
DRAINAGE MANUAL



2000

*With revisions through
November 2017*

CONNECTICUT DEPARTMENT OF TRANSPORTATION

**CONNECTICUT
DEPARTMENT OF TRANSPORTATION**

DRAINAGE MANUAL

**Prepared by the
OFFICE OF ENGINEERING
BUREAU OF ENGINEERING & HIGHWAY OPERATIONS
DIVISION OF DESIGN SERVICES
HYDRAULICS AND DRAINAGE SECTION**

OCTOBER 2000
includes revisions through November 2017

REVISIONS TO CONNDOT DRAINAGE MANUAL 2000

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1.1.3	January 2001	7.6.8	December 2003
1.2.6	May 2002	7.7.7	December 2003
		7.9.5	December 2003
3.2	May 2002	7.R	December 2003
3.5.2	May 2002	7.B	December 2003 (new)
3.6.3	May 2002		
3.7.5	May 2002	8.2.3	December 2003
3.7.6	December 2003	8.4.5	May 2002 (p. 3)
3.7.7	December 2003	8.5.2	December 2003
3.9.1	May 2002	8.5.4	December 2003
3.9.2	May 2002 (eliminated)	8.7.1	May 2002
3.11.1	May 2002	8.7.2	May 2002
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3.11.4	January 2001	8.7.4	May 2002
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6.B	November 2017	9.2.4	July 2016
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9.R	December 2003	11.13.4	May 2002
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		11.13.7	December 2003
10.1.3	December 2003	11.R	December 2003
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11.13.1	May 2002		

ALL REVISIONS TO CONNDOT DRAINAGE MANUAL (NOVEMBER 2017)

<u>SECTION</u>	<u>REVISION DATE</u>	<u>NOTES</u>
1.1.3	January 2001	correction computer program approval
1.2.6	May 2002	Major Traffic Generator section expanded
3.2	May 2002	section no. reference correction
3.5.2	May 2002	"Model Hydraulic Analysis" publication renamed
3.6.3	May 2002	added independent condition survey requirement
3.7.5	May 2002	added "model" in Final SCEL Report paragraph
3.7.6	May 2002	new section "Final Structure Plans"
3.7.6	December 2003	add requirement for tidal and temporary data for structure plans
3.7.7	May 2002	section no. change
3.7.7	December 2003	add requirement for approval of stormwater management standards
3.9.1	May 2002	refers to new Appendix C
3.9.2	May 2002	section eliminated
3.11.1	May 2002	"Model Hydraulic Analysis" publication renamed
3.11.2	May 2002	added paragraph re: FM Gen. Certification
3.11.4	January 2001	new section "Water Diversion Permits"
3.11.5	January 2001	new section "Dam Safety Permits"
3.R	May 2002	"Model Hydraulic Analysis" publication renamed
3.B	January 2001	add references (#2, 5, 6, 11, 12, 13, 14)
		correction (#6, 13) (p. 5, 6)
3.B	December 2003	add requirement to identify engineer who prepared and checked work (p. 6)
3.C	May 2002	new section re: Hydraulic Design Process
4.1.3	May 2002	added condition survey requirement
4.6.5	May 2002	added "of high density polyethylene pipe (HDPE)"
4.6.6	May 2002	corrected height of fill requirements
4.C	May 2002	updated HDPE Guideline
6.1.10	February 2001	changed km ² to ha
6.5	December 2003	reference revised HDS No. 2
6.7.3	December 2003	reference revised HDS No. 2 and remove reference to HEC-19
6.9.1	February 2001	changed km ² to ha
6.9.3	February 2001	changed km ² to ha
6.9.6	December 2003	remove reference to HEC-19
6.10.4	December 2003	correct equations
6.12.4	May 2002	Fig 6-5 corr. line label (p. 7)
6.13.1	December 2003	remove ref. to HEC-19, reference HDS No. 2
6.15.1	December 2003	sentence clarification
6.16	December 2003	new section – Average Daily and Spring Flows
6.R	December 2003	remove ref. to HEC-19, reference HDS No. 2
6.B	November 2017	removed rainfall tables superseded by NOAA Atlas 14
6.C.4	February 2001	correction eq. 6.C.2
6.D.1	February 2001	changed km ² to ha

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<u>SECTION</u>	<u>REVISION DATE</u>	<u>NOTES</u>
6.D.3	December 2003	add requirement for approved engineer's seal on Hydraulic Design Report
6.E	May 2002	note added (p. 1)
6.F	February 2001	
6.F	May 2002	drainage area units changed (p. 1)
6.F	December 2003	clarify Step 4
7.6.5	December 2003	spelling correction (p. 2)
7.6.7	December 2003	shear stress table (p. 11)
7.6.8	December 2003	reference to new Appendix B
7.6.9	December 2003	reference to new Appendix B (p.15)
7.7.7	December 2003	removed duplicate reference (p. 15)
7.9.5	May 2002	eq. 7.37 correction (p. 2)
7.9.5	December 2003	correction to nomograph (p. 6)
7.R	December 2003	typo. (p. 3)
7.B	December 2003	new section – Vegetative (Grass) Swales and Erosion Control Matting
8.2.3	December 2003	new section – Temporary Hydraulic Facilities (refers to 9.6.12)
8.4.5	January 2001	cutoff walls rewritten (p. 3)
8.4.5	May 2002	modified cutoff wall depth requirement (p. 3)
8.5.2	December 2003	reference correction
8.5.4	December 2003	add reference to "Connecticut Guidelines for Soil Erosion and Sedimentation Control"
8.7.1	May 2002	new section "Erosion, Sedimentation & Debris Control"
8.7.2	May 2002	new section "Scour at Inlets"
8.7.3	May 2002	new section "Scour at Outlets"
8.7.4	May 2002	new section "Sedimentation"
8.7.5	May 2002	previously 8.7.1 Assessment of Erosion Potential
8.8.1	May 2002	equation and figure nos. changed 8.8 through 8.12
8.8.2	May 2002	
8.8.3	May 2002	
8.9.1	May 2002	
8.9.2	May 2002	
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<u>SECTION</u>	<u>REVISION DATE</u>	<u>NOTES</u>
8.D	May 2002	typo. correction
8.F	December 2003	enhance Data Collection and Field Review Form
8.H	May 2002	eliminated-moved to Chapter 11, Appendix A
9.2.3	December 2003	added Flood Management information (p. 2)
9.2.3	July 2016	added note that SCEL were repealed October 1, 2013
9.2.4	July 2016	
9.3.8	May 2002	Table 9-2 (p. 5)
9.3.8	December 2003	reference to "Tidal Hydraulic Modeling for Bridges" manual
9.3.8	July 2016	Table 9-2 following this section revised (see 9.3.9)
9.3.9	July 2016	replaced with new section "Design Exceptions"
9.3.10	July 2016	added (former section 9.3.9 moved here)
9.4.2	July 2016	revised reference to 9.3.10
9.4.5	December 2003	section revised to explain hydraulic computer models
9.5.1	May 2002	add reference to Appendix C
9.5.3	May 2002	add NBIS rating information
9.5.8	May 2002	new section "Scour Eval. Studies for ConnDOT Projects"
9.6.11	December 2003	removed reference to temporary structures in Chapter 6
9.6.12	December 2003	new section – Temporary Hydraulic Facilities
9.A	May 2002	Hydraulic Data Form
9.A	December 2003	enhance Data Collection and Field Review Form and Hydraulic Data Form
9.C	May 2002	new section "Bridge Scour Eval. Report Format"
9.C	December 2003	add requirement for engineer's signature and seal on Bridge Scour Evaluation Report
9.R	December 2003	added references
10.1.3	December 2003	add reference of Section 10.13
10.4.2	May 2002	release rate changed to ½" for 30 hr.
10.4.2	December 2003	new section (DEP Requirements) (former 10.4.2 to 10.4.6 headings changed)
10.4.3	December 2003	(Release Rate section) – add reference to Section 10.11
10.4.5	December 2003	(Grading and Depth section)
10.4.7	December 2003	(Location) additional information provided
10.5	December 2003	all of section changed
10.5.1	December 2003	
10.5.2	December 2003	
10.5.3	December 2003	
10.5.4	December 2003	section deleted
10.10.1	December 2003	detention basin design moved to Section 10.11
10.10.2	December 2003	new section – Temporary Sediment Trap
10.11		all of section changed
10.11.1	December 2003	changed "dry pond" to "dry detention basin" and removed regulatory discussion (moved to Section 10.11.2)
10.11.2	May 2002	riser diameter changed to riser circumference (p. 4)

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<u>SECTION</u>	<u>REVISION DATE</u>	<u>NOTES</u>
10.11.2	December 2003	new section (Regulatory Considerations)
10.11.3	December 2003	new section (Sizing Requirements)
10.11.4	December 2003	new section (Outlets)
10.11.5	December 2003	new section (Embankment Cross Sections)
10.12	December 2003	all of section changed
10.12.1	December 2003	changed "wet pond" to "wet detention basin" and removed reference to recreational benefits
10.12.2	December 2003	new section (Regulatory Considerations) revise WQV, include 25 yr. storm, add forebay
10.12.3	December 2003	new section (Sizing Requirements)
10.12.4	December 2003	new section (Outlets)
10.12.5	December 2003	new section (Sediment Forebay)
10.12.6	December 2003	new section (Embankment Cross Sections)
10.13.1	December 2003	table number change add reference to Connecticut Stormwater Quality Manual
10.13.2	December 2003	add reference to Connecticut Stormwater Quality Manual infiltration rate paragraph change
10.13.3	December 2003	equation label change and other small changes
10.13.4	December 2003	Figure 10-32 modification and equation label change
10.R	December 2003	added reference
11.4.6	December 2003	added reference to Chapter 10
11.8.2	May 2002	revise Type "C" and "C-L" CB descriptions
11.9.5	January 2001	complete revision
11.9.6	May 2002	add factor of safety clogging % (p. 11)
11.9.6	December 2003	correct grate type (p. 12)
11.9.9	January 2001	correction Steps 11, 12, 13
11.9.10	January 2001	correction Step 10 (p. 22)
11.11.2	December 2003	"just-full" condition
11.11.5	December 2003	change storm drainage system design procedures
11.11.9	December 2000	column eliminated (Manhole Invert Drop) column correction (Flow Time T_c) (p. 12, 13) Storm Drain Computation Sheets (p. 13, 14)
11.11.9	December 2003	
11.13.1	May 2002	grammar change
11.13.2	May 2002	previously 8.7 outlet protection
11.13.3	May 2002	previously 8.7 outlet protection
11.13.4	May 2002	previously 8.7 outlet protection
11.13.5	May 2002	previously 8.7 outlet protection
11.13.6	May 2002	(p 10, 11 change) prev.8.7 outlet protection
11.13.7	May 2002	previously 8.7 outlet protection
11.13.7	December 2003	changed "riprap type" to "riprap size" (p. 16)
11.R	May 2002	added reference
11.R	December 2003	added references
11.A	May 2002	previously 8.H outlet protection form

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<u>SECTION</u>	<u>REVISION DATE</u>	<u>NOTES</u>
11.B	May 2002	new section "Typical Grading for Type "C-L" Catch Basins Located in a Swale"
11.C	December 2003	new section – Water Quality Flow & Flow Diversion Guidance

ACKNOWLEDGMENTS

The 2000 Connecticut Department of Transportation Drainage Manual was developed using the 1999 and 1991 editions of the American Association of State Highway and Transportation Officials (AASHTO) Model Drainage Manual and various other publications. The Department gratefully acknowledges the personnel of the Hydraulics and Drainage Section in the preparation of this manual and the many persons and agencies involved in the technical review.

PREFACE

The American Association of State Highway and Transportation Officials (AASHTO) Task Force on Hydrology and Hydraulics Model Drainage Manual serves as a basis for this manual. The Connecticut Department of Transportation (ConnDOT) Bureau of Engineering and Highway Operations, Division of Design Services, Hydraulics and Drainage Section has modified the Model Drainage Manual to apply specific principles that are applicable to Connecticut's resources. The ConnDOT Hydraulics and Drainage Section has incorporated many other references and their numerous years of experience into this comprehensive drainage manual to provide guidance on roadway drainage.

The objectives of drainage design are to provide thorough investigation and planning, adequate design, construction and durability of ConnDOT facilities while also providing for the safety of the travelling public and adjacent property owners; to minimize any undesirable effects which might be caused by the construction and maintenance of the facility; and to achieve an optimum balance between risk of damage to the facility and the cost of construction. Each of these objectives contains within it the implication of certain judgements and decisions which must be made by the designer.

This manual contains the design criteria to form a framework for achieving the goals listed above. It is the responsibility of the designer to develop the required hydrologic and hydraulic data for any given project or site. Using this base data, the best overall solution must be tailored to the situation when a particular design may require decisions involving conflicting value judgements such as economics vs. environmental considerations.

The need for proper field site investigations, collection of data, analysis of data, and exercise of judgement are the foundations of good drainage design that will enable proper use of the criteria set forth in this manual. It is with this goal in mind that this drainage manual has been prepared.

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1.1 Introduction

1.1.1 Background

The AASHTO Task Force on Hydrology and Hydraulics, in cooperation with the FHWA Rural Technical Assistance Program, has assisted in writing this Drainage Manual. This manual does not set AASHTO design policy and procedures. All design practices and procedures have been reviewed and set by the Connecticut Department of Transportation (ConnDOT).

1.1.2 Overview

The AASHTO Task Force has incorporated their numerous years of experience into their comprehensive drainage manual. This has been modified by the ConnDOT to provide guidance on various aspects of highway drainage for the specific conditions in the State of Connecticut. Design theories, concepts, manuals, practices and procedures, are condensed and written for use by the designer. Where appropriate, relevant existing AASHTO guidelines are given and/or referenced.

The manual has been developed to give the designer a basic working knowledge of hydrology and hydraulics. All basic design elements are included such 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 or engineering judgment.

1.1.3 Manuals and Computer Programs

References to specific computer programs, AASHTO guidelines, manuals and regulations will be noted within the 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 keep up-to-date with these programs and the latest drainage-related Federal regulations. Therefore, it is incumbent upon the hydraulic engineer to retain a current, working knowledge of all applicable statutes and regulations related to the discipline. Further, it is the responsibility of the engineer to obtain approval of any computer programs not specifically mentioned in this manual prior to initiating work on Department administered projects. In this manner, the designer will be assured that the Hydraulics and Drainage section concurs with the use of a specific computer program for the intended application.

1.1.4 Metric Conversion

This manual is presented in dual units, SI (metric) and English units. In most cases all equations, tables and figures are given in dual units. In a few instances empirical equations, based on historical data, could not be converted to metric units and therefore only English units are given. Where equations could be converted, hard conversions between units were used. In all cases, computer software is available to obtain the required information in metric units. All example problems are given in metric units. The English equations and units are presented in parentheses. If the equations are the same for both systems, only one equation is presented. Care must be used to avoid improper use of metric input data into the historic English equations noted above.

1.1.5 References

At points in the manual where the designer may need more detailed source material, references are given. The reference section at the end of each chapter has been organized to include these source documents as well as a short listing in bold print of those documents which are recommended additions to the designer's library of references.

1.2 User Instruction

1.2.1 Instructions

This manual has been prepared for the use of all engineers who become involved with drainage facilities funded through or owned by the ConnDOT. Its purpose is to present the best available methods, guidelines, and criteria in useable form for drainage design. It is expected that the criteria and methods set forth herein will be used in the design of the drainage facilities for the Department.

However, it is also recognized that new and improved design concepts or understandings are continually being devised, and the Department promotes and encourages the use of such improved procedures when they have been submitted to the Hydraulics and Drainage Section for review and approval prior to use. To the extent feasible, this manual will be kept current to include improved design procedures that have been proven to be sound and effective.

It must also be recognized that regardless of the degree of sophistication of drainage standards and criteria, each drainage design problem is unique and may require the use of considerable engineering judgment to fit or adjust a standardized design method to a particular site.

It is the job of the designer to develop the required hydrologic and hydraulic data for any given project or site. Using this base data, the designer must determine the best overall solution when a design may present facts involving conflicting value judgments such as economics vs. environmental considerations.

The need for proper field site investigations, collection of data, analysis of data, and exercise of judgement are the foundations of good drainage design that will enable proper use of the criteria set forth in this manual.

Ultimately, the submission of complete hydraulic reports documenting the basis for the hydrology and hydraulic design is a requirement for work to be performed or reviewed by the Department's forces. Such reports serve as valuable sources of information for evaluating the performance of the constructed facilities and also serve as aids in analyzing the needs for future improvements.

The Hydraulics and Drainage Section is available to provide assistance to users and would appreciate any constructive suggestions or comments that would improve the contents of this Drainage Manual. All users of this manual are encouraged to submit comments and/or corrections on the form provided in Appendix A.

1.2.2 Updates

The ConnDOT will issue update changes to this manual as required. These revisions will be disseminated to the user by memo from the Manager of Design Services. It is the designer's responsibility to ensure the latest revision is being used. The most current manual is also available on the Department's web site.

1.2.3 Consultant Engineers

Consultant Engineers should also refer to the current editions of the ConnDOT Bridge Design Manual, Consultant Engineers Manual, Highway Design Manual, and other pertinent ConnDOT manuals to ensure that the minimum standards as outlined are met. In addition, pertinent Department of Environmental Protection (ConnDEP) manuals and guidelines should be referenced to ensure the design satisfies permit requirements, where applicable.

1.2.4 Hydraulic Engineer Approval

In order to ensure Quality Control/Quality Assurance on hydraulic designs **for structures which convey watercourses greater than 2.6km² (1mi²), the engineer performing the analysis must be approved by ConnDOT on a project by project basis.** The Department requires that an individual responsible for the work be a licensed Connecticut Professional Engineer. The procedure for Department approval is outlined in the CE Manual. Approval requests for previously qualified engineers to work on other State projects will not require the resubmission of a resume. However, an approval request for the current project together with a copy of the Department's prior approval letter and an updated list of hydraulic designs performed by the candidate is required.

1.2.5 ConnDOT Highway Encroachment Permit

In accordance with the Highway Encroachment Permit Regulations, this manual shall be used in conjunction with all Highway Encroachment permits which require modifications to existing systems and/or new drainage facilities within the State's right of way. The facilities and supporting documentation shall conform to all requirements set herein or as required by the District Drainage Engineer.

1.2.6 Major Traffic Generators

This manual shall be used as a basis for all activities affecting State drainage facilities as related to the proposed development. As a minimum, the applicant shall include a narrative addressing the following:

1. The existing and proposed drainage patterns on the site. This section would specifically state the following; (a) Existing drainage patterns are being maintained on the proposed site and there is no diversion(s) of storm water runoff; (b) There is a proposed storm water diversion(s) that is directed away from a State drainage facility or (c) There is a proposed storm water diversion(s) that is directed to a State drainage facility. Diversion of storm water runoff to State drainage facilities is generally not approved unless appropriate drainage rights are obtained from affected downstream property owners.
2. The proposed drainage design and its relation to any adjacent existing State drainage facilities. The effect of the proposed site development and drainage design on State drainage facilities should be described. While it is recognized that it may not be possible to include final drainage design and computations early in the development of the project, sufficient information should be provided to demonstrate that the proposal is feasible.
3. The features included in the proposed drainage design to treat storm water and enhance storm water quality for sites involving a direct connection to a State drainage system.

Topographic plans with contours showing existing and proposed drainage areas, including any off-site areas draining toward the project, are required.

It should be noted that if a ConnDOT encroachment permit is required, then Section 1.2.5 must be complied with.

1.2.7 Abbreviations

Following are the abbreviations used in this manual.

AASHTO – American Association of State Highway and Transportation Officials
ACPA – American Concrete Pipe Association
AISI – American Iron and Steel Institute
AISC – American Institute of Steel Construction
ASTM – American Society of Testing and Materials
AREA – American Railway Engineering Association
BMP – Best Management Practices
CFR - Code of Federal Regulations
CGS – Connecticut General Statute
ConnDEP – Connecticut Department of Environmental Protection
ConnDOT – Connecticut Department of Transportation
EA – Environmental Assessment
EIS – Environmental Impact Statement
EPA - Environmental Protection Agency
FDSA – Federal Dam Safety Act
FEMA - Federal Emergency Management Agency
FHBM – Flood Hazard Boundary Map
FBFM – Flood Boundary and Floodway Map
FHPM - Federal-Aid Highway Program Manual
FHWA - Federal Highway Administration
FRA - Federal Railway Administration
FWS - Fish and Wildlife Service
IECA – International Erosion Control Association
IWRD – Inland Water Resources Division (ConnDEP)
LTEC – Least Total Expected Cost
MDC – Metropolitan District Commission
NBIS – National Bridge Inspection Standards
NCHRP – National Cooperative Highway Research Program
NCSPA – National Corrugated Steel Pipe Association
NFIP – National Flood Insurance Program
NMFS - National Marine Fisheries Service
NOAA – National Oceanic and Atmospheric Administration
NPDES – National Pollutant Discharge Elimination System
NPS - National Park Service
NRCS - Natural Resources Conservation Service; formerly Soil Conservation Service (SCS)
OCZM - Office of Coastal Zone Management
Stat. - Statute
U.S.C. - United States Code
USCE - U.S. Corps of Engineers
USCG - U.S. Coast Guard
USDA – United States Department of Agriculture
USFS - U.S. Forest Service
USGS – United States Geological Survey

1.3 General Hydraulic Design Practices

1.3.1 Introduction

An adequate drainage structure (culvert or bridge) may be defined as one which meets the following:

- the design of the structure meets or exceeds ConnDOT standard engineering practice
- the design is consistent with what a reasonably competent and prudent designer would do under similar circumstances
- the design is consistent with environmental concerns and regulations, where applicable

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

- hydrologic and hydraulic analysis
- engineering evaluation of selected alternatives

1.3.2 Hydrologic Analysis

Present state-of-practice formulas and models for estimating flood flows are based on statistical analyses of rainfall and runoff records. 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 values for the desired flood frequencies. The predicted value of the flood flows represents the designer's best estimate, with varying degrees of error. The expected magnitude of this variation can be determined for some formulas or models as a part of the hydrologic design procedure.

1.3.3 Hydraulic Analysis

The next step in the design process involves preliminary selections of 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.

1.3.4 Engineering Evaluation

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

- legal considerations
- flood hazards to highway users and neighboring property owners
- hydraulic efficiencies
- costs
- environmental and social concerns
- other site specific concerns

1.3.5 ConnDOT "Standard Engineering Practice"

Hydrologic analysis and hydraulic analysis set forth the design process representative of the ConnDOT 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 considerations are made in regard to this design 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 "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 water course 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 floodplains 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 variations caused by 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 for drainage structures which convey a watercourse greater than 1km² (mi²); however, flows larger than this value may be considered for complex, high risk or unusual cases that require special studies or risk analyses. See Chapter 8, Culverts and Chapter 9, Bridges.

1.4 Construction Considerations

Construction-related hydraulic considerations are a necessary part of the planning and design phases. Factors which will affect construction timing and methods need to be kept in mind as project development proceeds. Those responsible for contract administration and actual construction may need to coordinate their scheduling and construction procedures with the designer in order to achieve the results intended. Any special or unique construction requirements should be communicated to the designer prior to the final design phase of the project.

The designer should be present at the preconstruction conference to explain special features and planned construction phasing where these considerations are necessary for the proper functioning of the design. It may be advisable and necessary to specify certain time limits and special instructions as to how the work will be accomplished. Phased construction to accommodate seasonal variations, floods, fish passage, etc., may be needed. In addition, the need for special considerations related to temporary work, detours and public safety issues can be outlined and discussed. It should be emphasized by the designer that any and all revisions of drainage designs as contained on construction plans should be discussed with the designer prior to execution.

