to submit to the Department should be the same as required for a riverine site (see Section 12.6.6).

12.8 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 serves 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. 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). The Department has developed a computer program entitled "BRRIPRAP" which performs all necessary riprap design calculations in accordance with HEC-23. It is available upon request.

12.9 Removal of Existing Bridge and Approach Embankments

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 on Form LD-293B (Appendix 12B-3). 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 percent 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.

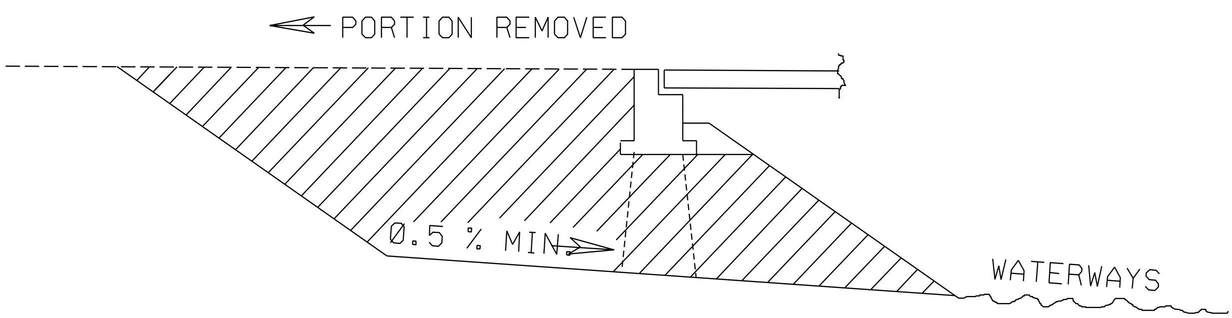

Figure 12-1. Removal of Approach Embankment

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.

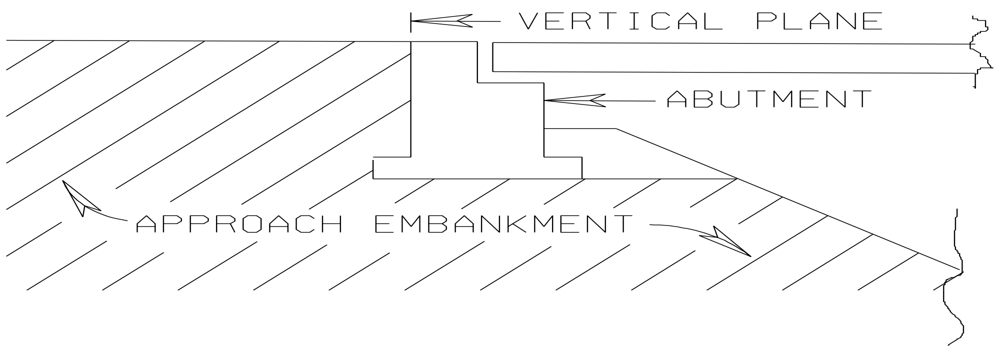

Figure 12-2. Quantifying Removal of Approach Embankment

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.

12.10 Temporary Construction Causeway Design

12.10.1 Background

The need to provide a construction access facility that will not have a significant impact on normal flow conditions has been identified by the Environmental Division.

12.10.2 Causeway Design

12.10.2.1 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 Ordinary High Water stage, will not significantly increase the velocity of flow through the causeway opening(s) for that flood, 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.

12.10.2.2 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 be Standard Class I Dry Riprap.

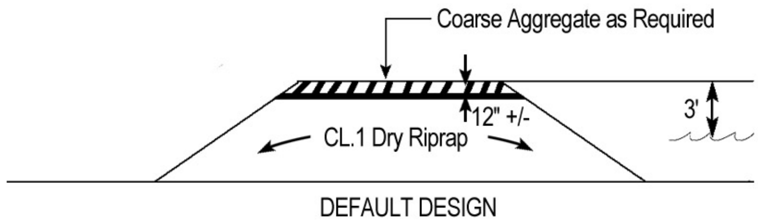

Figure 12-3. Temporary Construction Causeway Design

Show “Ordinary High Water” as the level that the top of the causeway is 3’ over.

12.10.2.3 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.

12.10.3 Design Procedure

Step 1 *Set the alignment of the causeway to facilitate construction activity. Set the finished grade 3'± above the Ordinary High Water elevation. Set the side slope angle at the natural angle of repose (approx. 1½:1).*

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-feet less than the causeway is high. In other words, if the causeway is 6-feet high, then use 48-inch pipe(s).*

12.11 Daily Stream Flow Information

12.11.1 Background

In instances where a VDOT project crosses and/or is in the floodplain of a major waterway, it will be necessary to provide the contractor (or others as appropriate) with a means of determining which times of year would be most suitable for in-stream work (i.e. periods of normally extended low flows) as well as those times when larger or flood flows can be expected. When such information is available, the best source is usually stream gaging information from gage stations which provide daily flow data.

12.11.2 Development of a Composite Stream Flow Hydrograph

To provide the needed information, it will be necessary to plot approximately 10 consecutive Water Years of daily stream flow hydrographs, superimposed one upon the other, for a given stream gage. The department has developed computer software for this purpose. A "Water Year" starts October 1st of the previous year and goes through September 30th of the year under consideration. It is therefore desirable, when generating these plots, to have them start with October of the first Water Year under consideration and end in September of the last (usually 10th) Water Year. It is also desirable to use the most recent 10 consecutive years for which uninterrupted daily flow data is available for the stream gage being employed. Ideally, a stream gage would be used which is located relatively near (either up or downstream) of the project. It may not always be possible or feasible to utilize a stream gage located on the same stream and/or in very close proximity to the project. In such instances it will be acceptable to utilize a gage on another nearby stream, which in the judgment of the hydraulic engineer, can provide more appropriate stream flow information. The most important objective is to provide an indication of those times of year when sustained periods of low flow or high flows can be expected.

After selecting a stream gage, it is highly recommended that the gaging records be reviewed prior to utilizing the plotting software to insure that the gage is of the recording type (i.e. that daily stream flow records are available) and to determine the most recent 10 consecutive years for which uninterrupted data is available. The usual references for this information are the U.S. Geological Survey's annual publications entitled WATER RESOURCES DATA VIRGINIA, VOLUME 1, SURFACE-WATER-DISCHARGE AND SURFACE-WATER-QUALITY RECORDS (for each Water Year under consideration) and their Internet web site which is entitled "NWISWeb Data for Virginia", the "URL" for which is http://waterdata.usgs.gov\va\nwis\.

The software necessary to generate these plots – COMPOSITE HYDROGRAPH – is located on the Central Office Location & Design Division's 0501COLND file server. Access to the software will normally be granted to any VDOT personnel involved in drainage design and is an integral part of the department's "Hydraulic

Engr. Package" of software. Consultants needing these hydrographs must currently request them from the designated drainage design coordinator. Permission for access to the software must be requested of the Central Office Location & Design Division's AES Manager but shall not be granted without the approval of the State Hydraulic Engineer.

The software's database contains daily stream flow records for all recording stream gages in the state of Virginia. This data will, for gages currently in operation, be available up through the most recent Water Year for which data has been published. The software can, at the user's option, generate the hydrograph either as a ".BMP" file saved to disk or as a letter size hard-copy printout. The ".BMP" file should be made available to the Road Designer so he can import it into MicroStation and convert into a plan sheet for inclusion in the plan assembly. Probably the quickest and most convenient way to do this will be to attach the file to the cover e-memo used to transmit the usual "LD-293B" memorandum (in the case of a bridged waterway) to the Road Designer. If no bridged waterway is involved, as would be the case when the floodplain involvement is by virtue of a major culvert or roadway encroachment, the file should be generated and transmitted at the conclusion of the hydrologic & hydraulic analysis. An example daily stream flow composite hydrograph plot is included in the Appendix 12-E1, "Example Daily Stream Flow Information".

12.12 References

The Federal Highway Administration Hydraulic Engineering Circular No. 17, "The Design of Encroachments on Flood Plains Using Risk Analysis" – October 1980.

Survey Instructions Manual – Virginia Department of Highways & Transportation.

Hydraulics of Bridge Waterways, Federal Highway Administration – 1970.

HEC-2 Water Surface Profiles – U. S. Army Corps of Engineers.

Highways in the River Environment – Hydraulic and Environmental Design Considerations – Federal Highway Administration – 1975.

Appendix 12A-1 Definitions and Abbreviations

Definitions:

Bridges are defined as:

- Structures that transport traffic over waterways or other obstructions,
- Part of a stream crossing system that includes the approach roadway over the floodplain, relief openings, and the bridge structure and
- Structures with a centerline span of 20 feet or more. However, structures designed hydraulically as bridges, as described above, are considered bridges in this chapter, regardless of length.

Abbreviations:

AASHTO	American Association of State Highway and Transportation Officials
DCR	Department of Conservation and Recreation
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
HDS	Hydraulic Design Series
HEC	Hydraulic Engineering Circular
LTEC	Least Total Expected Cost
NFIP	National Flood Insurance Program
NHS	National Highway System
NOAA	National Oceanic and Atmospheric Administration
NRCS	National Resources Conservation Service
TR	Technical Release
USCOE/USACE	United States Corps of Engineers
USGS	United States Geological Survey
VDOT	Virginia Department of Transportation

Appendix 12B-1 Hydrologic and Hydraulic Analysis Report Distribution List

Distribution of LD-293 Assembly and Associated Documentation

Note: Where possible and practicable, it is desirable to distribute this documentation electronically as an e-mail attachment.

Bridge designer who requested the analysis (district or Central Office as appropriate)
- LD-293 (including design & check scoured bed profile sketches as appropriate)
- LD-293C

Central Office Hydraulics Section – Asst. State Hydraulics Engineer for Bridge Hydraulics & River Mechanics Engineering
- LD293 (including design & check scoured bed profile sketches as appropriate)
- LD293B
- LD293C

Central Office Structure & Bridge Division
Assistant Structure & Bridge Engineer – District Coordination (if a district job other than a secondary)
- LD293 (including design & check scoured bed profile sketches as appropriate)
- LD293C

District River Mechanics Engineer (DRME) or District Drainage Engineer (DDE) as appropriate unless the H&HA was performed by the DRME or DDE
- LD293 (including design & check scoured bed profile sketches as appropriate)
- LD293B
- LD293C
- H&HA Outline

Road designer responsible for the associated roadway project (or the District L&D Engineer in the district in which the project is located if there is no associated road project)
- LD293B
- Hydrologic Data Sheet

District Environmental Manager
- LD293B
- LD293C
- Excerpt from the FEMA Flood Map covering the crossing site if applicable (Note that this can be done electronically by cropping and saving a portion of the FEMA map that brackets the crossing site using our FEMA FMT 2000 computer software.)

Central Office Public Involvement Section
Administrator – Public Involvement Section
- LD293B

Appendix 12B-2 LD-293D Hydrologic and Hydraulic Analysis Documentation Outline

DATE:		ENGINEER:		
HYDROLOGIC & HYDRAULIC ANALYSIS OUTLINE				
ROUTE:		PROJ. #:		
CITY/COUNTY:	STREAM NAME:			
DRAIN. AREA:	STATION:	LAT:		LONG:

EX #	**REFERENCE DATA**
	MAPS:
	PHOTOS:
	OTHER:
	APPLICABLE FLOOD PLAIN MANAGEMENT:
	STUDIES BY EXTERNAL AGENCIES:
	STUDIES BY INTERNAL SOURCES:
	GAGING DATA AVAILABLE:
	AVAILABLE SURVEY DATA:
	TECH. AIDES & FILE NAMES:
	OTHER DATA:

REMARKS:
Add any relevant comments concerning the data obtained and its quality (particularly if it is questionable).

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Appendix 12B-2 LD-293D Hydrologic and Hydraulic Analysis Documentation Outline

HYDROLOGY	
	METHODS USED FOR DISCHARGES:
	REASONS FOR FINAL SELECTION OF DISCHARGE VALUES:
	INFLUENCE AND CONTROL OF SITE:
	HIGH WATER ELEV: DATE & SOURCE:**
** See documentation data at the end of form for approximate discharge and frequency of event:	
REMARKS:	

STREAM STABILITY – LEVEL 1: QUALITATIVE ANALYSIS PER HEC-20	
	BRIDGE CHARACTERISTICS:
	STREAM CHARACTERISTICS:
	LAND USE CHANGES:
	OVERALL STABILITY:
	LATERIAL STABILITY:
	VERTICAL STABILITY:
	STREAM RESPONSE:
BASED UPON THE ABOVE ANALYSIS, IS A MORE DETAILED ANALYSIS NECESSARY: YES: NO:	
IF YES, WHAT LEVEL:	SEE EXHIBIT #:

Appendix 12B-2 LD-293D Hydrologic and Hydraulic Analysis Documentation Outline

REMARKS:
Please complete with general comments based on observations of the conditions at the site.

Appendix 12B-2 LD-293D Hydrologic and Hydraulic Analysis Documentation Outline

EX #	HYDRAULIC ANALYSIS OF EXISTING STRUCTURE		
	Computer Model:	FILE:	Plan:
	DESCRIPTION OF EXISTING STRUCTURE:		
	SPAN LENGTH:	PARAPETS:	
	ABUTMENT TYPE:	SKEW TO CL:_	TO FLOOD FLOWS:
	NO. OF PIERS & TYPE:		
	PIER WIDTH:	TOTAL PIER AREA:	
	ABUTMENT “A” STA:	FINISH GRADE ELEV:	
	ABUTMENT “B” STA:	FINISH GRADE ELEV:	
	ELLC ELEV:	ELLC FOR PRESSURE FLOW:	
	STREAM BED ELEVATION:		
	WEIR ELEV. ON EACH SIDE OF STRUCTURE:	LEFT:	RIGHT:
	EXPANSION COEF:	CONTRACTION COEF:	
	ENERGY S_O	“n” VALUES:	
	BRIDGE MODELING APPROACH:		
	REASON FOR SELECTION:		
	HIGH FLOW METHOD:		
	REASON FOR SELECTION:		

DISCHARGE	EST. EXC.	WSP ELEV. AT COMMON UPSTREAM SECTION SECNO #:	WSP ELEV. AT UPSTREAM FACE OF BRIDGE SECNO #:	VEL. AT DWNSTREAM FACE OF STRUCTURE SECNO #:
(cfs)	(%)	(ft)	(ft)	(fps)
	50			
	20			
	10			
	4			
	2			
	1-N			
	1-FW			
	0.2			
	OHW			
	H.W. EVENT			

EX.#	EVENT	STAGE ELEV.	DISCHARGE	EXC. PROB. (%)
	H.W Flood			
	Base Flood			
	Overtopping Flood			

REMARKS:

Comment on the modeling approach and correction or observations relative to the original analysis.

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DOCUMENTATION OF STEPS TAKEN TO CALIBRATE MODEL
If there is difficulty in calibrating the model to a historical event contact VDOT to see if there is additional information available regarding that particular event.

EX #	**HYDRAULIC ANALYSIS OF PROPOSED STRUCTURE**	
SCHEME #:		
	Computer Model: FILE: Plan:	
	DESCRIPTION OF PROPOSED STRUCTURE:	
	SPAN LENGTH:	PARAPETS:
	ABUTMENT TYPE:	SKEW TO CL:_ TO FLOOD FLOWS:
	NO. OF PIERS & TYPE:	
	PIER WIDTH:	TOTAL PIER AREA:
	ABUTMENT “A” STA:	FINISH GRADE ELEV:
	ABUTMENT “B” STA:	FINISH GRADE ELEV:
	ELLC ELEV:	ELLC FOR PRESSURE FLOW:
	STREAM BED ELEVATION:	
	WEIR ELEV. ON EACH SIDE OF STRUCTURE: LEFT: RIGHT:	
	EXPANSION COEF:	CONTRACTION COEF:
	ENERGY S_O	“n” VALUES:
	BRIDGE MODELING APPROACH:	
	REASON FOR SELECTION:	
	HIGH FLOW METHOD:	
	RESAON FOR SELECTION:	

DISCHARGE	EST. EXC. (%)	Diff. At Common SECNO	WSP ELEV. AT COMMON UPSTREAM SECTION SECNO #:	WSP ELEV. AT UPSTREAM FACE OF BRIDGE SECNO #:	VEL. AT DWNSTREAM FACE OF STRUCTURE SECNO #:
(cfs)	(%)	(ft)	(ft)	(ft)	(fps)
	50				
	20				
	10				
	4				
	2				
	1-N				
	1-FW				
	0.2				
	OHW				
	H.W. EVENT				

EX.#	EVENT	STAGE ELEV.	DISCHARGE	EXC. PROB. (%)
	Design Flood			
	Overtopping Flood			
	Base Flood			

REMARKS:

Appendix 12B-2 LD-293D Hydrologic and Hydraulic Analysis Documentation Outline

DOCUMENTATION OF STEPS TAKEN FOR PROPOSED MODEL - SCHEME #:
Comment on modification to existing conditions model to develop the proposed model

Appendix 12B-2 LD-293D Hydrologic and Hydraulic Analysis Documentation Outline

EX #	**HYDRAULIC ANALYSIS OF PROPOSED STRUCTURE**	
	SCHEME #:	
	Computer Model: FILE: Plan:	
	DESCRIPTION OF PROPOSED STRUCTURE:	
	SPAN LENGTH:	PARAPETS:
	ABUTMENT TYPE:	SKEW TO CL:_ TO FLOOD FLOWS:
	NO. OF PIERS & TYPE:	
	PIER WIDTH:	TOTAL PIER AREA:
	ABUTMENT “A” STA:	FINISH GRADE ELEV:
	ABUTMENT “B” STA:	FINISH GRADE ELEV:
	ELLC ELEV:	ELLC FOR PRESSURE FLOW:
	STREAM BED ELEVATION:	
	WEIR ELEV. ON EACH SIDE OF STRUCTURE: LEFT: RIGHT:	
	EXPANSION COEF:	CONTRACTION COEF:
	ENERGY S_O	“n” VALUES:
	BRIDGE MODELING APPROACH:	
	REASON FOR SELECTION:	
	HIGH FLOW METHOD:	
	REASON FOR SELECTION:	

DISCHARGE	EST. EXC. (%)	Diff. At Common SECNO	WSP ELEV. AT COMMON UPSTREAM SECTION SECNO #:	WSP ELEV. AT UPSTREAM FACE OF BRIDGE SECNO #:	VEL. AT DWNSTREAM FACE OF STRUCTURE SECNO #:
(cfs)	(%)	(ft)	(ft)	(ft)	(fps)
	50				
	20				
	10				
	4				
	2				
	1-N				
	1-FW				
	0.2				
	OHW				
	H.W. EVENT				

EX.#	EVENT	STAGE ELEV.	DISCHARGE	EXC. PROB. (%)
	Design Flood			
	Overtopping Flood			
	Base Flood			

REMARKS:

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Appendix 12B-2 LD-293D Hydrologic and Hydraulic Analysis Documentation Outline

DOCUMENTATION OF STEPS TAKEN FOR PROPOSED MODEL - SCHEME #:
Comment on modification to existing conditions model to develop the proposed model

EX #:	**SCOUR DATA**
	SCOUR POTENTIAL: SEE EX. # FOR COMPUTATIONS AND PLOT SUMMARY OF RESULTS:
	RIPRAP RECOMMENDATIONS: IF DESIRED, CLASS , D= , OVER FILTER CLOTH BEDDING WILL BE SATISFACTORY.

HISTORICAL RETURN PERIOD
The approximate frequency of the event that caused the highwater at the existing structure is the____Year or the _____% Exc. Event.

CAUSEWAY ANALYSIS RESULTS
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 Unit for analysis and documentation. Temporary construction access causeways for this project should be composed of: Armor layering will/will not be required on either side.. The ordinary highwater will be increased by___ft. The high flow profiles will not be affected. The causeway will not affect the water surface profile. The maximum causeway elevation is ____ft. From abutment A station________to station________. From abutment B station________ to station________. Only one will be in place at a time.
SUMMARY
Make a brief summary statement about the impact of the proposed bridge on the flooding.

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Appendix 12B-3 LD-293 Hydraulic Analysis Report

HYDROLOGIC AND HYDRAULIC ANALYSIS REPORT

LOCATION
Project :
Route :
County/City :
Waterway :

PREPARED BY
Name :
Organization :
Date :

STRUCTURE DESCRIPTION

Abutment A Station: Finished Grade Elevation_________ ft. (m)
Abutment B Station: Finished Grade Elevation_________ ft. (m)
Minimum Low Chord Elevation_______ ft. (m)
Skew_______ to centerline________ to flood flow
Span Length
Abutment Type
Number/Type Piers

HYDROLOGIC/HYDRAULIC DATA

Drainage Area _____________ Sq. Mi. (km^2)

HISTORICAL DATA

High Water Elevation ________ ft. (m) Date of Occurrence
Estimated Discharge __________ cfs. (m^3/s)
Estimated Exceedence Probability________ %

HYDRAULIC PERFORMANCE

The data presented herein is the result of statistical analysis and indicates an approximate estimate of the performance of this facility.

Discharge (cfs)(m^3/s)	Estimated Exceedence Probability (%)	Change in existing flood levels (ft.)(m)	Flood stage upstream of bridge (ft.)(m)	Velocity thru Bridge Structure (ft/s)(m/s)
	50%			
	20%			
	10%			
	4%			
	2%			
	1% Natural			
	1% Floodway			
	0.2%			

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Appendix 12B-3 LD-293 Hydraulic Analysis Report

DESIGN SUMMARY

	Exceedence Probability (%)	Stage Elevation (ft.)(m)	
Design Flood			
Overtopping Flood			
Base Flood			
Ordinary High Water			

DEBRIS POTENTIAL

ABUTMENT SLOPE PROTECTION RECOMMENDATIONS
26" Class I Dry Riprap over 4" no. 25 or 26 aggregate over filter cloth will be hydraulically satisfactory.
650 mm Class I Dry Riprap over 100 mm no. 25 or 26 aggregate over filter cloth will be hydraulically satisfactory.
950 mm Class II Dry Riprap over 150 mm no. 25 or 26 aggregate over filter cloth will be hydraulically satisfactory.
38" Class II Dry Riprap over 6" no. 25 or 26 aggregate over filter cloth will be hydraulically satisfactory.

SCOUR PLOTS
A sketch of the final scoured bed profile and the check scoured bed profile is attached. If scour countermeasures are required, a request must be submitted to the Hydraulics Unit for their design and documentation.

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 Unit for analysis and documentation.

Temporary construction access causeways for this project should be composed of <<specify>>.
The ordinary highwater will be increased by _______ ft. (m)
The high flow profiles will not be affected.
The causeway will not affect the water surface profile.
The maximum causeway elevation is __________ ft. (m)
From abutment A to station
From station _________ to abutment B.
Only one will be in place at a time.

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.

COMMENTS
Note any channel modifications, flood plain 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 structures(s) and approaches described. Any changes in these conditions may invalidate this analysis and should be reviewed by this office.

This design represents the smallest structure practicable for use at this site.

Appendix 12B-3 LD-293 Hydraulic Analysis Report

The existing structure and the existing approach roadways from station: to station: are to be removed and the land is to be regraded to its natural contour.

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:

If you have any questions or need additional information, please contact _____________ at _______________ or via electronic mail at __________________. The completed **"CONFIRMATION OF DESIGN"** should also be sent to _____________________________.

Appendix 12B-3 LD-293 Hydraulic Analysis Report

HYDROLOGIC DATA SHEET
The information presented hereon is to be transcribed to the Hydrologic Data sheet contained in the plan assembly.

LOCATION
Project :
Route :
County/City :
Waterway :

DESCRIPTION
Sheet No Station

Drainage Area ____________ sq. mi (km^2)

Structure Size

BASE FLOOD
Discharge______________ cfs (m^3/s)

Stage Elevation __________ft. (m)

DESIGN FLOOD
Discharge _______________cfs (m^3/s)

Estimated Exceedence Probability_________ %

Stage Elevation_______________ ft. (m)

OVERTOPPING FLOOD
Stage Elevation_______________ ft. (m)

Estimated Exceedence Probability________ %

HISTORICAL DATA
Date

Stage Elevation ________________ ft. (m)

Estimated Exceedence Probability___________ %

REMARKS

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Appendix 12B-3 LD-293 Hydraulic Analysis Report

CONFIRMATION OF DESIGN

The bridge designer will complete this form and forward it to the Hydraulics Unit confirming that the design that was analyzed is being used.