The designer should use the procedures outlined in Chapter 6, Section 6.15 for the hydrology for temporary hydraulic facilities. These facilities include all inlets, pipes, channels, culverts or bridges which are required for stage construction, temporary roads, channel relocation, culvert installations, bridge construction and haul roads. They are to be designed with the same care that is used for the primary facility.

1.5 Maintenance Considerations

Drainage facilities perform the function of removal of water from streets, highway sections, parking areas, and other drainage areas and the protection of these areas from the effects of the water. These drainage facilities include drop inlets, storm drains, bridges, culverts, underdrains, ditches, slope protection, detention facilities, and erosion control devices. In order for these facilities to function as designed and constructed, they must be properly maintained. Full consideration must therefore be given to this activity during their design. Designing drainage facilities, that are as maintenance free as practical, will often result in cost savings that, over the service life of the drainage feature, equal or exceed initial construction cost. Good drainage design practices recognize that all structures require periodic maintenance inspections and repairs. Reasonable access for maintenance personnel and equipment must be provided for this necessary function.

Communications between designers and maintenance personnel are essential. Design personnel are encouraged to contact maintenance personnel for their input on difficulties they identify in maintaining drainage facilities. Suggestions from maintenance personnel on how drainage facilities and future designs may be improved for efficient and effective maintenance should be invited.

Roadway maintenance operations that require lane closures on heavily traveled highways are costly and may compromise the safety of maintenance workers and the traveling public. Good drainage designs eliminate or reduce the need for traffic control measures for cleaning drainage systems and facilities. Drainage inlet types and pipe drains that are designed to be as self cleaning as possible should be specified wherever it is practical.

The routine cleaning and minor repairs of highway drainage features often require that labor intensive hand methods be used. Adequate access for maintenance personnel and equipment to get to the site and do work on drainage facilities should be provided for in the plans. In addition, most costly maintenance work might easily be avoided, or more efficiently accomplished, if designers were to give more attention and thought to the shape and location of drainage features. For example, a parabolic shaped roadside ditch that is contiguous to the shoulder can be efficiently reshaped and cleaned with a motor grader. Small trapezoidal and other shaped roadside ditches may require hand cleaning or special equipment.

Locating drainage inlets where trash and sand naturally tend to accumulate causes clogging of grate type inlets and pipe drains. Wherever practical, drainage inlets should be located where sweeping operations will not deposit additional debris in drainage facilities. Inlet and junction boxes and other minor drainage structures, that are subject to periodic cleaning, must be made large enough that maintenance workers can enter them and work with either hand tools or heavy duty vacuum equipment.

Several types of bank protection and erosion control materials are classified as flexible or self-adjusting and, as a rule, are less of a problem to repair and maintain than are the rigid category of linings. Whenever practical, materials requiring less maintenance attention should be specified.

1.6 Documentation

1.6.1 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)
- information leading to the development of defense in matters of litigation
- public information

Frequently, it is necessary to refer to plans, specifications and analyses 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.

1.6.2 Definition

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

1.6.3 Purpose

Documentation shall be included in the design files and on the construction plans. While documentation requirements for existing and proposed drainage facilities are similar, the data retained for existing facilities are often slightly different than that for proposed facilities and these differences are discussed. This section identifies ConnDOT's system for organizing the documentation of hydraulic designs and reviews so as 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 arrived at. Often there is expressed the 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);
- providing a continuous site history to facilitate future reconstruction;
- providing the file data necessary to quickly evaluate any future site problems that might occur during the facilities service life; and
- expediting plan development by clearly providing the reasons and rationale for specific design decisions.

1.6.4 Types

There are three basic types of documentation which shall be considered. The types are preconstruction, design and construction or operation.

1. Preconstruction documentation may include the following if available or within the budgetary restraints of the project.
 - aerial photographs
 - contour mapping
 - watershed map or plan including:
 - flow path for determination of times of concentration and watershed delineation
 - watershed boundaries
 - watershed areas
 - natural storage areas
 - surveyed data reduced to include:
 - existing hydraulic facilities
 - existing controls
 - profiles - roadway, channel, driveways
 - cross sections - roadway, channels, faces of structures
 - flood insurance studies and maps by FEMA
 - Natural Resource Conservation Service soil maps
 - field trip report(s) which may include:
 - video cassette recordings
 - audio tape recordings
 - still camera photographs
 - movie camera films
 - written analysis of findings with sketches
 - reports from other agencies (local, State or Federal), Department personnel, newspapers and abutting property owners
2. Design documentation shall include all the information used to justify the design, including:
 - reports from other agencies
 - hydrologic report with all relevant backup data and computations
 - hydraulic report with all relevant backup data and computations
 - approvals as required
 - design related correspondence (internal and external)

3. Construction or operation documentation shall include:

- plans
- revisions
- as-built plans and subsurface borings
- photographs
- record of operation including complaints and resolutions during flooding events

It is very important to prepare and maintain in a permanent file the as-built plans for every major 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.

1.6.5 Scheduling

Documentation shall 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 rather be an ongoing process and part of each step in the hydrologic and hydraulic analysis and design process. This will increase the accuracy of the documentation, provide data for future steps in the plan development process, and provide consistency in the design even when different designers are involved at different times of the plan development process.

1.6.6 Standard Practices

Following are the Department's standard practices related to documentation of hydrologic and hydraulic designs and analyses.

1. Hydrologic and hydraulic data, preliminary calculations and analyses and all related information used in developing conclusions and recommendations related to drainage requirements, including estimates of structure size and location, shall be compiled in a documentation file.
2. The designer shall document all design assumptions and selected criteria including the decisions related thereto.
3. The amount of detail of documentation for each design or analysis shall be commensurate with the risk and the importance of the facility.
4. Documentation shall be organized to be as concise and complete as practicable so that knowledgeable designers can understand years hence what was done by predecessors.
5. 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.
6. Documentation shall be organized to logically lead the reader from past history through the problem background, into the findings and through the performance.
7. An executive summary at the beginning of the documentation will provide an outline of the documentation file to assist users in finding detailed information.

The items outlined in the respective chapters of this manual shall be included in the documentation file. The intent is not to limit the data to only those items listed, but rather establish a 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 shall appear in the documentation file. Additionally, the designer shall include in the documentation file items not listed but are useful in understanding the analysis, design, findings and final recommendations which may not be included in the documentation sections of the various chapters.

All users of this manual are encouraged to submit comments/corrections to:

Date Submitted: _____

Organization: _____

[illegible]

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2.1 Overview

2.1.1 Introduction

Various drainage laws and rules applicable to highway facilities are discussed in this chapter. In any given project, designers may be required to defend their work in court. This chapter should not in any way 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 intention is only to provide information and guidance on the engineer's role in the legal aspects of highway drainage. If a designer has any concern as to the legal aspects or requirements of a particular item or situation, it is their responsibility to seek proper guidance prior to proceeding.

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.
- Even where some alteration in flow is legally permissible, the courts look with disfavor upon infliction of injury or damage that could reasonably have been avoided by a prudent designer.
- The laws relating to the liability of government entities are undergoing radical change, with a trend toward increased government liability.

2.1.2 Order Of Authority

The descending order of law supremacy is Federal, State, and local. 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, a lower level of government may promulgate a law, rule or regulation which would require an unreasonable or even illegal action by a higher level. An example is a local ordinance which would require an expenditure of State funds for a purpose not intended in the appropriation. Many such conflicts in the laws of different levels of government involve constitutional interpretation and must be determined case by case. Such conflicts should be referred to the Attorney General's Office for legal counsel before any action is taken.

2.1.3 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, among others:

Highway Drainage Guidelines, American Association of State Highway and Transportation Officials, Washington, D. C., 1999. Chapter V - The Legal Aspects of Highway Drainage.

Legal Research Digest, Transportation Research Board.

Water Resources Coordination and Permit Manual, Office of Environmental Planning, Bureau of Policy and Planning, ConnDOT, April 1997.

Model Drainage Manual, AASHTO, 1999 Metric Edition.

2.2 Federal Laws

2.2.1 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).

2.2.2 Drainage

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

- flood insurance and construction in flood hazard areas
- navigation and construction in navigable waters
- water pollution control
- environmental protection
- protection of fish and wildlife
- coastal zone management

Federal agencies formulate and promulgate rules and regulations to implement these laws, and highway hydraulic engineers should attempt to keep informed regarding proposed and final regulations.

2.2.3 Constitutional Power

The Congress of the United States is granted constitutional power to regulate "commerce among the several states." A part of that power is the right to legislate on matters concerning the instrumentalities of interstate commerce such as navigable waters. The definition of navigable waters expands and contracts depending upon the breadth required to adequately carry out the Federal purpose. The result is that Congress can properly assert regulatory authority over at least some aspects of waterways that are not in themselves subject to navigation.

When becoming involved in obtaining approvals from the Federal agencies, one should be aware that these agencies do not always work in concert. Quite often they will not be in agreement with each other. This can result in significant project delays unless early coordination is initiated and diligently pursued. These conflicts between Federal agencies occur as a result of their having different rules; some are "regulators" while others are "resource" motivated. For this reason they will have different goals and, in some instances, definitions of such things as wetlands. When

conflicts occur, it is best to quickly determine which agency has primary responsibility and attempt to satisfy their needs.

2.2.4 Fish And Wildlife Service

The Fish and Wildlife Act of 1956 (16 U.S.C. 742 et seq.), the Migratory Game-Fish Act (16 U.S.C. 760c-760g) and the Fish and Wildlife Coordination Act (16 U.S.C. 611-666c) express the concern of Congress for the quality of the aquatic environment as it affects the conservation, improvement and enjoyment of fish and wildlife resources. The Fish and Wildlife Coordination Act requires that "whenever the waters of any stream or body of water are proposed or authorized to be impounded, diverted, the channel deepened, or the stream or other body of water otherwise controlled or modified for any purpose whatever, including navigation and drainage, by any department or agency of the United States, or by any public or private agency under Federal permit or license, such department or agency shall first consult with the United States Fish and Wildlife Service, Department of the Interior, and with the head of the agency exercising administration over the wildlife resources of the particular state with a view to the conservation of wildlife resources by preventing loss of and damage to such resources as well as providing for the development and improvement thereof."

2.2.5 Executive Orders

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 highway departments, these requirements are usually implemented through general regulations.

- **E.O. 11988**

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

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

- **E.O. 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 to 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
- consider factors relevant to the proposal's effects on the survival and quality of the wetlands

2.3 National Flood Insurance Program

2.3.1 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. 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 which 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 0.3 m (1 ft.).

2.3.2 Flood Insurance

The National Flood Insurance Act of 1968, as amended, (42 U.S.C. 4001-4127) 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 which 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 0.3 m (1 ft.) 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 0.3 m (1 ft.) 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 towns 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.

2.3.3 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 the Federal Emergency Management Agency

(FEMA) for amendments to NFIP ordinances and maps in that community. 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.**

2.3.4 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). The FHBM is generally not based on a detailed hydraulic study and, therefore, the floodplain boundaries shown are approximate.
- Flood Boundary and Floodway Map (FBFM) The FBFM, 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.
- Flood Insurance Rate Map (FIRM). The FIRM is generally produced at the same time as the FBFM 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.

2.3.5 Coordination With FEMA

Early in the design stage, coordination should be established with FEMA through the ConnDEP 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, 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-ft (0.3-m) increase in the base flood elevation would be exceeded.
- 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 ft (0.3 m). Where insurable buildings are not affected, it is sufficient to notify FEMA of changes to base flood elevations as a result of highway construction.

The draft Environmental Impact Statement or Environmental Assessment (EIS/EA) should indicate the NFIP status of affected communities, the encroachments anticipated and the need for floodway or floodplain ordinance amendments. Coordination means furnishing to FEMA the draft EIS/EA and, upon selection of an alternative, 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. If a determination by FEMA would influence the selection of an alternative, a commitment from FEMA should be obtained prior to the final environmental impact statement (FEIS) or a finding of no significant impact (FONSI). Otherwise this later coordination may be postponed until the design phase.

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.

2.3.6 Consistency 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 the hydraulic model can demonstrate no increase in the water surface elevations, the project may normally be considered as being consistent with the standards.

2.3.7 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-ft (0.3-m) 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. **It should be noted that floodway revision is a lengthy process and should be avoided when possible.**

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 which 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. Increases in base flood elevations are referenced to the profile obtained for existing conditions when the floodway was first established.

2.3.8 Data For Revisions

The ConnDEP Inland Water Resources Division should be consulted regarding the data required for a floodway revision. The following references should also be reviewed.

- Revisions to National Flood Insurance Program Maps. Application/Certification Forms and Instructions of Conditional Letters of Map Revision, Letter of Map Revision and Physical Map Revisions (FEMA Form 81-89 series, Oct. 1994).
- Appeals, revisions and amendments to Flood Insurance Maps. A Guide for Community Officials (FIA-12, Jan. 1990)

2.3.9 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-ft (0.3-m) 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 ft (0.3 m).
- 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.

A floodway revision requires a considerable amount of time to accomplish. It starts with preliminary approval, prior to advertising a project for construction and concludes with reanalyzing the hydraulics using a survey of the constructed facility. This procedure is not recommended except for unusual encroachment circumstances and then, only with prior approval of the Hydraulics and Drainage Section.

Highway Encroachment on Unidentified Floodplains

Encroachments which 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.

2.4 State Drainage Law

2.4.1 Origin of Law

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

2.4.2 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.

2.4.3 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.

2.4.4 Precedence of Law

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. The designer has the responsibility to notify the Attorney General's Office if any portion of a project is expected to violate any accepted laws.

2.4.5 Classification Of Waters

State drainage laws originating from common law, or court-made law, first classified the water that was being dealt with, after which the rule that was pertinent to the particular classification was applied to obtain a decision.

The first step in the evaluation of a drainage problem is to classify the water 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 ground waters which have entered, and now flow in, a well-defined natural or man-made 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 which have escaped from a watercourse (and its overflow channels) and flow or stand over adjoining lands. They remain flood waters

until they disappear from the surface by infiltration or evaporation, or return to a natural watercourse.

- Ground Waters - In legal considerations, ground waters are divided into two classes, percolating waters and underground streams. The term "percolating waters" generally includes all waters which 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 through undefined paths and to take them out of the percolating class, the existence and course of a permanent channel must be clearly shown. Underground streams are waters passing through the ground beneath the surface in permanent, distinct, well-defined channels.

2.5 State Water Rules

2.5.1 Basic Concepts

Two major rules have been developed by the courts regarding the disposition of surface waters. One is known as the civil law rule of natural drainage. The other is referred to as the common enemy doctrine. Modification of both rules has tended to bring them somewhat closer together, and in some cases the original rule has 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. This 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 ground water 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.

2.5.2 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. The owner of upper lands has an easement over lower lands for drainage of surface waters. These natural drainage conditions can be altered by an upper proprietor provided the water is not directed in such 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 they 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 their land so as not to unreasonably or unnecessarily damage the property of others.

Under the reasonable use rule, each property owner can legally make reasonable use of their land, even though the flow of surface waters is altered thereby and causes some harm to others. However, liability attaches when their 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 their 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?

2.5.3 Stream Waters

Where natural watercourses are unquestioned in fact and in permanence and stability, there is little difficulty in application of the reasonable use 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 cause 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.

2.5.4 Flood Waters

Considering flood waters 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" has frequently been applied, 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.

The "English Rule" has been modified by the "Reasonable Use Rule" which states in essence that each landowner is restricted to a reasonable exercise of their own right and a reasonable use of their property in view of the similar right of their 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 their property for the benefit of agriculture, manufacturing, irrigation, etc. pursuant to the reasonable development of their 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.

2.6 Statutory Law

2.6.1 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. In general, statutes have been enacted that affect drainage in one way or another in the subject areas described below.

2.6.2 Eminent Domain

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

In Connecticut this is accomplished for highway purposes, through condemnation. It is important to remember, however, that whenever any property is taken under eminent domain, the private landowner must be compensated for the loss.

2.6.3 Drainage Statutes

There are several statutes which empower the Commissioner of ConnDOT to construct, drain, and control highways. Other statutes exist to specifically address drainage of properties, blockage of watercourses, etc. Some of these statutes are listed below for reference:

- Obligations pertaining to culvert capacity and length

CGS Section 19a-336

The State is obligated to maintain culverts of adequate capacity to conduct natural watercourses under state highways. The State is only obligated for piping that portion of the system within the highway area which is necessary to provide a safe highway for the traveling public. The State is not obligated to pipe the entire width of the right-of-way.

CGS Section 52-461

It is the obligation of each property owner to maintain the flow of a stream so as not to create an obstruction to the flow of the watercourse. Should an abutting owner desire to extend a state culvert which conducts a stream, it is their responsibility to accomplish this totally at their expense. The private installation must be of adequate capacity so as not to create an obstruction to the flow of the watercourse. Permits issued by the Department to private owners for such extensions should carefully state these requirements.

- Blockage of watercourses

There are two statutes relating to procedures for clearing watercourses:

CGS Section 13a-138 empowers the Commissioner of Transportation to clear a watercourse.

CGS Section 52-461 must be administered by the selectman in the particular town involved

- Upon discovering an obstructed watercourse causing a real or potential problem to state facilities, notify the property owner by certified mail of the obligation to remove the obstruction. State in this letter that if the obstruction is not removed within 30 days, we will refer the matter to the Attorney General's Office for legal resolution. If the obstruction has not been removed after 30 days, request the Attorney General's Office by memorandum to investigate the matter for appropriate action. The request should detail all the facts of the case and attach maps, photographs and all records and correspondence available.
- If the obstruction causes imminent danger to highway facilities or flooding which is hazardous to the highway user, take immediate action to remove the obstruction with state forces. A full report should be sent to the Attorney General's Office for such action as necessary to avoid future problems.

2.6.4 Water Rights

The water right itself which 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.
- **Relocation of watercourses** – Neither an individual nor a governmental agency may relocate a watercourse so as to deprive the abutters of their riparian rights without obtaining recorded releases from such abutters. Likewise, an individual and a government agency may obtain the rights necessary to change the location of the point of discharge of a watercourse onto an abutter's property.
- **Erosion of private property** – The erosion of private property caused by state highway drainage outlets such as leakoffs, culverts and storm sewers is the responsibility of the State to correct. These cases are quite frequent and may be costly if the condition is not corrected as soon as it is discovered. However, when land use at the outlet has been modified and erosion is caused by the owner or developer's activities, the State should not accept the responsibility of correcting the condition.

Where the State possesses drainage rights by prescription, it may be necessary to acquire a drainage right-of-way in order to resolve the problem, or to extend a present drainage right-of-way where the erosion occurred beyond the easement.

- **State's right to outlet highway drainage** – The State has the well-established right to outlet highway drainage into a natural watercourse within the highway right-of-way, provided such a watercourse is the eventual natural recipient of the collected runoff. This right is not impaired even though the State, in the performance of its governmental duties, increased the rate or quantity of water discharged from highway by changing the runoff characteristics of areas being drained or by installing pipe and other drainage facilities. The right is impaired only if the State increases the size of the drainage area from which the runoff is collected. If the natural drainage area is exceeded, then deeded drainage rights must be acquired from all owners adversely affected by the diversion. Should the State make drainage improvements within an existing drainage area draining to a watercourse, which results in an increased rate of flow causing an existing pipe on private property to be of inadequate capacity, the State is not obligated for improvements necessary on private property. The State also has the right to increase the size of a culvert conducting a watercourse under a state highway without acquiring drainage rights. The State must also conform to all applicable environmental laws, some of which are noted in Section 2.6.5.

Generally, what is important 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.

2.6.5 Environmental Law

In addition to Federal laws which affect designs with regard to pollution, dredging and filling, Connecticut has several statutes pertaining to these aspects as well as flooding, dams and wetlands. The present statutes are as follows and are under the jurisdiction of the ConnDEP.

- Inland Wetland and Watercourse - CGS 22a-36 to 45

This statute and regulations cover all inland wetlands and watercourses in the state. The wetlands are defined by soil type and the watercourses are defined as any body of water, standing or flowing, natural or artificial. Any impacts on wetlands or watercourses require a permit. However, if an activity requires a tidal wetlands or structure and dredging permit, then the ConnDOT is exempt from the inland wetland and watercourses process.

- Stream Channel Encroachment Lines - CGS 22a-342 to 349

The State has established encroachment lines along major flood prone rivers. Any encroachment or activity riverward of these lines regardless of elevation requires a permit.

- Water Diversion – CGS 22a-365 to 378

A diversion is any withdrawal, alteration, modification or diminution of the volume of water that would occur at a point at any given moment. Wells or well systems withdrawing more than 50,000 gallons per day, stream relocation, stream channelization, stream bypass and storm drainage system collecting surface water runoff from areas of 100 acres or more in size; and dams and impoundments are among the activities requiring permits. However, any DOT highway project is exempt from this permit process if other permits are required from D.E.P.

- Dam Maintenance and Construction - CGS 22a-401 to 411

Construction, alteration, rebuilding, substantially repairing, adding to, replacing or removing a dam requires a permit.

- Flood Plain Management - CGS 25-68b through 25-68h

Any action proposed by any state agency in, or affecting a floodplain, or which impacts storm drainage, including but not limited to structures, obstructions, site development, grants, loans, or programs regulating flood flows must be certified and approved by D.E.P. This statute enforces FEMA flood plain management standards.

- CGS 13a-94 CGS (Construction over or Adjacent Streams) requires that all structures to be built over, or structures or embankments to be built adjacent to, streams in connection with state highway projects conform to the requirements of DEP as determined by Sections CGS 22a-342 to 22a-348.
- Structures and Dredging Permit – CGS 22a-359 through 22a-363f for regulated activities conducted waterward of the high tide line in tidal, coastal, or navigable waters.
- Tidal Wetland Permit – CGS 22a-29 –for regulated activities conducted within tidal wetlands

2.6.6 Sewage Draining into Highway Area

When it is discovered that contaminated water has been directed to a state highway system or is outletting into a highway area, an investigation into the source should be conducted. The following procedure should be pursued:

1. Notify the property owner by certified letter that he is creating a nuisance within the state highway area in violation of the Sanitary Code of the State of Connecticut, and request that the nuisance be abated within a given period, generally 30 days.
2. If the property owner ignores this request, refer a copy of the correspondence to the local health officer asking that action be taken by him/her under their statutory authority. It may sometimes be necessary for the District Drainage Engineer to assist the local health officer in making a definite determination as to the source of contamination. The use of “Uranine Water Soluble” (sodium fluorescein) may be used as a tracing material for this purpose.
3. If the local health officer fails to make acknowledgement or to take action within 15 days, refer the entire matter to the Director of the Bureau of Sanitary Engineering, State Health Department, requesting positive action to insure the abatement of the nuisance.
4. During the foregoing procedure, send copies of all correspondence to: (1) Director of Environmental Health Services, State Health Department, (2) Department of Environmental Protection, when the case involves pollution of “waters” of the State.

2.7 Local Laws

Local governments have ordinances and codes which require consideration during design. For example, zoning ordinances can have a substantial effect on the design of a highway and future drainage from an area. On occasion, a question may arise as to whether the State must comply with local ordinances. Generally, the State is not legally required to comply with local ordinances except where compliance is required by specific state statute. The flood management regulations require that local requirements be followed if more stringent than state requirements.

2.8 Role Of The Designer

2.8.1 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 the designs; and, secondly, work closely with the Attorney General's office, as necessary, in the preparation and trial of drainage cases. Thirdly, become familiar with drainage rights and other property considerations. The following should be considered in the development of drainage designs.