LOCATION
Project :
Route :
County/City :
Waterway :

STRUCTURE DESCRIPTION

Abutment A Station: ______________Finished Grade Elevation __________ft. (m)

Abutment B Station: ______________Finished Grade Elevation __________ft. (m)

Minimum Low Chord Elevation ________ft. (m)

Skew ________to centerline ________ to flood flow

Span Length

Abutment Type

Number/Type Piers

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Appendix 12B-4 LD-293B Report to VDOT Road Designer

ROAD DESIGN NOTIFICATION OF HYDRAULIC ANALYSIS

LOCATION
Route :
Project :
County/City :
Waterway Name :

PREPARED BY
Name :
Organization :
Date :

HYDRAULIC DATA

The Hydrologic and Hydraulic Analysis has been completed for this site and the report has been furnished to the Bridge Designer. No recommendations were made that would affect the road plans. The following recommendations were made that may affect the road plans:

The estimated ordinary high water elevation is ft.(m)

ABUTMENT SLOPE PROTECTION RECOMMENDATIONS

26" Class I Dry Riprap over 4" no. 25 or 26 aggregate over Filter Cloth will be hydraulically satisfactory.
650 mm Class I Dry Riprap over 100 mm no. 25 or 26 aggregate over Filter Cloth will be hydraulically satisfactory.
38" Class II Dry Riprap over 6" no. 25 or 26 aggregate over Filter Cloth will be hydraulically satisfactory.
950 mm Class II Dry Riprap over 150 mm no. 25 or 26 aggregate with Filter Cloth will be hydraulically satisfactory.

REMARKS
The existing approach roadway embankments from station to station are to be removed and the land is to be regraded to its natural contour. The work will be included in the quantity for regular excavation.

This project will not exert a significant flood plain impact.

Appendix 12B-5 LD-293C Report to VDOT District Environmental Manager

LOCATION
Project :
Route :
County/City :
Waterway :

PREPARED BY
Name :
Organization :
Date :

ENVIRONMENTAL DATA

1. Identify involvement within the base flood plain:
   This is a skewed crossing of <<stream name >>.
   This is a perpendicular crossing of <<stream name >>
   The existing bridge will be removed.
   The existing bridge will remain in place.

2. Traffic service: ADT
   Detours available ________ Length _______ Miles. (km)
   Frequency of overtopping Flood
   Potential damage to the highway facility

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.

> 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.

For project permits in a FEMA floodplain carrying a **Zone A** 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. 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.

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. Note social, economic, ecological and human use of the flood plain:

5. Drainage area _______sq. mi. (km^2)

6. Overtopping flood

Discharge = ________ cfs (m^3/s)
Exceedence Probability ____ %
Stage _______ ft. (m)

7. Compare the hydraulic performance of the proposed action to the hydraulic performance of the existing conditions in terms of:

There will be no change in the flood levels or velocities.
The flood flow characteristics will not change.
This proposed bridge will replace an existing bridge.
There will be no increase in the level of the 1% flood.
There will be an increase of one foot or less for the 1% flood.
This is a proposed bridge in a new location.

HYDRAULIC COMMENTARY FOR PERMIT

HYDROLOGY

The hydrologic analysis for this project was predicated on Flood Insurance Data for <<county/city name>> county.
The hydrologic analysis for this project was predicated on data obtained by VDOT personnel.

Design Discharge _____cfs (m^3/s) 1% Discharge _____ cfs (m^3/s)

HISTORICAL DATA

Highwater elevations were obtained by field reconnaissance and were correlated with the hydraulic data.

Highwater Elevation _______ ft (m) _____ Date

HYDRAULIC

The hydraulic analysis was performed using FHWA water surface profile computer model WSPRO.
The hydraulic analysis was performed using USACE water surface profile computer model (HEC-2/HEC-RAS).
The hydraulic analysis was performed using accepted principals and techniques of river mechanics applicable to this site.
The proposed facility will not increase the 1% Flood Stage.
The proposed facility will not increase the 1% Flood Stage by more than 1.0 foot.

Design Flood Stage Elevation ______________ ft. (m)

1% Flood Stage Elevation _________________ ft. (m)

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 Unit for analysis and documentation.

Temporary construction access causeways for this project should be composed of <<specify>>.
The ordinary highwater will be increased by _____ft. (m)
The high flow profiles will not be affected.
The causeway will not affect the water surface profile.
The maximum causeway elevation is ____________ft. (m)
From abutment A to station
From station ________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 the Erosion and Sediment Control Law, the Erosion and sediment control Regulations, and the annual erosion and sediment control Standards and Specifications approved by the Department of Conservation and Recreation.

STORMWATER MANAGEMENT

Design of this project will be in compliance with the Stormwater Management Act, the Stormwater Management Regulations, and the annual stormwater management Standards and Specifications approved by the Department of Conservation and Recreation.

Appendix 12C-1 LD-23 Structure and Bridge Data Sheet

COMMONWEALTH OF 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 from Transportation Department County Map _________
____
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

1. EXISTING STRUCTURE

Existing structure is any structure at, upstream, or downstream from the proposed site have comparable drainage area.
Date of original construction:____
Was present bridge in place at time of extreme high water? ___________________________________
Has bridge ever been washed out?__________Date___________Mo.__________ Yr.__________
Explain what portion of bridge or approaches have been washed out: ___________________________
Elevation of maximum high water:
Upstream side of existing structure____
Downstream side of existing structure ____
________Ft. upstream of existing structure ____
________Ft. downstream of existing structure ____
At other locations on the flood plain (describe)
____
____
Date of maximum high water: ______Mo.______Yr._______Source of information________________

2. STREAM FLOW DATA AT PROPOSED SITE

Elevation of maximum high water of this stream at proposed location if different from data for existing site:
______Ft. on upstream side of Proposed____
______Ft. on downstream side of proposed____
At other locations on the floodplain (describe)____
Date:______Mo.______Yr.______Source of information____
____
Elevations of highest backwater caused by another stream_______________________________________
Date_____________________ Stream name____
Source of information____
Elev. of normal water:_______ (Average) Elev. of extreme low water ___________________________
Date:______________Mo.____________Yr.____________
Source of information____
Velocity of current at high water:_________ft./sec. Velocity of current at normal water___________ft./sec.

3. SITE CONDITIONS

Amount and character of drift during a freshet or flood: ____
____
Amount and character of ice:____
Do banks or bed show scour?_ ____
Description and location of scour:____

Appendix 12C-1 LD-23 Structure and Bridge Data Sheet

____
Bed of stream consists mainly of: mud, silt, clay, sand, gravel, cobbles, boulders, soft solid rock, stratified rock, hard rock, silt sedimentation, deposition of large stones, is this material loose or well compacted:

Comments on stream ecology and wild life habitat: ______________________________________

4. INFLUENCE & CONTROL OF SITE

Location and condition of dams upstream or downstream that will affect high water or discharge at this site:
____
____
Location and description of any water-gaging stations in the immediate vicinity: ______________________
____
____
Elevation________________ on gage corresponds to elev.______________________________________
____________________on survey datum.
Extent to which sinkholes affect runoff, etc.:____
____
Brief description of usage of stream for navigational purposes. By small boats, etc_________________
____
____
____

Railroad Grade Separation Structure Site Data

Railroad milepost_________________ No. of tracks____
Situation data for design of bridge on ________________ over___________________________________
Type of construction: ___________________________ New structure
___________________________ Replacement of existing structure
___________________________ Remodeling of existing structure
___________________________ Paralleling existing structure

Owner of existing structure ____

Owner of grade crossing to be eliminated____

Date of original construction of any railroad structure being replaced or within approximately 500 feet of the site of a proposed overpass____

Conditions of existing cut slopes, whether stable, eroded, et cetera_________________________________
______________________. Are ditches open, maintained, et cetera_________________________________
NOTE: Show cross-section of existing railroad is at right angles to centerline crossing, with all dimensions, on bridge situation plan. This cross-section should extend from top of cut to toe of fill.

REMARKS

____
____
____
____

Appendix 12C-2 Tidal Bridge Scour Data and Worksheet

VIRGINIA DEPARTMENT OF TRANSPORTATION
TIDAL BRIDGE SCOUR DATA & WORKSHEET

Hydraulic Engineer:_________
Date:_________

I. BRIDGE LOCATION

BRIDGE No. _________ Route: _________ County No. _________
Length: _________ Ft. River: _____________________

TIDAL BRIDGE CATEGORY: Islands Semi-Enclosed Bays & Inlets Estuary

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.
Note: Mean sea level (MSL), mean low water (MLW), mean tide level (MTL)
Skew Angle (Centerline of Bridge with Channel) Phi (ϕ) = _________° (Degrees)

II. DRAINAGE AREA CHARACTERISTICS

(Information per USGS Report 94-4148 for Virginia Department of Transportation dated 1995)
Drainage Area: ___Sq. Mi.; Forest: F = ____%; Average basin elevation: EL=_____Ft.
Main Channel Slope: SI=_____Ft/Mi; Main Channel length: L=_____Mi.
Peak Discharge Region Used:______________________________

Compute from USGS Regression Equation:
Q_{r100} = _________________CFS; $Q_{r500} = 1.7 (Q_{r100})$ = _________________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: As = _________Sq. Mi.
at _____Ft.: As = _________Sq. Mi.
at _____Ft.: As = _________Sq. Mi.

Compute Tidal Flows:
Q_{t100} = _________________CFS; $Q_{t500} = 1.7 (Q_{r100})$ = _________________CFS

IV. FLOW VELOCITY

a) Based on Cross Sectional Area at MSL/MLW
Cross Sectional Area, $A_1 = W_oD$ = _________________Ft^2
$V_{r100} = Q_{r100}/A_1$ = _________Ft/S $V_{r500} = Q_{r500}/A_1$ = _________Ft/S

b) Based on Cross Sectional Area at Midtide Elevation
$V_{t100} = (Q_{t100} + Q_{t100}) / (A_1 + W_oH_{100}/2)$ = _________ Ft/S
$V_{t500} = (Q_{t500} + Q_{t500}) / (A_1 + W_oH_{500}/2)$ = Ft/S

c) Based on Manning Equation (n = 0.025; s = 0.0005)
$V_{t100} = 1.2 ((Q_{r100} = Q_{t100}) / W_o)^{0.4}$ = _________ Ft/S
$V_{t500} = 1.2 ((Q_{r500} = Q_{t500}) / W_o)^{0.4}$ = _________ Ft/S

Attach a Sketch of Cross-Section at Upstream (U/S) Side of Bridge

Appendix 12C-3 Table of Storm Tide Description of Virginia Coast

VIRGINIA DEPARTMENT OF TRANSPORTATION
Table of Storm Tide Description of Virginia Coast

DERIVED CHARACTERISTICS	STORM TIDE DESCRIPTION			
	500-Year (P-0.002)	100-Year (P-0.01)	50-Year (P-0.075)	10- Year (P-0.1)
Suitable R/V Probability	p-0.50	p-0.12	p-0.075	p-0.05
Part A – General Values for Virginia Tidal Waters				
Correspondence R/V Value	2 hr	1 hr	0.8 hr	0.7 hr
Tide Duration, D – 10 R/V	20 hr	10 hr	8 hr	7 hr
Part B – Specific Example for Hampton Roads				
Storm Tide Elevation, E	11.2 ft	8.8 ft	7.8 ft	5.8 ft
E/D, ft/hr	0.56	0.88	0.97	0.83

Appendix 12C-4 Virginia Coastal Maps Showing Predicted Water Surface Elevations

VIRGINIA DEPARTMENT OF TRANSPORTATION
Maps of the Costal Regions of Virginia

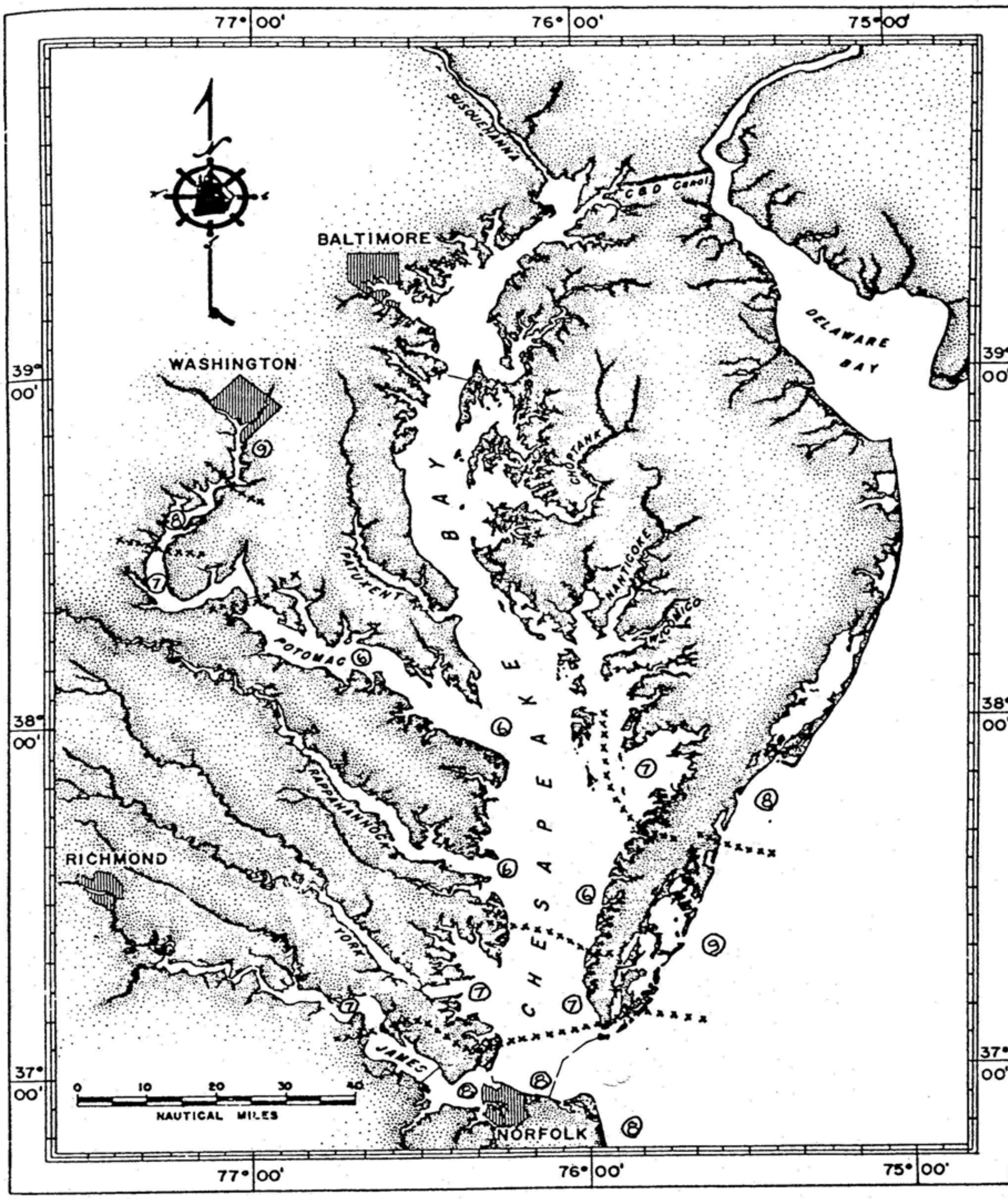


FIGURE 24A. ESTIMATED TIDAL FLOOD ELEVATIONS FOR 50-YEAR EVENT.

Appendix 12C-4 Virginia Coastal Maps Showing Predicted Water Surface Elevations

VIRGINIA DEPARTMENT OF TRANSPORTATION
Maps of the Costal Regions of Virginia

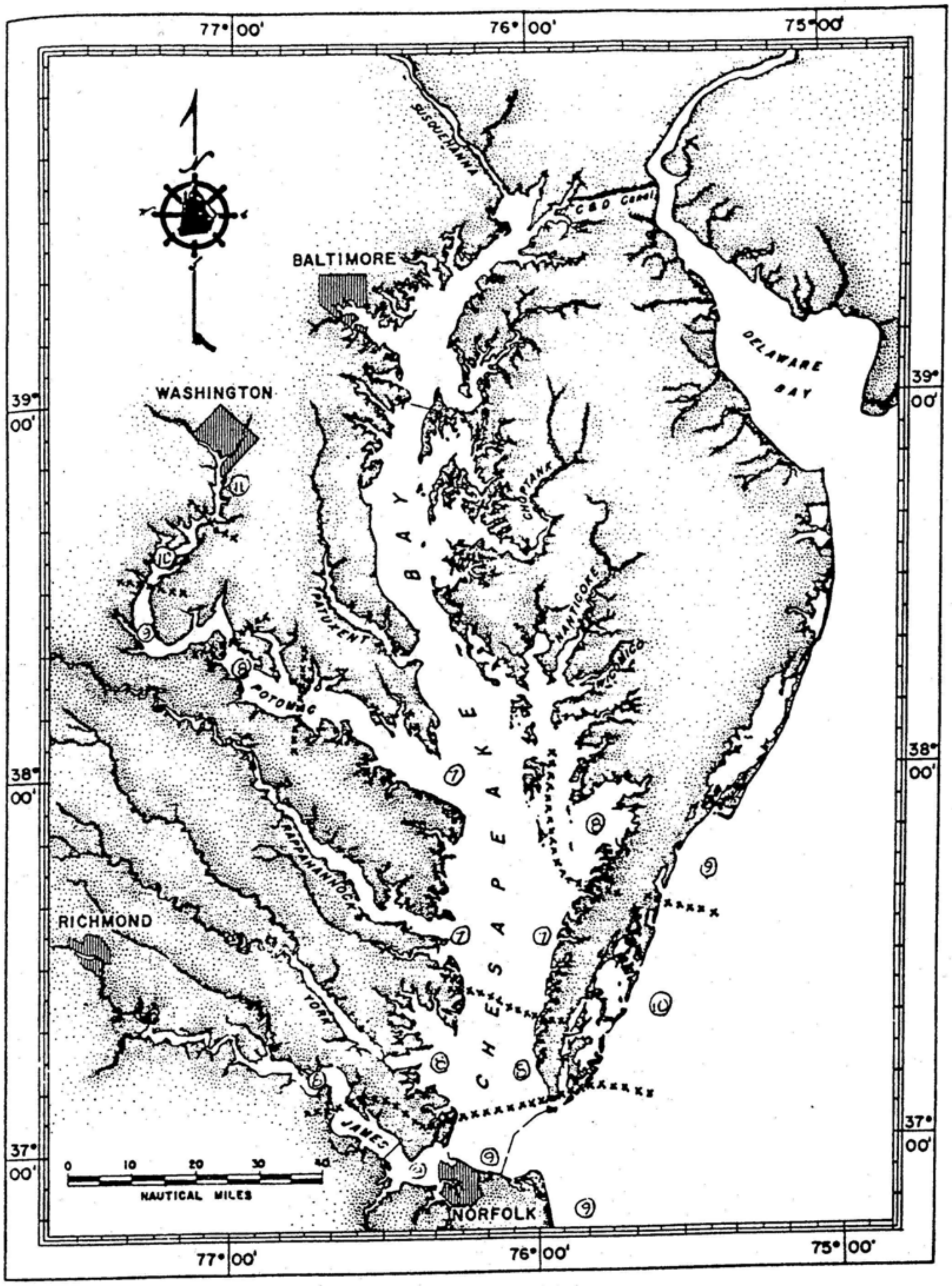


FIGURE 0. ESTIMATED TIDAL FLOOD ELEVATIONS FOR 100-YEAR EVENT, IN FEET ABOVE NGVD. 10 HOURS FOR FLOOD RISE AND FALL IS APPROPRIATE TO HURRICANE PASSAGES CAUSING THIS EVENT.

Appendix 12C-4 Virginia Coastal Maps Showing Predicted Water Surface Elevations

VIRGINIA DEPARTMENT OF TRANSPORTATION
Maps of the Costal Regions of Virginia

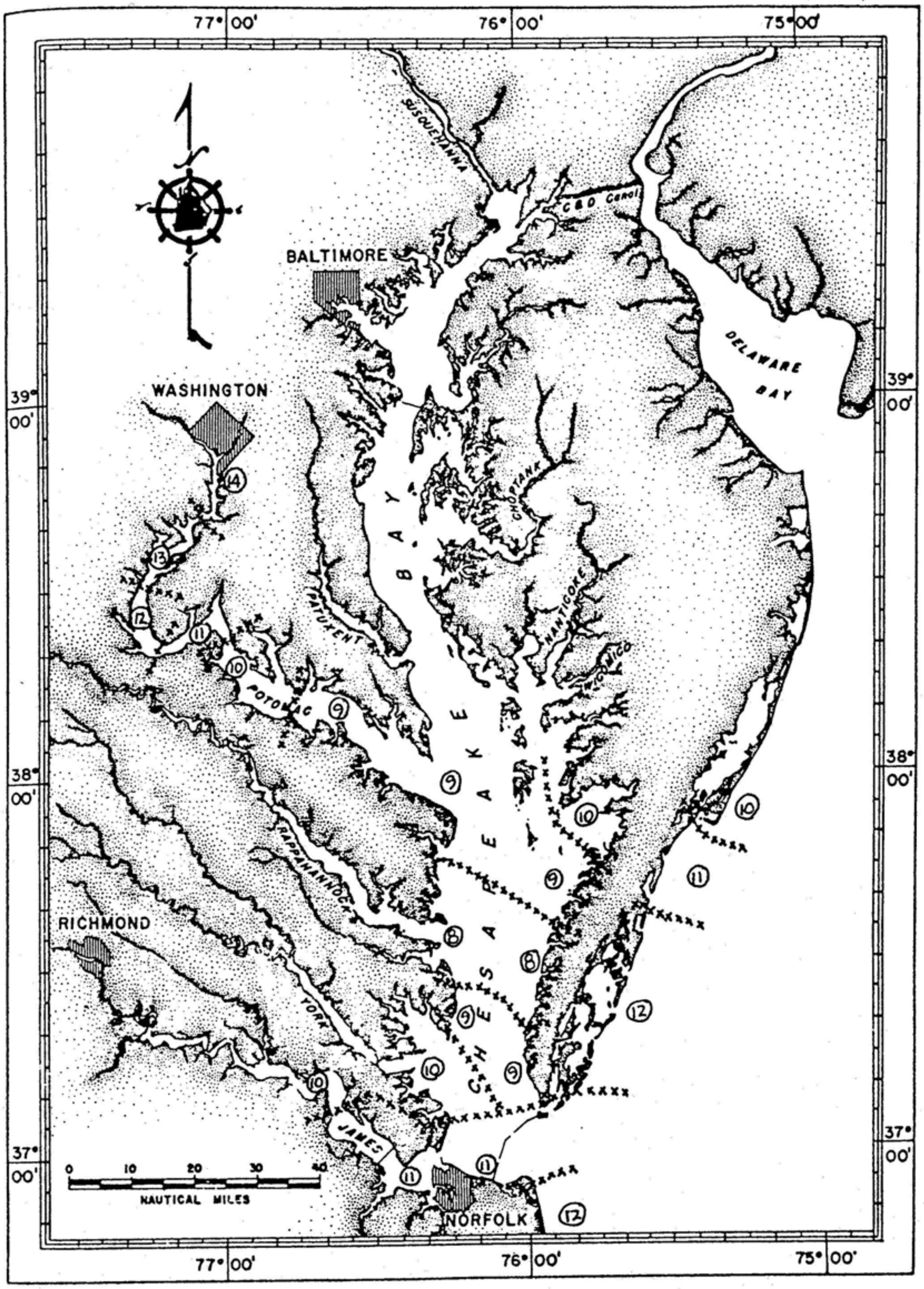


FIGURE 24C. ESTIMATED TIDAL FLOOD ELEVATIONS FOR 500-YEAR EVENT.

Appendix 12D-1 Guidelines for the Determination, Documentation, and Processing of Hydraulically Equivalent Replacement Structures (HERS)

12D-1.1 Background

This is to update the guidelines presented in memoranda from Mr. C.F. Boles, III on November 9, 1987 and November 18, 1987 for Hydraulically Equivalent Replacement Structures (HERS) and includes the supplemental scour evaluation recommendations presented in an e-memo by Mr. D.M. LeGrande on August 15, 2000. The purpose of Mr. Boles' original memoranda was to establish guidelines for the determination, documentation, and processing of proposed HERS facilities.

12D-1.2 Definition

A HERS determination is only applicable to the replacement of a culvert or bridge, either the complete structure or portions thereof, with a hydraulically identical culvert or bridge. It is not intended to be applied to the replacement of a bridge with a culvert or the replacement of a culvert with a bridge. The waterway opening of the proposed structure must provide the same height and width as the existing facility. An exception would be instances where existing bridge piers were being removed and not replaced. The proposed roadway grades on the approaches and parapet walls on the proposed bridge superstructure must be such that the same flood overtopping characteristics prevail as would be experienced by the existing facility. It should be noted that a larger proposed facility, though it may be more hydraulically efficient than the existing facility, would not necessarily qualify for a HERS determination as the scour characteristics and potential would normally be different from that of the existing facility. If it can be determined, via a field review by the structural engineer, the geotechnical engineer, and the river mechanics engineer, that a suitable foundation is present (i.e. rock of good quality is at or near the surface), a formal H&HA would not be needed. Whatever the decision, we need to be ever mindful of the potential consequences of not performing a formal H&HA, be they litigation or failure of the structure or both.