- At any location where highway drainage is to be discharged onto abutting property, except where such drainage is into an existing stream at a point within the limits of the highway right of way, appropriate rights must be secured. These include the concentrated discharge from any culvert, storm drainage system or channel regardless of whether the waters so discharged primarily originate within the highway limits or from land beyond the highway limits.
- The outlets shall be located, insofar as possible, on or adjacent to the boundary line between abutting owners. Where one owner has extensive frontage along the highway and there is a drainage depression or swale in that frontage, the outlet may be located at or near the depression. In order to minimize property damage, it may be necessary to install pipe to the rear of buildings or to minimum depth of a building lot.
- The course of flow of such drainage should be investigated for a sufficient distance, particularly in urban or suburban areas, to insure that other downgrade owners will not be adversely affected. It may be necessary to purchase drainage rights from owners not immediately adjoining the highway in such cases.
- The course of such outlet drainage shall be shown on the construction plans for a minimum distance of 61 m (200 ft.) or shown to terminate by dissipation or entrance into a watercourse or body of water. New or updated drainage rights should be dimensioned on the plans. No drainage shall be outletted into a wetland or natural depression which does not have an adequate outlet unless proper rights for storage in, or provision for an adequate outlet from, such an area are secured or provided.
- It should be noted that the damage done to a property, by the establishment of a drainage outlet, is determined by its location and its capacity. Frequently the rights to install a small or moderate size outlet can be obtained at a reasonable cost but rights for large outlets can be obtained only with great difficulty and at considerable expense. In court or referee decisions, the full capacity of the outlet pipe is usually the determining factor, with an almost invariable refusal to consider the lesser capacity of conduit lines leading to the outlet conduit or lesser capacities of inlets leading into the entire system.
- It is important that outlets be of adequate capacity to pass the inlet capacity and any reasonable increase in inlet capacity which may be provided in the future. No excess capacity should be provided. Careful adherence to this policy will minimize the cost of securing the needed drainage rights.
- The acquisition of property, flowage or flooding rights beyond the normal highway line may, in certain circumstances, result in a lower net cost of the project by allowing the installation of smaller culverts and storm drainage systems.

2.8.2 Investigating 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 complaining party, and not accepting anyone's preconceived conclusions. All facts must be assembled and analyzed before deciding on what happened and why it happened. Also, other agencies should be listed that could possibly have responsibility for a remedy to the flooding.

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

Determine Facts About The Complaint

- Show on a map the location of the problem on which the complaint is based.
- Clearly determine the basis for the complaint (what was flooded, complainer's opinion as to what caused the flooding, description of the alleged damages, dates, times and duration of flooding).
- Briefly relate the history of any other grievances that were expressed prior to the claim presently being investigated.
- Obtain approximate dates that the damaged property and/or improvements were acquired by those claiming damages.
- Collect facts about the specific flood event(s) involved.
- Rainfall data (dates, amounts, time periods and locations of gages). Rainfall data are often helpful regardless of the source.
- Document observed highwater information at or in the vicinity of the claim. Locate highwater marks on a map and specify datum. Always try to obtain highwater 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 the Department, such as newspaper accounts, witnesses, measurements by other agencies (USGS, Corps of Engineers, Natural Resources Conservation Service (NRCS) and individuals), maps and Weather Bureau rainfall records.
- State facts about the highway crossing involved.
- Show 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 town 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.
- Determine peak rates of flow for a range of return frequencies.

- Give the original drainage design criteria, or give capacity and related 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.)

This list of Determining Facts About the Drainage 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 beneficial to have available hydraulic design documentation as outlined in Chapter 1. A report should be prepared with a response to the complainer explaining the results of the investigation. Documentation of the facts and findings is important in the event there is future action.

2.8.3 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 ConnDOT should be able to undertake such improvements without 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
- the proposed improvement is "reasonable"

2.8.4 Witness Obligation

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.

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 loud 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.

References

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

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Appendix A – Pre-Submission Quality Assurance/Control Guideline	3.A–1	
Appendix B – Submission Check List	3.B–1	12/03 (p. 6)
Appendix C – In-House Development of Hydraulic Requirements for Crossings Conveying a Waterway Greater Than 2.59 km ² (1 mi ²)	3.C–1	5/02 (new)

3.1 Overview

3.1.1 Purpose

The purpose of this chapter is to provide guidance to the designer when advancing a hydraulic or drainage design through the development of a project. It is understood that each project is unique; however, the general guidelines discussed in this chapter can be applied as a project progresses from its preliminary design to its final completion.

The objective of the Hydraulics and Drainage Section is to ensure that all drainage and hydraulic features of highways are designed, constructed and maintained in accordance with sound engineering and in conformance with current design practices. This objective will be attained with proper application of engineering principles.

3.1.2 Project Development

The preliminary, semifinal and final design phases of a project discussed in this chapter focus on the hydraulic and/or drainage requirements at each stage of design. The process of developing a proper drainage design involves coordination with many engineering and environmental groups. The designer should be familiar with various aspects of highway projects including:

- planning and environmental assessment
- right of way acquisition
- surveying
- Departmental policy and procedures
- roadway design
- pavement design
- structural design
- environmental design
- geotechnical and soils aspects
- material properties (strengths/durability, etc.)
- quantity and cost estimation
- construction
- maintenance
- underground utilities
- legal issues

It is neither expected nor necessarily recommended that the designer become an expert in every area of highway engineering, but it is imperative that the engineer be able to understand and communicate with personnel in other disciplines.

Drainage characteristics are affected by roadway geometry and vice versa. The engineer performing the drainage design should not only understand the drainage aspects and drainage criteria of the project but should also be conversant with roadway geometric criteria and design procedures. This would ensure that an optimum balance is achieved between drivability, traffic safety, drainage and economics.

This chapter was prepared to assist the designer in developing and coordinating the drainage aspects of the project. The Consulting Engineers Manual, the Bridge Design Manual and the various pertinent sections of the ConnDOT Drainage Manual should be referred to as necessary.

3.2 Permit Determination

The Department's Office of Environmental Planning (OEP) coordinates, reviews and processes the state and federal permits involved on a specific project. Early in the project, prior to preliminary design, a request is sent to OEP to conduct an environmental review of a project. This process and associated permits that are required for a project are discussed in their guidelines entitled, "Water Resources Coordination and Permit processing Manual." The purpose of this initial environmental evaluation is to identify potential environmental, historical and socioeconomic concerns on a given project.

Chapter 800 of the Consulting Engineers Manual also discusses the various permits that may be involved. Potential permit involvement may include, but is not limited to, wetland impacts, fisheries, Army Corps permits and tidal impacts. The designer should be familiar with various permits that may be involved as the project advances through its design phase. This will ensure consistency between Department criteria and state/federal regulations.

The Hydraulics and Drainage Section's involvement with permits is limited to Floodplain Management (FPM) Stream Channel Encroachment Lines (SCEL), Drainage Diversions and Dam Construction. Section 3.11 of this chapter discusses these specific permit requirements in more detail.

For State Design projects, a request for a permit needs determination should be submitted early in the design phase to the Hydraulics and Drainage Section. Oftentimes it is requested at the same time the environmental review is requested from OEP. Include a location plan, scope of work and construction limits of the project. This will typically provide the necessary information to determine the potential environmental impacts associated with floodplain, encroachment, diversions or dams.

For Consultant Design projects, the potential permit involvement is addressed in the preliminary design report.

3.3 Scope Review Meeting

The Hydraulics and Drainage Section may be requested to attend a scope meeting for a project being designed. Pertinent information regarding the project should accompany the request. Appendix B of this chapter provides an outline of the information necessary to prepare for the scope meeting.

Typical issues that are discussed by the Hydraulics and Drainage Section may include Floodplain Management impacts, hydraulic design involvement, other projects within the project vicinity and other pertinent aspects related to the design of the project.

3.4 Drainage Design Development at the Preliminary Design Phase for Roadway Projects

3.4.1 General

The preliminary design submission is provided when plans are approximately 35% complete. The information that should be provided in this phase is outlined in the preliminary design checklist, found in Appendix B. This checklist should be completed and included in this submission. It is designed to serve as a tool for the designer, ensuring that many of the common drainage aspects found at this stage of design are addressed. This checklist also provides necessary information to the Hydraulics and Drainage Section to expedite the review process.

3.4.2 Preliminary Design Report

The preliminary design report should include a section on drainage and/or hydraulic involvement. The report should include an overview contour map showing the watershed areas to major cross culverts and all environmental permit requirements as outlined in Section 3.11. Any special conditions or drainage issues that the designer is aware of, (i.e. diversions, existing flooding problems, hydraulic controls or limitations) should be addressed in the preliminary design report.

3.4.3 Preliminary Plans

The plans should include a conceptual layout of the proposed drainage system. Proposed drainage outlets should be established and a preliminary estimate prepared. There should be adequate information to allow the reviewer to concur with the conceptual layout of the proposed system and determine how it may impact the existing drainage facilities. Proposed storm drainage systems, swales, ditches and channels should also be included on the plans to depict the drainage patterns specific to the site. The proposed system should be evaluated with respect to its effect on adjacent properties, the need for additional improvements to the existing system and to identify any diversions that may occur with the preliminary layout. Existing property lines and drainage rights, proposed taking lines and approximate slope limits should be shown.

3.4.4 Watercourses

The determination of whether a crossing conveys a watercourse is important. It defines the legal obligations by the State, establishes the right of way requirements and determines the storm frequency for the design discharge. The legal definition of what constitutes a watercourse is found in Chapter 7, Channels. Oftentimes the establishment of the watercourse can be best determined by field review. A stream can be either perennial or intermittent to be considered a watercourse. Watercourses should be shown and labeled on the plans. The designer should review Sections 7.2.1 and 7.2.2 of the Channels chapter to determine the design criteria and ConnDEP requirements.

3.4.5 Permits

The designer should be aware of environmentally sensitive areas within the project early in the design phase. Mitigative measures and alternative designs may need to be considered to comply with federal and state regulations. There should be ongoing coordination between the

designer and Office of Environmental Planning to ensure that the design of the project is consistent with state statutes and federal regulations. Section 3.2 of this chapter discusses the Hydraulics and Drainage Section's involvement with permits. The processing and coordination of the associated permits is discussed in Section 3.11, Permit Requirements.

3.4.6 Fish Passage

A meeting should be held with a fish biologist from ConnDEP at the project site to review all streams and determine which crossings and channels need to be designed for fish passage. **The meeting should be held prior to the completion of Preliminary Design.**

3.4.7 Watershed Area Maps

Contour maps should be provided showing watershed areas to all cross culverts. A list of existing culvert or bridge crossings should accompany the map.

3.4.8 Unusual Designs/Special Provisions

The designer should identify the need for any unusual designs. Early in the design phase the designer should be aware of possible storage facilities (detention basins), pump stations, tidal impacts or existing drainage problems that local property owners or the Town may be concerned with. The Hydraulics and Drainage Section may offer guidance to the designer regarding these issues. Special drainage provisions, if required, should be included in this submission. The designer should review the various chapters in this manual for additional guidance.

3.5 Hydraulic Design Development for Structure Type Studies

3.5.1 General

The preliminary design phase for structures is referred to as the Structure Type Study Phase. The purpose of this submission is to provide alternative structures that may be feasible based on design criteria and environmental conditions.

It is important at this stage of the structural design not only to develop water surface models with structures that are hydraulically adequate but also to develop models that consider the environmental permit requirements as related to fish passage, floodplain management and stream channel encroachment line involvement.

3.5.2 Procedure

The procedure to prepare the hydraulic studies associated with the Structure Type Study for crossings conveying a drainage area equal to or greater than 2.59 km² (1.0 square mile) includes the following:

Step 1... Ensure that the hydraulic engineer performing the hydraulic design is approved by ConnDOT. Given the complexity and time required for hydraulic analyses, it is important that the hydraulic engineer is familiar and understands the Department's procedures and requirements for advancing a hydraulic design. The procedure and guidelines for approval are outlined in Section 1.2.4 of this Manual.

Step 2... Submit the hydrology to the Department with recommended discharges for the crossing. Chapter 6 of this Drainage Manual discusses the various accepted methodologies that can be applied for drainage areas of this magnitude. The documentation that should be included in this submission is discussed in Section 6.1.5.

Step 3... Following approval of the design discharge rates, develop the preliminary hydraulic and environmental models as necessary. Chapter 9, (Bridges) outlines the requirements for the design model. The hydraulic engineer should also refer to the guidelines prepared by ConnDEP entitled "**Hydraulic Analysis Guidance.**" The engineer should prepare the necessary hydraulic computer models and incorporate the various alternates in the design runs.

The potential for scour should also be addressed for each structure. Chapter 9, (Bridges), discusses the scour aspects and requirements for a structure.

Step 4... Submit the preliminary hydraulic models, preliminary scour report and environmental models in accordance with the Documentation sections outlined throughout this manual. The foundation type should be selected with consideration to scour at the bridge.

For deck or superstructure widening or replacements, the scour vulnerability of the substructure should be evaluated. Countermeasures may be required or complete replacement may be recommended based on the scour history and safety aspects. Addressing the impacts of scour early in the design phase will provide a better understanding not only of the vulnerability of the substructure but also will provide a basis for modifying the scope of the project if a full replacement is economical.

The structure type selection must consider environmental permitting conditions and requirements. An alternate may be feasible with respect to hydraulic adequacy and scour, but may not be practicable with respect to floodway impacts, fish passage or floodplain involvement. Therefore the structure type study needs to address all the impacts to the site to ensure that any of the structure types recommended can be constructed and can meet the necessary permit requirements. The procedure for the design of culverts with tributary watershed less than 2.59 km² (1.0 mi²) is outlined in Section 3.8 in more detail.

3.5.3 Submissions

The Structure Type Study Report is typically submitted concurrently with the Preliminary Design Submission and should include:

- Hydrology Report which was approved prior to performing the hydraulic analysis
- A Preliminary Hydraulic Analysis including the approved design discharges with all the requirements outlined in the various chapters of this Manual.

For a structure crossing an established Floodway...

- A Floodway Model with all the associated runs is necessary to evaluate the potential impacts to the floodway. This model incorporates the FEMA discharges that were used to establish the floodway limits. **These discharges may be different from the design discharge used in the hydraulic model.**

For a structure within stream channel encroachment lines...

- A Stream Channel Encroachment Line model is necessary. **This model uses the SCEL discharges that were used to develop the water surface profile for the stream channel encroachment limits of the site.** Similar to the Floodway model, the proposed alternates should be evaluated using the stream channel encroachment line discharges.

For a new or rehabilitated bridge...

- [Chapter 9 (Bridges) describes the various reports generated under the Department's Bridge Scour Evaluation Program for all existing bridges over waterways with span lengths greater than 6.1 m (20 feet).] The designer should review this information, as it may prove useful in developing the scour design and recommended course of action for a specific structure. A scour analysis will not be required for existing structures that have an NBIS 113 rating of 8 and do not require substructure replacement or modification.
- For a structure with an NBIS rating less than 8 and new bridges, a preliminary scour report should be included with this submission. As scour parameters differ depending on the substructure being recommended, a qualitative scour assessment report should be developed to support the selection of foundation type for each recommended structure.
- A rehabilitated structure should be carefully evaluated to determine any evidence of scour or scour history. Scour corrective measures should be addressed in the report. Structures assessed as low risk, should be assigned an appropriate NBIS Item 113 rating.

In summary, the Structure Type Study and preliminary design submission should include enough information for the Department to evaluate the approach and feasibility of the design. The preliminary hydraulic models required for waterway crossings need to be developed at this stage to ensure that any of the proposed structure types would be suitable for the site. Based on all the available information, the designer should offer a recommended alternate.

3.6 Drainage/Semifinal Submission

3.6.1 General

The semifinal design is submitted when the plans are 60% to 70% complete. However, a separate drainage submission is recommended when the drainage aspects of the project appear to be complex or when the drainage facilities may involve right of way impacts. Concurrence of the drainage design by the Department for these types of projects should be obtained to ensure the project schedule is maintained. A pre-submission Quality Assurance/Control Guideline is included in Appendix A. This should be used as a guide to ensure the major aspects of the drainage design have not been overlooked. The design checklist for drainage/semifinal review found in Appendix B of this chapter provides the necessary information for the Hydraulics and Drainage Section to complete the review. The completed checklist should be included with the drainage and semifinal submission.

3.6.2 Resolution of Previous Drainage Comments

The reviewer's initial task is to check the previous comments made at the preliminary design stage and review the plans and accompanying responses to determine how the outstanding issues have been incorporated or addressed. If a comment cannot be incorporated (or the designer does not agree with a comment), then a clear response should be provided, justifying its non-inclusion.

3.6.3 Condition Survey

A condition survey should be conducted for existing drainage facilities (pipes, structures, swales, ditches) that are to remain in use within the project limits to ensure their condition is sound and replacement is not warranted. This should be performed early in the final design phase so that the proposed design includes all systems that are to be replaced. The approach and documentation with respect to verifying the condition of existing facilities that are to remain in use are discussed in Chapter 4, Appendices A and B. A separate independent report shall be included with the Drainage/Semifinal submission.

3.6.4 Drainage Computations

All roadway reconstruction projects require drainage computations. Design criteria and design elements pertinent to drainage facilities are referenced throughout this manual. Drainage design computations should include swale, outlet protection, gutter flow analysis, storm sewer design, and hydraulic gradeline (HGL) calculations as well as computations for other pertinent drainage facilities, as outlined in Chapter 1, Section 1.6. It is important to include the supporting documentation used to develop the drainage design so that the design assumptions and calculations can be verified. Drainage computations should identify structures by station and offset rather than by numerical identifier. If this is not feasible, include an index to correlate the structure with its location on the plans. Topographic mapping should be provided with the watershed areas delineated accordingly.

The drainage system should be brought to a suitable outlet. The drainage computations should show that the proposed improvements will not adversely impact the existing downstream storm system or property owners. If the existing drainage system is found inadequate, an upgrade will be

designed in conformance with the criteria outlined in this manual. Additional drainage rights may need to be acquired if upgrading the storm drainage downstream of the project is necessary.

3.6.5 Right of Way Involvement

Right of way acquisitions can prove to be one of the most expensive and time consuming aspects of a highway project's development. The designer should account for such costs and difficulties when determining suitable drainage appurtenances. Therefore the potential for new or additional drainage rights and easements should be a consideration in developing the drainage design. The existing and proposed rights should be shown on the plans. Properties affected by diversions should also be shown on the plans so that proper rights can be acquired.

3.6.6 Culverts

Chapter 8 (Culverts) discusses the specific design criteria related to culverts. The designer should also refer to Chapter 4, (Culvert Repair, Materials, and Structural Design) if the existing culvert will remain in use. Minor and small culvert design computations should be included with the documentation with a design data sheet similar to that found in Chapter 8, Appendix A.

In areas where cross culverts are being extended or replaced or where outlet protection is being proposed, a profile or cross section of the natural ground should be provided to show how the inverts will tie into the existing topography. The need for additional rights or easements should be addressed.

3.6.7 Outlet Protection

Riprap splash pads are required to dissipate the velocity and prevent erosion at the exit of a pipe. Guidelines for the design of riprap and bedding are discussed in Chapter 8, Section 8.7. Splash pads at inlets are typically not required. Riprap proposed at an inlet requires justification.

Outlet protection should be shown on the plans. A typical detail should also be included with the drawings.

3.6.8 Drainage Patterns

Existing and proposed drainage patterns with respect to pipes, ditches, channels and swales should be shown on the plans by directional flow arrows. The direction of flow should be shown 61 m (200 feet) beyond any drainage outlet, or shown to terminate by dissipation or entrance into a watercourse or body of water. This will show where the proposed drainage is discharging and establish whether additional drainage rights are required.

3.6.9 Grading Plans

Intersection grading plans should be included in the semifinal submission. The grading should be designed such that ponding and icing conditions are avoided. Inlets should be located at the low points. Include top of frame elevations on the grading plan.

3.6.10 Permits

The Hydraulics and Drainage Section reviews permit applications for Floodplain Management, Stream Channel Encroachment Lines, dam construction, and drainage diversions. Section 3.11 of this chapter discusses permit requirements. The necessary applications should be ***forwarded to the Office of Environmental Planning*** for submission to ConnDEP.

3.6.11 Temporary Drainage

Provisions for temporary drainage should be designed in accordance with the procedure outlined in Chapter 6, Section 6.15. These facilities should be designed with the same care as the permanent facilities using the reduced design discharge.

3.6.12 Meetings

There are occasions, depending on the complexity of a project, where a meeting is requested by the designer to discuss specific concerns or questions. Prior to the meeting, an agenda should be prepared by the designer to facilitate the meeting and allow the Hydraulics and Drainage Section time to review and become familiar with the material. It will also ensure that the appropriate parties are represented.

3.7 Final Plans for Review Submission (Roadways and Structures)

3.7.1 General

The final plans are submitted for review when they are approximately 85% to 90% complete. Should the project involve both structure and highway design, then separate but coincident submissions should be prepared. The highway design submission shall be accompanied by the final design checklist. (See Appendix B.)

3.7.2 Resolution of Previous Drainage Comments

Comments made previously in the semifinal design phase should be incorporated or addressed accordingly. As discussed in the previous section, justification is required for all comments that have not been incorporated.

3.7.3 Drainage Report and Plans

A final drainage report and final plans should be submitted for review.

3.7.4 Permits

At this stage of design, the necessary permit applications should have been forwarded to ConnDEP through the Office of Environmental Planning for processing and approval.

3.7.5 Structures

The following final design submission requirements refer to structures over waterways. The final design submission includes the various water surface models discussed in Section 3.5. However rather than several alternates evaluated for the Preliminary Design phase, only the proposed structure chosen during the structure type selection process is reflected in the hydraulic, FEMA, and SCEL models. The designer should provide the following design reports for final submission:

- Final Hydraulic Design Report – reflecting the proposed structure chosen to be advanced to final design.
- Final Scour Report – if the proposed structure is a bridge. A scour report is not required for a culvert with a concrete bottom. Chapter 9 (Bridges) discusses the requirements for the Scour Report.
- Final Floodway Analysis Report – if there is an established floodway
- Final SCEL Report – if there is involvement within Stream Channel Encroachment Lines. These water surface models are similar to those submitted during the Structure Type Study phase however, only the proposed structure model is compared with the existing condition model rather than all the feasible alternates.

3.7.6 Final Structure Plans

The following data for riverine (inland) structures, as shown in Table 3-1, shall be included in tabular form on the general plan sheet:

Table 3-1

Drainage Area	sq. km (sq. mile)
Design Frequency	year
Design Discharge	cms (cfs)
Average Daily Flow Elevation	m (ft) - <i>indicate</i> Estimated or Observed. <i>For observed, show date.</i>
Upstream Design Water Surface Elevation	m (ft)
Downstream Design Water Surface Elevation	m (ft)
Maximum Scour Elevation	m (ft)
Frequency	year
Discharge	cms (cfs)
Worst case scour sub-structure unit	<i>indicate</i> Abutment # or Pier #

- The design water surface and average daily flow elevations, from Table 3-1, shall be shown graphically on the cross section of elevation view. The upstream water surface elevation is typically taken at the approach cross section.
- Scour information not applicable for culverts (enclosed conduits).

The following data for tidal structures, as shown in Table 3-2, shall be included in tabular form on the general plan sheet in lieu of Table 3-1 above:

Table 3-2

Mean Low Water	m (ft)	
Mean High Water	m (ft)	
High Tide Line (1-Year Tide)	m (ft)	
10- Year Tide	m (ft)	
100- Year Tide	m (ft)	
Design Frequency/Event*	Tidal: year/event	Riverine: year/event
Design Discharge	cms (cfs)	
Design Water Surface Elevation – Ebb Direction	m (ft)	
Design Water Surface Elevation – Flood Direction	m (ft)	
Maximum Scour Elevation	m (ft)	
Frequency/Event*	Tidal: year/event	Riverine: year/event
Discharge	cms (cfs)	
Worst case scour sub-structure unit	<i>indicate</i> Abutment # or Pier #	

* Event means other occurrence such as average daily flow, mean high water, mean low water, etc.