12D-1.3 Authorization

A HERS determination can only be made by one of the Department's River Mechanics Engineers or, if a River Mechanics Engineer is unavailable, by one of the Department's Hydraulic Engineers (including the District Drainage Engineers), provided such Hydraulic Engineer has been properly trained and is experienced in river mechanics engineering and bridge hydraulics. If neither a River Mechanics Engineer or Hydraulic Engineer having the necessary training and/or experience is available, the determination as to whether or not a proposed structure qualifies for a HERS determination shall be as directed by the Department's State Hydraulics Engineer.

Appendix 12D-1 Guidelines for the Determination, Documentation, and Processing of Hydraulically Equivalent Replacement Structures (HERS)

12D-1.4 Required Hydraulic Studies

When the proposed bridge is determined to be the hydraulic equivalent of the existing facility and it has also been determined that scour won't be a problem (refer to "Scour Evaluation Guidelines" shown below), no formal H&HA will be required. However, in the event a previous H&HA has been prepared either for or by the Department or by virtue of a FEMA flood insurance study, this information should be used to complete all required documentation and, in the event a scour evaluation must be performed, as a source of information necessary to perform such evaluation. Regardless of whether or not the proposed structure has been determined to qualify for a HERS classification, if it is found that a formal scour analysis is needed, such H&HA as is necessary to perform the scour analysis shall be conducted. In the event a FEMA hydraulic model is not available or cannot be used, it shall be necessary to conduct a complete H&HA to determine those factors necessary to conduct the scour analysis.

12D-1.5 Scour Assessment and/or Analysis

If the proposed facility has been determined to qualify for a HERS classification and no hydraulic data is available from any source, an assessment of the scour potential may be made using the following guidelines:

(1) A careful review of the bridge inspection reports for the existing bridge
(2) Checking for the availability of scour data for the existing bridge that may have been obtained during the FHWA mandated bridge scour evaluation program
(3) Checking (or having the bridge designer check) the "Item 113" code in the Bridge Inventory under the HTRIS system
(4) An on-site inspection, if possible accompanied by a Bridge Engineer and a Geotechnical Engineer

If there is <u>any evidence whatsoever,</u> that the existing bridge is experiencing scour problems, the designer should assume that a proposed HERS replacement will experience those same problems. In such cases a detailed hydraulic analysis <u>shall be performed</u> to determine such information as will be necessary to perform a detailed scour analysis. If time constraints, no FEMA or other previous hydraulic study is available, lack of or limited survey, etc. or other restrictions prevail, the analysis can be abbreviated but must still be performed. The engineer's findings regarding scour potential could be sufficient justification for the consideration of an entirely different replacement bridge layout.

Appendix 12D-1 Guidelines for the Determination, Documentation, and Processing of Hydraulically Equivalent Replacement Structures (HERS)

12D-1.6 Documentation

If no hydraulic data was available and/or none was performed, and scour was determined not to be a consideration, only the following documentation need be prepared:

(1) The “Hydrologic Data Sheet” (extracted from the LD-293 document) prepared in accordance with the attached example – it should be sent to the bridge designer and the road designer if an associated roadway project is involved

(2) LD-293B – to serve as a cover letter to convey the “Hydrologic Data Sheet” to the road designer (if applicable), to serve as a cover letter to convey the LD-293C document to the District Environmental Manager, and notify the Public Involvement Section of no hydraulic impacts

(3) LD-293C – to be completed with such information as is available and sent to the District Environmental Manager

If hydraulic data was available or if a scour analysis had to be performed (if it was determined that scour potential was significant) then all the usual documentation shall be prepared and distributed.

Appendix 12D-1 Guidelines for the Determination, Documentation, and Processing of Hydraulically Equivalent Replacement Structures (HERS)

EXAMPLE HYDROLOGIC DATA SHEET FOR USE ON HERS FACILITIES

HYDROLOGIC DATA SHEET

The information presented hereon is to be transcribed to the Hydrologic Data Sheet contained in the plan assembly.

DESCRIPTION

Sheet No. 4 Station 31+90

Stream Name Accotink Creek

Drainage Area 17 sq. mi

Structure Size 45' single span bridge

BASE FLOOD

Discharge 11,600[1] cfs

Stage Elevation 265.6[1] ft.

DESIGN FLOOD

Discharge no H&HA performed[2] cfs

Estimated Exceedence Probability no H&HA performed[2] %

Stage Elevation no H&HA performed[2] ft.

OVERTOPPING FLOOD

Stage Elevation 261.8 ft.

Estimated Exceedence Probability no H&HA performed[2] %

HISTORICAL DATA

Date Feb. 1979

Stage Elevation 258.6 ft.

Estimated Exceedence Probability no H&HA performed[2] %

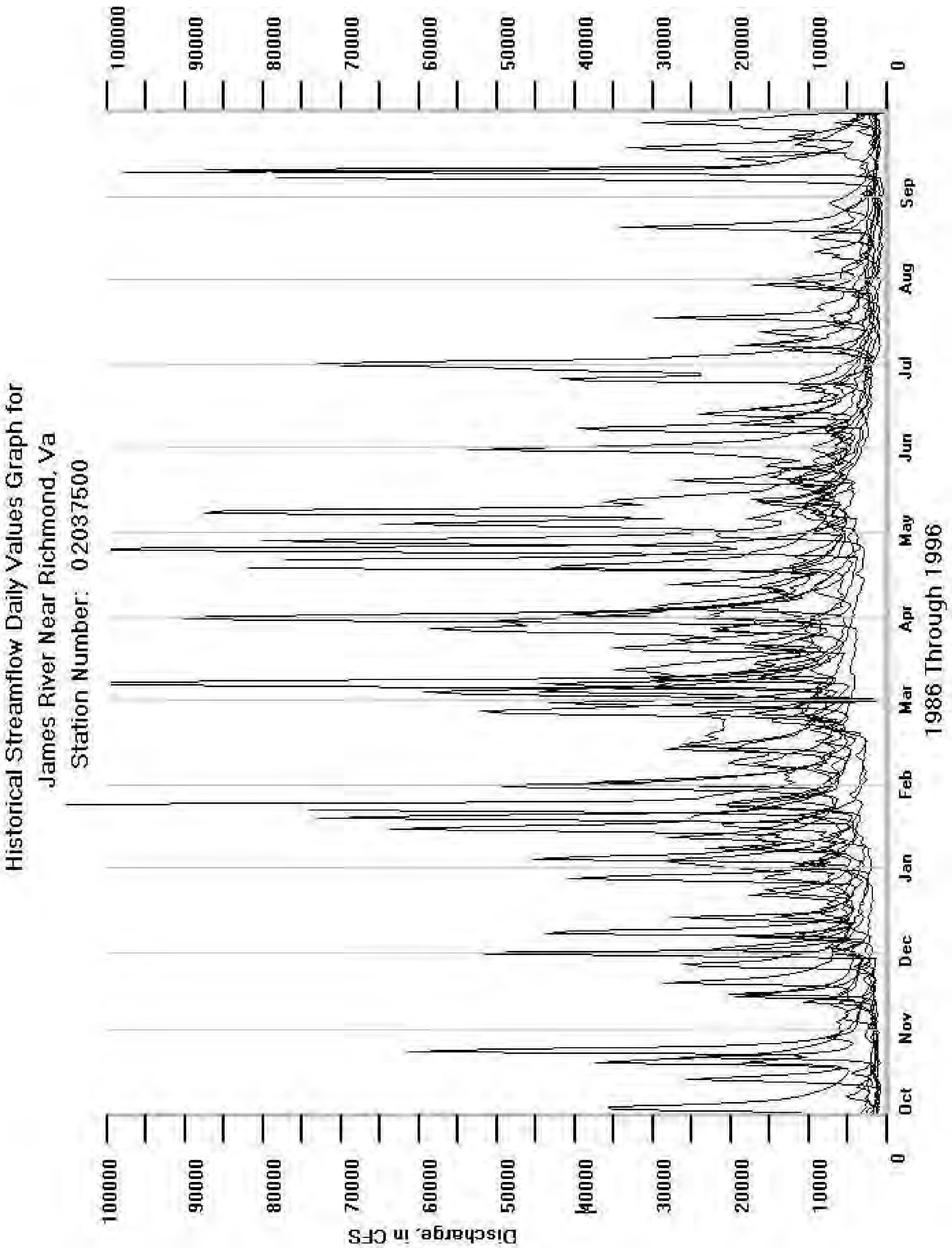
REMARKS

[1] Discharges and elevations are taken from U.S.G.S. open file report 76-442 dated 1977.

[2] This structure and its approaches are the hydraulic equivalent of the existing facility and no formal H&HA was prepared.

No significant hydraulic impact is anticipated as a result of this project.

Appendix 12E-01 **Daily Stream Flow information**



Chapter 13 - Shore Protection

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Chapter 13 - Shore Protection

13.1 Introduction

13.1.1 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.

13.1.2 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.

13.1.2.1 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 cans be subject to high wave attack from wind-generated waves.

13.1.2.2 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.

13.1.2.3 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.

13.1.2.4 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.

13.1.2.5 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.

13.1.2.6 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 off shore devices like groins may be used to aggrade the beach at embankment toe.

13.1.3 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 rushing 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.

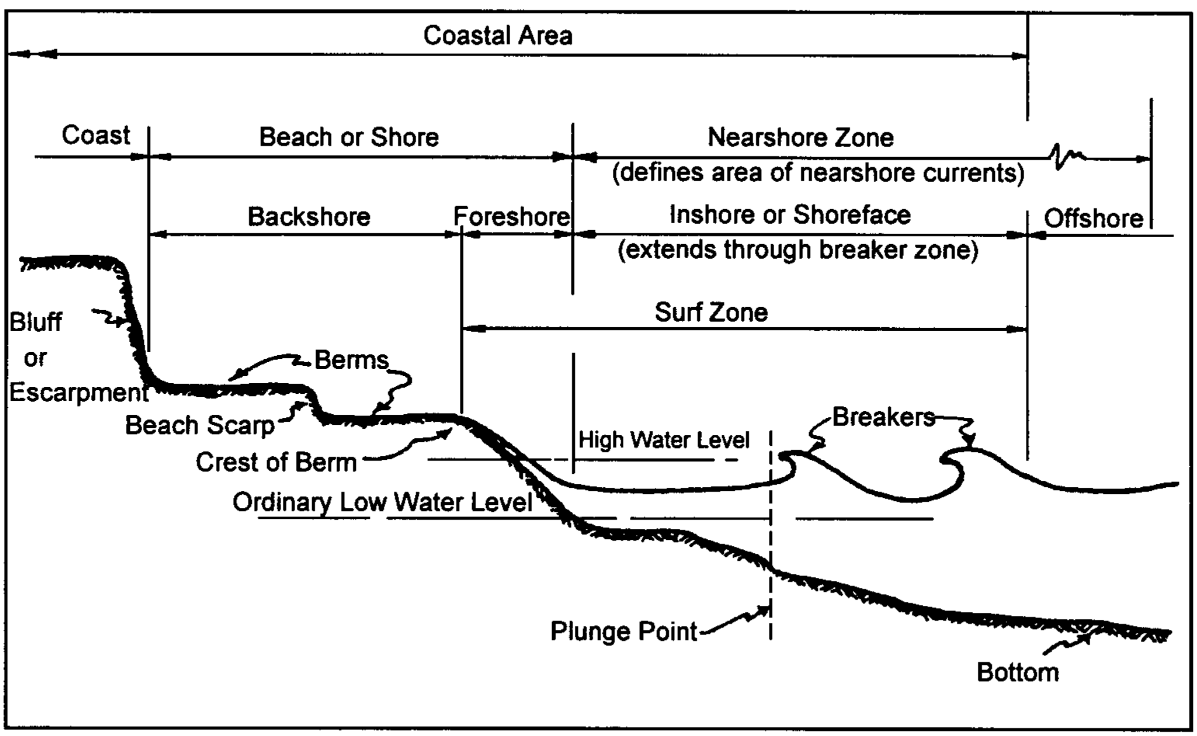

Figure 13-1. Visual Definition of Terms Describing a Typical Beach Profile

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 been updating the SPM. The updated manual is called the Coastal Engineering Manual and expected to be published in fall 2001. Because VDOT methods are based on the SPM, references are made to the SPM rather than the Coastal Engineering Manual.

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).

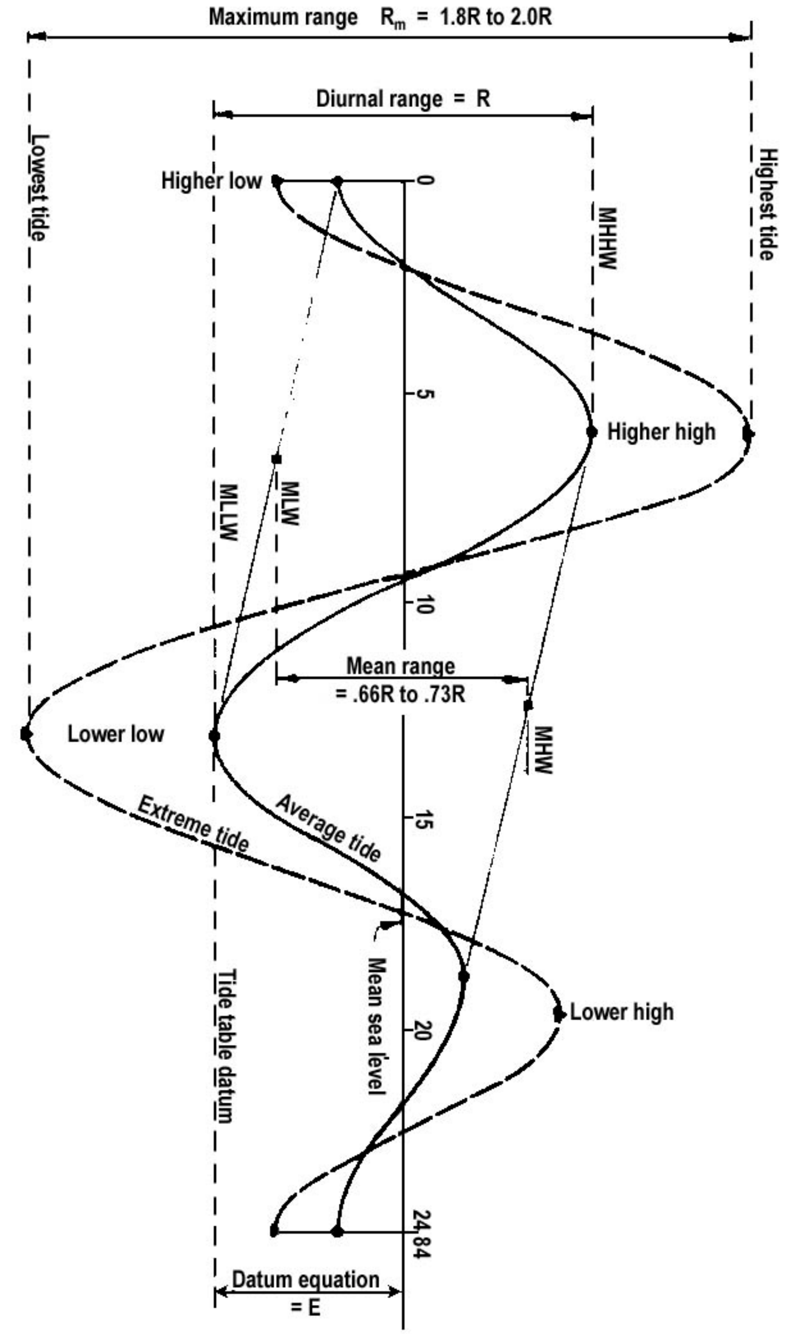

Figure 13-2. Nomenclature of Tidal Ranges

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.

The average of all tidal elevation is Mean Sea Level (MSL). The term MSL is often incorrectly used to refer to the zero datum for topographic maps. Usually MSL and the National Geodetic Vertical Datum are not the same and a correction must be made to convert tidal elevations to elevations in NGVD (see below).

The elevation of the Design High Tide may be taken as MSL plus one-half the maximum tidal range (R_m).

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). 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 shown in Appendix 13B-2.

13.2.3 Tidal Elevation Conversion

A conversion must be made to relate the elevation of Design High Water to the National Geodetic Vertical Datum (NGVD 1929) used in Highway Design[1]. A series of geodetic bench marks have been established which permit conversion of MLW datum to the more accepted NGVD. The relationships between MLW and NGVD 1929 for gage locations in Virginia are listed in Appendix 13B-1. Conversions for intermediate points may be made by interpolating between stations. Additional information on MLW and NGVD may be found by locating NOAA gages on the National Ocean Service web page (www.nos.noaa.gov), or 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

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

[1] NGVD 1929 has been superseded by the North American Vertical Datum (NAVD 1988). Use of NGVD 1929 is still common and many maps are referenced to NGVD 1929.

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 low water 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 NOAA's internet web site at the URL address:

<http://www.co-ops.nos.noaa.gov/bench_mark.shtml?region=va.>

The usual procedure for determining Mean High Water (MHW) and Mean Low Water (MLW) elevations is to ascertain (from NOAA's web site) the closest tidal bench mark to the point of interest and extract from it the NGVD 29 or the NAVD 88 (as appropriate) conversion factor and Mean Tide Level (MTL). Subtract the appropriate conversion factor from the Mean Tide Level (MTL) to establish the actual elevation for Mean Tide Level (MTL). The next step is to extract the Mean Tide Range (MTR) from the NOAA tide tables for that same location. Add half that value to the elevation established for the Mean Tide Level (MTL) to determine the elevation of Mean High Water (MHW). Subtract half that value from the elevation established for the Mean Tide Level (MTL) to determine the elevation of Mean Low Water (MLW).

Example: Find the mean low and high water elevations on the James River near Claremont, Virginia*.*

Step 1 *Consult NOAA's internet web site, locate the tidal benchmark for the James River near Claremont, and find the National Geodetic Vertical Datum-1929 (NGVD) conversion factor to be 0.68 ft. and the Mean Tide Level (MTL) to be 1.05 ft.*

Step 2 *Subtract the NGVD value of 0.68 ft. from the MTL value of 1.05 ft. to establish an MTL elevation of 0.37 ft.*

Step 3 *Consult NOAA's Tide Tables, locate Claremont on the James River, and the Mean Tide Range (MTR) to be 1.8 ft.*

Step 4 *Add one half the MTR to the elevation established for MTL:*

0.9 + 0.37 = 1.27 ft. Use 1.3 ft. for Mean High Water (MHW)

Step 5 *Subtract one half the MTR from the elevation established for MTL:*

0.37 – 0.9 = -0.53 ft. Use –0.5 ft. for Mean Low Water (MLW)

Answer: Mean Low Water (MLW) elevation = -0.5 ft.

Mean High Water (MHW) elevation = 1.3 ft.

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-foot high will break in a depth of about 1.3 feet. 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).

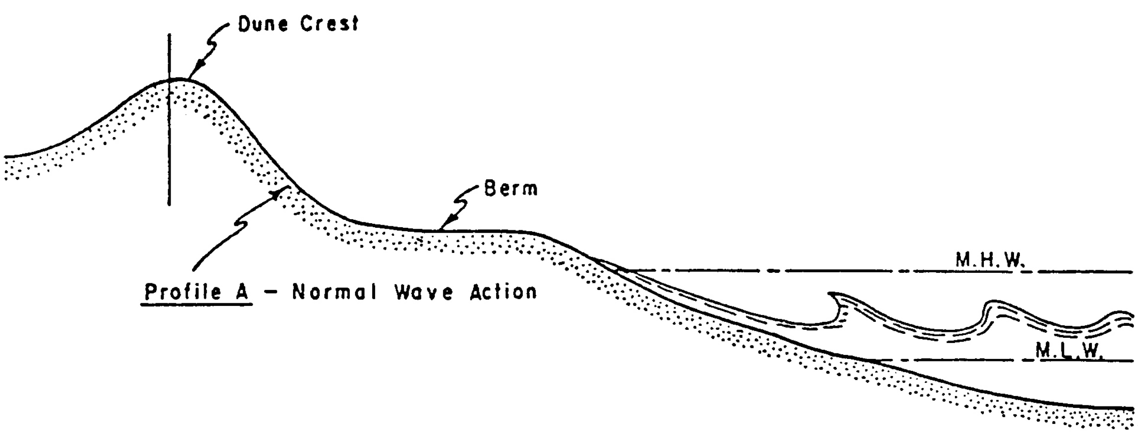

Figure 13-3. Normal Wave Action on Beach and Dune

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.

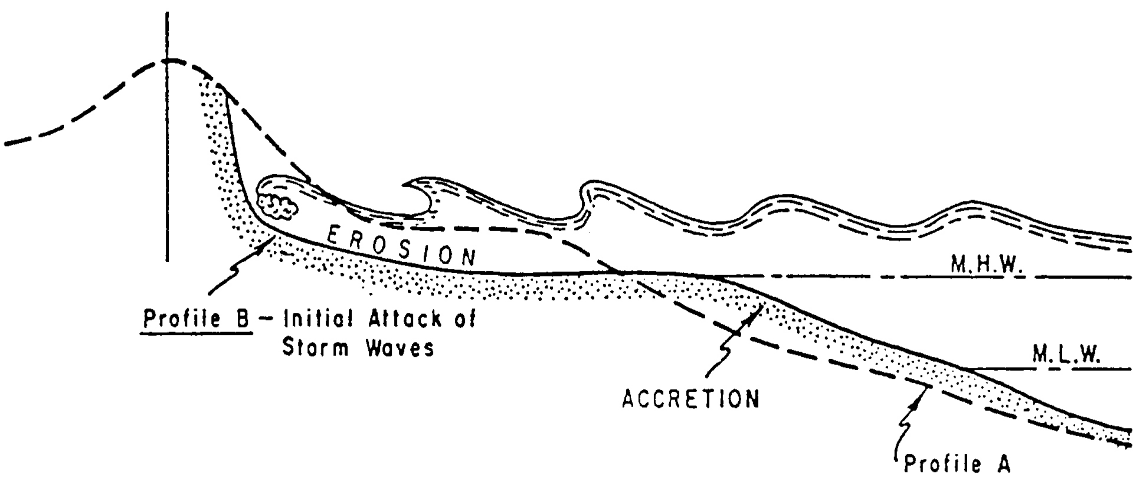

Figure 13-4. Initial Attack of Storm Waves on Beach and Dune

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 feet wide dunes to disappear in a few hours. Storm surges are especially damaging if they occur concurrently with high astronomical tides (Figure 13-5).

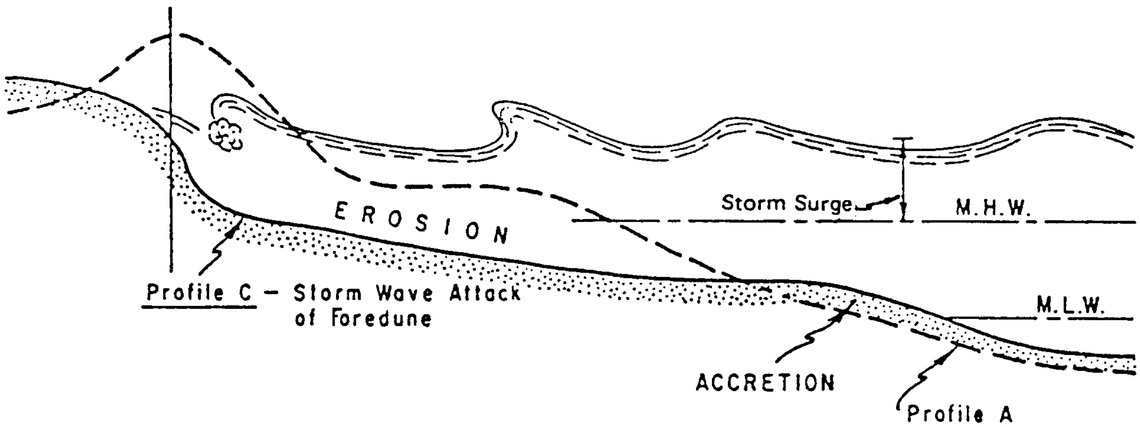

Figure 13-5. Storm Wave Attack of Foredune

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).