- Design frequency/event for tidal structures may be based on riverine floods, tidal storm surge or a combination of tidal storm surge and riverine flooding. See Sections 9.3.8 and 9.4.5 for further discussion.

Examples:

- (1) The **Design Discharge Elevation** at a structure was determined based on a 100-year riverine flood in combination with a 10-year tide. The Design Frequency/Event information in Table 3-2 would be completed as follows:

Design Frequency/Event	Tidal: 10-year	Riverine: 100-year

- (2) The **Maximum Scour Elevation** is based on a 500-year tidal storm surge in combination with an average daily riverine flow. The Design Frequency/Event information in Table 3-2 would be completed as follows:

Frequency/Event	Tidal: 500-year	Riverine: Average Daily Flow

- The design, mean high and mean low water elevations shall be shown on the cross section view.

For projects requiring the use of temporary hydraulic and/or water handling facilities, the tabulated stage/discharge data shown on Table 3-3 for riverine structures or Table 3-4 for tidal structures, shall be included, typically on the stage construction/water handling plans. The temporary design water surface elevations shall also be shown graphically on the cross section or elevation view. The minimum top elevation of cofferdams being utilized to isolate or divert the stream flow away from the work area shall be indicated on the plan. This information is provided to assist the contractor during construction. Section 9.6.12 of Chapter 9, Bridges, discusses the hydraulic design of temporary hydraulic facilities.

Table 3-3

Average Daily Flow	m ³ /s (cfs)
Average Spring Flow	m ³ /s (cfs)
2-Year Frequency Discharge*	m ³ /s (cfs)
Temporary Design Discharge	m ³ /s (cfs)
Temporary Design Frequency	Year
Temporary Water Surface Elevation Upstream	m (ft)
Temporary Water Surface Elevation Downstream	m (ft)

* not included if same as temporary design discharge

- The average daily and spring flows are determined by the method described in Chapter 6, Hydrology, Section 6.16. The temporary design discharge is determined by the procedure described in Section 6.F.1.

Table 3-4

Mean Low Water	m (ft)	
Mean High Water	m (ft)	
High Tide Line (1-Year Tide)	m (ft)	
Temporary Design Frequency	Tidal: year	Riverine: year
Temporary Design Discharge	cms (cfs)	
Temporary Design Elevation	m (ft)	

3.7.7 Environmental Permit Aspects

The Hydraulics and Drainage Section offers technical assistance in reviewing the various water surface profile models associated with structures.

Upon concurrence by the Hydraulics and Drainage Section, the FPM and/or SCEL applications should be forwarded to the Office of Environmental Planning. The application is then submitted to ConnDEP for certification. Permit requirements for a structure are discussed in Section 3.11 of this chapter.

The following standard conditions for approval of stormwater management standards taken from Section 25-68h-3(f) of the Flood Management Statutes and Administrative Regulations shall be incorporated into all ConnDOT projects:

- (1) All construction work shall incorporate best management practices to minimize soil erosion and sedimentation and conform with the "Connecticut Guidelines for Soil Erosion and Sediment Control."
- (2) All fill shall be clean, material free of stumps, rubbish, hazardous, and toxic material.
- (3) Contractor shall remove equipment and materials from the floodplain during periods when flood warnings have been issued or are anticipated by a responsible federal, state or local agency. It shall be the contractors responsibility to obtain such warnings when flooding is anticipated.
- (4) Contractor shall notify the Commissioner seven days prior to starting work on-site.
- (5) Once work is initiated, it shall proceed rapidly and steadily until completed and stabilized in order to minimize use of temporary structures and to minimize soil erosion.
- (6) Work shall not be conducted in or adjacent to watercourses and reservoirs used as public drinking water supply sources without further coordination with the water supply utility and Department of Health Services.
- (7) All temporary structures, cofferdams, and fill shall not impede the movement of flood flows and shall be removed at the completion of their use. The design of such temporary structure, cofferdams and fill shall be based on this manual, where applicable.
- (8) The applicant or his agent shall permanently maintain the proposed facility.

3.8 Process for Design of Culverts (State Design)

The following applies to projects which are not assigned to Consultant Engineers:

- **Drainage areas less than or equal to 81 ha (200 ac)...**The State Design Section typically designs cross culverts conveying drainage areas less than or equal to 81 ha (200 acres). Culverts should be designed in accordance with the criteria discussed in Chapter 6 (Hydrology) and Chapter 8 (Culverts).
- **Drainage areas between 81 ha (200 ac) and 259 ha (1 sq. mi)...** Culverts conveying drainage areas between 81 ha to 259 ha (200 acres to 1 sq. mile) can be designed either in State Design or in Hydraulics and Drainage. The methodologies used to develop the design discharges are different from an 81 ha (200 acre) culvert crossing as discussed in Chapter 6 of this manual. If requested, the Hydraulics and Drainage Section can assist State Design with the analysis. The design includes an abbreviated analysis. Coordination is necessary between the Hydraulics and Drainage Section and State Design to determine the survey requirements for the site.
- **Drainage areas greater or equal to 259 ha (1 sq. mi)...** Culverts conveying drainage areas of this magnitude require detailed hydraulic designs. The analysis is performed by the Hydraulics and Drainage Section as requested. The advancement of hydraulic designs for projects assigned to the State Design Section is outlined in Section 3.9.

3.9 Hydraulic Design Process with Hydraulics and Drainage Performing the Design (Structures)

3.9.1 General

The Hydraulics and Drainage Section typically performs the hydraulic design for structures conveying drainage areas 259 ha (1 sq mi) or greater. A step by step flow chart and description of the development of the hydraulic design for structures and preparation of the relevant portions of the hydraulic permit application can be found in Appendix C.

3.10 Drainage Guidelines for Resurfacing Projects

3.10.1 General

The Department's drainage requirements for a resurfacing project are different from a reconstruction project. With the understanding that every project is unique, there are typical guidelines that should be considered for resurfacing projects. These guidelines are outlined in this section and are discussed below.

3.10.2 Adequate Cross Slope for Proper Drainage

The designer should ensure that the cross slope of the proposed overlay is constructed to current roadway design standards and consistent with the Department's current specifications. Adequate information should be shown on the plans to allow the proper construction of the roadway cross slope. This may include typical sections, gutterline profiles, superelevation grade lists, etc. A typical detail for the normal crown and superelevated section should be included on the plans.

It is recommended that pavement elevations be verified by survey prior to the final overlay to ensure the proper cross slopes have been established. Pavement milling or bituminous wedge courses may be required.

3.10.3 Modifications to the Existing Drainage System

Catch basin tops may need to be reset. If curbing is to be removed, the existing Type "C" inlets should be converted to Type "C-L" inlets. The designer should ensure that the slope is well established to intercept the pavement runoff. Catch basins and other pertinent drainage structures should not be removed if they connect subdrains.

3.10.4 Drainage Computations

Detailed storm drainage computations are typically not required for resurfacing projects. However the condition of the existing pipes should be determined by visual inspection to ensure that they do not warrant replacement. Guidelines discussed in Chapter 4 of this manual should be reviewed to perform culvert inspections and condition surveys. Pipes and drainage structures should be cleaned as necessary.

3.10.5 Existing Problems or Concerns

Existing drainage problems should be investigated, documented and addressed. The district drainage engineer and the maintenance garage responsible for the area should be contacted to determine if there are any areas of concern such as flooding, icing conditions, evidence of erosion at outlets or any other drainage conditions that may be remedied under the scope of the project.

3.11 Permit Requirements

3.11.1 General

The Hydraulics and Drainage Section is involved in the following environmental permit applications.

- Floodplain Management Certification
- Stream Channel Encroachment Permits
- Water Diversion Permits
- Dam Construction Permits

All environmental permit applications are coordinated through the Department's Office of Environmental Planning. The Hydraulics and Drainage Section offers technical assistance in the review and preparation of the permits specified above. The permit submittals to ConnDEP are performed through the OEP.

The permit applications for projects administered by the Consultant Design Section are typically prepared by the consultant and forwarded to the Hydraulics and Drainage Section for review. For State Design projects, the designer prepares the permits or requests that the application or portions thereof be prepared by the Hydraulics and Drainage Section. The ConnDEP permit application package includes instructions on preparing the various permit applications.

The intent of the permit application review by the Hydraulics and Drainage Section is to ensure that it is consistent with ConnDEP guidelines and is in agreement with the design prior to it being forwarded to ConnDEP. The permit process is discussed in several ConnDOT and ConnDEP guidelines. The designer should review and be familiar with them prior to the start of the design. They include the following:

- **Consulting Engineers Manual** – Chapter 800
- **Water Resources Coordination and Permit Processing Manual by ConnDOT Office of Environmental Planning** - See Chapter 4, (State Permits) and Chapter 6 (The Process) which includes a flowchart illustrating the permit process and how it relates to the various stages of design.
- **Hydraulic Analysis Guidance (DEP-IWRD-GUID-001)** - ConnDEP publication intended as a supplemental guideline for preparing hydraulic analyses for various permit applications.

These are good sources of information for those not familiar with the process or those in need of a review. Early coordination with ConnDEP and familiarity with permit requirements is stressed throughout this Drainage Manual. Typically, a State project cannot be advertised until all permits are in place and certified. Therefore, submitting applications late in the design process could substantially delay the project schedule.

3.11.2 Floodplain Management Certification

Floodplain Management involvement may be related to proposed work either in the floodplain or in the floodway. The definition of both of these is discussed in the Legal Chapter of this manual, (Chapter 2). These limits are depicted on FEMA mapping. The limits of a floodplain

are defined by the 100-year water surface elevation of the watercourse. The limits of a floodway are defined by a regulatory water surface elevation. A FEMA hydraulic water surface profile analysis is used to establish the limits of the floodway.

Roadway Impacts...Floodplain Management involvement for a roadway design project is typically attributed to work being proposed in a floodplain of a watercourse that conveys a drainage area of 259 ha (1.0 sq. mi.) or greater. Associated impacts may include but are not limited to fill being proposed in the floodplain, a storm drainage outlet discharging drainage into a floodplain or any activity on a roadway that is within the regulatory limits (Zone A) as depicted on the FEMA mapping. A hydraulic analysis is typically not required unless the amount of fill proposed to be placed within a floodplain is substantial. The floodplain impacts should be minimized as much as possible.

Fill within an established floodway is **strongly** discouraged. ConnDEP may require a hydraulic analysis if there is concern with substantial impacts to the floodway (even if a cross culvert is not being proposed for the project).

Structure Impacts...The Floodplain Management documentation requirements for a structure differ from a roadway project, as there may be considerably greater impacts attributed to a new structure spanning a watercourse. A hydraulic analysis needs to be prepared not only for the design of the structure but also for ConnDEP to ensure that an unacceptable increase in the water surface elevation is not created by the new structure. The proposed activities associated with the construction of the structure must comply with ConnDEP regulations. The Hydraulic Analysis Guidance referenced in Section 3.5.2 above is conducive to understanding the requirements and necessary steps for certification. The limits of the FEMA floodplain and floodway should be shown on the project plans to aid in determining potential impacts.

Flood Management General Certification is available for minor activities located within the regulated floodplain. This general certification granted by ConnDEP can be obtained for certain activities in lieu of an individual Flood Management permit application. The Hydraulics and Drainage section is responsible for reviewing all requests and issuing approval. A description of the minor activities that would qualify for general certification can be obtained from the offices of Hydraulics and Drainage or Environmental Planning.

When seeking Flood Management Certification under the general permit, at a minimum the following information should be provided:

- Request memorandum "Request for FM General Certification Approval"
- Project description with a statement of hydraulics and drainage involvement
- Location plan
- Design plan
- Copy of flood map (FEMA map panel for area of concern)
- Justification of why the request qualifies under FM General Certification
- Available supporting reports, computations, etc.

3.11.3 Stream Channel Encroachment Line Permit

Stream Channel Encroachment Lines are established by the State for many flood prone rivers and watercourses. Any activity riverward from the established lines requires a permit from ConnDEP. A listing of regulated areas may be obtained from ConnDEP's Bureau of Water

Management, Inland Water Resources Division. Copies of the maps are on file with the town clerks of the respective towns, the Hydraulics and Drainage Section, and the office of ConnDEP's Inland Water Resources Division.

Roadway Impacts...Unlike the Flood Management Certification process where involvement is limited to the 100-year water surface elevation of the watercourse, Stream Channel Encroachment Lines have no limit with respect to elevation. For example, should a project involve work being performed on a travelway that is situated well above the 100 year water surface elevation of an adjacent river, and the project falls within the limits of the SCEL, a permit will be required.

Structure Impacts...A project that includes a structure within these regulated SCEL limits requires a hydraulic analysis. The Stream Channel Encroachment water surface model is used incorporating the project's proposed condition. Hydraulic reports used to establish the encroachment lines are also available for review at the ConnDEP Inland Water Resources Division. **Note however that this hydraulic model is independent from the hydraulic model used to design the structure and from that described for Floodplain Management certification.**

3.11.4 Water Diversion Permits

The need for a water diversion permit is addressed in the Water Resources Coordination and Permit Processing Manual and in Section 804.07 of the Consulting Engineers Manual.

3.11.5 Dam Safety Permits

A Dam Safety Permit is required for any work on or affecting a dam, dike, or similar structure subject to DEP jurisdiction. Under State regulations, these structures are broadly defined as "barriers of any kind whatsoever which are capable of impounding or controlling the flow of water" and, by breaking away or otherwise failing, could endanger life or property. Chapter 4 of the Water Resources Coordination and Permit Processing Manual and Section 10.5 of this manual discuss this permit application.

References

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

Water Resources Coordination and Permit Processing Manual, Office of Environmental Planning, Bureau of Policy and Planning, ConnDOT, April 1997.

Hydraulic Analysis Guidance, Inland Water Resources Division, Bureau of Water Management, ConnDEP, 02/14/02.

Appendix A – Pre-submission Quality Assurance/Control Guideline

These guidelines were developed to ensure important aspects of drainage design are considered in the project development. Prior to submitting a project to the Hydraulics and Drainage Section for review, the designer should review these guidelines to ensure they are considered in the design.

Chapter 1 – Introduction

- Documentation is an important aspect of the design or analysis of any drainage/hydraulic facility. The major purpose of providing good documentation is to define the design procedure that is used and to show how the final design and decisions were arrived at.
- Consider construction and maintenance related issues when developing the drainage/hydraulic design.
- Temporary hydraulic facilities are to be designed with the same attention to detail that is used for the primary facility.

Chapter 2 – Legal Aspects

- Existing and proposed drainage rights should be shown on the plans.
- The course of flow beyond drainage outlets should be investigated for a sufficient distance to ensure that other downgrade owners will not be adversely affected.
- The direction of flow from the drainage outlet should be shown on the construction plans a minimum distance of 61 m (200 ft) or shown to terminate by dissipation or entrance into a watercourse or body of water. This will aid the Department in determining any property impacts and right of way obligations.
- If a pipe upgrade downstream of the project is necessary, then additional rights may need to be acquired.
- Any concentration of flow upon adjacent property requires an easement to discharge. Any changes in size or location of the outlet require new drainage rights. Diversions may require rights from all affected owners and thus should be avoided.
- Outlets should be appropriate to present land use and adaptable to possible future uses. Avoid any outlets where ponding will be created. Existing outlets should be reviewed carefully before utilizing them in a new design. Outlets onto steep slopes should be avoided as they lead to erosion problems. Extending the outlet to a flatter slope is desirable.
- Watercourses (perennial or intermittent) should be shown and labeled on the plans. This will define appropriate drainage rights that may be required. It will also define the design storm event. (See Chapter 6, Hydrology.)
- If Floodplain Management Certification or a Stream Channel Encroachment Line permit is required, we recommend coordination with the ConnDEP, Inland Water Resources Division early in the design phase to maintain the project schedule.
- Diversions should be avoided as they increase the likelihood of erosion and can cause changes to wetland characteristics.
- Any disturbances to septic systems or water supply wells must be avoided. Health regulations stipulate separation distance from septic systems to drainage systems and may control drainage system or channel locations. Maintenance chemicals splashed onto or flowing over adjacent surfaces can affect wells.

Chapter 3 – Design Development

- If a project involves work within a watercourse, the designer should coordinate with the ConnDEP Fisheries Division early in the design phase to address fish passage concerns.
- A separate drainage submission is recommended when the drainage aspects of the project appear to be complex or the drainage facilities may involve right of way impacts.
- Provide responses and justification for all review comments.

Chapter 4 – Culvert Repair, Materials, and Structural Design

- Existing pipes that are to remain in use should be visually inspected and analyzed to determine if they are structurally and hydraulically adequate or require replacement. The condition of the pipes should be documented accordingly.
- Use existing pipes if in good condition. With regard to existing systems which are to be discontinued, metal pipes under traveled ways in poor condition should be removed and concrete pipe should be plugged at ends.
- Use the appropriate type pipe in tidal areas.
- Check strength, installation conditions and bedding requirements if the cover is over 4.6m (15 ft.) or under 0.6m (2 ft.) for concrete pipes and any depth for metal pipe.
- Parallel pipe lines must be separated at least 1/2 the pipe diameter or 1 m (3 ft.), whichever is less.
- Connections of pipe to structures should be designed with all dimensions and structural details considered.

Chapter 5 – Data Collection

- A well planned data collection program leads to a more orderly and effective analysis, and a design that is commensurate with project scope, project cost, the complexity of site hydraulics and regulatory requirements.

Chapter 6 - Hydrology

- Rational Method - This method shall be used only for drainage areas less than 81 hectare (200 acres).
- Stream Gage Data - The U.S. Geological Survey maintains a network of stream gaging stations throughout Connecticut. From long term periods of observation and through statistical analysis of the resultant data, peak flow rates for various return frequencies can be developed. This data shall be used wherever possible in the design of hydraulic facilities.
- U.S.G.S. Regression Equations shall be used for all routine designs at sites greater than 2.59 km² (1 mi²) in tributary area unless stream gage data is available or unique watershed characteristics, such as significant detention storage dictate otherwise. Refer to Water Resources Bulletin No. 36.

- Computer Models for Hydrograph Generation - Many watersheds exhibit characteristics which require the use of a computer model to estimate design discharge quantities from run-off hydrographs. Additionally, hydrograph generation models are useful in the analysis of "before and after" conditions as related to proposed activities within a given basin. Where use of a computer model is appropriate, the methods described in Section 6.14 shall be used.
- FEMA Flood Insurance Study Discharge Rates - Discharges specified in the appropriate Flood Insurance Study shall be used for ANALYSIS where the regulatory floodway is adopted. The DESIGN discharge may vary from the Flood Insurance Study quantity if the results of an independent analysis so indicate.
- SCEL – Stream Channel Encroachment Discharge Rates – Discharges specified in the appropriate Stream Channel Encroachment study shall be used for analysis where SCEL are established. The **Design**, FEMA and SCEL discharges may vary, as they are all developed independently and may use different methods.
- The hydrology for cross culverts conveying drainage areas greater than 81 ha (200 ac) should be submitted for review and approval prior to completing the hydraulic design.

Chapter 7 - Channels

- The channel design should ensure that the design flow in its flattest section is accommodated.
- Flexible lining is preferred over rigid to reduce velocity and for lower maintenance and installation cost. A closed drainage system should be used instead of steeply graded channels.
- Safety requirements will often dictate the location of roadside ditches or channels.
- Sharp bends and sudden grade changes should be analyzed for possible overtopping of channel lining during high flows. A lining change may be needed at bends due to velocity increases.
- At intersections of side and main channels, consider the velocity head and need for additional lining depth, or heavier lining, on the main channel opposite the junction.
- The location and alignment of a culvert should be consistent with the flow tendency of a stream. Inlet and outlet rechanneling should be held to a minimum.
- At severe transitions from cut to fill slopes, erosion can occur where roadside swales are used, unless short riprap channels, catch basins and pipes or paved ditches are employed to control the flow.

Chapter 8 - Culverts

- Available head should be checked by using cross sections, profiles, contour maps and a thorough field check.
- Allowable headwater elevation should be at least 0.3m (1 ft.) below the controlling elevation of a building or point where overflow could run over the road or to another watershed.
- Safety of the structure location must be considered in relation to pedestrian and vehicular traffic. Culvert ends are generally preferred to endwalls.
- Consider possible erosion around outlet structures from both high and low water in the stream. Provide riprap and channel improvements or energy dissipation where needed.
- Culverts providing for the passage of area drainage from one side of the highway to the other usually shall not be smaller than 600mm (24 in) for interstate systems and 450mm (18 in) for other systems.

- Fish passage through culverts should be considered where appropriate. Minimum depths of flow, low velocities and outlet pools are some methods employed.
- Avoid long skew crossings under traveled ways.

Chapter 9 - Bridges

- Occasionally, the waterway opening(s) for a highway-stream crossing can be provided for by either culvert(s) or bridge(s). Estimates of costs and risks associated with each will dictate which structure alternative should be selected. Other considerations which may influence structure-type selection are listed in Table 9-1 and Chapter 8, Culverts.
- The hydraulics of the highway-stream crossing system should be given considerable study in choosing the preferred design from the long list of available alternatives. **The selection should be based on a collective effort of bridge, soils and foundations, highway, and hydraulic engineers. This process is further outlined in Chapter 3, Design Development.**
- The final design selection should consider the maximum backwater allowed by the National Flood Insurance Program and Stream Channel Encroachment Lines unless exceedence of the limit can be justified by special hydraulic conditions.
- The final design should not significantly alter the flow distribution in the floodplain.
- A specified clearance should be established to allow for passage of ice and debris. For navigation channels, a vertical clearance conforming to Federal requirements should be established based on normally expected flows during the navigation season.
- Degradation or aggradation of the river as well as contraction and local scour shall be estimated. Appropriate positioning of the foundation, below the total scour depth, if practicable, shall be included as part of the final design.

Chapter 10 – Storage Facilities

- Preliminary approval of storage considerations should be obtained from Hydraulics and Drainage, before the actual storage computations are performed.
- Consider permanence of the storage, if already existing.
- Sedimentation basins and dams should not be used on watercourses.
- Sedimentation basins should be used to clarify runoff only from areas disturbed by construction. Runoff from adjacent undisturbed areas should be channeled or piped around the basins where possible.
- Field check the need for trash racks. If used they should not completely enclose the inlet but allow for overflow when clogged. Consider access for maintenance if used.

Chapter 11 – Storm Drainage Systems

- Any existing inlet structures should be checked for capacity if they will be used in a new design.
- Utility conflicts may require design changes. Keep at least 0.3m (1 ft) vertical and horizontal clearance between drainage systems and utilities.
- The most economical routing of pipes should be used. A different layout could save many feet of pipe.