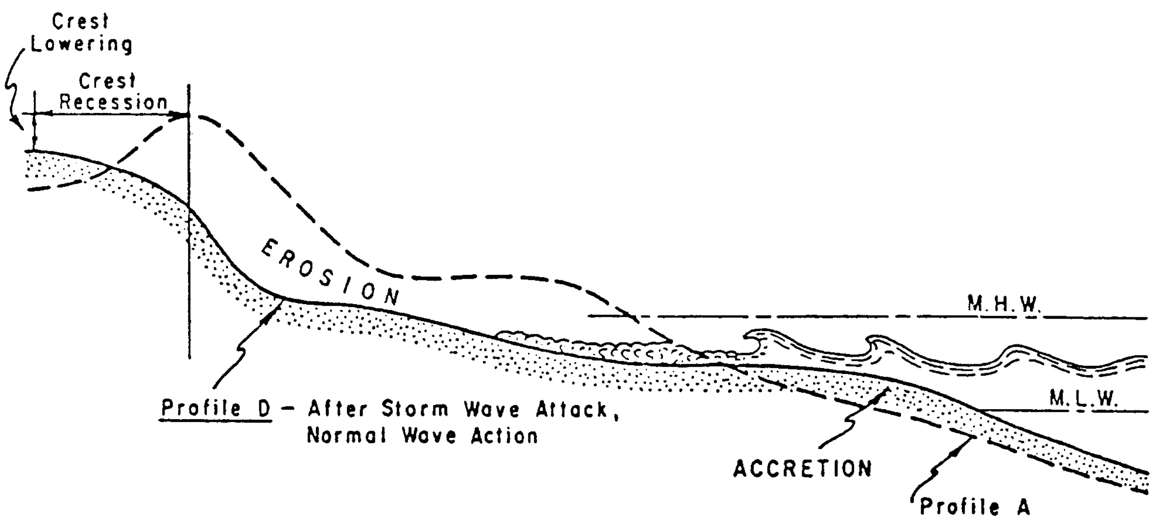

Figure 13-6. After Storm Wave Attack of Beach and Dune

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, "Shore Protection Manual," Volume 1, Chapter 3, 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 percent of all waves, and the H_1 design wave is the average of the highest 1 percent 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.

$$H_{10} = 1.27H_s \qquad (13.1)$$

$$H_1 = 1.67H_s \qquad (13.2)$$

Economics and risk of catastrophic failure are the primary considerations in designating the design wave average height.

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 "Shore Protection 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.

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 "Shore Protection 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 13C-1. If the estimated wave height from the nomograph is greater than 2.0 feet 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 13C-2.

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 "Shore Protection Manual," (SPM) the site maximization procedure consists of the following steps.

- Adjust wind information to 33 feet 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 (SPM Figures 3-24)
- Determine if deepwater or shallow-water conditions are present
- For shallow-water conditions, forecast shallow-water significant wave height and period (SPM Figures 3-27 through 3-36)
- 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 13C-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-feet 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 feet and the near-shore slope is 1V:10H.

Solution

$$\frac{d_s}{gT^2} = \frac{2.0}{32.2(3^2)} = 0.007$$

From Appendix 13C-2, $\dfrac{H_b}{d_s} = 1.4$

$$H_b = 2.8 \text{ ft}$$

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 feet.

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 feet or less wave run-up can be estimated by using Appendix 13C-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

Slope Surface - Material Type	Correction Factor
Concrete pavement	1.00
Concrete blocks (voids < 20%)	0.90
Concrete blocks (20% < voids > 40%)	0.70
Concrete blocks (40% < voids > 60%)	0.50
Grass	0.35 - 0.90
Rock riprap (angular)	0.60
Rock riprap (round)	0.70
Rock riprap (hand placed or keyed)	0.80
Grouted rock	0.90
Wire enclosed rocks/gabions	0.80

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, "Shore Protection Manual - Volume 1," pages 3-66 through 3-77.

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 percent 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. Procedures used by the Federal Emergency Management Agency (FEMA) for estimating wave heights in coastal high hazard areas in the Atlantic and Gulf Coast regions, are contained in FEMA's publication TD-3, 1981. This publication includes the NAS study above.

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 feet. For design waves greater than 2.0 feet, rock riprap usually provides the most economical and effective protection. Design procedures suitable for waves between 2.0 feet and 5.0 feet are provided below. Alternate design procedures are contained in Corps of Engineers Shore Protection Manual that should be used for design waves greater than 5.0 feet. 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 13C-4. The nomograph is derived from Equation 13.3.

$$W = \frac{0.003 d_s^3 sg_r \csc^3(\rho - \alpha)}{\left(\frac{sg_r}{sg_w} - 1\right)^3} \qquad (13.3)$$

Where:

d_s = Maximum depth of water at toe of the rock slope protection or bar, ft
sg_r = Specific gravity of stones

sg_w = Specific gravity of water (sea water = 1.0265)
α = Angle of face slope from the horizontal, deg
ρ = Constant -- 70° for randomly placed rubble
W = Minimum weight of outside stones for no damage, tons

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 13C-5. The nomograph is derived from Equation 13.4.

$$W = 0.00231 H_s^3 sg_r \csc^3 \left(\frac{sg_r}{sg_w} - 1 \right)^3 \qquad (13.4)$$

Where:

H_s = Significant wave height (average of the highest $\frac{1}{3}$), 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.

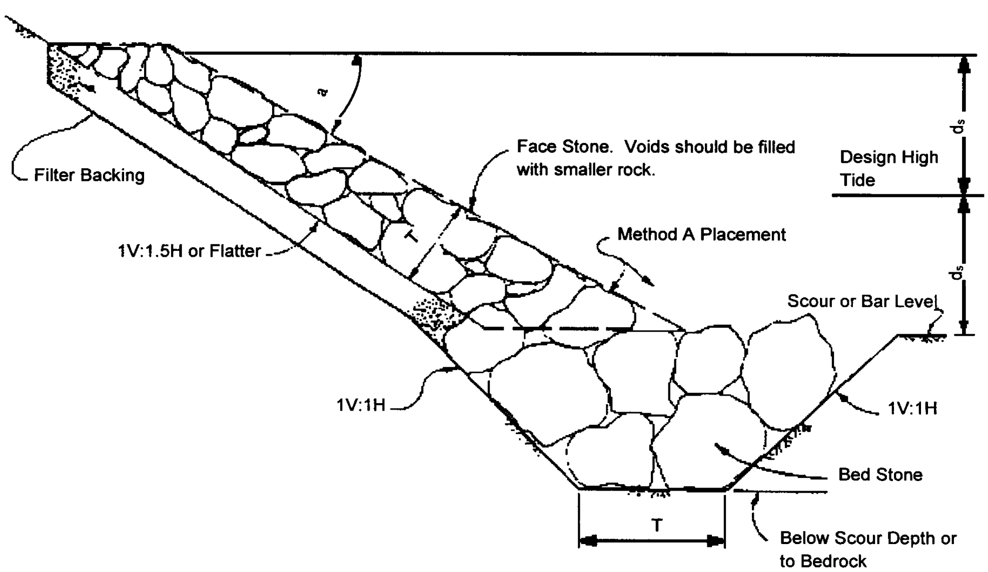

Figure 13-7. Riprap Rock Shore Protection Typical Design Configuration

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 that 5.0 ft and there is no major overtopping of the embankment and is defined by Equation 13.5.

$$W_{50} = \frac{1.67H^3}{\cot\theta} \qquad (13.5)$$

Where:

W_{50} = Weight of the 50% size stone, lbs
H = Wave height, ft.
θ = Angle of the embankment with respect to horizontal, deg.

Expressing the equation in terms of median grain diameter produces

$$D_{50} = \frac{0.57H}{\cot^{\frac{1}{3}}\theta} \qquad (13.6)$$

Where:

D_{50} = Mean spherical diameter of the 50% size stone, ft.

Equation 13.6 can be solved with the Hudson relationship nomograph in Appendix 13C-6.

13.8 References

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Appendix 13A-1 Definitions and Abbreviations

Definitions:

Backshore	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.
Barrier beach	A bar essentially parallel to the shore, which has been built up so that its crest rises above the normal high water level.
Deep-water wave	A deep-water wave is a wave in which the depth of water is greater than one-half the wavelength.
Downcast	The direction of predominant movement of littoral currents and transport.
Embayment	An indentation in a shoreline forming an open bay.
Fetch	The length of unobstructed open sea surface across which the wind can generate waves.
Foreshore	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.
Headland	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 or erosion resistant materials.

Appendix 13A-1 Definitions and Abbreviations

Lakeshore	Lakeshore, like coastal zone, refers to 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 mi^2 or more in area, have shores that require many of the same type considerations as ocean bays and estuaries. Analogous to coastal zone tides, some lakeshores are subject to significant changes in water surface elevation due to operation practices.
Longshore	Longshore generally refers to currents or sediment transport that move parallel to the shoreline.
Mean Higher High Water (MHHW)	MHHW is the average tidal elevation of the highest tidal elevation in a tidal day experienced over the 19-year metonic cycle.
Mean High Water (MHW)	MHW is the average high water elevation (both Higher High Water and Lower High Water) experienced over the 19-year metonic cycle.
Mean Lower Low Water (MLLW)	MLLW is the is the average tidal elevation of the lowest tidal elevation in a tidal day experienced over the 19-year metonic cycle.
Mean Low Water (MLW)	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.
Metonic Cycle	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.

Appendix 13A-1 Definitions and Abbreviations

Seiche	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.
Still-water Level (SWL)	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.
Surf Zone	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.

Abbreviations:

CEM	Coastal Engineering Manual
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
MHW	Mean High Water
MHHW	Mean Higher High Water
MLW	Mean Low Water
MLLW	Mean Lower Low Water
MSL	Mean Sea Level
MTL	Mean Tide Level
MTR	Mean Tide Range
NAS	National Academy of Sciences
NAVD	North American Vertical Datum
NGVD	National Geodetic Vertical Datum
NOAA	National Oceanic and Atmospheric Administration
SPM	Shore Protection Manual
SWL	Still Water Level
USACE or USCOE	United States Army Corps of Engineers
WES	Waterways Experiment Station

Appendix 13A-2 Symbols

Symbol	Definition	Units
D_{50}	Mean spherical diameter of the 50% size stone	ft
d_s	Maximum water depth at toe of rock slope protection or bar	ft
H	Wave height	ft
H_b	Design wave height	ft
H_s	Significant (design) wave height	ft
H_1	Average design wave height for highest 1%	ft
H_{10}	Average design wave height for highest 10%	ft
Sg_r	Specific gravity of stone	-
Sg_w	Specific gravity of water	-
W	Minimum weight of outside stones for no damage	tons
W_{50}	Weight of the 50% size stone	lbs
a	Angle of face slope from the horizontal	deg
?	70° for randomly placed rubble	deg
?	Angle of embankment with respect to the horizontal	deg

Appendix 13B-1 Tidal Data and Conversions

National Geodetic Vertical Datum Above Mean Low Water

Locality	NGVD-MLW (ft.)
Assateage beach Coast Guard Station, Chincoteague Isle	1.85
Chincoteague Island (north end of Chincoteague Channel)	0.77
Chincoteague Point, Chincoteague Bay	1.27
Wishart Point, Bogues Bay, Chincoteague Inlet	1.09
Wallops Island	1.86
Gargathy Neck	1.40
Oyster (off Sand Shoal Channel)	2.16
Kiptopeke Beach, Chesapeake Bay	1.42
Cape Charles Harbor, Chesapeake Bay	1.22
Gaskins Point, Occohannock river	0.94
Nandau Creek, Chesapeake Bay	0.93
Harborton	0.76
Onancock, Onancock Creek, Chesapeake Bay	0.77
Lewisette, Potomac River	0.48
Dahlgren, Potomac River	0.55
Aquia Creek, Potomac River	0.35
Quantico, Potomac River	0.55
Alexandria, Potomac River	0.87
Washington, D.C.	0.91
Glebe Point Highway Bridge	0.48
Windmill Point, Rappahannock River	0.59
Mill Creek, Rappahannock River	0.62
Tappahannock, Rappahannock River	0.63
Saunders Wharf, Tappahannock River	0.39
Massapaponax, Sand and Gravel, Fredericksburg, Rappahannock River	0.63
Gloucester Point, York River	1.25
West Point, York River	1.42
Old Point Comfort	1.30
Newport News, James River Entrance	1.31
Huntington Park, James River	1.39
Chuckatuck Creek Entrance	1.52
Menchville, James River	1.38
Burwell Bay, James River	1.17
Claremont, James River	0.78
Ferry Point, Chickahominy River	0.81
Lanexa, Chickahominy River	0.83
Norfolk Naval Shipyard, Portsmouth	1.44
Berkley, Norfolk Harbor	1.53
Sewells Point, Hampton Roads	1.38
Willoughby Spit, Chesapeake Bay	1.26
Little Creek Entrance, Chesapeake Bay	1.46
Little Creek	1.38
Virginia Beach	1.82

Appendix 13B-2 Tidal Differences and Other Constants

The publication of full daily tide predictions is necessarily limited to a comparatively small number of stations. These stations are referred to as "reference stations". Tide predictions for other locations can be obtained by applying certain differences to the daily tide predictions for the reference stations.

These pages provide a listing of the "subordinate stations" in Virginia for which such predictions can be made, the differences or ratios to be used, and a link to the appropriate reference station. The stations in the listing are arranged in geographical order to make it possible to find stations which are available for a specific area.

Caution: The time and height differences and ratios are derived from a comparison of simultaneous tide observations at the subordinate station and its reference station. Because these figures are constant, they may not always provide for the daily variations of the actual tide, especially if the subordinate station is some distance from the reference station. Therefore, although the application of time and height differences will generally provide reasonably accurate approximations, they cannot result in predictions as accurate as those listed for the reference stations which are based on much larger periods of analysis.

Time Differences: To determine the time of high and low tide at any station listed in this table, refer to the columns headed "Time Differences" in which the hours and minutes to be added or subtracted from the time of high or low tide of the reference stations. A plus sign (+) indicates that the tide at the subordinate station occurs later than at the reference station and the difference should be added; a minus sign (-) indicates that it is earlier and should be subtracted.

To obtain the tide at a subordinate station on any date, apply the difference to the tide at the reference station for that same date. In some cases, however, to obtain an AM tide it may be necessary to use the preceding day's PM tide at the reference station or to obtain a PM tide it may be necessary to use the following day's AM tide. For example, if a high tide at a reference station occurs at 0200 on July 17, and the tide at the subordinate station occurs 5 hours earlier, the high tide at the subordinate station will occur at 900 PM on July 16. For the second case, if the high water at a reference station occurs at 1000 PM on July 2, and the tide at the subordinate station occurs 3 hours later, then high tide will occur at 100 AM on July 3 at the subordinate station.

The results obtained by application of the time differences will be in local time for the subordinate station. The necessary allowances for the change in date when crossing the international date line, or for different time zones have been included in the time differences listed.

Appendix 13B-2 Tidal Differences and Other Constants

Height Differences: The height of the tide, referred to the datum of nautical charts, is obtained by means of the height difference or ratios. A plus sign (+) indicates that the difference should be added to the height at the reference station, and a minus sign (-) indicates that it should be subtracted. For most stations, use of a predicted height difference would give unsatisfactory predictions. In such cases they have been omitted and one or two ratios, indicated by an asterisk (*), are given. To obtain the height of tide at the subordinate station in these cases, multiply the height of tide at the reference station by the ratio listed. The result is normally rounded to the nearest 0.1-foot.

For some subordinate stations there is given, in parentheses, a ratio as well as a correction. In those instances, each predicted high and low water at the reference station should be first multiplied by the ratio and then the correction is added or subtracted from each product.

Example Tide Calculations

For Port Royal on the Rappahannock River, the time and height adjustments listed in the tables are:

-2 21 -2 49 *0.68 *0.68

and the reference station is Washington. If the times in column 1 are the tides for a day at Washington, column 2 are the time corrections and column 3 are the height corrections, column 4 will be the predicted tides at Port Royal.

(1)	(2)	(3)	(4)
403am L -0.1	-2 49	*0.68	114am -0.1ft
919am L 2.4	-2 21	*0.68	658am 1.6ft
401pm H -0.1	-2 49	*0.68	112pm -0.1ft
930pm L 2.6	-2 21	*0.68	709pm 1.7ft

Appendix 13B-2 Tidal Differences and Other Constants

TABLE 2 – TIDAL DIFFERENCES AND OTHER CONSTANTS

No.	PLACE	POSITION Latitude	POSITION Longitude	DIFFERENCES Time High Water	DIFFERENCES Time Low Water	DIFFERENCES Height High Water	DIFFERENCES Height Low Water	RANGES Mean	RANGES Spring	Mean Tide Level
		North	West	h m	h m	ft	ft	ft	ft	ft
	MD., VA. and DISTRICT OF COLUMBIA–cont. Potomac River–cont. Time meridian, 75° W			on Washington, p.84						
2393	Key Bridge, D.C.	38° 54'	77° 04'	+0 25	+0 06	*1.03	*1.00	2.8	3.2	1.6
2395	Chain Bridge, one mile below, D.C.	38° 55'	77° 06'	+0 30	+0 11	*1.03	*1.00	2.8	3.2	1.6
2397	Chain Bridge, D.C.	38° 56'	77° 07'	+0 35	+0 16	*1.03	*1.00	2.8	3.2	1.6
	VIRGINIA–cont. Chesapeake Bay, western shore–cont.			on Hampton Roads, p.68						
2399	Smith Point Light	37° 53'	76° 11'	+3 30	+3 30	*0.50	*0.50	1.2	1.4	0.7
2401	Sunnybank, Little Wicomico River	37° 53'	76° 16'	+5 42	+5 40	*0.31	*0.31	0.8	1.0	0.4
2403	Great Wicomico River Light	37° 48'	76° 16'	+2 59	+3 06	*0.44	*0.44	1.1	1.3	0.5
2405	Fleet Point	37° 49'	76° 16'	+2 59	+3 09	*0.42	*0.42	1.1	1.3	0.6
2407	Glebe Point, Great Wicomico River	37° 51'	76° 22'	+3 16	+3 32	*0.50	*0.50	1.2	1.4	0.7
2409	Dividing Creek	37° 44'	76° 16'	+2 38	+2 36	*0.44	*0.44	1.1	1.3	0.5
2411	Windmill Point Light	37° 36'	76° 14'	+1 48	+2 06	*0.42	*0.42	1.1	1.3	0.5
2413	Stingray Point	37° 34'	76° 18'	+1 05	+1 27	*0.50	*0.50	1.2	1.4	0.7
	Rappahannock River									
2415	Windmill Point	37° 37'	76° 17'	+1 56	+2 13	*0.50	*0.50	1.2	1.5	0.7
2417	Mill Creek	37° 35'	76° 25'	+2 23	+2 37	*0.54	*0.54	1.3	1.6	1.4
2419	Orchard Point	37° 39'	76° 27'	+2 29	+2 46	*0.58	*0.58	1.4	1.7	0.8
2421	Millenbeck, Corrotoman River	37° 40'	76° 29'	+2 32	+3 00	*0.54	*0.54	1.3	1.6	0.7
2423	Urbanna	37° 39'	76° 34'	+2 45	+3 04	*0.58	*0.58	1.4	1.7	0.8
2425	Bayport	37° 45'	76° 40'	+3 17	+3 46	*0.65	*0.65	1.6	1.9	0.9
2427	Bowlers Rock	37° 49'	76° 44'	+3 53	+4 20	*0.73	*0.75	1.8	2.2	1.0
2429	Wares Wharf	37° 52'	76° 47'	+4 02	+4 28	*0.73	*0.75	1.8	2.1	1.0
2431	Tappahannock	37° 56'	76° 51'	+4 35	+5 13	*0.69	*0.69	1.7	1.9	1.0
				on Washington, p.84						
2433	Leedstown	38° 07'	77° 00'	−4 31	−4 49	*0.53	*0.53	1.5	1.7	0.8
2435	Saunders Wharf	38° 05'	77° 02'	−4 11	−4 35	*0.54	*0.54	1.5	1.7	0.9
2437	Green Bay	38° 09'	77° 05'	−3 21	−3 39	*0.61	*0.61	1.7	2.0	0.9
2439	Port Royal	38° 10'	77° 11'	−2 31	−2 49	*0.68	*0.68	1.9	2.2	1.1
2441	Hopyard Landing	38° 15'	77° 14'	−1 25	−1 51	*0.76	*0.76	2.1	2.3	1.2
2443	Corbins Neck	38° 13'	77° 17'	−1 18	−1 20	*0.96	*0.94	2.6	3.0	1.5
2445	Massaponax	38° 15'	77° 25'	−0 57	−0 35	*0.90	*0.88	2.5	2.8	1.4
2447	Fredericksburg	38° 18'	77° 27'	−0 48	−0 51	*1.03	*1.00	2.8	3.2	1.6
				on Hampton Roads, p.68						
	Piankatank River									
2449	Cherry Point	37° 31'	76° 18'	+0 47	+1 02	*0.50	*0.50	1.2	1.4	0.7
2451	Jackson Creek	37° 33'	76° 20'	+1 31	+1 59	*0.50	*0.50	1.2	1.4	0.7
2453	Dixie	37° 30'	76° 25'	+1 29	+2 07	*0.54	*0.50	1.3	1.6	0.7
2455	Wolf Trap Light	37° 23'	76° 11'	−0 07	+0 27	*0.65	*0.65	1.6	1.9	0.9
	Mobjack Bay									
2457	New Point Comfort	37° 18'	76° 17'	−0 07	+0 01	*0.93	*0.92	2.3	2.8	1.2
2459	Mobjack, East River	37° 22'	76° 21'	−0 22	−0 03	*0.97	*1.00	2.4	2.9	1.3
2461	Belleville	37° 25'	76° 26'	−0 11	−0 05	*1.00	*1.00	2.5	3.0	1.4
2463	Browns Bay	37° 18'	76° 24'	−0 13	+0 01	*1.00	*1.00	2.4	2.9	1.4
2465	SW Branch, Severn River	37° 18'	76° 27'	−0 02	+0 06	*1.00	*1.00	2.5	3.0	1.1
2467	York Spit Light	37° 13'	76° 15'	−0 17	−0 09	*0.92	*0.92	2.3	2.8	1.3
	York River									
2469	Tue Marshes Light	37° 14'	76° 23'	−0 05	−0 02	*0.88	*0.92	2.2	2.6	1.2
2471	Perrin River	37° 16'	76° 25'	+0 13	+0 03	*1.00	*0.92	2.3	2.8	1.2
2473	Goodwin Neck	37° 14'	76° 26'	+0 13	+0 10	*0.88	*0.92	2.2	2.6	1.2
2475	Quarter Point	37° 15'	76° 27'	+0 08	+0 06	*0.93	*0.92	2.3	2.8	1.2
2477	Gloucester Point	37° 15'	76° 30'	+0 11	+0 07	*0.98	*0.98	2.4	2.8	1.3
2479	Yorktown	37° 14'	76° 30'	+0 11	+0 06	*0.96	*1.00	2.4	2.9	1.3
2481	Mumfort Islands	37° 16'	76° 31'	+0 19	+0 12	*1.00	*1.00	2.5	3.0	1.3
2483	Penniman Spit	37° 17'	76° 35'	+0 41	+0 44	*1.00	*1.00	2.5	3.0	1.3
2485	Cheatham Annex	37° 18'	76° 35'	+0 43	+0 35	*1.00	*1.00	2.5	3.0	1.3
2487	Queen Creek (2 miles above entrance)	37° 18'	76° 39'	+1 00	+0 59	*0.96	*0.96	2.4	2.9	1.3
2489	Clay Bank	37° 21'	76° 37'	+0 50	+0 49	*1.12	*1.12	2.8	3.4	1.1
2491	Allmondsville	37° 23'	76° 39'	+0 59	+1 02	*1.14	*1.16	2.8	3.3	1.5
2493	Roane Point	37° 27'	76° 42'	+1 42	+1 45	*1.12	*1.17	2.8	3.4	1.5
2495	West Point	37° 32'	76° 48'	+2 07	+2 33	*1.11	*1.17	2.8	3.4	1.5
	Mattaponi River									
2497	Wakema	37° 39'	76° 54'	+3 29	+3 52	*1.38	*1.33	3.4	3.9	1.9
2499	Walkerton	37° 43'	77° 02'	+4 26	+4 54	*1.58	*1.58	3.9	4.5	2.1
	Pamunkey River									
2501	Sweet Hall Landing	37° 34'	76° 54'	+3 48	+4 06	*1.08	*1.08	2.7	3.1	1.4
2503	Lester Manor	37° 35'	76° 59'	+4 40	+4 55	*1.12	*1.17	2.8	3.2	1.5
2505	White House	37° 35'	77° 01'	+5 09	+5 24	*1.23	*1.25	3.0	3.4	1.7
2507	Northbury	37° 37'	77° 07'	+5 58	+6 13	*1.35	*1.33	3.3	3.8	1.8
	Chesapeake Bay, western shore–cont.									
2509	York Point, Poquoson River	37° 10'	76° 24'	−0 07	+0 01	*0.96	*1.00	2.4	2.9	1.3
2511	Messick Point, Back River	37° 06'	76° 19'	−0 26	−0 05	*0.93	*0.92	2.3	2.8	1.3
	Hampton Roads									
2513	Old Point Comfort	37° 00'	76° 19'	−0 04	−0 14	*1.00	*1.00	2.5	3.0	1.4
2515	Hampton River	37° 01'	76° 20'	+0 02	−0 07	*1.04	*1.08	2.6	3.1	1.4