- Consider hydraulic effects of tides on a drainage system outlet. Return frequency of tides vs. storm design frequency should be considered.
- Pipes on steep grades may require energy dissipation at the outlet - check possibility of reducing velocity through flatter grades, using rougher pipe or drop structures.
- Catch basins not located precisely in a low point of the gutter profile will create ponding and lead to icing conditions.
- Intersection grading plans (contours) may be required to assure proper grading to and around catch basins to ensure inlets are located at low points and to avoid snow melt and consequent icing across traveled ways.
- Catch basins are usually needed just upgrade from a bridge to avoid excess flow across the bridge or into expansion joints.
- Catch basins may be needed at downgrade ends of bridges where curbing is not used, to avoid erosion down the fill slope.
- Consider the effects of snow melt and refreezing at all traffic islands and superelevated sections.
- A reverse cross slope should be provided for superelevated sections to contain the flow in the shoulder and therefore not allow runoff or snow melt to sheet across the travelway.
- The designer should exercise particular care that medians, shelves, and channelization islands are drained so as not to cause icing hazards. Snow stored in these areas can melt and flow across a traveled roadway. A subsequent freezing can lead to dangerous icing conditions requiring constant attention by maintenance forces.
- On **depressed expressways**, where curbing is used, flanker basins will be placed in locations approximately 0.06m (0.2 ft) higher than the sag catch basin. These will collect debris and provide additional capacity if needed. Flanking inlets are not usually considered as intercepting flow in the design computations.
- Catch basins should be located a minimum of 1.5m (5 ft) from driveway returns to prevent loss of vehicular traction.
- All catch basins shall have standard 0.6m (2 ft) sumps to collect roadway sand except at pipes conducting watercourses or pipes 900mm (36") in diameter or larger.
- At an expressway low point where curbing is not used, a 'C-L' catch basin shall be located on the shoulder with the far edge of the catch basin at the outer edge of the shoulder. This inlet is intended for collection of snow melt and rain when plowing operations prevent water from leaving the pavement. No interception is to be calculated for this inlet. A 300mm (12") pipe will be used for the outlet.
- For sags in the gutter profile where excess flow cannot be relieved (cut sections), use a 25-year design frequency (50 for expressways) for the grate capacity of the inlet structure and for pipes leading away from the inlet structure to a clear outlet.
- For residential and business areas where grading will produce small pockets not drained, lawn drains of small pipe elbows connected to the nearest catch basins may be economical.
- Where a change in size of pipe is made at a structure, the crowns of these pipes will normally be aligned on grade.
- As a preliminary rough check on hydraulic design, pipe sizes should increase consistently with the direction of flow and added flow from branches to the system.
- Hoods at catch basins should be avoided as they create higher losses than a conventional inlet and are a clogging concern. When hoods are used in the storm system, then the losses and clogging potential associated with these structures must be considered when developing the hydraulic gradeline computations.

- Drainage should be brought to a suitable outlet. The system should be shown to its outlet or to its hydraulic control.
- Computations should be provided to verify that the proposed drainage will not adversely impact the existing downstream storm system.

Appendix B – Submission Check List**State Highways and Local Roads Checklist
for Hydraulics and Drainage Design Reviews**

The following is a checklist to be completed and included with all State Highways and Local Roads projects that have hydraulics and drainage involvement. To expedite the review process, meet project schedules, and avoid possible delays due to incomplete design submissions, the following checklist outlines the necessary material (for each design phase) that should be provided to the Hydraulics and Drainage Section for review. Also, approximate review time frames with each submission are provided for the designer to consider. The procedures and criteria of the current Consulting Engineers Manual, Bridge Design Manual, and Drainage Manual should be followed with special attention to Sections 302, 303, 304, 403, 404, 502, 707, 711, 712, 713, 802, and 804 of the Consulting Engineers Manual; Sections 2, 5, 11, and 13 of the Bridge Design Manual; and all the sections within the Drainage Manual.

The following checklist(s) are to be submitted for all design projects at the appropriate design phases:

Scope Review Meeting/Permit Determination Request

Preliminary Design Checklist (Plans 35% Complete)

Drainage Design Checklist (Plans 50% Complete) – *If Necessary*

Semi-Final Design Checklist (Plans 60% to 70% Complete)

Final Design Checklist (Plans 85% to 90% Complete)

Project No.	_____
Roadway	_____
Town	_____
Date	_____
Signature of Engineer	_____

Scope Review Meeting/Permit Determination Request

Allow a 2-3 week review time.

- a.** Project description with a brief statement of hydraulics and drainage involvement.

☐ Included ☐ N/A

- b.** Request for determination of permit involvement.

☐ Included ☐ N/A

- c.** Location plan.

☐ Included ☐ N/A

- d.** Available plans, profiles, cross sections, etc...

☐ Included ☐ N/A

Project No.	_____
Roadway	_____
Town	_____
Date	_____
Designed By	_____
Signature of Engineer	_____

Preliminary Design Checklist (Plans 35% Complete)

Allow a 3-4 week review time.

The Preliminary Design Submission should include the following:

a. Preliminary Design Statement

The design statement should contain a narrative for the following:

1. Description of hydraulics and drainage involvement.
☐ Included ☐ Not Included ☐ Not Applicable
2. A list of existing culvert or bridge crossings for tributary areas:
 a.) > 81 ha (200 Acres) and < 2.59 km² (1 mi²)
 b.) > 2.59 km² (1 mi²)
☐ Included ☐ Not Included ☐ Not Applicable
3. Available fish passage information from the ConnDEP Fisheries Division.
☐ Included ☐ Not Included ☐ Not Applicable
4. Water diversions.
☐ Included ☐ Not Included ☐ Not Applicable
5. Justification for drainage design not consistent with the Drainage Manual criteria.
☐ Included ☐ Not Included ☐ Not Applicable
6. List of permit requirements (ie: FPM, SCEL, DAM, CAM, etc...).
☐ Included ☐ Not Included ☐ Not Applicable
7. Required detention designs.
☐ Included ☐ Not Included ☐ Not Applicable
8. Any unusual designs such as pumping stations.
☐ Included ☐ Not Included ☐ Not Applicable
9. Tidal Involvement.
☐ Included ☐ Not Included ☐ Not Applicable
10. Investigation of existing drainage concerns and/or problems. Contact Town/City maintenance departments for local road projects or ConnDOT Maintenance departments and district drainage engineer for State highway projects.
☐ Included ☐ Not Included ☐ Not Applicable

b. Plans, Profiles and available Cross Sections

1. Watercourses, perennial and intermittent should be shown and labeled.
☐ Included ☐ Not Included ☐ Not Applicable
2. Special drainage provisions such as sedimentation basins and/or non standard drainage structures.
☐ Included ☐ Not Included ☐ Not Applicable
3. Approximate slope limits.
☐ Included ☐ Not Included ☐ Not Applicable
4. Taking lines and property lines.
☐ Included ☐ Not Included ☐ Not Applicable
5. Existing ground line and existing structures.
☐ Included ☐ Not Included ☐ Not Applicable
6. Existing drainage rights from survey information.
☐ Included ☐ Not Included ☐ Not Applicable
7. Preliminary conceptual layout of proposed drainage system including channels, ditches and swales.
☐ Included ☐ Not Included ☐ Not Applicable
8. FEMA 100 year floodplain and/or SCEL shown and labeled.
☐ Included ☐ Not Included ☐ Not Applicable

c. Watershed Area Map

1. An overview contour map (100 or 200 scale contour map or USGS 2000 scale topographic map) for the project showing watershed areas for all cross culverts.
☐ Included ☐ Not Included ☐ Not Applicable

d. Structures with drainage areas > 2.59 km² (1 mi²)

1. Hydrology computations determining the drainage area and design discharge.
☐ Included ☐ Not Included ☐ Not Applicable
2. Preliminary hydraulic and scour computations for the structure type study alternatives (the above hydrology computations must be approved prior to the start of the preliminary hydraulic computations).
☐ Included ☐ Not Included ☐ Not Applicable

Provide justification for items **Not Included**. Justification should correspond to letter and number (e.g.: **a.1., b.1., c.1.**, etc...).

Project No.	_____
Roadway	_____
Town	_____
Date	_____
Designed By	_____
Signature of Engineer	_____

Drainage Design Checklist (Plans 50% Complete)

Allow a 6-8 week review time

See **Note** below.

Semi-Final Design Checklist (Plans 60% to 70% Complete)

Allow a 5-6 week review time

Note: A separate, earlier drainage submission (at approximately 50% completion) may be required if the drainage design is particularly complicated, requires significant right of way and/or otherwise might jeopardize the schedule of the project. **This checklist MUST accompany both of these submissions.**

Indicate which submission this checklist is for and include the following information:

☐ *Drainage Design Submission*

☐ *Semi-Final Design Submission*

a. Draft Drainage Report

- Disposition of Preliminary Design/Drainage Design Submission comments with written responses justifying comments not incorporated.
☐ Included ☐ Not Included ☐ Not Applicable
- A condition survey of the existing drainage pipes and structures that are to remain in use should be investigated for structural adequacy and documented. (See Section 3.6.3.)
☐ Included ☐ Not Included ☐ Not Applicable
- The condition of existing ditches that are to remain in use should be field inspected, analyzed and results documented to verify their stability and the need for cleaning and reshaping.
☐ Included ☐ Not Included ☐ Not Applicable
- The condition of the outlet at the existing discharge points should be investigated and documented to ensure no erosion or sediment problems exist. If outlet protection is required, it should be incorporated into the project and computations submitted.
☐ Included ☐ Not Included ☐ Not Applicable

5. A condition survey report including items 2, 3, and 4 above. (See Appendix A and B, Chapter 4)
☐ Included ☐ Not Included ☐ Not Applicable
6. Drainage design computations should include gutter flow analysis, storm sewer design, and hydraulic gradeline (HGL). The hydraulic gradeline should be analyzed to ensure 0.3m (1 ft) freeboard is maintained at drainage structures. This analysis should consider all friction, entrance, junction, exit and bend losses. Designer to verify that the proposed drainage will not adversely impact the existing downstream storm system or property owners. (See Chapter 11, Storm Drainage Systems.)
☐ Included ☐ Not Included ☐ Not Applicable
7. Drainage computations should identify structures by station and offset rather than by a numerical identifier. If station and offset is not feasible for the computations then include an index with the location of the structure corresponding to its numerical identifier. The watershed map should be prepared accordingly.
☐ Included ☐ Not Included ☐ Not Applicable
8. Existing drainage systems shall be analyzed for hydraulic adequacy to meet the proposed conditions and, if found inadequate, an upgrade will be designed in conformance with the criteria established in the Drainage Manual.
☐ Included ☐ Not Included ☐ Not Applicable
9. All roadway drainage systems should be brought to a suitable outlet.
☐ Included ☐ Not Included ☐ Not Applicable
10. If upgrading of pipes downstream of the project is necessary, then additional rights may need to be acquired.
☐ Included ☐ Not Included ☐ Not Applicable
11. The need for temporary drainage should be addressed. Temporary drainage computations should be prepared in accordance with criteria in the Drainage Manual. (See Section 3.6.11.)
☐ Included ☐ Not Included ☐ Not Applicable
12. Proposed swales, ditches and channels should be designed in accordance with HEC-15 for discharges 1.42 m³/s (50 ft³/s) and less or HEC-11 for discharges in excess of 1.42 m³/s (50 ft³/s). (See Chapter 7, Channels.)
☐ Included ☐ Not Included ☐ Not Applicable
13. Minor and small cross culvert design computations with culvert data sheet. (See Chapter 8, Culverts.)
☐ Included ☐ Not Included ☐ Not Applicable
14. Topographic mapping with watershed area delineated for each inlet and/or cross culverts as required to perform the drainage calculations. The flow path used in the time of concentration calculation and coefficient of imperviousness should be shown for each area. (See Chapter 6, Hydrology.)
☐ Included ☐ Not Included ☐ Not Applicable
15. Diversion identified.
☐ Included ☐ Not Included ☐ Not Applicable
16. All plans, computations and reports identify the responsible engineers who prepared and checked the work.
☐ Included ☐ Not Included ☐ Not Applicable
17. Alternate types of drainage pipe material have been considered and documented.
☐ Included ☐ Not Included ☐ Not Applicable

b. Plans, Profiles and Cross Sections

1. The existing and proposed storm drainage shown to their outlets.
☐ Included ☐ Not Included ☐ Not Applicable
2. Size and type of existing drainage pipes/structures and disposition of pipes/structures to be abandoned.
☐ Included ☐ Not Included ☐ Not Applicable
3. Properties affected by diversions should be shown on the plans so that proper rights can be acquired.
☐ Included ☐ Not Included ☐ Not Applicable
4. Drainage Rights and Easements.
☐ Included ☐ Not Included ☐ Not Applicable
5. Outlet Protection shown on plans and details provided.
☐ Included ☐ Not Included ☐ Not Applicable
6. Intersection grading plans to ensure inlets are located at the low points to alleviate ponding/icing conditions. Top of frame elevation should be shown.
☐ Included ☐ Not Included ☐ Not Applicable
7. In areas where cross culverts are being extended, replaced, or where outlet protection is proposed a profile or cross section of the natural ground should be provided to show how the inverts will tie into the existing topography.
☐ Included ☐ Not Included ☐ Not Applicable
8. The top of frame and invert elevations for each storm drainage structure shown. Proposed drainage structures shall be identified by station and offset on cross sections.
☐ Included ☐ Not Included ☐ Not Applicable
9. Existing and proposed drainage patterns (flow arrows) of pipes, ditches, channel and swales.
☐ Included ☐ Not Included ☐ Not Applicable
10. Details for any special drainage structures not found in the Standard Drawings.
☐ Included ☐ Not Included ☐ Not Applicable
11. The direction of flow should be shown by arrows to 61m (200 ft.) beyond any drainage outlet, or shown to terminate by dissipation or entrance into a watercourse or body of water.
☐ Included ☐ Not Included ☐ Not Applicable

c. Structures with drainage areas > 2.59 km² (1 mi²)

1. Draft hydraulic design report.
☐ Included ☐ Not Included ☐ Not Applicable
2. Draft scour report when the proposed structure spans the waterway.
☐ Included ☐ Not Included ☐ Not Applicable
3. Draft floodway report.
☐ Included ☐ Not Included ☐ Not Applicable
4. Draft SCEL report.
☐ Included ☐ Not Included ☐ Not Applicable
5. Draft scour report if required.
☐ Included ☐ Not Included ☐ Not Applicable

Provide justification for items **Not Included**. Justification should correspond to letter and number.

Project No.	_____
Roadway	_____
Town	_____
Date	_____
Designed By	_____
Signature of Engineer	_____

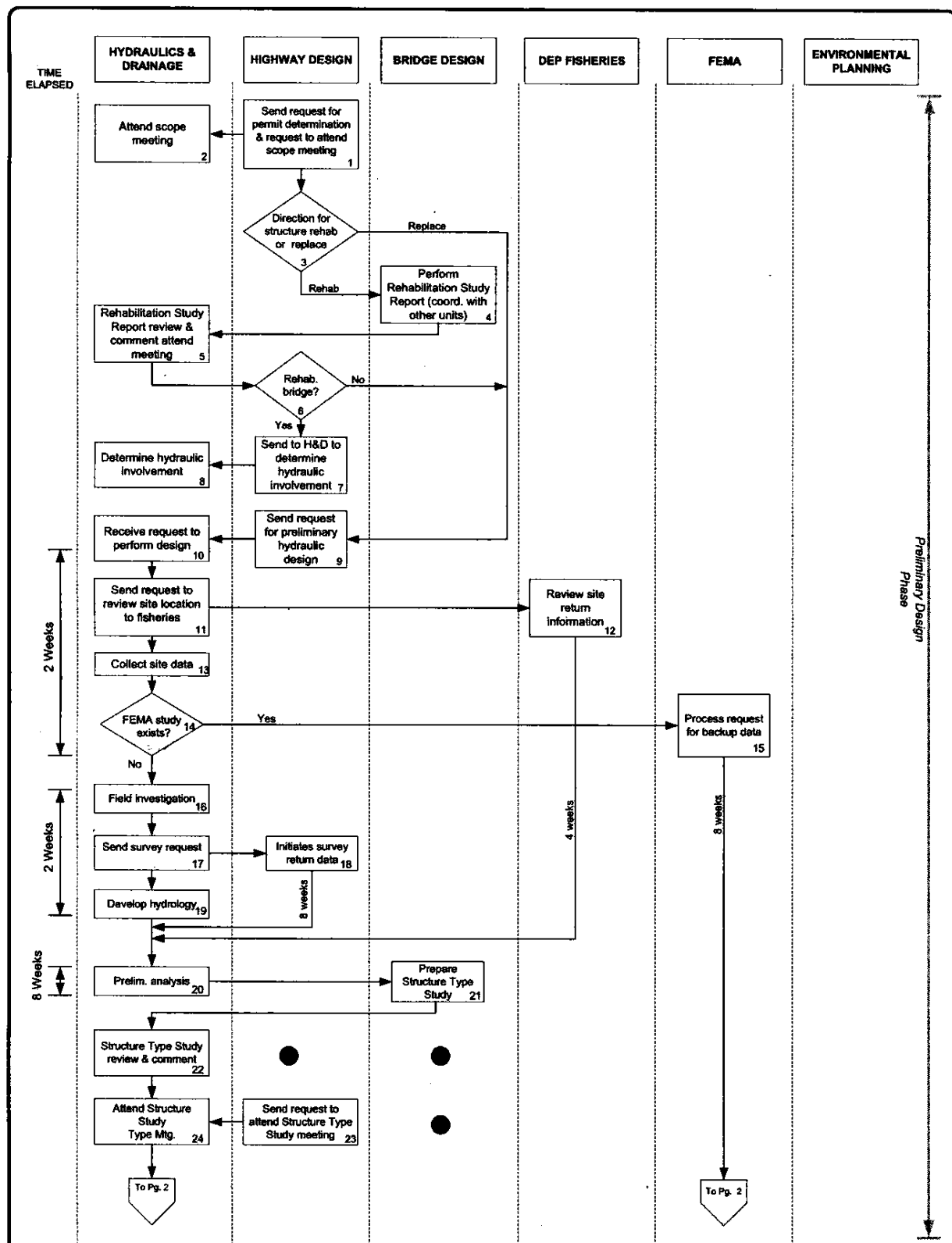
Final Design Checklist (Plans 85% to 90% Complete)*Allow a 4-5 week review time.*

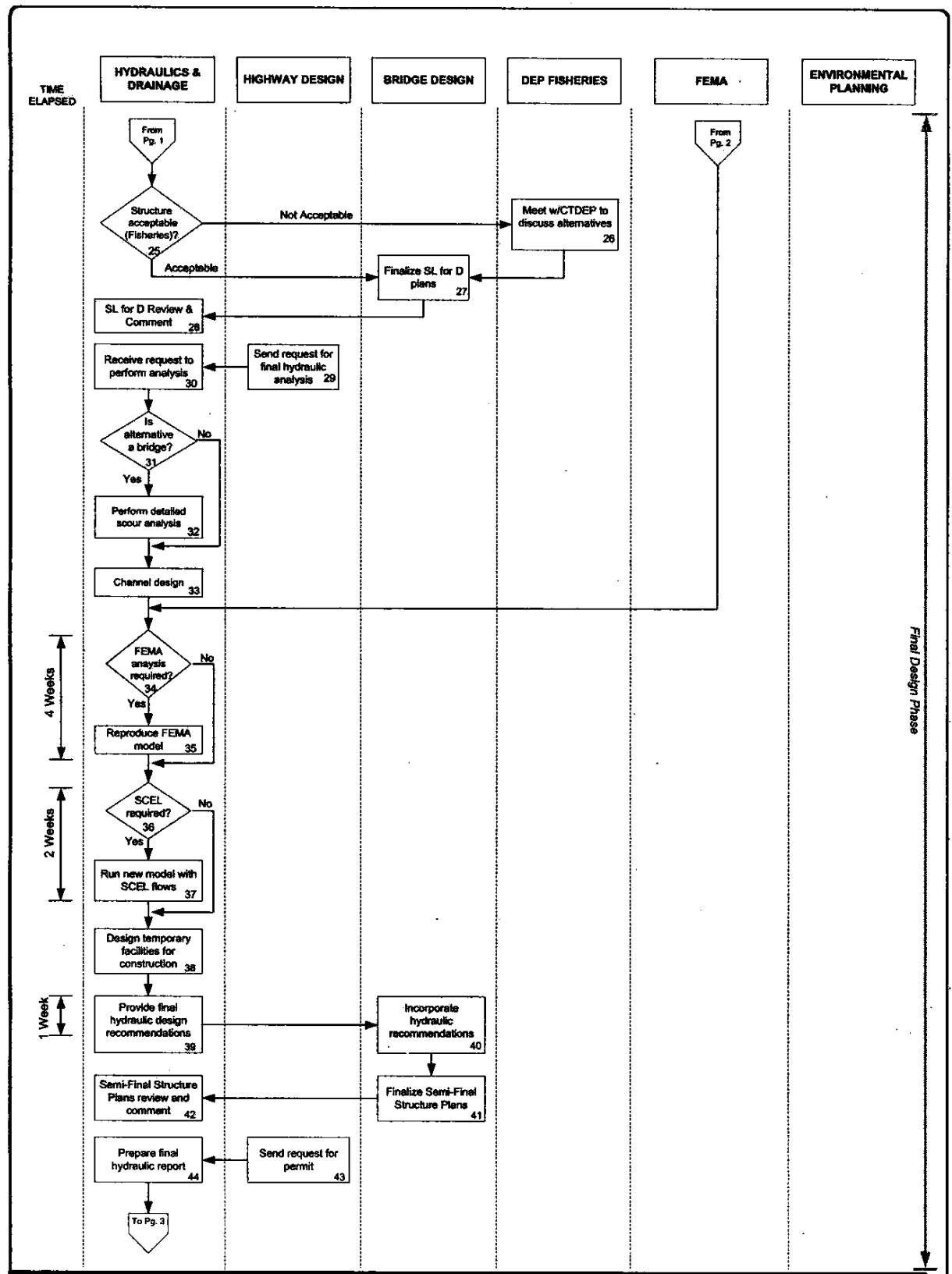
The Final Design Submission should include the following:

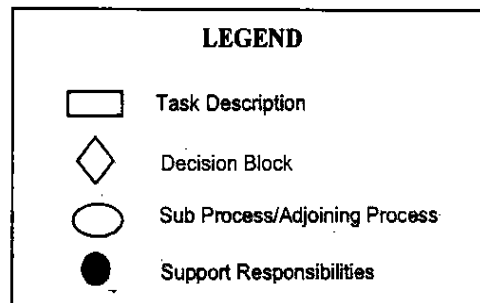
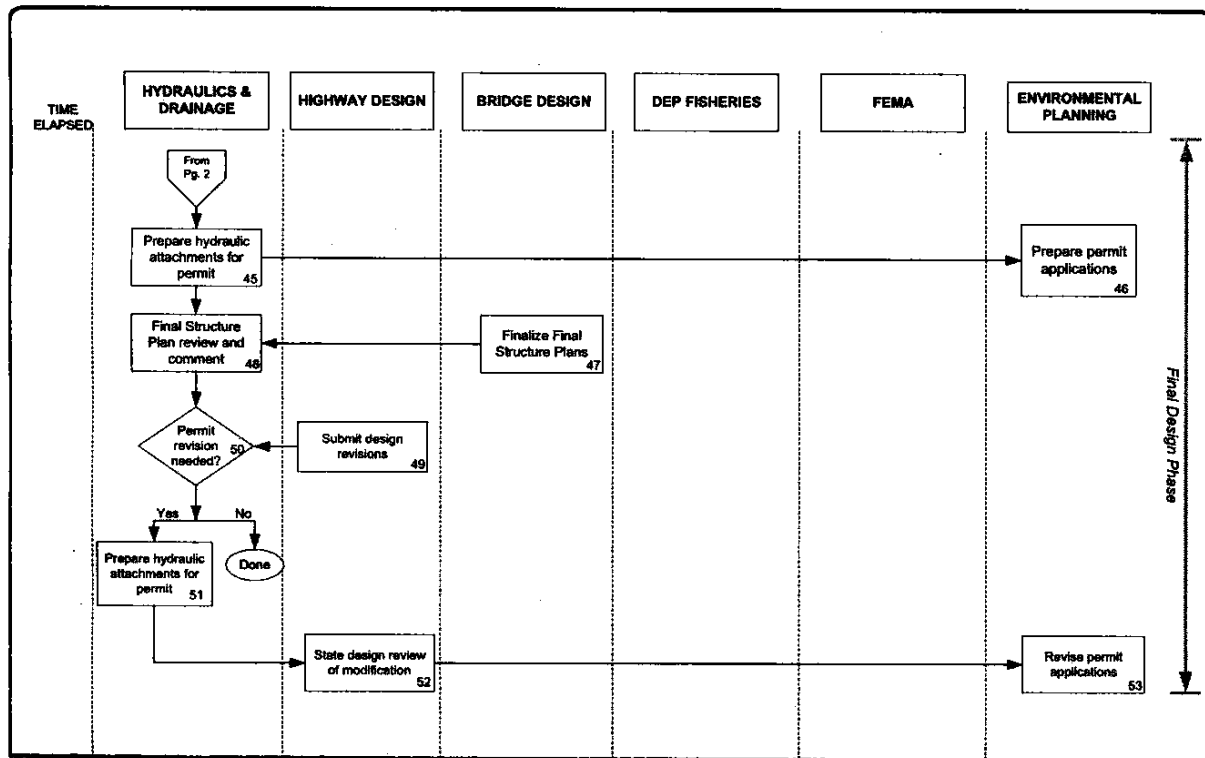
a.	Disposition of Semi-Final Design comments with written responses justifying comments not incorporated.		
	☐ Included	☐ Not Included	☐ Not Applicable
b.	Final Drainage Report and Final Plans.		
	☐ Included	☐ Not Included	☐ Not Applicable
c.	Final scour report.		
	☐ Included	☐ Not Included	☐ Not Applicable
d.	Final floodway analysis report.		
	☐ Included	☐ Not Included	☐ Not Applicable
e.	Final SCEL report.		
	☐ Included	☐ Not Included	☐ Not Applicable
f.	Final hydraulic design report.		
	☐ Included	☐ Not Included	☐ Not Applicable

Provide justification for items **Not Included**. Justification should correspond to the designated letter.

Appendix C – In-House Development of Hydraulic Requirements for Crossings Conveying a Waterway Greater Than 2.59 km² (1 mi²)







Box #	Task Name	Owner	Duration	Description
1	Request permit determination and request H&D to attend scoping meeting	Highway Design	1 day	Highway design sends permit determination request to H&D and requests that H&D attend scoping meeting.
2	Attend scoping meeting	Engineer	1 day	H&D attends scoping meeting with other units at request of Highway design to discuss project issues and direction of structure rehab or replacement.
3	Determine if structure is rehab or replacement	Highway Design	1 wk	Highway Design will determine if structure will be rehab or replacement.
4	Rehabilitation Study Report	Bridge Design	4 wks	Prepare Rehabilitation Study Report. Coordinate and obtain information from other units as necessary.
5	Review and comment on rehabilitation study report	Engineer	1 wk	Provide written comments on rehabilitation report and attend review meeting.
6	Determine if structure can be rehabilitated	Highway Design	1 day	After completion of rehabilitation study, Highway Design will determine if structure can be rehabilitated.
7	Request H&D involvement	Highway Design	1 day	Highway design will send written request to H&D to determine the hydraulic involvement for the rehab project.
8	Determine hydraulic involvement	Engineer	1 wk	H&D will determine the extent of H&D's involvement for the structure rehabilitation.
9	Request preliminary hydraulic design	Highway Design	1 day	Highway Design sends request to perform preliminary hydraulic design to H&D.
10	Project initiation	Section Head	1 day	Receive request for preliminary hydraulic design from Highway Design. Review request, information, schedule, funds, and assign to Engineer.
11	Request DEP Fisheries review	Engineer	1 day	Send request letter to DEP Fisheries to review site location.
12	Site review	DEP Fisheries	4 wks	Perform site review and return any special requirements to H&D.

Box #	Task Name	Owner	Duration	Description
13	Collect site data	Engineer	15 day	Collect existing data from H&D: FEMA, H&D files, River Study, Scour, USGS, SCEL. BS&E: Bridge inspection reports, underwater reports. ER&R: project plans and as-built. Soils & Foundations: search folders by town. Contact: Towns, Maintenance (District Office or Town), Environmental Planning, Central Surveys, DEP, USDA Natural Resource Conservation Service, USGS.
14	FEMA study?	Engineer	1 day	If a detailed FEMA study exists for the site, request backup data from FEMA with a prepaid vendor form.
15	FEMA process	FEMA	8wks	Assemble and return detailed FEMA backup data to H&D.
16	Field investigation	Engineer	1 day	Perform field visit. Send notification to and invite other disciplines. Complete field review form and identify survey requirements.
17	Survey request	Engineer	1 day	Prepare a request for hydraulic survey and send to Highway Design.
18	Initiate survey	Highway Design	8wks	Highway Design will initiate the survey and return the completed plans and cross-sections to H&D.
19	Develop hydrology	Engineer	1wk	Develop flows using information available.
20	Preliminary analysis	Engineer	8wks	Review information from Fisheries, Survey, and FEMA. Work with Bridge Design on types of structures to be studied. Perform preliminary analysis of alternative structures, including a qualitative scour evaluation and prepare data for the Structure Type Study. Forward data to Bridge Design.
21	Structure Type Study	Bridge Design	3wks	Bridge Design will prepare a Structure Type Study and send a copy to H&D for review and comment.
22	Review and comment on structure type study	Engineer	1wk	Prepare written comments on Structure Type Study and return to Highway Design.
23	Send request to attend structure type study meeting	Highway Design	1 day	Highway Design will request H&D to attend the Structure Type Study Meeting.
24	Attend structure type study meeting	Engineer	1 day	Attend Structure Type Study Meeting with other sections for structure selection.

Box #	Task Name	Owner	Duration	Description
25	Review structure acceptability to DEP Fisheries	Engineer	1 day	Investigate if selected structure is consistent with previous letter of recommendation from DEP Fisheries.
26	Meet with DEP Fisheries to discuss alternatives	Highway Design, DEP Fisheries and Engineer	1wk	If selected structure is not consistent with DEP Fisheries recommendation then Highway Design will meet with DEP Fisheries and discuss mitigation measures or alternatives. May have to coordinate with H&D and make a presentation to Fisheries with the most reasonably solution.
27	Finalize SLD plans	Bridge Design	8 wks	Bridge Design prepares SLD plans for the culvert/bridge including stage construction and water handling concepts and sends them to other units for review. Structure design is 30% complete.
28	Review and comment on SLD plans	Engineer	1 wk	Prepare written comments on the SLD plans and return to Bridge Design.
29	Final structure selected	Highway Design	1 day	Highway Design will send letter to H&D requesting they proceed with final design of selected alternative. Submission should include plan, profiles, cross sections, and available bridge plans.
30	Project advancement	Section Head	1day	Received request to advance project to final design.
31	Is alternative a bridge?	Engineer	1 day	If alternative is a bridge then H&D will conduct a Detailed Scour Evaluation early in Final Design for use in foundation design.
32	Scour analysis	Engineer	1 wk	Prepare detailed scour analysis and forward results to Highway Design, Bridge Design and Soils and Foundations.
33	Channel design	Engineer	2 wks	If channel protection or restoration is required, perform design and develop plans.
34	Detailed FEMA study required?	Engineer	1 month+	Was a detailed FEMA study performed?
35	FEMA model	Engineer	4 wks	Recreate FEMA model using FEMA backup information. Compare flood elevations to the original FEMA flood elevations. Impose proposed condition and re-run model. No increase in encroached condition flood elevation is allowed. If there is an increase in flood elevation, change model to increase conveyance, compensate for increased flood by increasing storage, or process request for map revision. (A map revision can take a year or more).

Box #	Task Name	Owner	Duration	Description
36	Do SCEL exist within the project area?	Engineer	1 day	If Stream Channel Encroachment Lines analysis data exists and the discharge varies from the FEMA values, run another model.
37	SCEL model	Engineer	2 wks	Re-run the existing and proposed model using the flows from the SCEL report to show the change in flood elevations.
38	Design temporary facilities	Engineer	1 wk	Run models for all stages of construction. Work with Highway Design to develop a workable construction sequence and size temporary work. Time varies depending on complexity of construction.
39	Final hydraulic design recommendations	Engineer	1 wk	H&D to provide final hydraulic design recommendations to Bridge Design.
40	Incorporate hydraulic recommendations	Bridge Design	1 wk	Bridge Design incorporates final hydraulic design recommendations in structure design.
41	Finalize semi-final design	Bridge Design	1 wk	Bridge Design prepares semi-final plans for the culvert/bridge and sends them to other units for review. Structure design is 90% complete.
42	Review and comment on semi-final plans	Engineer	1 wk	Prepare written comments on the semi-final plans and return to Highway Design.
43	Request permit	Highway Design	1 wk	Highway Design will request H&D to prepare permits.
44	Prepare final hydraulic report	Engineer	2 wks	Prepare final hydraulic report including scour, design, FEMA, SCEL.
45	Permit hydraulics attachments	Engineer	1 wk	Prepare and assemble attachments and supporting documentation for permit application. Which could include attachment A-1, attachment H Part 1 and Part 2. Forward to Highway Design who will complete permit application and forward to Environmental Planning.
46	Permit application	Environmental Planning	2 wks	Environmental Planning will finalize and submit the permit applications to Connecticut DEP. Connecticut DEP approval usually takes 6-12 months.
47	Finalize structure plans	Bridge Design	8 wks	Bridge Design prepares final plans for the culvert/bridge and sends them to other units for review. Structure design is 100% complete.
48	Review and comment on final structure plans	Engineer	1 day	Review final structure plans to assure they are consistent with hydraulic design and the permits.
49	Project revision	Highway Design	Varies	If Highway Design changes any aspect of the project that has any relation to the hydraulic aspects, they must forward changes to H&D.

Box #	Task Name	Owner	Duration	Description
50	Permit revision needed?	Engineer	Varies	H&D will review the revision, re-run the hydraulic models, if necessary, and determine if a revision to the permit is needed.
51A	No permit revision	Engineer	1 day	If no revision is needed, send letter to Highway Design.
51	Permit revision	Engineer	1 wk	If a revision to the permit is needed, prepare the hydraulic portion of the revision to the permit and submit to Highway Design
52	Review of modification	Highway Design	1 wk	Highway Design reviews modification made by H&D and determines whether to amend the permit or prepare a technical revision. Highway Design then prepares the full application and forwards to Environmental Planning.
53	Revise permit applications	Environmental Planning	1 wk	Environmental Planning reviews permit revision and submits to ConnDEP.

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4.1 Overview

4.1.1 Introduction

There are thousands of culverts under the highways of Connecticut, many over 50 years old. Culverts do not receive much attention, primarily because they are generally hidden from view from the travelling public. Occasionally, however, an incident occurs that serves as a reminder that the failure of a culvert can have serious consequences.

Over the past twenty years, most of the larger highway culverts, defined as “Qualifying Structures” in Section 1.4 of the ConnDOT Bridge Inspection Manual have been inspected at least once every two years. Currently culverts that qualify are those with multiple openings measuring over 6m (20 ft.) and single culverts measuring over 3m (10 ft.) in span. These inspections, conducted on a wide variety of culvert types and materials, revealed numerous types of problems. The inspections have not only documented the problems but rated the culverts with regard to their serviceability and structural safety. Recommendations for repair of structures are normally determined following the review of the inspection reports and are kept in the Department’s Bridge Safety and Evaluation Unit. However, inspection information should supplement and not take the place of the requirement of performing a condition survey should the culvert remain in use as part of a construction project.

One of the problems associated with repair, rehabilitation, and replacement of culverts is that the work frequently is approached strictly as a maintenance problem without consideration of the underlying structural or hydraulic conditions for which the deterioration originated. Moreover, there has never been enough money to maintain culverts, properly, and, over the years, the overall condition of culverts in the highway system has steadily worsened.

Although there is considerable information available on the design and construction of new culverts of many materials, there is little information in the literature on how to repair culvert problems and even less on how to rehabilitate, strengthen, or retrofit upgrade culverts. There are essentially no criteria for selecting the most cost-effective alternative for the work that should be done.

4.1.2 Overview of the Problem

There are many culverts in the ConnDOT highway system that are in various states of deterioration. Maintenance personnel are faced with the question of deciding whether to repair, rehabilitate, retrofit, or replace these culverts and how to do so with available funds. Experience has shown that it is often economically feasible to restore or upgrade culverts, and procedures have been developed for doing such work. In order to decide which repair procedures are most appropriate, it is necessary that a basic understanding of culvert design, construction, and maintenance and, more importantly, how to evaluate causes and solutions to the problems.

Culverts are primarily structures that carry surface and stream water under highway pavements. However, the same types of structures are also used to facilitate highway undercrossings by animals, people, and occasionally vehicles.

4.1.3 Construction Projects with Existing Drainage Facilities

A condition survey should be conducted for existing drainage facilities (pipes, structures, swales, ditches) that are to remain in use within the project limits, to ensure their condition is adequate and replacement is not warranted. This should be performed early in the design phase so that the proposed design includes all systems that are to be replaced. Individuals responsible for performing the inspections must possess the minimum qualifications in accordance with the National Bridge Inspection Standards. The approach and documentation with respect to verifying the condition is outlined in Appendix A (Inspection Guidelines) and Appendix B (Drainage Facility Condition Surveys Guideline). A separate report shall be prepared and submitted to the Hydraulics and Drainage Section early in the final design phase of the project.

4.1.4 Basic Characteristics

The structural and hydraulic design of culverts is substantially different from that of bridges, as are the construction, maintenance, repair, and replacement procedures. A few of the more significant characteristics of water-carrying culverts are:

- Hydraulic – Culverts are usually designed to operate at peak flows with a submerged inlet to improve hydraulic efficiency. The culvert constricts the flow of the stream and may cause ponding at the upstream or inlet end. The resulting rise in elevation of the water surface produces a head at the inlet that increases the hydraulic capacity of the culvert. The effects of ponding and flow on appurtenant structures, embankments, and abutting properties are important considerations in the design of culverts.

- Structural – Culverts are buried in soil and are designed to support the dead load of soil over the culvert as well as live loads of traffic. Either the live load or the dead load may be the most significant load element, depending on the type of culvert, type and thickness of cover, and amount of live load. However, live loads on culverts are generally not as significant as the dead load unless the cover is shallow. Box culverts with shallow cover are examples of the type of installation where live loads are important.

In most culvert designs, the soil or embankment material surrounding the culvert plays an important structural role. Lateral soil pressures enhance the culvert's ability to support vertical loads. The stability of the surrounding soil is important to the structural performance of most culverts.

- Maintenance – Because culverts usually constrict flow, there is an increased potential for waterway blockage by debris and sediment, especially for culverts subject to seasonal flow. Multiple barrel culverts are particularly susceptible to debris accumulation. Scour caused by high outlet velocity or turbulence at the inlet end is of concern. As a result of these factors, routine maintenance for culverts primarily involves the removal of obstructions and the repair of erosion and scour. Other defects from weathering, loads, and age will occur and require routine maintenance.

- Traffic Safety – A significant safety feature of many culverts, as compared to bridges, is the elimination of a constriction in the roadway. Culverts can economically be extended so that the standard roadway cross section can be carried over the culvert. However, when the ends are located near traffic lanes or adjacent to a shoulder, guiderail may be required to protect the traffic.

- Construction – One of the most significant factors is that culverts are constructed in and through the roadway embankment, and vehicle loads are carried by the combined strengths of the culvert and the surrounding embankment. The trench width, bedding, compaction, and amount of fill over the culvert are important factors that influence the ability of the culvert to carry the design loads. Thus, the construction techniques and quality control of workmanship are critical to the ultimate serviceability and life expectancy of culverts.

- Durability – Durability of materials is a significant problem in culverts and other drainage structures. In hostile environments, corrosion and abrasion can cause deterioration of all commonly available culvert materials. Many types of serviceability problems may occur because of scour of streambeds and erosion of embankments adjacent to the culverts.

- Inspection – Highway bridges and culverts designated as “Qualifying Structures” in Section 1.4 of the ConnDOT Bridge Inspection Manual must be inspected at least once every two years. There is no mandated criteria for the length of time between inspections for culverts that are less than 6.1 m (20 ft.) in roadway span. However, ConnDOT inspects all culverts over 1.8m (6 ft.) on State highways. It is recognized that all other culverts may not be routinely inspected and that problems are reported on an as-noticed basis.

4.1.5 Types of Culverts

Although there is a very wide range of styles and designs of culverts in service, all culverts may be classified into two basic types: rigid and flexible. This classification is based on the primary difference in the manner in which structural loads are carried by the culvert and the interrelationship between the culvert structure and the surrounding soil. Rigid culverts are designed to resist bending moment; flexible culverts are not.

Culverts are also often described by their shape, which may be circular, arch, elliptical, or box. The box shape may be made more torsionally rigid by adding internal web walls between the top and bottom surfaces. Culverts may also be made with multiple barrels for additional flow capacity. Most modern culverts are made from either corrugated metal, plastic, or reinforced concrete. Concrete culverts may be of either precast or cast-in-place construction, which may be post-tensioned in the field. These materials may be used to construct most of the mentioned structural shapes.

4.1.6 General Problems with Culverts

There is a wide variety of types of problems that occur with culverts. The problems may be classified by serviceability and strength-related criteria. Listed below are general types of culvert problems:

Serviceability-related problems

- Scour and erosion of streambed and embankments
- Inadequate flow capacity
- Corrosion and abrasion of metal culverts
- Abrasion and deterioration of concrete and masonry culverts
- Sedimentation and blockage by debris
- Separation and/or dropoff of sections of modular culverts

- Inadequate length

Strength-related problems

- Cracking of rigid culverts
- Undermining and loss of structural support
- Loss of the invert of culverts due to corrosion or abrasion
- Over-deflection and shape deformation of flexible culverts
- Stress cracking of plastic culverts

4.1.7 Need for Economical Methods of Repair and Replacement

Many culverts are reaching the end of their design life. In many cases, these culverts have been constructed under high fill or are under roadways that carry high volumes of traffic. Replacement of these culverts can be very costly as well as cause severe disruption to traffic. These circumstances require that viable methods of repair and rehabilitation be investigated.

The information provided in this chapter was obtained from FHWA-RD-94-096, Culvert Repair Practices Manual, Volume 1, May 1995. This publication should be used as a guide in the repair of culverts and appurtenances on ConnDOT facilities.

4.2 Structural Design of Culverts

4.2.1 Introduction

Structural design of a culvert must be performed to ensure that the culvert is strong enough to resist the loads that will be imposed upon it. The strength of a culvert depends on the strength of the materials that are used and the shape of the culvert barrel. For example, a circular shape carries and resists loads differently than a box shape.

4.2.2 Loads

In addition to fulfilling their hydraulic functions, culverts must also support the weight of the embankment or fill covering the culvert and loads on the roadway. There are two general types of loads that must be carried by culverts: dead loads and live loads. The amount of both dead and live load that is actually exerted on a culvert depends upon whether it is a rigid or flexible material, the height of the embankment above the culvert, the type of material surrounding the culvert, the degree of compaction of the material, and whether special types of structural members are built around the culvert to resist and distribute soil pressures.

Dead loads on a culvert include the earth load or weight of the soil over the culvert and any added surcharge loads such as buildings or additional earth fill placed over or adjacent to the culvert alignment. The live loads on a culvert include the loads and forces that act upon the culvert due to vehicular or pedestrian traffic plus an impact factor. Actual loads for specific cases are assigned by the designer. The effect of live loads decreases as the height of cover over the culvert increases. For single-span culverts, the effects of live load may be neglected where the depth of fill is more than 2400mm (8 ft) and exceeds the span length; for multiple span culverts, the effects may be neglected where the depth of fill exceeds the distance, between faces of endwalls. Loading determinations should follow the procedures set forth in AASHTO LRFD Bridge Design specifications section 12.6 with an earth load of 2000 kg/m^3 (125 pounds per cubic foot) if the actual weight of earth is not known.

4.2.3 Flexible Culvert Behavior

A flexible culvert is a composite structure made up of the culvert barrel and the surrounding soil. The barrel and the soil are both vital elements to the structural performance of the culvert.

Flexible pipe has relatively little bending stiffness or bending strength on its own. As loads are applied to the culvert, it attempts to deflect. In the case of a round pipe, the vertical diameter decreases and the horizontal diameter increases, as shown in Figure 4-4.

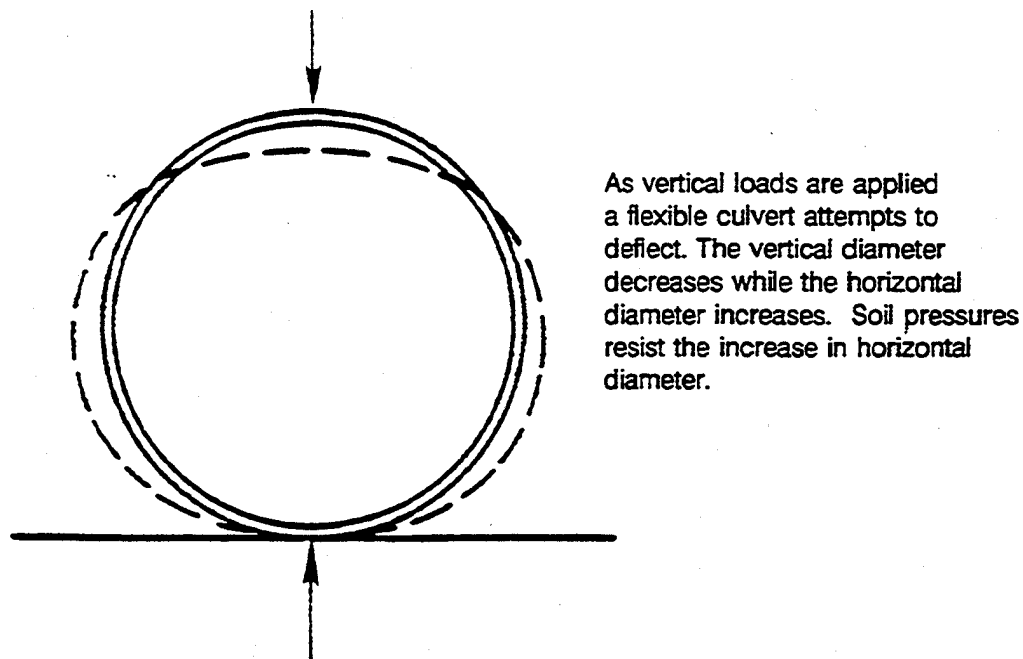
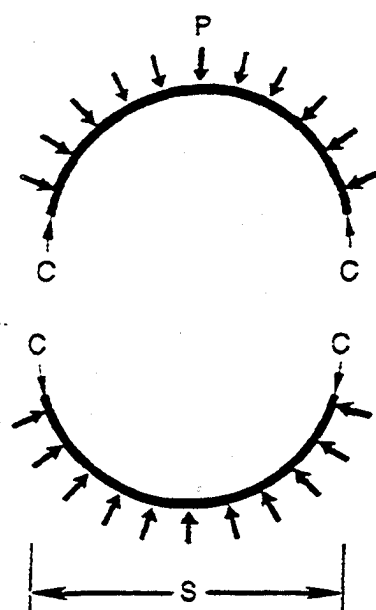


Figure 4-4 Deflection of flexible culverts.

When good embankment material is well compacted around the culvert, the increase in horizontal diameter of the culvert is resisted by the lateral soil pressure. With round pipe, the result is a relatively uniform radial pressure around the pipe that creates a compressive thrust in the pipe walls. As illustrated in Figure 4-5, the compressive thrust is approximately equal to vertical pressure times one-half the span length ($C = P \times S/2$ or $C = P \times R$).