Appendix 13B-2 Tidal Differences and Other Constants

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TABLE 2 – TIDAL DIFFERENCES AND OTHER CONSTANTS

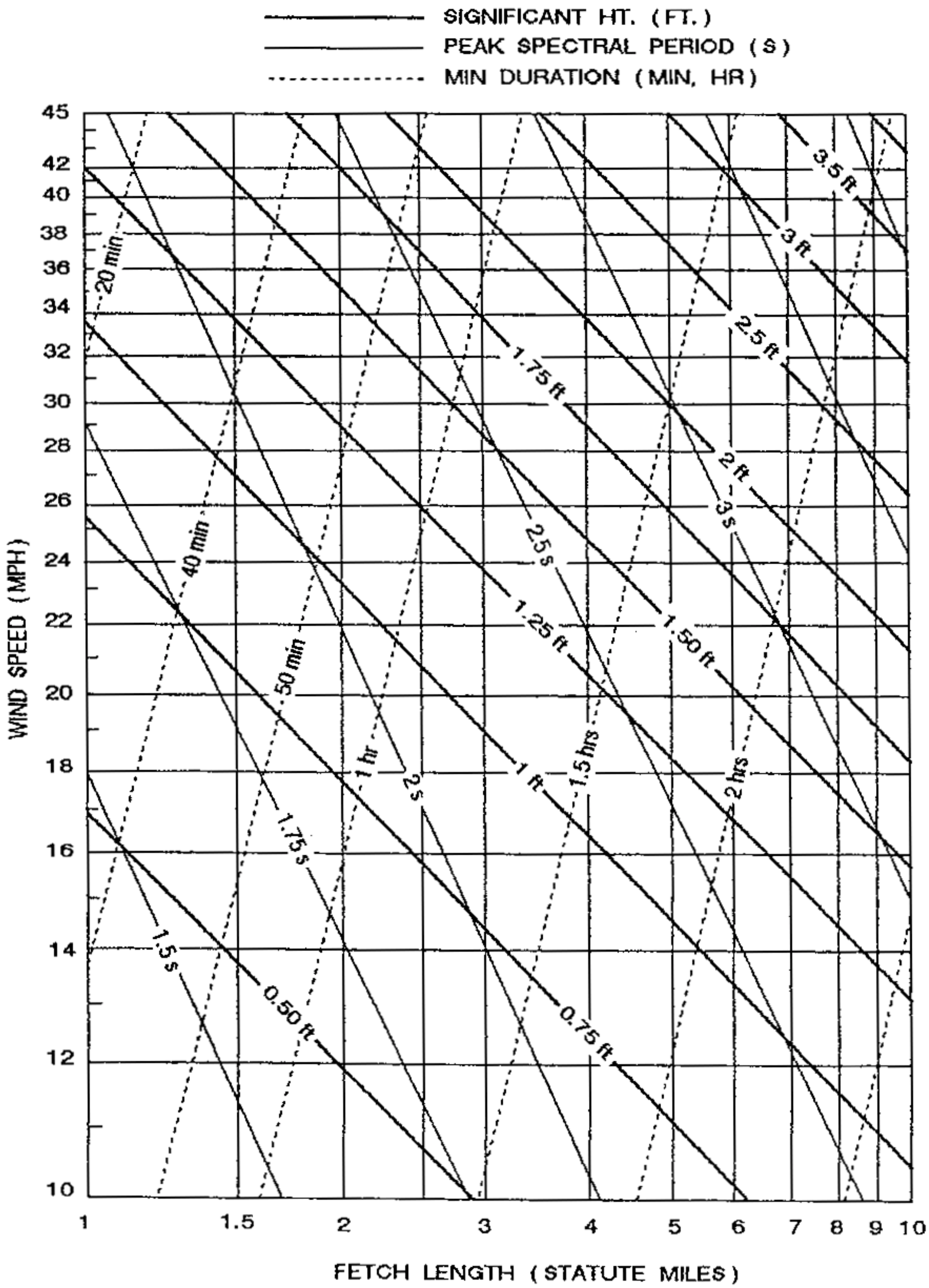
No.	PLACE	POSITION: Latitude	POSITION: Longitude	DIFFERENCES: Time: High Water	DIFFERENCES: Time: Low Water	DIFFERENCES: Height: High Water	DIFFERENCES: Height: Low Water	RANGES: Mean	RANGES: Spring	Mean Tide Level
		North	West	h m	h m	ft	ft	ft	ft	ft
	VIRGINIA–cont. Chesapeake Bay, western shore–cont. Time meridian, 75° W			on Hampton Roads, p.88						
	Hampton Roads–cont.									
2517	HAMPTON ROADS (Sewells Point)	36° 57'	76° 20'	*Daily predictions*				2.5	2.9	1.4
2519	Lafayette River	36° 54'	76° 18'	+0 11	+0 20	*1.04	*1.08	2.6	3.1	1.4
2521	Lafayette River, Granby St. Bridge	36° 53'	76° 17'	+0 26	+0 32	*1.08	*1.08	2.7	3.2	1.5
	Elizabeth River									
2523	Craney Island	36° 54'	76° 20'	+0 13	−0 01	*1.04	*1.08	2.6	3.1	1.4
2525	Port Norfolk, Western Branch	36° 51'	76° 20'	+0 17	+0 24	*1.04	*1.08	2.6	3.1	1.4
2527	Norfolk	36° 51'	76° 18'	+0 16	+0 15	*1.12	*1.17	2.8	3.4	1.5
2529	Portsmouth, Southern Branch	36° 49'	76° 18'	+0 07	+0 09	*1.14	*1.17	2.8	3.3	1.5
	Nansemond River									
2531	Pig Point	36° 55'	76° 26'	+0 37	+0 35	*1.12	*1.17	2.8	3.4	1.5
2533	Town Point	36° 53'	76° 30'	+0 33	+0 39	*1.16	*1.17	3.0	3.6	1.6
2535	Hollidays Point (bridge)	36° 50'	76° 33'	+0 51	+0 58	*1.23	*1.25	3.0	3.6	1.6
2537	Suffolk	36° 44'	76° 35'	+1 37	+1 30	*1.54	*1.50	3.8	4.6	2.1
	James River									
2539	Chuckatuck Creek entrance	36° 55'	76° 30'	+0 45	+0 52	*1.12	*1.12	2.8	3.4	1.5
2541	Newport News	36° 58'	76° 26'	+0 24	+0 23	*1.04	*1.08	2.6	3.1	1.4
2543	Huntington Park	37° 01'	76° 26'	+0 40	+0 39	*1.04	*1.08	2.6	3.1	1.5
2545	Menchville	37° 05'	76° 32'	+0 58	+1 14	*1.04	*1.08	2.6	3.1	1.4
2547	Smithfield, Pagan River	36° 59'	76° 38'	+1 29	+1 23	*1.12	*1.17	2.8	3.4	1.5
2549	Burwell Bay	37° 03'	76° 40'	+1 20	+1 39	*0.96	*1.00	2.4	2.9	1.3
2551	Mulberry Point	37° 08'	76° 38'	+2 00	+2 21	*0.96	*1.00	2.4	2.9	1.3
2553	Hog Point	37° 12'	76° 41'	+2 15	+2 33	*0.85	*0.83	2.1	2.5	1.1
2555	Scotland	37° 11'	76° 47'	+2 51	+3 20	*0.77	*0.75	1.9	2.1	1.0
2557	Jamestown Island	37° 12'	76° 47'	+2 58	+3 31	*0.81	*0.81	2.0	2.4	1.1
2559	Dillard Wharf	37° 12'	76° 52'	+3 33	+4 10	*0.77	*0.75	1.9	2.3	1.0
	Chickahominy River									
2561	Ferry Point (bridge)	37° 16'	76° 53'	+3 56	+4 21	*0.77	*0.75	1.9	2.3	1.0
2563	Wright Island Landing	37° 21'	76° 52'	+4 39	+4 56	*0.89	*0.92	2.2	2.6	1.1
2565	Mount Airy	37° 21'	76° 55'	+5 05	+5 33	*0.88	*0.92	2.2	2.6	1.2
2567	Lanexa	37° 24'	76° 54'	+5 35	+6 03	*1.04	*1.08	2.6	3.1	1.4
2569	Claremont	37° 14'	76° 57'	+3 58	+4 30	*0.73	*0.75	1.8	2.0	1.0
2571	Sturgeon Point	37° 18'	77° 00'	+4 32	+5 04	*0.85	*0.83	2.1	2.5	1.1
2573	Windmill Point	37° 18'	77° 06'	+5 26	+5 51	*0.92	*0.92	2.3	2.7	1.2
2575	Willcox Wharf, Charles City	37° 19'	77° 08'	+5 25	+5 45	*0.88	*0.92	2.2	2.4	1.1
2577	Westover	37° 19'	77° 09'	+5 47	+6 12	*0.96	*1.00	2.4	2.8	1.3
2579	Jordan Point	37° 19'	77° 13'	+6 11	+6 34	*1.00	*1.00	2.5	2.9	1.4
				on Washington, p.84						
2581	City Point (Hopewell)	37° 19'	77° 16'	−4 40	−5 06	*0.96	*0.94	2.6	3.0	1.5
2583	Petersburg, Appomattox River	37° 14'	77° 24'	−4 10	−3 54	*1.08	*1.06	2.9	3.3	1.6
2585	Bermuda Hundred	37° 20'	77° 16'	−4 35	−4 59	*0.96	*0.94	2.6	3.0	1.5
2587	Hexall	37° 22'	77° 15'	−4 28	−4 48	*1.00	*0.94	2.7	3.1	1.6
2589	Curles, 1 mile north of	37° 24'	77° 18'	−4 10	−4 20	*1.03	*1.00	2.8	3.2	1.6
2591	Chester	37° 23'	77° 23'	−3 57	−3 53	*1.03	*0.82	2.9	3.2	1.6
2593	Meadowville	37° 23'	77° 19'	−4 19	−4 27	*1.06	*1.06	2.9	3.3	1.7
2595	Kingsland Reach	37° 24'	77° 23'	−4 17	−4 22	*1.10	*1.06	3.0	3.5	1.7
2597	Falling Creek entrance	37° 26'	77° 26'	−4 06	−4 02	*1.17	*1.18	3.2	3.7	1.8
2599	Richmond Deepwater Terminal	37° 27'	77° 25'	−4 03	−3 55	*1.21	*1.18	3.3	3.8	1.8
2601	Lower Rocketts	37° 30'	77° 25'	−3 37	−3 26	*1.17	*1.18	3.2	3.6	1.8
2603	Richmond (river locks)	37° 32'	77° 25'	−3 34	−3 20	*1.17	*1.18	3.2	3.6	1.8
	Chesapeake Bay, southern shore			on Hampton Roads, p.88						
2605	Little Creek (RR. Terminal)	36° 55'	76° 11'	−0 48	−0 50	*1.04	*1.04	2.6	3.1	1.4
	Lynnhaven Inlet									
2607	Highway bridge, east of	36° 54'	76° 05'	−0 09	+0 06	*0.81	*0.83	2.0	2.4	1.1
	Lynnhaven Bay									
2609	Bayville	36° 54'	76° 06'	+0 50	+1 43	*0.69	*0.67	1.7	2.0	1.0
2611	Buchanan Creek entrance	36° 52'	76° 07'	+1 00	+1 51	*0.77	*0.75	1.9	2.3	1.0
2613	Long Creek	36° 54'	76° 04'	+0 48	+1 19	*0.32	*0.32	0.8	1.0	0.4
2615	Brown Cove	36° 52'	76° 04'	+0 46	+1 43	*0.69	*0.67	1.7	2.0	0.9
2617	Cape Henry	36° 56'	76° 00'	−0 48	−1 10	*1.11	*1.17	2.8	3.4	1.5
	VIRGINIA, outer coast									
2619	Virginia Beach	36° 51'	75° 58'	−1 28	−1 30	*1.38	*1.33	3.4	4.1	1.9
2621	False Cape	36° 36'	75° 53'	−1 41	−1 40	*1.46	*1.42	3.6	4.3	2.0
	NORTH CAROLINA, outer coast									
2623	Currituck Beach Light	36° 23'	75° 50'	−1 46	−1 45	*1.39	*1.42	3.6	4.3	2.0
2625	Duck Pier	36° 11.0'	75° 44.8'	−1 46	−1 49	*1.32	*1.25	3.28	3.97	1.79
2627	Albemarle and Pamlico Sounds <9>	— — —	— — —	— — —	— — —	— — —	— — —	— —	— —	— —
2629	Kitty Hawk (ocean)	36° 06'	75° 43'	−1 50	−1 49	*1.31	*1.33	3.2	3.8	1.8
2631	Jennetts Pier (ocean)	35° 55'	75° 36'	−1 54	−1 50	*1.32	*1.33	3.3	3.9	1.8
2633	Roanoke Sound Channel	35° 48'	75° 35'	+0 27	+0 37	*0.19	*0.17	0.5	0.6	0.3
2635	Oregon Inlet Marina	35° 48'	75° 33'	−0 38	+0 26	*0.23	*0.23	0.6	0.7	0.3
2637	Oregon Inlet	35° 46'	75° 31'	−1 13	−1 07	*0.77	*0.83	2.0	2.4	1.1
2639	Oregon Inlet Bridge	35° 46'	75° 32'	−1 27	−1 35	*0.77	*0.75	1.9	2.3	1.1

Appendix 13B-2 Tidal Differences and Other Constants

TABLE 2 – TIDAL DIFFERENCES AND OTHER CONSTANTS

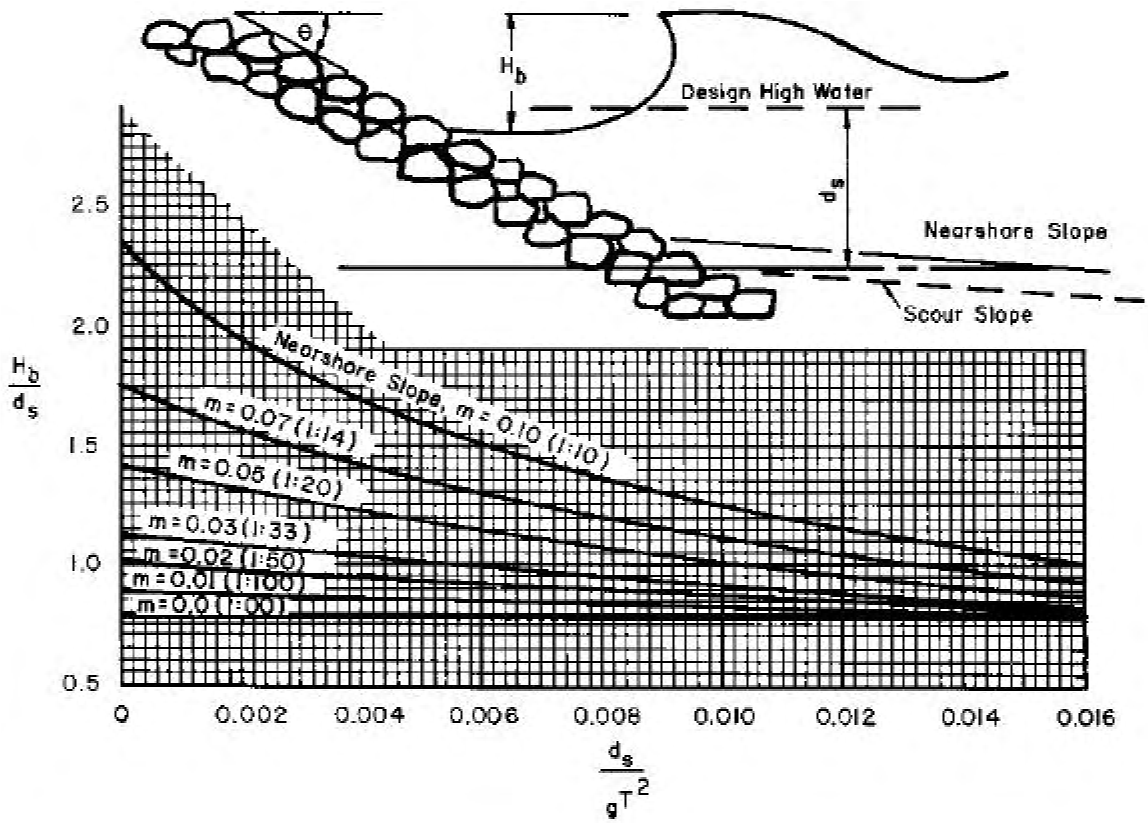
No.	PLACE	POSITION: Latitude	POSITION: Longitude	DIFFERENCES Time: High Water	DIFFERENCES Time: Low Water	DIFFERENCES Height: High Water	DIFFERENCES Height: Low Water	RANGES: Mean	RANGES: Spring	Mean Tide Level
		North	West	h m	h m	ft	ft	ft	ft	ft
	MARYLAND–cont. Chesapeake Bay, western shore–cont. Time meridian, 75° W				on Baltimore, p.80					
	Severn River									
2251	Cedar Point	39° 04'	76° 34'	−0 47	−0 49	*0.62	*0.62	0.7	0.8	0.6
2253	Brewer Point	39° 02'	76° 32'	−1 00	−1 00	*0.71	*0.71	0.8	0.9	0.6
2255	Annapolis	38° 59'	76° 29'	−1 38	−1 49	*0.81	*0.83	0.9	1.1	0.7
2257	Bay Ridge	38° 56'	76° 27'	−2 01	−2 05	*0.74	*0.70	0.8	1.0	0.6
2259	Thomas Point Shoal Light	38° 54'	76° 26'	−2 04	−2 16	*0.81	*0.83	0.9	1.0	0.6
2261	Edgewater, South River	38° 57'	76° 33'	−2 01	−2 12	*0.81	*0.83	0.9	1.0	0.6
2263	Gingerville Creek, South River	38° 58'	76° 33'	−2 09	−2 00	*0.90	*0.92	1.0	1.1	0.7
2265	Rhode River (County Wharf)	38° 53'	76° 32'	−2 04	−2 14	*0.91	*0.92	1.0	1.1	0.7
2267	Shady Side, West River	38° 51'	76° 31'	−1 58	−1 48	*0.81	*0.83	0.9	1.0	0.6
2269	Galesville, West River	38° 50'	76° 32'	−1 48	−1 38	*0.81	*0.83	0.9	1.0	0.6
2271	Fairhaven, Herring Bay	38° 45'	76° 33'	−2 55	−3 01	*0.81	*0.83	0.9	1.0	0.6
2273	Rose Haven	38° 43'	76° 32'	−2 47	−2 49	*0.81	*0.83	0.9	1.0	0.6
2275	Chesapeake Beach	38° 41'	76° 32'	−2 52	−3 04	*0.91	*0.92	1.0	1.1	0.7
2277	Plum Point	38° 37'	76° 31'	−3 30	−3 33	*0.82	*0.83	0.9	1.0	0.6
2279	Long Beach	38° 28'	76° 28'	−4 10	−4 12	*0.90	*0.92	1.0	1.2	0.7
2281	Cove Point	38° 23'	76° 23'	−4 05	−4 27	*1.17	*1.17	1.3	1.5	0.9
	Patuxent River									
2283	Drum Point	38° 19'	76° 25'	−4 51	−5 01	*1.11	*1.08	1.2	1.4	0.9
2285	Solomons Island	38° 19'	76° 27'	−4 45	−4 52	*1.07	*1.08	1.2	1.3	0.9
2287	Broomes Island	38° 25'	76° 33'	−4 29	−4 21	*1.17	*1.17	1.3	1.5	0.9
2289	Benedict	38° 31'	76° 40'	−4 10	−3 56	*1.45	*1.45	1.6	1.9	0.8
2291	Lower Marlboro	38° 39'	76° 41'	−2 59	−2 55	*1.62	*1.63	1.8	2.0	1.3
2293	Nottingham	38° 43'	76° 42'	−2 47	−2 29	*2.29	*2.29	2.5	2.9	1.8
2295	Hills Bridge (Route 4)	38° 48'	76° 43'	−1 07	−0 38	*2.17	*2.17	2.4	2.8	1.7
2297	Cedar Point	38° 18'	76° 22'	−4 54	−5 02	*1.08	*1.08	1.2	1.4	0.9
2299	Point No Point	38° 08'	76° 18'	−5 27	−5 39	*1.18	*1.17	1.3	1.5	0.9
2301	Point Lookout	38° 02'	76° 19'	−5 26	−5 38	*1.11	*1.08	1.2	1.4	0.9
	MD., VA. and DISTRICT OF COLUMBIA Potomac River				on Washington, p.84					
2303	Cornfield Harbor, Md.	38° 04'	76° 22'	−6 29	−7 29	*0.48	*0.47	1.3	1.5	0.8
2305	Lewisetta, Va.	38° 00'	76° 28'	−6 22	−7 19	*0.46	*0.46	1.3	1.5	0.8
2307	Travis Point, Coan River, Va.	38° 00'	76° 28'	−6 18	−6 59	*0.44	*0.41	1.2	1.4	0.7
	Yeocomico River									
2309	Lynch Point, Va.	38° 03'	76° 31'	−6 09	−6 52	*0.48	*0.47	1.3	1.5	0.7
2311	Kinsale, Va.	38° 02'	76° 35'	−6 04	−6 47	*0.44	*0.41	1.2	1.4	0.7
	St. Marys River									
2313	Kitts Point, Md.	38° 06'	76° 25'	−6 36	−7 17	*0.55	*0.53	1.5	1.7	0.8
2315	St. Marys City, Md.	38° 11'	76° 26'	−6 21	−7 02	*0.55	*0.53	1.5	1.7	0.8
2317	Piney Point, Md.	38° 08'	76° 32'	−6 12	−7 10	*0.52	*0.53	1.4	1.6	0.8
2319	Ragged Point, Coles Neck, Va.	38° 08'	76° 37'	−5 53	−6 57	*0.55	*0.53	1.5	1.7	0.9
2321	Coles Point, Va.	38° 09'	76° 38'	−5 55	−6 49	*0.66	*0.64	1.8	2.0	1.0
2323	Leonardtown, Breton Bay, Md.	38° 17'	76° 38'	−5 50	−6 33	*0.62	*0.59	1.7	2.0	0.9
2325	Shipping Point, St. Clements Bay, Md.	38° 16'	76° 42'	−5 45	−6 28	*0.64	*0.64	1.8	2.1	0.9
2327	Mount Holly, Nomini Creek, Va.	38° 06'	76° 44'	−5 09	−5 48	*0.55	*0.55	1.5	1.7	0.8
2329	Colton's Point, Md.	38° 13'	76° 45'	−5 31	−6 38	*0.66	*0.65	1.8	2.0	1.0
	Wicomico River									
2331	Cobb Point Bar Light, Md.	38° 15'	76° 50'	−5 41	−6 22	*0.69	*0.71	1.9	2.2	1.0
2333	Rock Point, Md.	38° 16'	76° 50'	−5 36	−6 17	*0.69	*0.71	1.9	2.2	1.0
2335	Bushwood Wharf, Md.	38° 17'	76° 48'	−5 36	−6 17	*0.69	*0.70	1.9	2.2	1.0
2337	Wicomico Beach, Md.	38° 20'	76° 52'	−5 23	−5 59	*0.66	*0.65	1.8	2.0	1.0
2339	Colonial Beach, Va.	38° 15'	76° 58'	−6 27	−6 12	*0.58	*0.58	1.7	1.9	0.9
2341	Dahlgren, Upper Machodoc Creek, Va.	38° 19'	77° 02'	−5 00	−5 45	*0.59	*0.59	1.6	1.8	0.9
2343	Lower Cedar Point, Md.	38° 20'	76° 59'	−5 05	−5 50	*0.55	*0.53	1.5	1.7	0.8
2345	Mathias Point, Va.	38° 24'	77° 03'	−4 17	−4 50	*0.45	*0.45	1.2	1.4	0.7
2347	Goose Bay, Port Tobacco River, Md.	38° 27'	77° 03'	−4 20	−5 01	*0.55	*0.53	1.5	1.7	0.8
2349	Upper Cedar Point Light, Md.	38° 24'	77° 05'	−4 08	−4 47	*0.43	*0.43	1.2	1.4	0.6
2351	Riverside, Md.	38° 23'	77° 09'	−3 35	−4 19	*0.48	*0.47	1.3	1.5	0.7
2353	Maryland Point Light, Md.	38° 21'	77° 12'	−3 12	−3 38	*0.41	*0.41	1.1	1.3	0.7
2355	Aquia Creek, Va.	38° 25'	77° 21'	−1 46	−2 26	*0.45	*0.45	1.3	1.5	0.7
2357	Clifton Beach, Md.	38° 25'	77° 16'	−2 00	−2 40	*0.43	*0.41	1.1	1.3	0.6
2359	Liverpool Point, Md.	38° 28'	77° 16'	−2 07	−2 33	*0.48	*0.47	1.3	1.5	0.7
2361	Quantico Creek, Va.	38° 31'	77° 17'	−1 04	−1 59	*0.51	*0.47	1.4	1.6	0.8
2363	Deep Point, Mattawoman Creek, Md.	38° 34'	77° 13'	−1 12	−1 38	*0.58	*0.59	1.6	1.6	0.9
2365	High Point, Occoquan Bay, Va.	38° 37'	77° 12'	−1 02	−1 28	*0.57	*0.59	1.6	1.8	0.9
2367	Indian Head, Md.	38° 36'	77° 11'	−0 26	−1 28	*0.66	*0.65	1.8	2.0	1.0
2369	Glymont, Md.	38° 37'	77° 08'	−0 47	−1 13	*0.65	*0.65	1.8	2.1	1.0
2371	Gunston Cove, Va.	38° 40'	77° 08'	−0 28	−0 54	*0.72	*0.71	2.0	2.3	1.1
2373	Marshall Hall, Md.	38° 41'	77° 06'	−0 02	−0 50	*0.83	*0.82	2.3	2.6	1.3
2375	Mount Vernon, Va.	38° 42'	77° 05'	−0 17	−0 42	*0.79	*0.82	2.2	2.5	1.2
2377	Fort Washington, Md.	38° 43'	77° 02'	−0 07	−0 32	*0.88	*0.88	2.4	2.8	1.4
2379	Riverview, Md.	38° 43'	77° 02'	−0 07	−0 32	*0.92	*0.88	2.5	2.9	1.4
2381	Alexandria, Va.	38° 48'	77° 02'	+0 08	−0 17	*1.00	*1.00	2.8	3.2	1.6
2383	Bellevue, D.C.	38° 50'	77° 02'	+0 16	−0 04	*1.03	*1.00	2.8	3.2	1.6
2385	Washington National Airport	38° 51'	77° 02'	+0 16	−0 02	*1.07	*1.06	2.9	3.3	1.7
2387	WASHINGTON, Washington Channel, D.C.	38° 52'	77° 01'		Daily predictions			2.8	3.0	1.6
	Anacostia River									
2389	Anacostia Bridge, D.C.	38° 52'	77° 00'	+0 25	+0 06	*1.06	*1.06	2.9	3.3	1.7
2391	Benning Bridge, D.C.	38° 54'	76° 58'	+0 31	+0 10	*1.07	*1.06	2.9	3.3	1.2