An arc of a flexible round pipe or other shape will be stable as long as adequate soil pressures are achieved, and as long as the soil pressure is resisted by the compressive force C on each end of the arc. Good quality backfill material and proper installation are critical in obtaining a stable soil envelope around a flexible culvert.



Summing the vertical forces on half of the pipe at a time shows that

$$C = P \times S/2$$

where

C = Compressive thrust in the culvert wall.

P = Sum of soil pressure acting on the culvert.

S = The span or diameter.

$S/2$ = The radius (R).

Figure 4-5 Formula for ring compression

In long span culverts the radius (R) is usually large. To prevent excessive deflection due to dead and/or live loads, longitudinal or circumferential stiffeners are sometimes added. The circumferential stiffeners are usually metal ribs bolted to the outside of the culvert. Longitudinal stiffeners are reinforced concrete, as shown in Figure 4-6. The thrust beams are added to the structure when backfill reaches their location. The use of concrete stress-relieving slabs is another method used to achieve longer spans or reduce minimum cover. A stress-relieving slab is cast over the top of the backfill above the structure to distribute live loads to the adjacent soil.

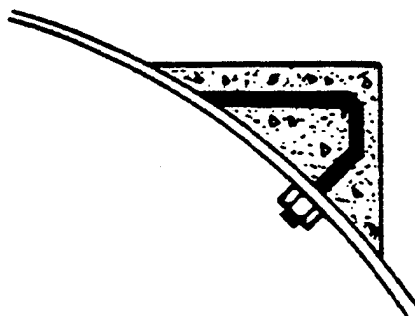


Figure 4-6 Concrete thrust beam used as a longitudinal stiffener

4.2.4 Rigid Culvert Behavior

The load carrying capacity of rigid culverts is essentially provided by the structural strength of the pipe itself and little benefit from the surrounding earth is required. When vertical loads are applied to a rigid pipe, zones of tension and compression are created as illustrated in Figure 4-7. With the exception of non-reinforced circular pipe, reinforcing steel is added to the tension zones to increase the tensile strength of concrete pipe. Shear stress in the haunch area can be critical for heavily loaded rigid pipe on hard foundations, especially if the haunch support is inadequate. Because rigid pipe is stiffer than the surrounding soil, it carries a substantial portion of the load.

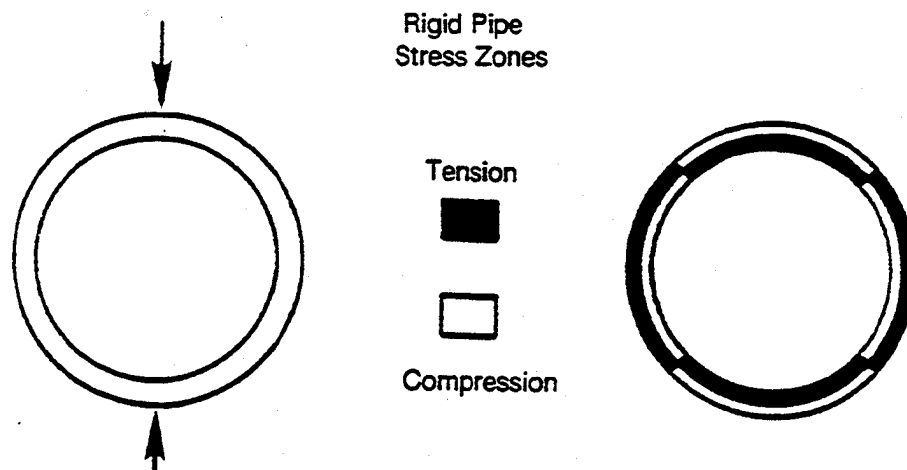


Figure 4-7 Zones of tension and compression in rigid pipes

The weight of each that must be carried varies with soil characteristics and installation conditions. The installation conditions can have a significant influence on the loads that must be carried by a rigid culvert. There are two major classes of installation conditions: 1) trench, where culverts are placed in natural ground or compacted fill with a controlled trench width and 2) embankment, where culverts are placed in or covered by an embankment.

In narrow trench installations, the pipe is placed in a relatively narrow trench and covered with backfill material. The backfill tends to settle more than the undisturbed soil beside the trench. Friction between the backfill material and the sides of the trench tends to help support the backfill material, reducing the load on the pipe. In effect the width of the soil column over the pipe is decreased. This concept is illustrated in Figure 4-8.

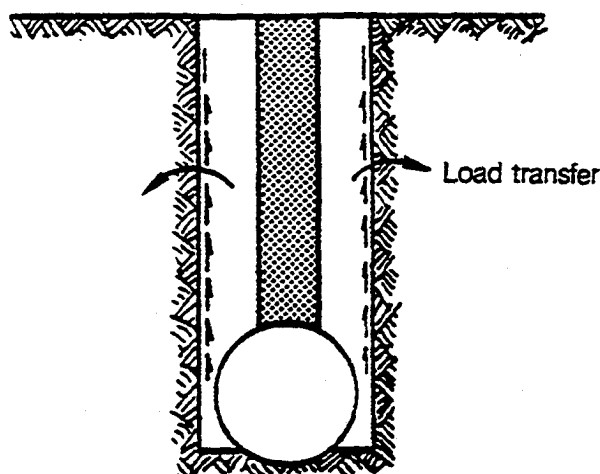


Figure 4-8 Trench installation. Friction on trench sides reduces the size of the column of fill carried by the pipe.

As the trench width increases, the effect of the friction at the sides of the trench is reduced and dead load on the pipe is increased. The amount that the loading is increased depends on trench width and the amount of backfill settlement, which is related to compaction. Poorly compacted soil will settle more than well compacted soil. In a trench that is too wide, poor compaction can result in an increase in the dead load on the pipe. Pipes placed in a shallow bedding on top of the original ground surface and then covered by the embankment material will have loads similar to the very wide trench. Pipes placed in trenches in the original ground prior to being covered by embankment have reduced earth loads similar to those described for the narrow trench.

4.2.5 Durability

Although structural condition is a very important element in the performance of culverts, durability problems are probably the most frequent cause of replacement. Culverts are more likely to “wear away” than fail structurally. Durability is affected by two mechanisms: corrosion and abrasion. Each are discussed in the following sections:

Corrosion

Corrosion is the deterioration of metals due to chemical or electrochemical reaction to the environment. Corrosion of culvert materials may occur in many different soils and waters. These soils and waters may contain acids, alkalis, dissolved salts, organics, industrial wastes or chemicals, mine drainage, sanitary effluents, and dissolved or free gases. However, culvert corrosion is generally related to water and the chemicals that have reacted to, become dissolved in, or been transported by the water.

Certain soil and water conditions have been found to be particularly aggressive or hostile to culverts. Extremes in acidity or alkalinity are much more aggressive than more neutral conditions. The term pH is a measure of the relative acidity or alkalinity: 7.0 is neutral, values less than 7.0 are

acid, and values greater than 7.0 are alkaline. For culvert purposes, values of less than 5.0 are strongly acid and those greater than 8.5 are strongly alkaline. Acid water stems from two sources: mineral and organic. Mineral acidity comes from sulfurous wells and springs and drainage from coal mines, with water containing dissolved sulfur and iron sulfide that may form sulfurous and sulfuric acids. Mineral acidity with a pH as strong as 2.3 has been encountered. Organic acidity, which may be found in swampy land and barnyards, may have a pH as low as 4.0. Alkalinity in water is caused by strong minerals and limed and fertilized fields. Acid water is more common to wet climates and alkaline water is more common to dry climates.

Along with pH, water resistivity (R) is an effective way to measure how aggressive the water's chemistry may be. Resistivity of low flow samples can accurately predict the potential corrosivity of drainage runoff. Resistivity above 1500 ohm-cm are considered normal. Resistivity between 500-1500 ohm-cm are considered below normal. Resistivity below 500 ohm-cm should be considered aggressive.

The electrical resistivity of soil, which depends largely on the nature and amounts of dissolved salts, also influences the potential for corrosion. The greater the resistance the less the flow of electrical current associated with corrosion. High moisture content and temperature lower the resistivity and increase the potential for corrosion. The use of granular backfill around the entire pipe will increase electrical resistivity and reduce the potential for galvanic corrosion.

Corrosion can attack the inside or outside of the culvert barrel. The chemicals in drainage water can attack the material on the interior of the culvert. Culverts subject to continuous flows or standing water with aggressive chemicals are more likely to be damaged than those with intermittent flows. The exterior of culverts can be attacked by chemicals in the ground water that can originate in the soil, be introduced through contaminants in the backfill soil, or be transported by subsurface flow.

Corrosion affects all metals and alloys, although the rates can vary widely depending both upon the chemical and physical properties of the metal and upon the environmental condition to which it is exposed. When a metal corrodes, a very low voltage electrical current is established between two parts of a metal surface that have different voltage potential. The difference in voltage potential may be caused by slight variations in the material, changes in surface condition, or the presence of foreign materials. The current removes metallic ions from one location and deposits them at another location, causing corrosion, as shown in Figure 4-9. The chemicals present in the water greatly influence its effectiveness as an electrolyte.

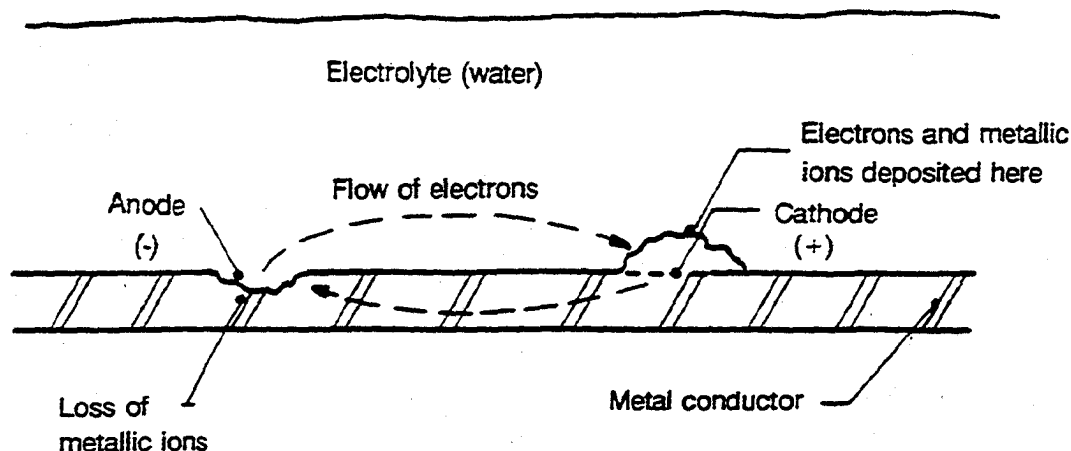


Figure 4-9 The corrosion process

Although less common than with metal pipe, corrosion can occur in concrete culverts. Metallic corrosion can take place in the reinforcing steel when it is exposed by cracking or spalling, when the concrete cover is inadequate, or when the concrete is porous enough to allow water to contact the reinforcing steel.

If steel corrodes, the corrosion products expand and may cause spalling of the concrete. Corrosion can also take place in the concrete itself. It is not, however, the same type of electrochemical reaction that occurs in metal. Other reactions between the concrete materials and the chemicals present in the stream flow or ground water are involved and can result in deterioration of the concrete.

Abrasion

Abrasion is the process of wearing down or grinding away the surface material of culverts as water laden with sand, gravel, or stones flow through a culvert. Abrasion forces increase as the velocity of the water flowing through a culvert increases; for example, doubling the velocity of a stream flow can cause the abrasive power to become approximately four-fold.

Often corrosion and abrasion operate together to produce far greater deterioration than would result from either alone. Abrasion can accelerate corrosion by removing protective coatings and allowing water-borne chemicals to come into contact with corrodible culvert materials.

4.2.6 Economic Considerations

For the design of new culverts and major culvert repairs, an economic analysis usually includes factors such as construction cost, estimated service life, maintenance cost, replacement cost, risk of failure, and risk of property damage. The most economical culvert is not the one with the lowest initial cost nor the culvert with the longest service life. The importance is that short and long term costs should be considered in both original designs and in repairs or replacements.

4.2.7 Maintenance

It is appropriate to emphasize the need to consider maintenance needs in the design of culverts. That is, the designs should be such that the need for maintenance and repair work is minimized through the selection of the culvert type and the quality of the materials and construction methods that are used. For example,

- If abrasion problems are anticipated, then the designs should minimize the potential problem by flattening the slopes, providing stilling basins, or providing a tough, abrasion-resistant invert.
- If a problem with sedimentation is expected, it may be possible to steepen slopes or select a culvert shape (such as a box) that is easier to clean out with mechanized equipment.

4.2.8 Safety

Personnel safety should always be of concern during the construction, inspection, maintenance, repair, and rehabilitation of culverts. To the maximum extent possible, the designer should identify low flow periods for construction for certain types of structures. Inspection, maintenance, and repair personnel should be aware of the possibilities of poor air quality, toxic and chemical contaminants, animals, and the potential for collapse of unstable structures.

4.2.9 Geotechnical

During design, particularly for larger culverts, 0.9m (3 ft) span or greater, the foundation conditions should be investigated to determine such factors as allowable bearing pressure, bedding requirements, and any condition requiring special treatments. In addition, determinations should be made concerning any unusual construction conditions such as groundwater, slope stability, and rock excavation. These factors apply to the end treatments, approaches, and barrel elements. The type, strength, slope, and bedding of soils and rocks all influence the design, construction and maintenance/repair operations.

4.3 Culvert Shapes

4.3.1 General

A wide variety of standard shapes and sizes are available for most culvert materials. Since equivalent openings can be provided by a number of standard shapes, the selection of shape may not be critical in terms of hydraulic performance. Shape selection is often governed by factors such as depth of cover or limited headwater elevation. In such cases a low profile shape may be needed. Other factors such as the potential for clogging by debris, the need for a natural stream bottom, or structural and hydraulic requirements may influence the selection of culvert shape. Each of the common culvert shapes is discussed in the following paragraphs. More details are included in the section on materials.

4.3.2 Circular Pipes

The circular shape is the most common shape manufactured for pipe culverts. It is hydraulically and structurally efficient under most conditions. Possible hydraulic drawbacks are that circular pipe generally causes some reduction in stream width during low flows. It may also be more prone to clogging than some other shapes due to the diminishing free surface as the pipe fills beyond the midpoint. With very large diameter corrugated metal pipes, the flexibility of the structure dictates that special care be taken during backfill construction to maintain uniform curvature.

4.3.3 Pipe Arch and Elliptical Shapes

Pipe arch and elliptical shapes are often used instead of circular pipe when distance from channel invert to pavement surface is limited or when a wider section is desirable for low flow levels. These shapes may also be prone to clogging as the depth of flow increases and the free surface diminishes. Pipe arch and elliptical shapes are not as structurally efficient as a circular shape. They are normally used in areas with limited vertical clearance and low cover conditions.

4.3.4 Arches

Arch culverts have no culvert barrel material at the bottom and offer less of an obstruction to the waterway than pipe arches and can be used to provide a natural stream bottom where the stream bottom is naturally erosion and abrasion resistant. The structure should also meet scour design requirements.

4.3.5 Box Sections

Rectangular or square cross section culverts are easily adaptable to a wide range of site conditions, including sites that require low profile structures. Due to the angular corners, boxes are not as structurally and hydraulically efficient as other culvert shapes.

4.3.6 Multiple Barrels

Multiple barrels are used to obtain adequate hydraulic capacity under low embankments or for wide waterways. In some locations they may be prone to clogging as the area between the barrels tends to catch debris and sediment. When a channel is artificially widened, multiple barrels placed beyond the dominant channel are subject to excessive sedimentation. **The span or opening length of multiple barrel culverts includes the distance between barrels as long as that distance is less than half the opening length of the adjacent barrels.**

4.4 Culvert Materials

4.4.1 General

Culverts are primarily made with reinforced concrete, corrugated metal, and more recently, solid-wall, profile wall, and reinforced plastic. The strength and physical characteristics of the materials depend upon their chemistry and the interrelationship between the constituent materials. Metals and plastic are homogeneous isotropic materials whereas concrete and masonry is a mixture or combination of materials. The method by which the materials are connected significantly influences whether the strength of the materials may be utilized structurally.

4.4.2 Concrete

Culverts may be made with either precast or cast-in-place reinforced concrete. This selection depends on the size and complexity of the culvert design. Precast sections are uniform in size and shape and are made in sections that can easily be transported, lifted, and installed. Cast-in-place concrete construction is often used when ready-mix concrete is available and when the culvert should be constructed without joints. Precast concrete culverts may be made with high strength concrete, whereas cast-in-place concrete culverts may have special reinforcement at critical locations to resist high loads and stresses.

- Precast – Precast concrete pipe is manufactured in eight standard shapes: circular, arch, horizontal elliptical, vertical elliptical, pipe arch, box sections, three-sided arch top, and flat top sections, as shown in Table 4-1. With the exception of box culverts, concrete culvert pipe is manufactured in up to five standard strength classifications. The higher the classification number the higher the strength. Box culverts are designed for various depths of cover and live loads. All of the standard shapes are manufactured in a wide range of sizes. Circular and elliptical pipes are available with standard sizes as large as 3600 mm (144 inches) in diameter, with larger sizes available as special designs. Standard box sections are also available with spans as large as 3600 mm (144 inches). Precast concrete arches on cast-in-place footings are available with spans up to 12.2 m (40 feet).
- Cast-in-place – Reinforced culverts that are cast-in-place are typically either rectangular or arch-shaped. the rectangular or box shape is more common and is usually constructed with multiple cells (barrels) to accommodate longer spans. One advantage of cast-in-place construction is that the culvert can be designed to meet the specific requirements of a site. Due to the longer construction time of cast-in-place culverts, precast concrete or corrugated metal culverts are often selected. However, in many areas cast-in-place culverts may be more practical.
- Shapes – By the very nature of it, reinforced concrete may be used to make virtually any structural shape desired. Thus, if necessary and feasible, it is possible to make almost any shaped culvert with either precast or cast-in-place reinforced concrete.

Table 4-1 Standard concrete pipe shapes

SHAPE	RANGE OF SIZES	COMMON USES
CIRCULAR 	300 to 400mm (12 to 180 in) reinforced 100 to 900mm (4 to 36 in) non-reinforced	Culverts, storm drains, and sewers.
PIPE ARCH 	375 to 3300 mm (15 to 132 in) equivalent diameter	Culverts, storm drains, and sewers. Used where head is limited.
HORIZONTAL ELLIPSE 	Span X Rise 450 to 3600 mm (18 to 144 in) equivalent diameter	Culverts, storm drains, and sewers. Used where head is limited.
VERTICAL ELLIPSE 	Span X Rise 900 to 3600 mm (36 to 144 in) equivalent diameter	Used where lateral clearance is limited.
RECTANGULAR (box sections) 	Span 0.9 to 3.6 m (3 to 12 ft)	Culverts, storm drains, and sewers. Used for wide openings with limited head.
ARCH 	Span 7.2 to 12.3 m (24 to 41 ft)	Culverts and storm drains. For low, Wide waterway enclosures.
FLAT TOP 3-SIDED 	Span 4.2 to 10.5 m (14 to 35 ft)	Culverts and storm drains. For low, Wide waterway enclosures.
ARCH TOP 3-SIDED 	Span 4.8 to 10.8 m (16 to 36 ft)	Culverts and storm drains. For low, Wide waterway enclosures.

4.4.3 Corrugated Steel

Corrugated steel culverts are made with factory-produced corrugated sheet steel. Corrugated pipe culverts are made with factory-produced corrugated pipe sections. Large corrugated culverts are normally field-assembled using structural plate products. Structural plate steel products are available as structural plate pipes, box culverts, or long span structures. Standard shapes for corrugated steel culverts are shown on Table 4-2.

- **Material** – Corrugated steel pipe is fabricated from sheets coated with zinc or aluminum. It is reasonably lightweight for shipping and comes in a large range of thicknesses and corrugations to provide the appropriate strength. However, it requires controlled backfill for proper soil support. Other options include various coatings and/or pavings for added protection.
- **Shapes** – Corrugated steel may be used for a wide variety of shapes, sizes, and lengths of culverts. The culverts may be made from prefabricated sections that are factory produced or assembled in the field from specially fabricated plates. The shapes may be made from various thicknesses of plate stock.

Pipe – Corrugated steel pipe is factory made in two basic shapes: round and pipe arch. Both round and arch shapes are available in a wide range of standard sizes. Round pipe is available in standard sizes up to 3600 mm (144 inches) in diameter. Standard sizes for pipe arch are available in sizes up to the equivalent of 3000 mm (120 inch) diameter round pipe. Both shapes are produced in several wall thicknesses, several corrugation sizes, as shown in Figure 4-10 and with annular (circumferential) or helical (spiral) corrugations.

Pipes with annular corrugations have riveted, spot welded, or bolted seams. Pipes with helical corrugations have continuously welded seams or lock seams. Corrugated steel pipe and pipe arch are usually coated with zinc (galvanized) or aluminum. Additional protective coatings are used with the metallic coating when there are potential corrosion or abrasion problems.

Structural plate – Structural plate steel pipes are field assembled from standard corrugated galvanized steel plates. Standard plates have corrugations with a 150mm (6-inch) pitch and a depth of 50 mm (2 inches). Plates are manufactured in a variety of thicknesses and are pre-curved for the size and shape of the structure to be erected. Standard plates have a nominal length of either 3 m or 3.7m (10 or 12 feet) and are produced in standard widths of 3N, 5N, 6N, 7N, and 8N, where N equals 3 pi or 244 mm (9.6 inches). Widths are measured along the circumference of the structure. Since the circumference of a circle equals pi times the diameter, the use of dimensions expressed in N or pi permits easy conversion from pipe circumference of 60 pi or 20N and would normally be assembled from four 5N plates. Structural plate pipes are available in six basic shapes: round, pipe arch, arch, vertical ellipse, horizontal ellipse, and underpass. The standard sizes available range in span from 1.5 m to 7.9m (5 feet to 26 feet).

Box – Steel box sections use standard 150 by 50 mm (6 by 2 inch) corrugated galvanized steel plates with special reinforcing elements applied to the areas of maximum moment or 375 by 140 mm (15 by 5 1/2 inches) corrugated plate without ribs. Steel box culverts are available with spans that range from 3m (9 feet 8 inches) to 6.3m (20 feet 9 inches).

Long span – Long span steel structures are assembled using conventional 150 by 50 mm (6 by 2 inch) corrugated galvanized steel plates with longitudinal or circumferential stiffening members or 375 by 140 mm (15 by 5 ½ inch) corrugated plate without ribs. There are five standard shapes for long span structures: horizontal elliptical, pipe arch, low profile arch, high profile arch, and pear shape. The long span pipe arch is not commonly used. The span lengths of typical sections range from 5.9m (19 feet 4 inches) to 12.2 m (40 feet). Longer spans are available for some shapes as special designs.