Appendix 13C-1 Nomographs of Significant Wave Height Prediction Curves as Functions of Windspeed, Fetch Length, and Wind Duration



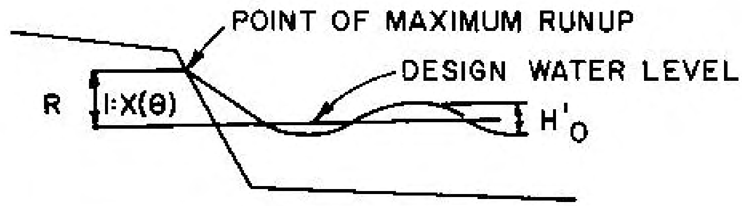
Source:

Appendix 13C-2 Design Breaker Wave



Source:

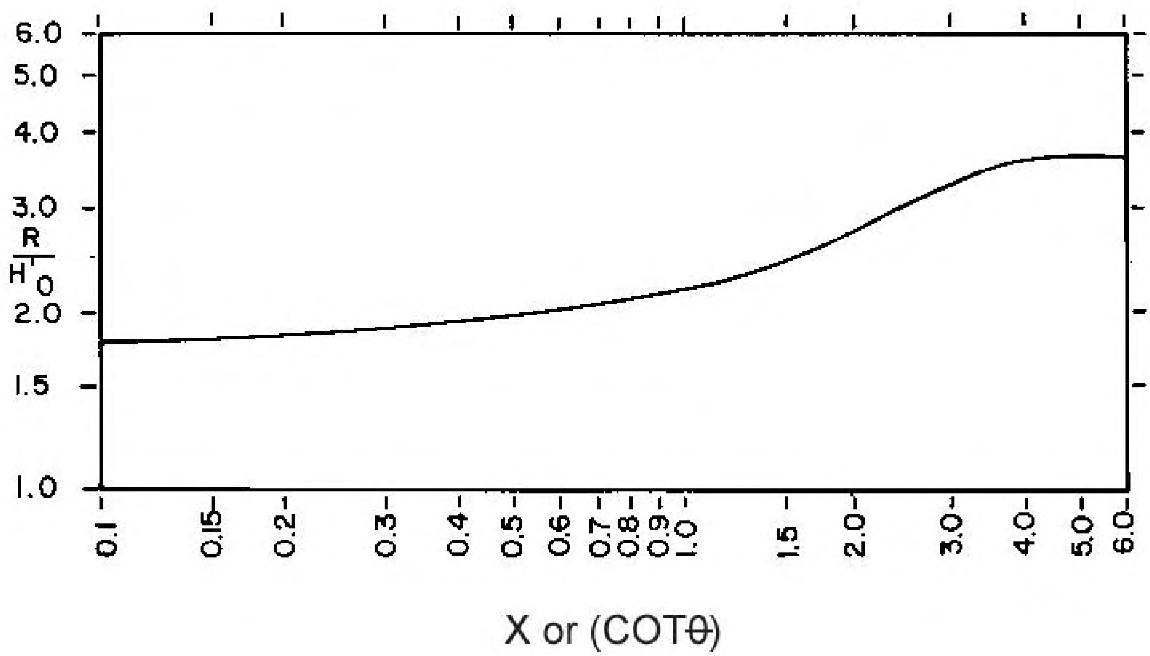
Appendix 13C-3 Wave Run-up on Smooth Impermeable Slope



R = WAVE RUN UP HEIGHT (ft)

H'O = WAVE HEIGHT (ft)

θ = BANK ANGLE WITH THE HORIZONTAL



Source:

Appendix 13C-4 Nomographs for Design of Rock Slope Shore Protection (For Shoal Water)

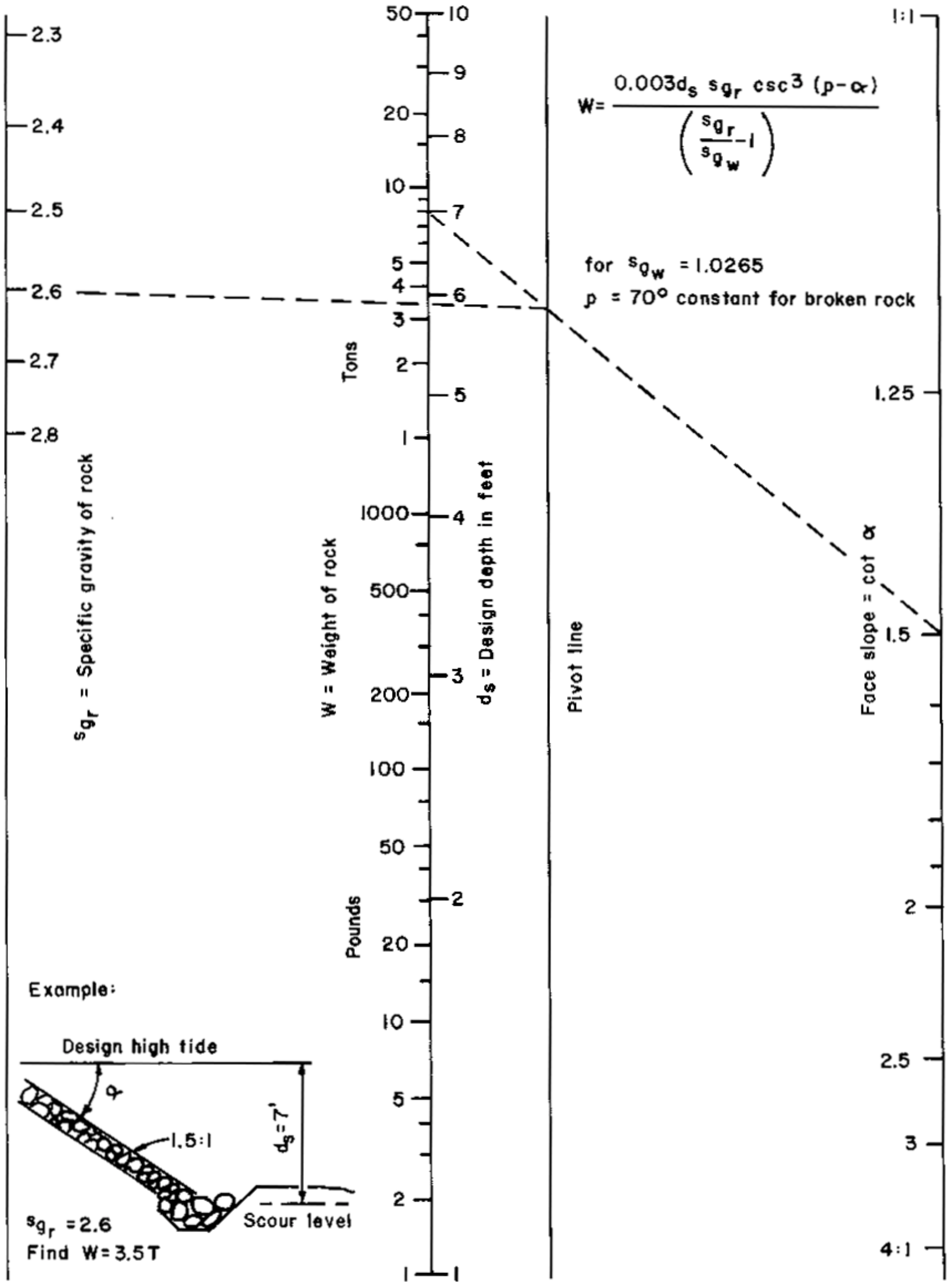
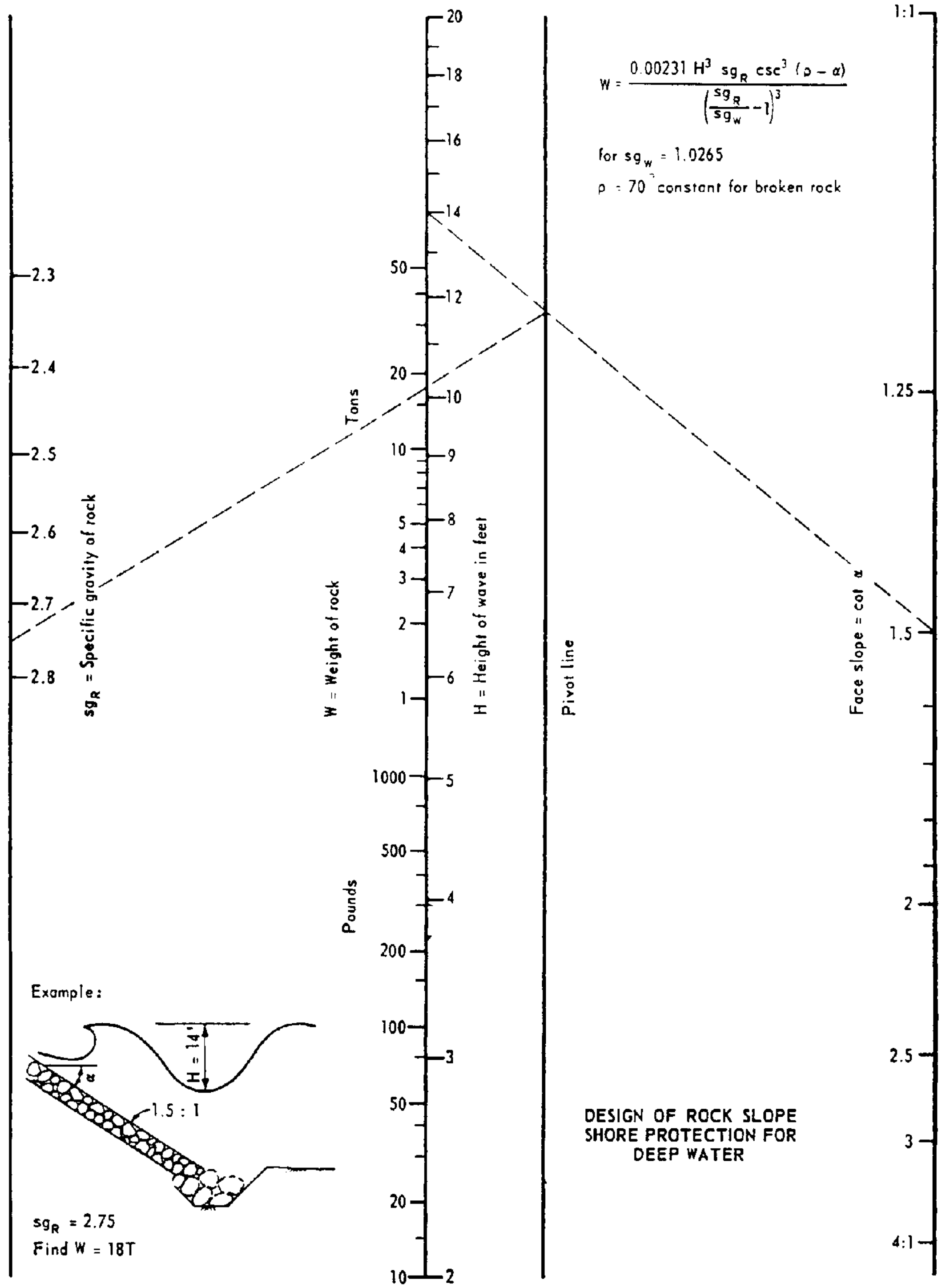


Figure 18-8 Nomographs For Design Of Rock Slope Shore Protection (For Shoal Water)

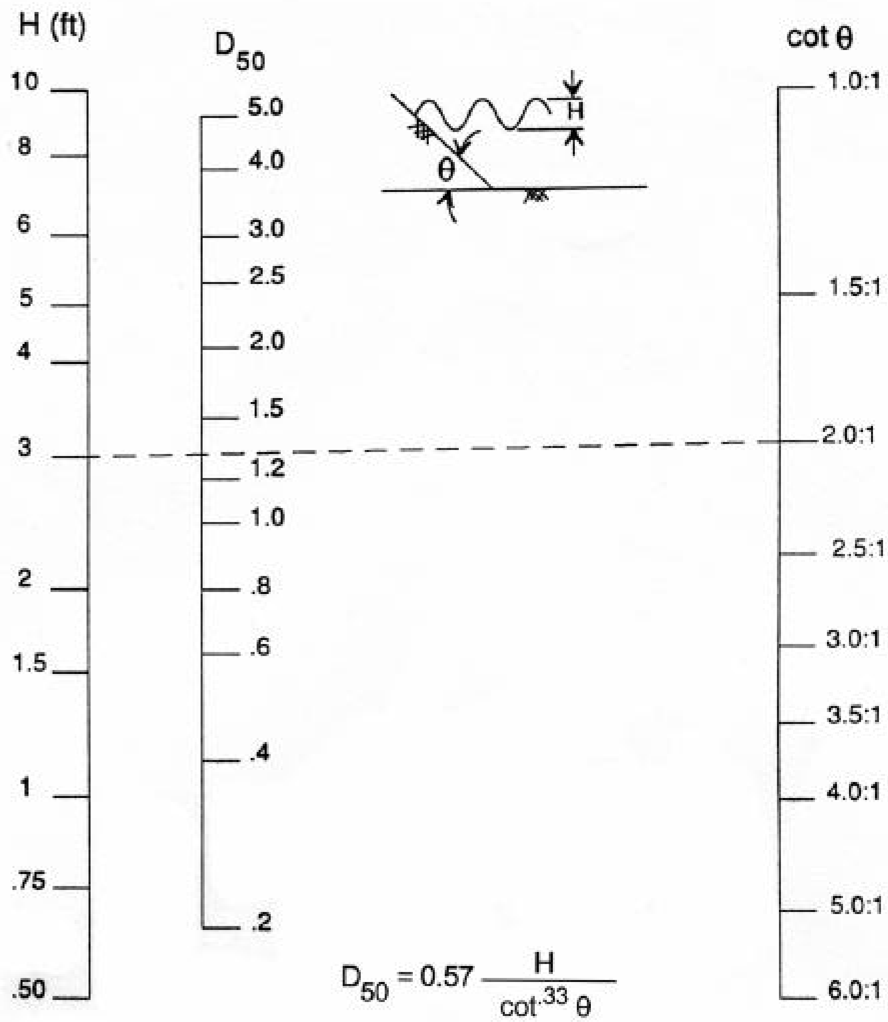
Source: Bank and Shore Protection in California Highway Practice

Appendix 13C-5 Nomographs for Design of Rock Slope Shore Protection (For Deep Water)



Source: Bank and Shore Protection in California Highway Practice

Appendix 13C-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

θ = Bank Angle with Horizontal

Example

Given:	Find:	Solution:
$\cot\theta = 2{:}1$ $H = 3$ ft.	D_{50}	$D_{50} = 1.33$ ft.

Hudson relationship for riprap size required to resist wave erosion

Source: HEC-11

Chapter 14 - Subdivisions

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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 Secondary Roads Division in Richmond, phone (804) 786-2746.

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.

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 country 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 assure 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 Secondary Roads 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.

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 Secondary Roads, 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 secondary roads 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 the Culverts chapter.

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 the Storm Drain Chapter.

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 or, in lieu thereof, 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 the Ditches and Channels Chapter.

14.3.4 Structural Design of Culverts, Storm Drains, and Bridges

Pipes for culverts and storm drains shall comply with the current Drainage Design Memoranda (DDM), the current VDOT Road and Bridge Specifications, and the current Road Designs Manual and 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 for retention basins 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 Secondary Roads Division.

An alternate way of ingress and egress is not necessary when the detention dam is designed and constructed for the temporary storage of floodwater.

Acceptable dams for retention basins shall comply with the applicable General Instructions and Criteria established in Appendix 14D 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 following criteria:

- All dams that are twenty-five feet or greater in height and that create an impoundment capacity of fifteen acre-feet or greater, and
- All dams that are six feet or greater in height and that create an impoundment capacity of fifty acre-feet or greater.

For additional information, see the DCR web site for Dam Safety Programs at http://www.dcr.state.va.us.

14.3.6 Drainage Easements

Drainage easements should be provided from all drainage outfalls to extend to a natural watercourse, defined in the Ditches and Channels Chapter, 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.

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.

Two example copies of subdivision review checklists that are used by some drainage engineers in the plan review process are included as Appendix 14B and 14C. The checklist is an indication of the pertinent data considered in the design and design review of subdivision plans.

14.5 References

Virginia Department of Highways and Transportation Policy Memorandum DPM 7-5.5

Guide for Additions, Abandonments, and Discontinuances – Secondary system of State Highways, VDOT Secondary Roads Division

Virginia Law, Dam Safety Act, Article 2, Chapter 6, Title 10.1

Appendix 14A-1 Definitions and Abbreviations

Definitions:

CBR Tests	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. (Lindeburg reference noted in the reference section)
Detention Basins	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.
Retention Basins	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.

Abbreviations:

AASHTO	American Association of State Highway and Transportation Officials
DCR	Department of Conservation and Recreation
VDOT	Virginia Department of Transportation

Appendix 14B-1 Checklist

CHECKLIST FOR PLANS SUBMITTED TO VDOT FOR APPROVAL

Yes	No	I. Plans
___	___	1. North arrow shown on each sheet
___	___	2. Centerline, __ __ curve data, __ __ stations, __ __ lot lines & numbers shown.
___	___	3. R/W lines including dimensions (radius or radial lines) shown.
___	___	4. Slope easements for fill slopes where slopes exceed proposed right of way limits shown.
___	___	5. Names of adjacent subdivisions and/or property owners denoted.
___	___	6. Subdivision limits shown.
___	___	7. Existing state route numbers and subdivision street names shown.
___	___	8. Approved or proposed street names shown.
___	___	9. Destination of routes and streets shown.
___	___	10. Radii for easements, turn-arounds, cul-de-sac, flares, etc., shown.
___	___	11. Guardrail where needed shown.
___	___	12. Vicinity map shown with location referenced 0.01 mile to nearest intersection.
___	___	13. General notes satisfactory. (Also see sheet 6 of 6)
___	___	14. Location of paved ditches by station shown.
___	___	15. Entrance pipe sizes shown for each lot.
___	___	16. Proposed drainage layout and description shown.
___	___	17. Erosion control stone and type shown.
___	___	18. Channel change and outlet ditches and easements shown.
___	___	19. Zoning shown.
___	___	20. Projected traffic count for each street between intersections shown.

Appendix 14B-1 Checklist

Yes	No	**II. PROFILE & GRADE**
___	___	1. Existing ground line shown.
___	___	2. Grade line (mainline and connections) shown.
___	___	a. % of grade, CG, VC, and VSD shown.
___	___	b. Finished grade elevations (50’ on tangent, 25’ on vertical curve).
___	___	c. Station on profile agrees with plan stations.
___	___	d. Name of streets shown.
___	___	3. Invert elevations on proposed drainage structures shown.
___	___	4. Profile and grades for outfall ditches shown.
___	___	5. Grades for special channels and ditches shown.
___	___	6. Profile for proposed storm sewers shown. (hydraulic grade line to be introduced where applicable).
___	___	7. Sight distance at intersection and entrance satisfactory.

Yes	No	**III. GRADING PLAN**
___	___	1. Existing contours shown.
___	___	2. Proposed grading contours shown.
___	___	3. Contour of computed headwater shown.
___	___	4. Foundation elevations of existing dwellings shown if questionable.
___	___	5. Design layout for drainage system shown.

Yes	No	**IV. TYPICAL SECTIONS** - Geometrics shall be governed by traffic counts. (Minimum TC = no. of lots x 7.5)
___	___	1. Projected traffic count shown for each street between intersections shown.
___	___	2. Centerline shown.
___	___	3. Point of finished grade noted.

Yes	No		Yes	No		Yes	No	
___	___	4. Surface *			Base *			Subbase*
__	__	a. width	__	__	a. width	__	__	a. width
__	__	b. cross slope	__	__	b. cross slope	__	__	b. cross slope
__	__	c. type	__	__	c. type	__	__	c. type
__	__	d. depth	__	__	d. depth	__	__	d. depth

Appendix 14B-1 Checklist

(* According to secondary roads current criteria.)

___	___	5. Shoulders			Ditches			Slopes
__	__	a. cut	__	__	a. width	__	__	a. ratio
__	__	b. fill	__	__	b. depth	__	__	b. rounding
__	__	c. slope ratio	__	__	c. slope ratio	__	__	c. seeding

Same as above for each different road typical section required.

___ ___ 6. Same as above for each different road typical section required.

___ ___ 7. Channel changes, outlet and inlet ditches shown.

___	___	a. bottom width	___	___	d. length
___	___	b. side slopes	___	___	e. If lined, show depth and type of lining
___	___	c. depth			

___ ___ 8. Special design typical sections included.

___ ___ 9. Stations to which typical sections apply shown under typical section.

Yes **No** **V. DRAINAGE GENERAL**

___ ___ 1. Existing or proposed drainage easements shown to water course.

___ ___ 2. Direction of flow by arrows for pipe and ditches shown.

___ ___ 3. Drainage areas in acres (Supported by outlined contour map).

___ ___ 4. Pipe sizes to be in accordance with VDOT standard sizes.

___ ___ 5. Items not VDOT standard are to be detailed.

___ ___ 6. Type of pipe to be installed (on and off right of way noted).

___ ___ a. Design cover shown

___ ___ 7. Design computations computed according to current VDOT criteria included.

Yes **No** **VI. DRAINAGE DESIGN**

1. Hydrologic breakdown.

___ ___ a. Reference to material used if not VDOT

b. Rational method: Q = CIA

___ ___ 1. "C" – breakdown of planned development and related areas shown.

Appendix 14B-1 Checklist

___ ___ 2. "I" – (a) overland flow (slope, type cover included)
(b) channel flow (typical section lining, slope, and included velocity)
___ ___ 3. "A" – DA in acres (supported by outlined contour map)
___ ___ c. Other methods – (supported by factors used in computation of Q)

2. Hydraulic breakdown

a. Open channels (capacity)
___ ___ 1. Computation to support design included.
___ ___ 2. Capacity (lined, unlined, and combination lining)
___ ___ (a) depth
___ ___ (b) type of lining
___ ___ (c) velocity of flow
b. Open channels (erosion control)
___ ___ 1. Computation supporting lined ditches included (See also sheet 6 of 6)
c. Curb and gutter (computation to support the following):
___ ___ 1. Depth and Spread of water in gutter.
___ ___ 2. Length of slots and placements of inlets.
___ ___ 3. Type of inlet proposed.
d. Storm sewers (computation to support the following) to include:
___ ___ 1. Velocity of flow.
___ ___ 2. Minimum velocity of 3 fps maintained.
___ ___ 3. Elevation of the hydraulic grade line at key points (drop inlets, manholes, etc.) noted.
___ ___ 4. Size of pipe.
e. Special design structures (computations to support the following include):
___ ___ 1. Special design entrance for culverts.
___ ___ 2. Flumes (Q-dn, vn, freeboard)
___ ___ 3. Energy dissipators.
___ ___ 4. Special channels (curve, mitered, channel confluences, etc.)
f. Culverts
___ ___ 1. Invert ___ ___ 6. Outlet velocity
___ ___ 2. Length ___ ___ 7. Erosion control &
___ ___ 3. Type ___ ___ type (where outlet
___ ___ 4. Headwater velocity exceeds 6 fps)
depth ___ ___ 8. Size
___ ___ 5. End ___ ___
treatment
g. Flood control structures
___ ___ 1. When proposed, consult District Drainage Engineer

<u>Yes</u> **<u>No</u>** **<u>VII. DRAINAGE OUTFALLS</u>**

___ ___ 1. Has been field checked for location and found satisfactory?

___ ___ 2. Is the length of easement shown satisfactory?

Appendix 14B-1 Checklist

___ ___ 3. Is width of easement shown satisfactory for maintenance?

___ ___ 4. Is easement shown on plat?

___ ___ 5. Is erosion a problem?
___ ___ a. Will this development create an erosion problem?

___ ___ 6. Is siltation now a problem?
___ ___ a. Will this development create a siltation problem?

___ ___ 7. Are there obstructions in or is downstream maintained by the department?
___ ___ a. Are there existing problems?
___ ___ b. Will this development cause problems?

___ ___ 8. Is there drainage being diverted to this outfall?

___ ___ 9. Are there other developments (future or proposed) which affect outfall?

___ ___ 10. Are there any proposed road projects which could adversely affect or be affected by this proposed development?

<u>Yes</u> **<u>No</u>** **<u>VIII. GENERAL NOTES</u>**

___ ___ A.. Are the following notes included as a part of the plans?
If not, should they be included?
Which ones? ____

1. VDOT approval of subdivision road plans does not preclude the right to add additional facilities such as seeding, paving, sediment control items, etc., as may be deemed necessary by the Department prior to the acceptance of such roads in order to limit siltation and pollution of nearby lakes, ponds, streams, and adjacent property.

   a. Are provisions included in plans to protect adjacent property from siltation?

2. The Department's approval of these plans expires three years from data approval.

<u>Yes</u> **<u>No</u>** **<u>IX. DESIGN CRITERIA</u>**

A. Drainage Design

___ ___ 1. Are spans, culverts, storms sewers, and channel capacity being computed on a minimum 10-year flood frequency?

___ ___ 2. Are the size of curb drop inlet slots being computed with the proper rainfall?

Appendix 14B-1 Checklist

___ ___ 3. Is the following policy being followed in computing and proposing paved ditches:

a. Ditches shall be paved where computed velocities exceed the appropriate allowable velocity unless supporting soils or channel lining analysis indicate greater velocities can be tolerated.

b. Ditches shall be paved when conditions and unusual situations are encountered and/or directed by the Highway Engineer in writing.

c. Additional paved ditches shall be provided where necessary to meet field conditions to prevent erosion as directed by the engineer in writing.

d. Paved ditches may be deleted as deemed necessary as directed by the engineer in writing.

Appendix 14C-1 Land Development Manual

FORWARD

The Virginia Department of Transportation (VDOT) has for many years worked with local jurisdictions to review site plans, evaluate traffic impacts, and recommend roadway improvements needed to serve proposed development sites. This manual is a guide for site plan and subdivision reviews by VDOT staff and the staffs of local jurisdictions and can also be used by engineering consultants to ensure that all the information necessary for a review has been provided. This is a compilation of selected excerpts from Volume I of the three volume VDOT Land Development guide.

Careful reviews of proposed development plans are important because in the past the traffic impacts of new developments have been very costly for both VDOT and the local jurisdictions The General Assembly has in recent years passed legislation for local jurisdictions, through their zoning ordinances, to allow voluntary proffers of “off site” improvements by developers to alleviate the traffic impacts caused by their developments. (Code of Virginia, Section 15.491.2:1.)

VDOT must use all available transportation funds for maintaining or improving existing highways. The transportation needs far exceed the available funding and usually it takes several years for transportation improvements to be programmed and constructed. Also, the costs for maintenance of our transportation facilities have risen rapidly during the past several years.

Because of this, VDOT’s limited funds cannot be used to correct transportation problems created by new developments. Therefore, it becomes extremely important that the Department’s review of development plans include a thorough analysis of the traffic impacts and recommendations for improvements to alleviate those impacts.

This document is intended as a guideline. It does not constitute a regulation or requirement. If subsequent legislation is enacted that conflicts with any of the guidelines presented in this publication, the legislative provisions shall govern.

In the site plan review process, VDOT personnel examine the site plan to determine if development plans provide designs adequate to accommodate trips generated by the proposed site without adversely impacting state maintained roads. VDOT’s comments and recommendations on the site plan are transmitted to the local jurisdiction, which has the authority to approve the site development. As part of the site development proposal, a traffic impact study may be required (by the local jurisdiction or VDOT) to describe how the traffic generated by the site will be served by the existing or future road network. This study of the site must analyze the traffic impacts of the fully developed site and provide solutions that will be implemented to accommodate the site traffic.

Since there are numerous land development activities throughout Virginia, VDOT has a substantial number of site plan reviews under way at any given time. This manual provides guidelines to ensure that the site plan reviews are both thorough and uniform throughout the state.

This procedural guide for review of site plans and subdivisions is presented in the following sections:

- The Roles of VDOT Offices: This section delineates the responsibilities of the Residency, the District and, the Central Offices in the site plan review process.
.
- Review of Preliminary Development Plans/Rezoning Applications: This section outlines the information required for review of preliminary development plans that are submitted with rezoning applications.

- Subdivision and Site Plan Final Reviews: This section gives a step by step procedure for review of the final subdivision and site plans. A checklist is included to help ensure a thorough review.
.
- Guideline for a Traffic Impact Study: These guidelines are provided for use in reviewing traffic impact studies of proposed developments. This outline should aid in improving the quality and uniformity of site plan reviews throughout the state.

- VDOT Policies and Standards: This section is a reference for VDOT policies and standards that are applicable to subdivision and site plan reviews.

- Glossary: This section presents the definitions of certain words, phrases, and abbreviations as used throughout the manual.

ROLES OF VDOT OFFICES IN SITE PLAN REVIEWS

Residency Offices

The Virginia Department of Transportation has two roles in the review of site plans: regulator and advisor. The regulator role includes (a) issuing permits for work performed within VDOT's right-of-way (including entrances to state highways), and (b) regulating subdivision street development for streets to be included in the secondary system. The advisor role includes advising local governments on the transportation impacts of proposed developments that the government body must approve as the land use regulator. In all VDOT districts, except Northern Virginia, the focal point for site plan review is the Residency Office. All communications should be between the residency office and the local jurisdiction.

As part of maintaining a continuing liaison with local governments, the residency office should work cooperatively with the local government staff in the development of the transportation elements in the comprehensive plan. Additionally, the residency office assists the locality by reviewing proposed rezonings and new development site plans for conformity to the comprehensive plan and VDOT policies.

The residency staff person performing site plan reviews should be familiar with Title 15 of the Code of Virginia, which enables local jurisdictions to develop their comprehensive plan and zoning ordinances. The reviewer should also be familiar with the transportation statute in Title 33 of the Code of Virginia. These and other relevant statutes are contained in the publication Highway Laws of Virginia issued by the Department of Transportation for use by VDOT personnel.

The VDOT residency office serves as the clearing-house for all site plans and traffic impact studies, except in Northern Virginia where the site plans are sent directly to the Northern Virginia District office. Unless otherwise agreed upon, any site plans sent directly to the district or central office should be returned to the appropriate residency office for review. In performing site plan reviews, the reviewer must apply the appropriate VDOT policies or standards to ensure that the plan is acceptable. The following are activities or tasks associated with the residency office reviews of site plans.

1. Initiate and maintain a file on all site plans that are submitted.
2. Check the site plan for completeness using the appropriate checklist (completeness includes a determination of the adequacy and accuracy of the plan).
3. Request additional information or data from submitter on incomplete or deficient plans.
4. After reviewing the site plan, prepare written review comments for return to the submitter or forward the plan to the district office for additional review. The factors to consider in this determination include:
    a. The capabilities of the residency staff
    b. The size and density of the development
    c. The level-of-service on the existing or proposed highways that will provide access
    d. The complexity of the drainage system design
5. A traffic impact study on the site is to be reviewed. The residency compiles and transmits all review comments to the county. The residency also must coordinate any future plan review activities with the county and other VDOT offices.

District Offices

Sections within the district offices, primarily Traffic Engineering and Location and Design, assist the residency in the review of site plans. The reviewing section(s) will prepare written comments for the residency to use in responding to the submitter. If more than one section reviews the submission, they should coordinate their response to the residency.

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The district reviewing section will take the steps necessary to provide a thorough review of the site plan. These steps should include the following:

1. Perform the site plan review and prepare written comments.
2. Determine if the site plan should be forwarded to the central office for a limited or comprehensive review. The factors considered in this determination include:
   a. The size of the development
   b. The stage of the review (preliminary site plans at rezoning vs. final site plans, etcetera)
   c. The level of service on the existing or proposed highways that will provide access
   d. Impact on an Interstate highway or any limited access facility
   e. The complexity of the road and drainage designs
   f. The development impacts on major planned roadway improvements
   g. A policy change is needed
   h. The district staff has questions on the plan
3. If the district office requests a central office review, the following should be considered:
   a. For a comprehensive review, forward the plan to the Location and Design Division, indicate the divisions that should review the plan, flag issues of special concern, and give the review a deadline.
   b. For a limited review, forward the plan to the division that should review the plan and flag issues of particular concern. Send a copy of the letter to the Location and Design Division.
4. When reviewing a traffic impact study:
   a. Check for adherence to the guidelines for a traffic impact study
   b. If the study does not satisfy the guidelines, return it to the originating Residency
   c. The district reviewing section will determine if the study should be reviewed by the Transportation Planning Division. The factors to be considered are outlined in item 2 above for the district office
   d. Perform the review and prepare written comments or forward the review to the Transportation Planning Division, flagging issues of concern and giving a review deadline
5. When the Central Office review is complete, the district office should consolidate all comments and transmit them to the residency for responding to the submitter.

Central Office

For comprehensive plan reviews, the Location and Design Division will coordinate the review with the related divisions as requested by the district office. The Location and Design Division is responsible for forwarding the plans to the appropriate divisions, compiling the review comments from the divisions, and forwarding the comments to the district offices.

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For a limited review in the central office, the reviewing division receives the plan from the district office and reviews the plan using the Site Plan Review Checklist, and other references deemed appropriate by the division, and prepares a written response that is forwarded to the district. The areas of site plan responsibility of the divisions are as follows:

Location and Design:

(a) reviews road geometries
(b) reviews drainage designs
(c) examines how the proposed site may impact planned road projects

Transportation Planning Division:

(a) reviews plans for traffic impact on existing roads and future transportation improvements (b) determines adequacy of traffic impact studies, especially the capacity analysis
(c) reviews and develops recommendations for improvements
(d) reviews the developers proffers for consistency with long range plans

Traffic Engineering Division:

(a) evaluates unusual proposals for operational and safety functions
(b) reviews entrance designs
(c) highway crossovers
(d) standards for unusual traffic control devices

Secondary Roads: evaluates unusual proposals and determines compliance with the subdivision street requirements.

Maintenance Division: has final approval authority for permits.

Materials Division:

(a) occasionally reviews pavement structures
(b) reviews the geotechnical plans of roadway dams

Environmental Quality Division:

(a) provides information on environmental problems with proffers
(b) answers questions regarding plantings on VDOT right-of-way
(c) advertising signs off of right-of-way

On rare occasions, other divisions may be requested to review a particular aspect of the site plan that involves their areas of responsibility.

Appendix 14C-1 Land Development Manual

SUBDIVISION - SITE PLAN REVIEWS

General

This section outlines VDOT's review process for subdivisions and site plans for commercial or office complexes. This review process is based on the Subdivision Street Requirements as adopted by the Commonwealth Transportation Board, and Minimum Standards of Entrances to State Highways as approved by the Commissioner of Transportation. The purpose of this section is to provide guidance and to ensure uniformity in the review of subdivision and final site plans.

Subdivision plans and site plans must be reviewed at the same level of detail as VDOT's construction plans for two reasons: safety and adequacy of the proposed design. On any connections that are made to existing roads, it is VDOT's responsibility to ensure that the safety of the traveling public is not diminished. Sight distances must meet VDOT's standard and any roadway widenings must be made in such a manner that the implementation and the final product will not adversely affect the roadway's safety or create a hazard. It is, therefore, important that the entrances meet VDOT's standards and specifications as delineated in Minimum Standards of Entrances to State Highways, and Road and Bridge Specifications.

Subdivision and site plans are, in fact, final construction plans after they are approved by the VDOT Residency Office. Therefore, any errors that are overlooked in the subdivision or final site plan review may result in sub-standard construction or safety problems that will require correction, frequently through the use of highway maintenance funds.

Pre-Review Process

In order to ensure that a timely and efficient review is made of submitted subdivision and site plans, a three-part pre-review process has been established.

1. A subdivision and site plan should be submitted by the developer with a an inventory checklist as part of the review package. This inventory checklist is a certification by the developer that the plan has been designed in accordance with VDOT standards and specifications. Plans submitted with inventory checklists should be given priority.
2. All submittals should be reviewed by the local jurisdiction staff before they are submitted to VDOT. Any minor discrepancies should be noted. Any plans with major errors should be returned to the developer/submitter for correction.
3. The first step in VDOT's review is to verify the information on the inventory checklist. If the information is complete, VDOT and the local jurisdiction, where applicable, will proceed with a thorough review of the subdivision/site plan as required in the Subdivision Street Requirement manual.

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DRAINAGE DESIGN REVIEW

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
  3. Check for proper treatment at ends of drainage facilities (riprap, paved ditches, etcetera)
  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

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DRAINAGE PLAN SUBMITTAL CHECKLIST

Project Name_______________________________________ Date____________ __________

Developer/Owner_______________________________________

Address_______________________________________ Zip_____________ __________

	GENERAL INFORMATION			
	Plan Sheet to Include:	INCLUDED		COMMENTS
		YES	NO	
1	Project Name.			
2	Date of Plan.			
3	Existing and/or proposed dams, detention basins & any extrinsic structures.			
4	Grading plan: existing contours, proposed contours, finished floor elevations, design layout for drainage system.			
5	Legend detailing graphic descriptions for all road items, drainage & utility items shown.			
6	Show total acreage, current zoning and proposed zoning by acres.			
7	Road classification schedule with pavement designs.			

Project Name_______________________________________

PROFILE AND GRADE

	Plan Sheet to Include:	INCLUDED		COMMENTS
		YES	NO	
1	K max < 167			

Project Name____ Page 2 of 5

HYDRAULICS

	To Include:	INCLUDED		COMMENTS
		YES	NO	
1	Detailed drainage area map defining corresponding sub-areas used for computations showing centerline stationing at 100' intervals, intersections, PC's, PT's, etc., & the proposed storm sewer layout.			
2	Reference to the hydrologic methodology used including supporting data used in computation of "Q".			
3	Complete design computations per the following criteria: A. Spans, culverts, & open channel capacity computations for a 10-year & 100-year frequency storm. B. Closed storm sewer system capacity computations for a 10-year frequency storm to include flow velocity, size of pipe, type of pipe & hydraulic grade line elevation at all junction locations. C. Culvert computations showing size, inverts, length type of pipe, design cover, headwater & tailwater depth/elevation, and treatment & outlet velocity. D. Open channel computations showing the type of lining, design depth, velocity & location of paved ditches (by stationing). E. Size of curb drop inlets slots. Curb & gutter computations showing spread depth of water in gutter, length of slots, placement of inlets (referenced by stationing), & type of inlet proposed. F. Support computations for all special design structures including culvert entrances, flumes, energy dissipators, & channels.			
4	Detailed description of all proposed storm sewer structures.			
5	Graphic details for all non-standard drainage facilities.			
6	Directions of drainage flow for streets, storm sewer, valley gutters, subdrains, etc.			

	Field location for all natural watercourses or drainageways affected by construction, including direction of flow.			
8	All existing & Proposed storm drainage systems in plan & profile views.			
9	Field located limits of 100-year flood zones & backwater inundation.			
10	Existing and/or proposed VDOT drainage easements dimensioned & labeled.			
11	Driveway entrance culvert sizing computations for each lot.			
12	Show all storm sewers and appurtenances by type and number. Station on plan must conform to stations shown on profile. Show top and invert elevation on each structure. Tabulation in the plan view may be permitted.			
13	Provide the contributing area, in acres, at all culvert, curb inlets and other entrances exclusive of driveway pipes.			
14	Show type or class of pipe to be installed both in right-of-way and outside of right-of-way.			
15	Dams-all structures, pipes, stabilization, calculations, design and appropriate data.			
16	Retention-detention ponds, show all structures, add note: VDOT will not be responsible for repair of pond or structures. Show fencing, if any is to be used.			
17	Show the size of all driveway entrance culverts, i.e. 15" or 18" according to computed size.			
18	Show paved ditches & easements at toe of fills.			
19	Stormwater management fact sheet.			
20	Access to stormwater management facility within a proposed easement provided.			
21	Show all existing or proposed drainage easements.			
22	Show all proposed/planned building pads.			
23	Show direction of drainage flow for streets, storm sewer, valley gutters, subdrains and the like, and all existing streams.			
24	Show the location of all streams or drainageway related to construction.			
25	Show proposed drainage ditches for full length in all easements, furnish detailed typical section and type of stabilization to be provided.			

26	Show proposed stream relocations, show existing and proposed locations, furnish detailed typical section and type of stabilization to be provided.			
27	Show protection for erosion control and any notes or information necessary to explain intent and purpose.			
28	Erosion control sheet completed in its entirety. Symbols shall conform to the Virginia uniform coding system.			
29	Calculations for drainage, stormwater management ditches, pipes, structures, retention, detention, ponds and outfall ditches.			
30	Provide profiles on outfall ditches and pipe and indicate natural drainage.			
31	10-yr. and 100-yr. elevations shown on detention ponds with distance to property line.			
32	Existing storm drainage system & proposed major drainage structures. Drainage easement, list use if other than drainage.			

Appendix 14C-1 Land Development Manual

Project Name____ Page 5 of 5

EROSION CONTROL

	Plan Sheet to Include:	INCLUDED		COMMENTS
		YES	NO	
1	Erosion control plan when disturbing over 10,000 sq. ft. within existing VDOT right-of-way			

Notes:

(1.) The developer is responsible for supplying sufficient information for the Department to determine entrance and road design features to adequately serve the existing roadway and the proposed development.

(2.) Subdivision plans shall be designed in accordance with VDOT's <u>Subdivision Street Requirements</u> and <u>Road Design Manual.</u>

(3.) All commercial entrances must meet VDOT standards and specifications as designated in <u>Minimum Standards of Entrances To State Highways.</u>

(4.) The submission is to include 2 copies of the plans for review. An additional copy of the plan is required once final approval is received.

(5.) Explain all "no" answers if information is not included in the site plan.

<u>Certification</u>

I certify that the above stated information is included in the attached plans.

____

Engineer's Signature

____

Date

Appendix 14D-1 General Instructions and Criteria Pertaining to Use of Highway Embankment as Dams

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

Appendix 14D-1 General Instructions and Criteria Pertaining to Use of Highway Embankment as Dams

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.

Appendix 14D-1 General Instructions and Criteria Pertaining to Use of Highway Embankment as Dams

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 it 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 following criteria will be followed:

      1. (Class A) Where failure may damage minor buildings and roads. The freeboard hydrograph will be developed for a minimum six-hour precipitation which varies between 8 and 9 inches for the State.

      2. (Class B) Where failure may damage homes, major buildings, highways, etc. The freeboard hydrograph will be developed for a minimum six-hour precipitation which varies between 14 and 16 inches for the State.

Appendix 14D-1 General Instructions and Criteria Pertaining to Use of Highway Embankment as Dams

3. (Class C) Where failure may cause the loss of life, serious damage to homes, industrial buildings, major transportation or important public utilities .The freeboard hydrograph will be developed for a minimum six-hour precipitation which varies between 27 and 29 inches for the State.

   The above is general and subject to variation depending upon the attendant circumstances.

   The above is the basic criteria, however, 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 15 – Drainage Design Memorandums

TABLE OF CONTENTS

DDM1
Drainage Instructions

VIRGINIA DEPARTMENT OF TRANSPORTATION

LOCATION AND DESIGN DIVISION

DRAINAGE DESIGN MEMORANDUM

GENERAL SUBJECT: DRAINAGE INSTRUCTIONS	NUMBER: DDM 1.1
SPECIFIC SUBJECT: CULVERTS, STORM SEWERS AND MISC. DRAINAGE ITEMS	DATE: September 1, 2005
	SUPERSEDES: DDM1, IIM-LD-01 (D) 223, HDA-01-02, HDA-03-02, HDA-04-01, Road Design Manual Section 2E-2
ADMINISTRATOR APPROVAL: R. T. Mills State Hydraulics Engineer	

TYPE OF STRUCTURE SELECTION – Culverts

- 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 metal 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

  - Structural plate pipe arch culverts:

    - concentrate considerable pressure in the haunch area.

- require near perfect backfill and compaction in haunch area during construction.
- 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

STORM DRAINS

- On projects that involve some storm drains and some cross drains (especially in areas which allow optional pipe material) the pipe description shall specify concrete pipe if the pipe is, or could become, a component part of a storm drain system under a roadway classification that requires concrete pipe only for storm drain systems.

CULVERT END TREATMENT

- End Treatments will be provided, regardless of the highway classification, on:

  o all culverts conveying a live stream

  o all circular culverts with a diameter of 48 inch or greater

  o all culverts of an arch or elliptical shape with a hydraulic opening of 12 sq. ft. or greater.

  o all multiple line structures with a combined hydraulic opening of 12 sq. ft. or greater.

- Types of End Treatments:

  o Standard endwall

  o Modified endwall or special design endwall

  o Special Design Concrete Slab End Treatment, Special Design Drawing No. isd-2045 and msd-2045

  o Other types of end treatment with a foundation of sufficient width and depth to protect the culvert bedding material from seepage.

PIPE IN HIGH FILLS

- Concrete pipe with a height of cover exceeding 30 feet 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.

SKEWED BOX CULVERT DETAILS

- Where box culverts are to be constructed on a skew, 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 five (5) degree increment.

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.

SMALL BOX CULVERTS

- Box culverts with heights and widths less than 4 feet should be avoided due to concerns with inspection and maintenance. If a box culvert with a height or width less than 4 feet 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.

PILE FOUNDATION DESIGN FOR BOX CULVERTS

- When the Materials 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.

JACKING PIPE

- There are certain cases where it is not feasible to install pipe through the existing embankment by the usual open trench method. The alternative is to jack the pipe through the embankment. The Drainage Designer is to specify the pipe as "Jacked Pipe" on the plans. The contractor then has the option of tunneling or boring the pipe in accordance with Section 302.03 of VDOT's Road and Bridge Specifications. Foundation information shall be requested for any size pipe that is to be jacked or bored in order to determine the feasibility of this installation method.

- Concrete pipe is normally employed in a jacking operation. In some cases, it is preferred to jack a Smooth Wall Steel Pipe (See Sec. 232 of VDOT'S Road and Bridge Specifications) through the embankment as the encasement structure. A concrete (or occasionally metal) pipe is then threaded inside of the steel pipe to act as the carrier for the stormwater. The void between the two pipes is to be pressure grouted in accordance with Section 302.03 of the VDOT Road and Bridge Specifications.

- On some specific occasions, it has been deemed appropriate to install only the Smooth Wall Steel Pipe and to let it serve as the drainage pipe. THIS IS NOT TO BE CONSIDERED A UNIVERSALLY ACCEPTABLE PRACTICE.

- The use of Smooth Wall Steel Pipe as the drainage pipe must conform to Notes 1, 2 and 4 for Table A of "Allowable Types of Pipe" as shown in St'd. PC-1. Any deviation from this policy must be approved by the State Location and Design Engineer and the District Materials Engineer.

FISH PASSAGE

- 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 inches minimum below the streambed.

  - Maintain a depth, width and velocity of flow in the culvert that matches the depth, width and velocity of flow in the natural channel adjacent to the culvert.

NON-STANDARD ROADSIDE DITCHES

- Safety, appearance, and economy necessitate that non-standard roadside ditches not be used or their use be minimized to the greatest extent reasonable for all highway projects.

- 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:

  - Using an enclosed drainage system, where economically feasible, in order to eliminate the need for the non-standard roadside ditch or channel.

  - Minimizing the size and depth of the proposed non-standard roadside ditch or channel.

  - Flattening the front slope (the slope adjacent to the highway shoulder) of the non-standard roadside ditch or channel. Where right of way is available, or can reasonably be obtained, the front slope of the non-standard 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 roadside ditches or channels as far from the proposed highway shoulder as the existing or proposed right of way will reasonably allow.

BERM/TOE DITCH LOCATIONS

- Except where severe right-of-way limitations exist, a minimum of 5 feet is to be provided between the end of the cut slope round-off and the front slope of a berm ditch. 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 feet is to be provided between the toe of the fill slope and the front slope of the ditch. Additional right of way is to be obtained for construction and maintenance of the ditch.

CONCRETE PIPE ON RADIUS

- Concrete 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.
- 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

  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.

GUIDELINES FOR MINIMUM RADII – CONCRETE PIPE			
Pipe Diameter	MINIMUM RADIUS BASED ON 8 foot PIPE JOINT LENGTH		
	Open Joint*	Full Bevel	Full Bevel Plus Open Joint *
Inch	Feet	Feet	Feet
12	240	95	70
15	280	125	90
18	295	125	90
21	340	125	90
24	350	120	90
27	390	120	90
30	395	120	90
33	400	120	90
36	405	120	90
42	410	120	90
48	530	120	95
54	505	120	100
60	500	120	100
66	515	120	100
72	560	120	100
78	570	120	100
84	650	120	100
90	655	120	100
96	730	120	100
108	615	120	100

* Maximum of 25% of spigot length

PIPE ON STEEP SLOPES

- Concrete Pipe
    - The gradient of concrete pipe should be limited to no more than 16%. If a grade of greater than 16% is required, the design should incorporate a "step down" manhole system (See in this DDM "Step Down Manhole" section) or 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
    - o Corrugated pipe may be used on steep slopes in situations similar to those where shoulder slot inlets are propsed. 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 Standard PC-1 of the VDOT Road and Bridge Standards.
    - o See VDOT's 2001 Road and Bridge Standard PI-1, Sheet 104.37 for Anchor Details for Corrugated Pipe.
- Step Down Manholes – In situations where the pipe grade needs to be more than 16%, structures at the upper end of the pipe system may be made deeper to help reduce the gradient. Additional "step down" manholes may also be added to the system to reduce the gradient. 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. This protection should be considered for use if the vertical difference between the inverts of the inlet pipe and outlet pipe is 4 feet and **<u>any</u>** one of the following factors are present or anticipated:
    - o The flow is expected to carry any abrasive material.
    - o Continuous live flow or live flow lasting several days may be expected.
    - o The size of the main pipes is 48" or greater in diameter (for circular pipe) or the hydraulic opening is 12 sq. ft. or greater (for shapes other than circular).
- 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.

EXISTING DRAINAGE 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, 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 104.05 of the Road & Bridge Specifications) or may be included in the cost of Regular Excavation. (See IIM-LD-110 & General Note D-10)

- 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, Cu. Yds." 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. The pipe location/structure number should be shown in the Drainage Summary and the pipe size should be noted in the remarks column.

  EXAMPLE:

D R A I N A G E S U M M A R Y	
FLOWABLE BACKFILL	REMARKS
C.Y.	
25	48 inch concrete pipe to be abandoned

- General Note D-13 (See IIM LD-110) is to be included on the General Note Sheet in all applicable project assemblies.

PROTECTIVE COATING FOR CULVERTS, STORM SEWERS AND CONCRETE 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.

REQUESTING CULVERT DATA AND MATERIALS DIVISION RECOMMENDATIONS

- The Drainage Designer will determine locations where foundation investigation and other culvert data/recommendations are required.

- Foundation, 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.

FOUNDATION INVESTIGATION

- Foundation data will be requested for all culvert installations with a diameter or span of 36 inch 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. Foundation data may be requested for culvert installations with smaller spans if deemed necessary. Foundation data should be requested on all pipes of any size that are to be bored or jacked.

- Foundation 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 305.01(a) of the Materials Division Manual of Instructions. For large basins, more than one (1) boring for the dam and one (1) boring for the area of the basin may be needed. The number and locations of the borings are to be determined/requested by the Drainage Designer.

- The existing foundation 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.

- Locations that require a pH and resistivity analysis as well as an evaluation of the abrasive bedload potential should be noted on the plans that are used to request culvert foundation information from the Materials Division. The pH and resistivity analysis of the soil and water and the potential for an abrasive bed load are to be requested for each culvert location allowing a metal culvert where **any** of the following conditions exist:

    - Diameter or span of 36 inches or greater (including multiple pipe installations)
    - Culvert is to be installed in a live stream environment
    - Culvert is to be installed in an area of known premature pipe failure

- At each location where a foundation investigation is requested for pipe or box culvert installations, the Materials Division is to 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

- In areas of known premature pipe failure, the pH and resistivity analysis is to be requested for any type of proposed pipe material.

PIPE CAMBER

- Construction of longitudinal camber in a pipeline shall be considered when **all** of the following conditions are present:
    - o Grade of the pipe is less than 0.5%
    - o Fills (not height of cover) greater than 20 feet
    - o Diameter or span 36 inch or greater
    - o Foundation 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 foundation data. The plan description for the structure will then note a camber equal to the amount of anticipated settlement.

GENERAL NOTES

- The Drainage Designer should refer to IIM LD-110 for the general notes pertaining to drainage, stormwater management and erosion and siltation control and shall select the appropriate notes to be used on each project.

DRAINAGE SUMMARIES

- 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 a specific type of pipe is required, such as concrete for the extension of an existing pipe or corrugated for a shoulder slot inlet, etc., the type of pipe required is to be specified in both the Streamlined and Detailed Summary.
- 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 following methods of listing pipe in the Standard Summary and the Streamlined Summary are to be used to eliminate a possible contractor's error when ordering the pipe.

  - o Standard Summary Example:

DRAINAGE SUMMARY					
	PIPE	CORRUGATED PIPE	CONCRETE PIPE	CONCRETE PIPE	REMARKS
	15 in	15 in	15 in	24 in	
	L.F.	L.F.	L.F.	L.F.	
	20				
				20	EXTEND EXIST. PIPE
			20		EXTEND EXIST. PIPE
		20			EXTEND EXIST. PIPE
		20			FOR SHOULDER SLOT INLET
TOTALS	20	40	20	20	

  - o Streamline Summary Example:

800 L.F.   15 inch Pipe
200 L.F.   15 inch Pipe (Corrugated)
200 L.F.   15 inch Pipe (Conc.)
100 L.F.   24 inch Pipe   (Conc.)
200 L.F.   72 inch Pipe   (Special Design Conc.)

- The total linear feet of all like size pipe (regardless of type) are generally combined for the purposes of the estimate.

- The <u>type</u> of pipe is to be specified on the estimate <u>only</u> when:

  - o It is the <u>only</u> type of pipe required for that particular size.

  - o It is the <u>only</u> type of pipe required for the project (e.g., concrete pipe on an urban storm sewer project).

- Projects on which the allowable pipe type tabulation allows optional materials for new pipe installations and end sections are required, the Drainage Summary shall have a column indicating the optional standard, "St'd. ES-1 or St'd. ES-2 ", for the end sections. A separate column on the Drainage Summary is required when specifying only a St'd. ES-1 or St'd. ES-2 end section for pipes of a particular material.

STRUCTURES

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 to be used in situations 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.

- 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.

CONCRETE GUTTERS

- 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).

GRATES

- 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:

<u>DI-5</u>	<u>DI-7</u> Grate A	<u>DI-12</u>	
Type I	Type I	Type I	Limited Access and Rural Unlimited Access- Pedestrian Access Unlikely
Type III	Type III	Type II	Urban Areas-Pedestrian Accessible Areas

- 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 DI-7 in an area subject to occasional traffic (e.g., shoulders, parking areas, etc.) the load carrying Grate B shall be specified.

END SECTIONS FOR PIPE CULVERTS

- The Standard ES-2 drawing in the <u>Road and Bridge Standards</u> 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 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.

PIPE ENDWALLS WITH LOAD CARRYING GRATE

- Pipe endwalls with load carrying grates (St'ds. 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.

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.

DI-13 Shoulder Slot Inlets

- The DI-13 was specifically designed to:
  - o Collect water running along the bituminous curbing used under a guardrail system in high embankment areas.
  - o 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.
  - o 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.
- 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. 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 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 feet 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:
  - o Type A Nose Required.
  - o Concrete gutter and grate elevation at curb line to be one inch below normal shoulder elevation.
  - o Structure to be cast-in-place only.

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.

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 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’ 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’ or the interior diameter is less than 4’. 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 he often does), the precast option would have a chamber dimension 4' 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.

STORMWATER CONVEYENCE DOWN STEEP SLOPES

- Due to the substantial number of failures and continual maintenance problems associated with PG-4 flumes on fill slopes, it is recommended that flumes not be used on fill slopes.

- In lieu of paved fumes, it is recommended that the appropriate type of drop inlet and pipe be used in all possible situations. For design considerations of pipe on steep slopes see “Pipe on Steep Slopes” section in this DDM.

- 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 should explore all feasible alternatives to develop a design that will address both constructibility and future maintenance concerns.

DDM2

Drainage Descriptions

VIRGINIA DEPARTMENT OF TRANSPORTATION

LOCATION AND DESIGN DIVISION

DRAINAGE DESIGN MEMORANDUM

GENERAL SUBJECT: DRAINAGE DESCRIPTIONS	NUMBER: DDM 2
SPECIFIC SUBJECT: BASIC DRAINAGE DESCRIPTION FORMATS FOR HYDRAULIC PLAN ITEMS	DATE: September 1, 2005
	SUPERSEDES*: IIM-LD-01 (D) 223, Road Design Manual, HDA 02-02, HDA-02-03, DDM1
ADMINISTRATOR APPROVAL: *R. T. Mills* State Hydraulics Engineer	

* - The information noted in this DDM supplants only specified individual items contained in the listed memorandums.

INSTRUCTIONS

- 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 2001 VDOT Road and Bridge Standards.

PLAN MEASUREMENTS

- The length of culverts and storm sewer pipe shall be shown to the nearest one foot.
- Invert elevations for culverts and appurtenances shall be shown to the nearest 0.1 foot.
- Invert elevations for storm sewer pipe and appurtenances shall be shown to the nearest 0.01 foot.
- Linear footage of manholes and heights of junction boxes and drop inlets shall be shown to the nearest 0.1 foot.

- The design height of cover for culverts and storm sewer pipe shall be shown to the nearest one foot.

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

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.

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.

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.

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.

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 (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. This allows the Contractor to determine the proper strength, sheet thickness, or class of pipe from the 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-15 should be included in the description for the structure.

- 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 Cu. Yds. 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 inches 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.

- o 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 DDM3)

- The strength, thickness, gage, class of pipe or method of bedding will not be noted on the plans except in those cases where, for specific reasons, the Road and Bridge Standards PC-1 and PB-1 Tables will not govern.

- 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.

TYPICAL CULVERT DESCRIPTIONS

- These descriptions allow the Contractor the option of utilizing any of the pipe materials specified in the Allowable Pipe Type Table for a particular location. If there is only one type of allowable culvert material, the type of pipe material should be specified in the description (e.g., 100'-48" Conc. Pipe Req'd.).

(2-3) 100'-48" Pipe Req'd. (6' Cover)(20°Skew)
Inv.(In) 435.0 Inv.(Out) 434.0
2 St'd. EW-2 Req'd.
11 Tons Erosion Control Stone Class I, St'd. EC-1 Req'd. Lt.
378 Cu. Yds. Minor Structure Excavation

(2-5) 100'-24" Pipe Req'd. (3'Cover)
Inv.(In) 435.0 Inv.(Out) 434.0
1 St'd. ES-1 or 2 Req'd. Lt.
1 St'd. EW-11 Req'd. Rt. 4:1 Slope

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 <u>not</u> be specified as concrete radial pipe. See DDM1 for the minimum radius for each method for various pipe sizes.

- o OPEN JOINT METHOD

  (2-3) 100'-48" Pipe Req'd. (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 St'd. EW-2 Req'd.
  1 Tons Erosion Control Stone Class I, St'd. EC-1 Req'd. Lt.
  378 Cu. Yds. Minor Structure Excavation

- o BEVEL PIPE METHOD

  (3-1) 100'-48" Conc. Radial Pipe Req'd. (6' Cover)
  (120' Radius – using 8' pipe joint lengths with full bevel)
  Inv.(In) 435.0 Inv.(Out) 434.0
  2 St'd. EW-2 Req'd. Lt.
  11 Tons Erosion Control Stone Class I, St'd. EC-1 Req'd.
  378 Cu. Yds. Minor Structure Excavation

- o BEVEL PIPE WITH OPEN JOINT METHOD

  (6-7) 100'-48" Conc. Radial Pipe Req'd. (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 St'd. EW-2 Req'd.
  11 Tons Erosion Control Stone Class I, St'd. EC-1 Req'd. Lt.
  378 Cu. Yds. Minor Structure Excavation

JACKED PIPE

(5-6) 80'-48" Jacked Conc. Pipe Req'd. (25' Cover)
Inv.(In) 197.6 Inv.(Out) 197.0
2 St'd. EW-2 Req'd.
11 Tons Erosion Control Stone Class I, St'd. EC-1 Req'd. Rt.

MULTIPLE PIPE INSTALLATION

(8-9) 300'-48" Pipe Req'd. (7' Cover)
(Triple Line – 100' each line)
Inv.(In) 164.8 Inv.(Out) 164.1
2 St'd. EW-7 Req'd.
34 Tons Erosion Control Stone Class I, St'd. EC-1 Req'd. Rt.
1,134 Cu. Yds. Minor Structure Excavation

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 Req'd. (7' Cover)
Inv.(In) 435.0 Inv.(Out) 434.0
1 St'd. EW-1 Req'd.

STORM SEWER PIPE

(2-3) T0 (3-3) 195'-24" Conc. Pipe Req'd. (11' Cover)
Inv.(In) 15.2 Inv.(Out) 14.5

BOX CULVERT DESCRIPTIONS

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'd. (25' Cover)(15° Skew)
Inv.(In) 60.0 Inv.(Out) 57.0
St'd. BCS-DT, BCS-30, & BCW-21
4 St'd. Type I Wings Req'd.
46 Tons Erosion Control Stone Class I, St'd. EC-1 Req'd. Rt.
527 Cu. Yds. Minor Structure Excavation

PRECAST

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

  (4-8) 150’- 6’ X 8’ Precast Box Culvert Req’d. (25’ Cover)(15° Skew)
  Inv.(In) 60.0 Inv.(Out) 57.0
  2 Headwalls Req’d. (Cost to be included in price bid for linear feet of box culvert) Reference St’ds. BCS-DT & BCS-30
  4 Wings Req’d. Reference St’d. BCW-21, Type 1(K)
  46 Tons Erosion Control Stone Class I, St’d. EC-1 Req’d. Rt.
  527 Cu. Yds. Minor Structure Excavation

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*

- 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 DDM1 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. 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 foot.

- 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.

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 St’d. DI-4D Req’d.
L=8’, H=5.2’ Inv. 197.6
St’d. IS-1 Req’d.

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

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.

(9-16) 1 St’d. DI-7 Req’d. Grate A Type II Req’d.
H=5.3’ Inv. 23.6

- 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 St’d. DI-5 Req’d. Type I Grate Req’d.
St’d. PG-2A Type E Cover
H=4.8’ Inv. 13.6

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 Lin. Ft. St'd. MH-1 or 2 Req'd.
1 St'd. MH-1 Frame & Cover Req'd.
Inv. 83.4
1 St'd. SL-1 Req'd.

JUNCTION BOXES

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

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' St'd. SWM-1 Req'd.
    Bottom Elev. 23.8
    3" Diameter Water Quality Orifice Req'd., Inv. 26.8
    10" Diameter Orifice Req'd., Inv. 28.8
    See Sheet 2G For Details.

  - STORMWATER MANAGEMENT DAM

    (11-9) 1 SWM Dam Req'd.
    See sheet 2E for details.

- o MANUFACTURED WATER QUALITY STRUCTURES

  (7-7) 1 Water Quality Structure Req'd.
  Top Elevation 26.3
  Inv. Pipe (In) 20.3, Inv. Pipe (Out) 20.0
  Minimum WQV=2,345 Cu. Ft.
  Minimum WQQ=8.5 CFS

EXISTING STRUCTURES

- "Modify" should be used when a major work effort is required (e.g., connecting or removing pipes, adjusting height more than 1 foot, etc.).

  (4-11) Modify Existing Drop Inlet
  Adjust To Grade. Raise 2.3'
  Add DI-3B, L=6' Top.
  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 foot or less).

  (5-18) Adjust Existing MH
  Adjust To Grade. Raise 0.5'
  1 St'd. MH-1 Frame & Cover Req'd.
  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:

  - o *Modify To (Accept/Remove) 15" Conc. Pipe*
  - o *Connect UD-4 To Structure*
  - o *Convert Existing DI to Manhole*
  - o *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 <u>field evaluated</u> 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.

DDM3

Minor Structure Excavation

VIRGINIA DEPARTMENT OF TRANSPORTATION

LOCATION AND DESIGN DIVISION

DRAINAGE DESIGN MEMORANDUM

GENERAL SUBJECT: MINOR STRUCTURE EXCAVATION	NUMBER: DDM 3.1
SPECIFIC SUBJECT: Measurement of Excavation for Pipe and Box Culverts and Appurtenances	Date: September 1, 2005 SUPERSEDES: DDM3 & IIM-LD-91 (D) 71.8
ADMINISTRATOR APPROVAL: *R. T. Mills* State Hydraulics Engineer	

POLICY

- Quantities for minor structure excavation will be computed for pipes and box culverts with a diameter or span of 48 inch 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 inches 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.

PROCEDURE

- 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 inches outside the neatlines of the periphery of the culvert are not to be included in the 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."

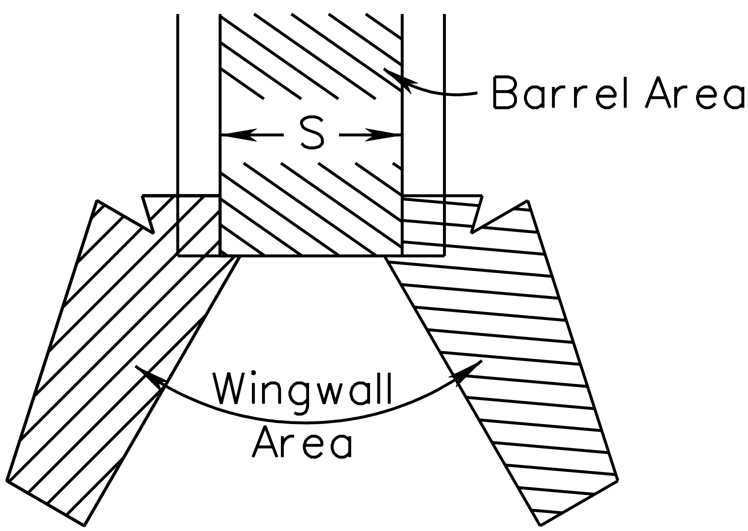
- Where there is not sufficient survey data to accurately determine minor structure excavation quantities, additional survey must be secured and incorporated before making final quantity determinations.

MEASUREMENT/PAYMENT

- 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.



TYPICAL BOX CULVERT

DDM4

Drainage Design At Railroads

VIRGINIA DEPARTMENT OF TRANSPORTATION

LOCATION AND DESIGN DIVISION

DRAINAGE DESIGN MEMORANDUM

GENERAL SUBJECT: DRAINAGE DESIGN AT RAILROADS	NUMBER: DDM4
SPECIFIC SUBJECT: GUIDELINES AND CRITERIA FOR DRAINAGE DESIGN UNDER OR ADJACENT TO RAILROADS	DATE: September 1, 2005 SUPERSEDES: IIM-LD- 229
ADMINISTRATOR APPROVAL: *R. T. Mills* State Hydraulics Engineer	

OVERVIEW

- 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.

CRITERIA

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 criteria, “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.

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.

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.

    o 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:

      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 <u>(See Table A)</u>.

Table A

Smooth Wall Steel Casing Pipe Minimum Wall Thickness For Installation Under Railroads	
Pipe Size Inches	**Minimum Wall Thickness Inches**
24	0.500
30	0.500
36	0.500
42	0.625
48	0.625
54	0.750
60	0.875
66	0.875
72	1.000

- 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.

0 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.

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.

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 storm but rather the effects of the 100 year storm 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. A computer printout that is not clearly presented or explained is not usually acceptable to the Railroad Company.

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.

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.

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.

- 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.

DDM5

Underdrains

VIRGINIA DEPARTMENT OF TRANSPORTATION

LOCATION AND DESIGN DIVISION

DRAINAGE DESIGN MEMORANDUM

GENERAL SUBJECT: UNDERDRAIN	NUMBER: DDM 5.1
SPECIFIC SUBJECT: DRAINAGE FOR PAVEMENT STRUCTURE; UNDERDRAINS IN GORE AREAS	DATE: September 1, 2005
	SUPERSEDES: DDM5,IIM-LD-01 (D) 130.8, IIM-LD-89 (D) 74.1
ADMINISTRATOR APPROVAL: *R. T. Mills* State Hydraulics Engineer	

GUIDELINES

- 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.

DESIGN PROCEDURES

- 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.

- Prior to submitting a request to the Hydraulics Unit for drainage design, the Roadway 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 Roadway Designer will depict only those underdrains that parallel the roadway centerline. A copy of the Materials Division's report will be included in the data forwarded to the Hydraulics Unit with the request for the drainage design.

- The Roadway 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:

  - o 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.

- o Determine outlet pipe locations for all parallel underdrain systems. Unless otherwise approved by the State Materials and the State Hydraulics Engineer, the following criteria will apply to spacing of outlet pipes:

  - UD-1 – Variable spacing
  - UD-2 – 500 feet maximum spacing
  - UD-3 – 1000 feet maximum spacing
  - UD-4 – 350 feet maximum spacing
  - UD-5 – 350 feet maximum spacing
  - UD-7 – 350 feet maximum spacing

- o For Rural (shoulder/ditch design) projects:

  - Determine the modifications required (If any) to the ditch typical section in order to provide a minimum 12 inches 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 inches of freeboard between the invert of the outlet pipe and the invert of the receiving structure.

- o For Urban (curb and gutter/storm sewer design) projects:

  - Design the storm sewer system to provide the minimum 12 inches of freeboard between the invert of the outlet pipe connection and the invert of the receiving structure.

- o Specify EW-12 Endwall at end of outlet pipe or specify connection to another structure (manhole, drop inlet, etc.)

- o 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.

TYPES AND USAGE

- Drainage for Pavement Subbase:

STANDARD	**USAGE AND PURPOSES**
UD-1	As recommended by materials division to lower ground water table in cuts
UD-2	Drains raised grass median strips as recommended by Materials Division
CD-1 & 2	Drains subsurface water from cuts and fills according to road and bridge standards and as recommended by Materials Division
UD-3	Drains area under sidewalk
UD-4	Provides drainage for pavement structure as recommended by Materials Division
UD-5	Same as UD-4; more easily added to previously constructed projects
UD-7	Provides pavement structure drainage as recommended by Material Division for existing pavements
EW-12	Used at outlet ends of all underdrains which do not tie to other drainage structures (inlets, manholes, etc.)

- 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.

PLAN DETAILS

- When showing EW-12's on plans, label as follows showing appropriate slope:

  1 – St'd. EW-12 Req'd. (4:1 Slope)

SUMMARY

- Following is a typical method of summarizing underdrains:

UNDERDRAIN SUMMARY							
STA. to STA.	UD-1	UD-4		CD-1	OUTLET PIPE	EW-12	
		Perforated	Non-Perforated			2:1	4:1
	L.F.	L.F.	L.F.	L.F.	L.F.	Each	Each
20+00 To 31+00 Rt.	1100				500	1	1
25+00 To 51+00 Rt.		2350	250		400	2	
31+50 Lt.				200	250	1	1
TOTALS	1100	2350	250	200	1150	4	2