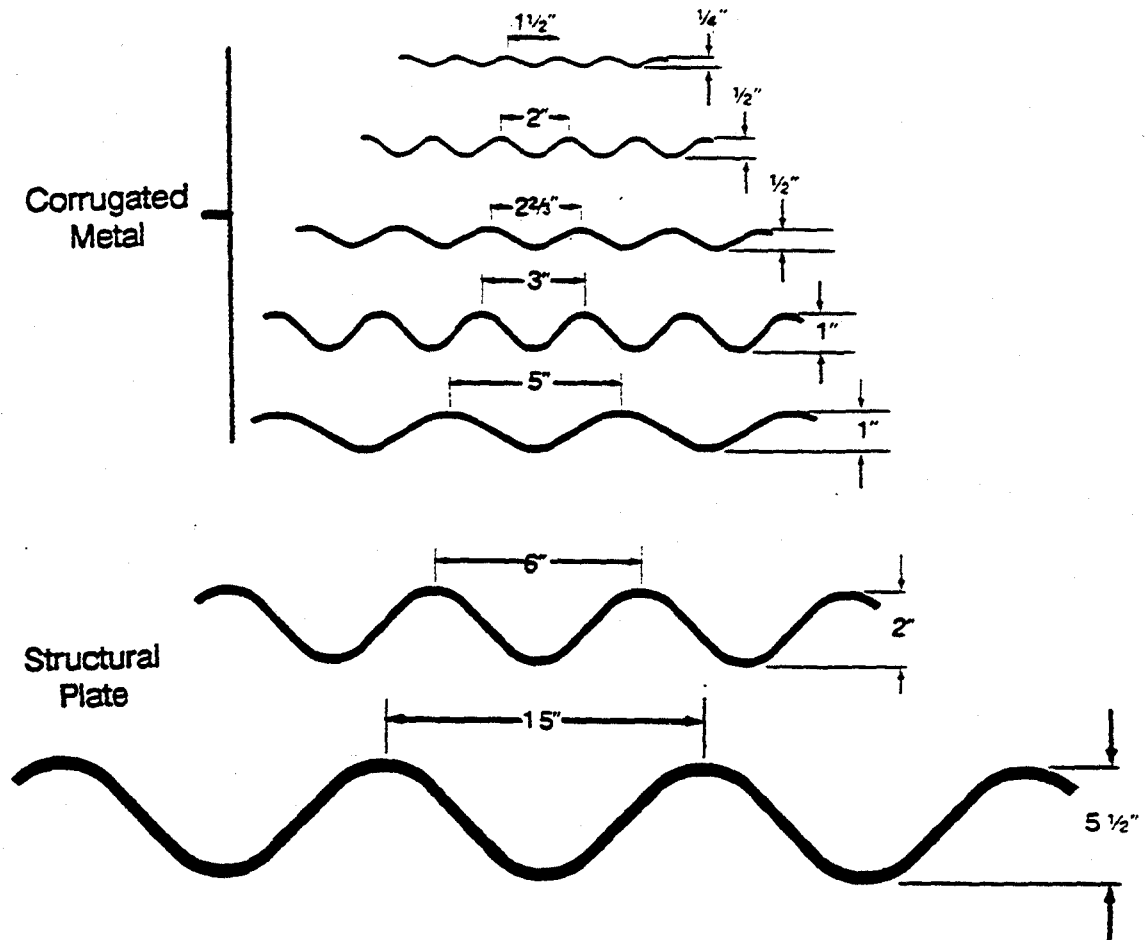
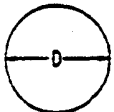
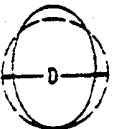
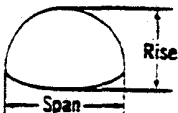
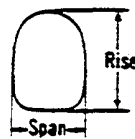
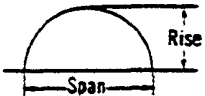
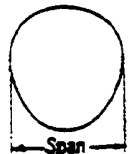
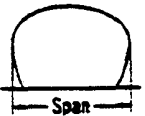


Figure 4-10 Common corrugated patterns (not to scale) (English only)

Table 4-2 Standard corrugated steel culvert shapes (English only)

Shape	Range of Sizes	Common Uses
Round 	6 in. – 26 ft.	Culverts, subdrains, sewers, service tunnels, etc. All plates same radius. For medium and high fills (or trenches).
Vertically-elongated (ellipse) 5% is common 	4 – 21 ft. nominal; before elongating	Culverts, sewers, service tunnels, recovery tunnels. Plates of varying radii; shop fabrication. For appearance and where backfill compaction is only moderate.
Pipe-arch 	Span x Rise 17 in. x 13 in. to 20 ft. 7 in. x 13 ft. 2 in.	Where headroom is limited. Has hydraulic advantages at low flows. Corner plate radius. 18 inches or 31 inches for structural plate.
Underpass* 	Span x Rise 5 ft. 8 in. x 5 ft. 9 in. to 20 ft. 4 in. x 17 ft. 9 in.	For pedestrians, livestock or vehicles (structural plate).
Arch 	Span x Rise 6 ft. x 1 ft. 9 1/2 in. to 25 ft. x 12 ft. 6 in.	For low clearance large waterway opening, and aesthetics (structural plate).
Horizontal Ellipse 	Span 7 – 40 ft.	Culverts, grade separations, storm sewers, tunnels.
Pear 	Span 25 – 30 ft.	Grade separations, culverts, storm sewers, tunnels.
High Profile Arch 	Span 20 – 45 ft.	Grade separations, culverts, storm sewers, tunnels, ammo ammunition magazines, earth covered storage.
Low Profile Arch 	Span 20 – 50 ft.	Low-Wide waterway enclosures, culverts, storm sewers.
Box Culverts 	Span 10 – 26 ft.	Low-Wide waterway enclosures, culverts, storm sewers.
Specials	Various	For lining old structures or other special purposes. Special fabrication.

*For equal area or clearance, the round shape is generally more economical and simpler to assemble.

4.4.4 Corrugated Aluminum

Corrugated aluminum culverts are constructed from factory assembled corrugated aluminum pipe or field assembled from structural plates. Structural plate aluminum culverts are available as conventional structural plate structures, box culverts, or long span structures.

- **Material** – Corrugated aluminum pipe is fabricated from aluminum-alloy sheets. It is very lightweight for shipping and handling. It has good resistance to corrosion, especially in brackish waters but is subject to abrasion in fast-flowing streams with a significant load of sand or rock. It is generally more flexible than steel, requires greater care in installation, and is less tolerant of less-than-normal cover.

- **Shapes** – Corrugated aluminum may be used for a wide variety of shapes, sizes, and lengths of culverts. The culverts may be made from prefabricated sections that are factory produced or assembled in the field from specially fabricated plates. The shapes may be made from various thickness of plate stock.

Pipe - Factory assembled aluminum pipe is available in two basic shapes: round and pipe arch. Both shapes are produced with several different wall thicknesses, several corrugation patterns, and with annular (circumferential) or helical (spiral) corrugations. Round aluminum pipe is available in standard sizes up to 3000 mm (120 inches) in nominal diameter. Aluminum arch pipe is available in sizes up to the equivalent of a 2400 mm (96-inch) diameter round pipe.

Structural plate - Structural plate aluminum pipes are field assembled with 228 mm (9-inch)-pitch by 64 mm (2.5-inch)-depth corrugations. Plates are manufactured in a variety of plate thicknesses and are pre-curved for the specific size and shape of the structure to be erected. Plates are manufactured in lengths of SN through 18N, where N equals 3 pi or 244 mm (9.625 inches). Plate length is measured along the circumference of the structure. Standard plates have a net width of 1.4 m (4.5 ft.). Structural plate aluminum pipes are produced in five basic shapes: round, pipe arch, arch, pedestrian/animal underpass, and vehicle underpass. A wide range of standard sizes is available for each shape. Spans as large as 7.9 m (26 feet) can be obtained for the arch shape.

Box - The aluminum box culvert utilizes standard aluminum structural plates with aluminum rib reinforcing added in the areas of maximum moments. Ribs are bolted to the exterior of the aluminum shell during installation. Aluminum box culverts are suitable for shallow depths of fill and are available with spans ranging from 2.7 m (8 feet 9 inches) to 7.7m (25 feet 5 inches).

Long Span - Long span aluminum structures are assembled using conventional 225 by 64 mm (9- by 2.5-inch) corrugated aluminum plates and aluminum rib stiffeners. Long span aluminum structures are available in the same five basic shapes as steel long spans: including horizontal ellipse, pipe arch, low profile arch, high profile arch, and pear shape. The typical sizes for aluminum spans are essentially the same as the typical sizes available for steel long span structures. Spans range from 5.9 m (19 feet 4 inches) to 12.2 m (40 feet).

4.4.5 Plastic

“Plastic” pipe is as unspecified a term as is “metal” pipe. There are many types of materials that may be used to produce plastic pipe, and the resulting pipe will have strength and other properties that vary accordingly. The properties of the plastic will depend primarily on the type of base resin that is used as well as the blend (or formulation) of chemicals in the final resin material that is used to produce the pipe. Just as with the design of concrete mixes, it is a common practice to use special additives with the basic resin to facilitate the production process and/or to alter the resulting physical and chemical properties of the finished product.

In general, plastics may be divided into two basic groups: (1) thermoplastics and (2) thermosetting plastics. The primary difference between these classes of material is that thermoplastics may be remelted and reshaped whereas thermosetting plastic cannot be remelted. Thus, the strength and other properties of thermoplastics will depend on the ambient temperature, and thermosetting plastics will retain their strength properties under a wide range of temperatures. The strength of these plastics will depend more on the types of resins that are used than on whether they are thermoplastics or thermosetting plastics.

Although both types of plastic may be used for culvert and drainage products, they are usually constructed from thermoplastic-type materials, which are less expensive and more easily used to manufacture. Two of the most popular types of material that are used are polyvinyl chloride (PVC) and polyethylene (PE). Thermosetting type resins are commonly used for pipe that must handle fluids at high temperatures.

Plastic drainage products may also be classified according to whether they are made just of plastic or whether the plastic is reinforced with fibers, typically glass fibers. The latter may be called “fiberglass” pipe. Since glass fibers have a filament strength of over 2067 N/mm² (300,000 psi), pipe products that are made with long continuous glass fibers will have greater strength properties over unreinforced plastic pipe.

- Polyvinyl Chloride (PVC) - Polyvinyl Chloride piping is made only from compounds that do not contain plasticizers and minimal quantities of other ingredients. It has been labeled as rigid PVC in the United States to distinguish it from flexible or plasticized PVCs from which such items as laboratory tubing, luggage, and upholstery are made. This pipe exhibits good long-term strength with high stiffness. It is for this reason that PVC has become an important material for both pressure and nonpressure pipe applications. There is a much broader range of PVC fittings, valves, and appurtenances available than in any other plastic. The pipe is manufactured in both solid wall and profile wall in sizes up to 1200 mm (48 inches).

- Polyethylene (PE) - Polyethylene is perhaps the most well known of the plastics in the polyolefin group. These are plastics that are formed by the polymerization of straight chain hydrocarbons that are known as olefins. They include ethylene, propylene, and butylene. PE piping is tough and flexible, even at subfreezing temperatures. PE pipe has good abrasion resistance and is available in solid wall and profile wall with diameters up to 2400 mm (96 inches). It is often used to replace deteriorating pipes.

4.4.6 Other Materials

- Masonry - Stone and brick are durable, low maintenance materials. Prior to the 1920's, both were used frequently in railroad and road construction projects because they were readily available from rock cuts or local brickyards. Although stone and brick are seldom used for

constructing culvert barrels, stone is used occasionally for this purpose in locations that have very acid runoff. The most common use of stone is for headwalls where a rustic or scenic appearance is desired. Brick is frequently used in the construction of manholes and inlets in storm drainage systems, because it may easily be built up without the need for formwork.

- **Vitrified Clay Pipe** - Vitrified clay pipe is manufactured from clays and shales that are the mineral aggregates remaining after the weathering process of nature. This weathering process leaches out the soluble and reactive minerals from the rock and soil, leaving an inert material. This chemically inert material is then burned in kilns at 1000-2100 degrees Fahrenheit at which "vitrification" occurs and the clay particles become fused into an inert chemically stable compound.

Vitrified clay pipe is resistant to internal and external attack from acids, alkalies, gases, and solvents. It is resistant to abrasion and scour and will not corrode.

- **Cast Iron** - Cast iron is iron in which carbon has been dissolved. It is generally no longer used for culvert construction. It has poor tensile strength and is brittle and susceptible to cracking. The shapes are cast and are bulky in comparison to steel. Cast iron does, however, exhibit good corrosion resistance.

4.4.7 Coatings for Culvert Materials

A variety of types of coatings may be used either singularly or in a combination of layers to protect culverts from chemical and/or abrasion attack. The type(s) of coatings will depend upon the type of culvert material and the types of deterioration or distress they incur. The necessity for protective coatings depends upon a number of factors, including:

- Chemistry and acidity (pH) of the adjacent soil
- Chemistry and acidity (pH) of the water passing through the culvert
- Particle size and velocity of the solid material being transported through the culvert
- Environmental effects including freezing and thawing

- **Coatings for metal culverts** - Corrugated steel culverts are protected with metallic coatings of zinc (galvanized) or aluminum. Protective coatings for metal culverts also include bituminous coatings, bituminous paving, fiber-bonded bituminous coatings, polymer, concrete paving, and concrete coatings. Additional protective coatings are used with the metallic coating when there are serious corrosion or abrasion problems.

Bituminous - This is the most common material used to protect corrugated steel pipe against corrosion. This procedure can also increase the resistance of metal pipe to acidic conditions if the coating is properly applied and it remains in place. Careful handling during transportation, storage, and installation is required to avoid damage to the coating. Bituminous coatings can also be damaged by abrasion. Field repairs should be made when bare metal has been exposed. Inert fibers may be embedded in the zinc coating to improve the adherence to metallic-coated bituminous material pipe. It should be noted that the durability of bituminous coatings is dependent on strict adherence by the fabricator to proper coating procedures.

Polymer - There are several types of polymer coatings that may be applied for corrosion and/or abrasion protection. The term polymer generally refers to a variety of types of plastic that

may be used either plain "neat" or as a matrix for binding aggregates together, much the same as portland cement or asphaltic cement are used to make those respective types of concrete. Plain plastic coatings, often epoxies, may be applied directly to the metal or to other surface coatings. Culverts may also be coated with a polymer concrete, which is a mixture of plastic and aggregate. There have also been recent developments for coating metal culverts with fiberglass, which are (for these types of applications) short glass fibers held in a resin matrix. However, the 10 mil thick PVC and polyolefin plastic coatings that may be used to coat metal culverts do not provide increased resistance to abrasion, although polyethylene will to some extent.

Concrete/mortar - Metal culverts may be coated with a Portland Cement mortar or concrete for corrosion and abrasion resistance. Concrete of good quality is resistant to many corrosive agents. When the effluent has a pH of 5.0 or less, protective measures are generally required. One problem with using this type of coating is getting a good bond or connection between the metal pipe and the mortar or concrete lining.

Galvanizing - Galvanizing refers to the process of coating steel with a layer of zinc. Bare, uncoated, galvanized steel pipe generally performs well when the pH of the soil immediately adjacent to the pipe and the pH of the flow that the pipe will carry are between 6 and 10 and when the electrical resistivity of the soil is 2,000 ohm-cm or greater. Bare galvanized steel pipe should not be used in salt or brackish environments.

Aluminum coating Type 2 - Steel may also be coated with aluminum for corrosion protection. Aluminum generally performs adequately when the pH of the soil immediately adjacent to the pipe and the pH of the flow that the culvert will carry are between 5 and 9, and when the electrical resistivity of the flow and the minimum electrical resistivity of the soil is 1500 ohm-cm or greater. When backfilled with a clean, granular, well-drained soil, aluminum coated pipe has shown excellent resistance to corrosion, except when exposed to seawater and tidal flow. Aluminum coatings may not perform well in very acid or heavy metal (copper, iron, etc.) environments. If the pH is between 6.0 and 8.0, aluminum coated Type 2 is acceptable with resistivity of 100 ohm-cm or greater.

Type 1 aluminum coatings are inappropriate for drainage applications.

- **Coatings for concrete culverts** - Concrete culverts are rarely coated when they are constructed. However, when they are installed in particularly aggressive chemical environments, they may be coated with epoxy resins or special high density, low porosity concrete materials that have a high resistance to chemicals and chemical attack.

- **Invert protection** - The inverts of corrugated metal culverts are frequently paved to extend the life of the culvert by protecting the invert against corrosion and abrasion. The paving also smoothes the inside of the culvert, which improves the hydraulic capacity of such culverts.

Bituminous paving - Paving of CMP inverts with bituminous materials has been a common practice for many years. The bituminous coating is usually at least 3 mm (1/8-inch) thick over the inner crest of the corrugations. Generally only the lower quadrant of the pipe interior is paved. Fiber binding is sometimes used to improve the adherence of bituminous material to the metallic-coated pipe. Although bituminous paving has been widely used, it has been found that the coating may deteriorate and spall off after a number of years, particularly in some environments. After the coating starts to deteriorate, corrosion of the culvert will begin.

Concrete paving - The invert of culverts may also be paved with plain or reinforced Portland Cement concrete. For both new and repair situations this type of paving would normally be applied after the culvert is installed. Although this would normally be done only for corrugated metal pipe culverts, it is occasionally used for precast concrete culverts, to provide additional thickness to resist abrasion and/or corrosion. Metal culvert sections may also be factory produced with a complete concrete lining.

4.5 Culvert Construction

4.5.1 General

There are two major classes of culvert installations, based upon the conditions that influence loads: 1) trenched, where culverts are placed in natural ground or compacted fill with a controlled trench width and 2) embankments, where culverts are usually placed in natural ground but are covered by a constructed embankment. A third method of installation for placing culverts is boring and jacking, used where deep installations are necessary or where conventional open excavation is not practical.

- Trenched

Trench installations are made in relatively narrow excavations on a carefully prepared bedding to distribute the load and the culvert is covered with earth backfill that extends to the ground surface. The trench load theory is based on the following assumptions:

- Earth loads on the pipe develop as the backfill settles.
- The resulting earth load on the pipe is equal to the weight of the material above the top of the pipe minus the shearing (frictional) forces on the side of the trench.
- Cohesion is negligible because, with cohesive soils, considerable time must elapse before effective cohesion between the backfill material and the sides of the trench can develop, and with cohesionless soils, would never develop. The assumption of no cohesion yields the maximum probable load on the pipe.
- For a rigid pipe, the side fills may be relatively compressible and the pipe will carry a large portion of the load over the entire width of the trench.
- For rigid pipe, active lateral pressure is neglected, which, in effect, increases the required pipe strength. (However, it should be taken into account if investigations and experience indicate such pressure is significant.

For flexible culverts, a well-compacted soil envelope of adequate width is needed to develop the lateral pressures required to maintain the shape of the culvert. The width is a function of the strength of the surrounding in-situ soil and the size of the pipe.

The backfill load ultimately transmitted to the pipe is a function of the trench width. With rigid culvert placement, the determination of the backfill load is based on the trench width and a pipe strength is selected to withstand that load. **If the actual trench width exceeds the width assumed in design, the load on the culvert will be greater than estimated and structural distress may result.**

Figure 4-13 illustrates the load carried by a rigid culvert installed in a normal trench installation.

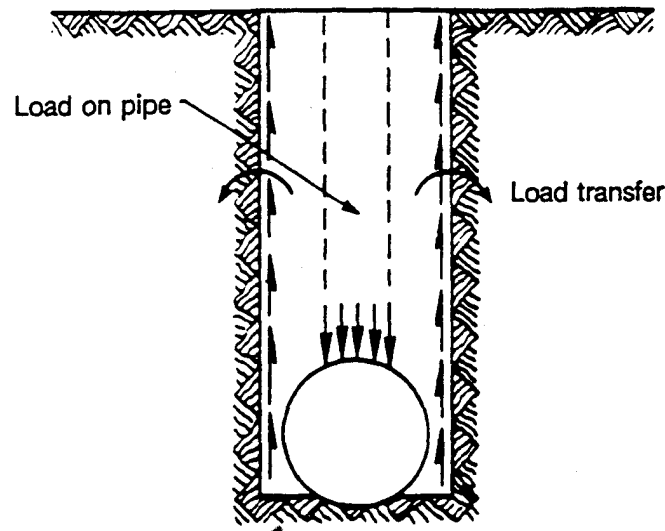


Figure 4-13 Trench installation

Figure 4-14 illustrates the increased load on the rigid culvert if the width of the trench is increased.

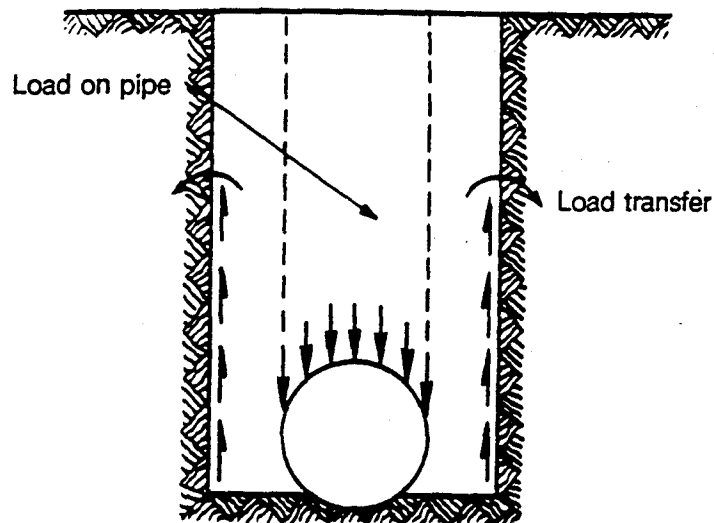


Figure 4-14 Wide trench installation

If an excessively wide trench is excavated or if the sides are sloped back, the culvert can be installed in a narrow subtrench excavated at the bottom of the wider trench, as shown in Figure 4-15, to avoid an increase in the backfill load.

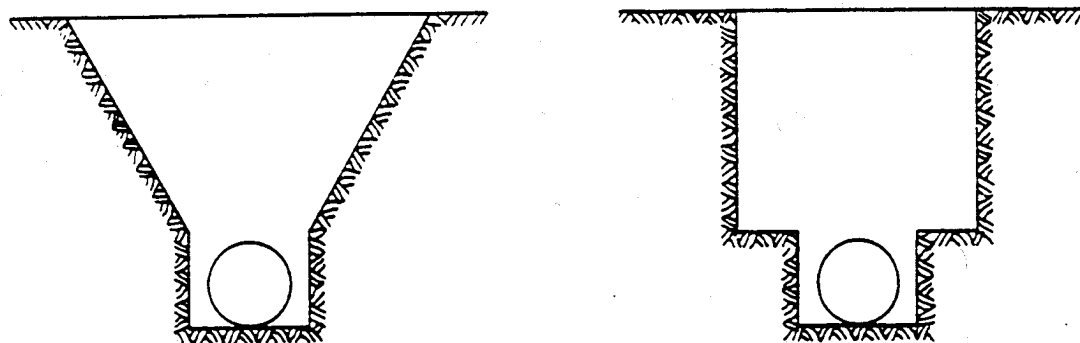


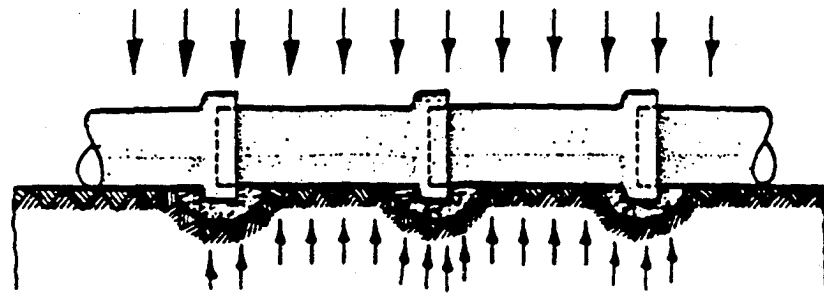
Figure 4-15 Subtrench installation in a wide trench

- **Bedding** - A stable and uniform foundation is necessary for the satisfactory performance of any culvert. Once a stable and uniform foundation is provided, the bedding should be prepared in accordance with the plans and specifications.

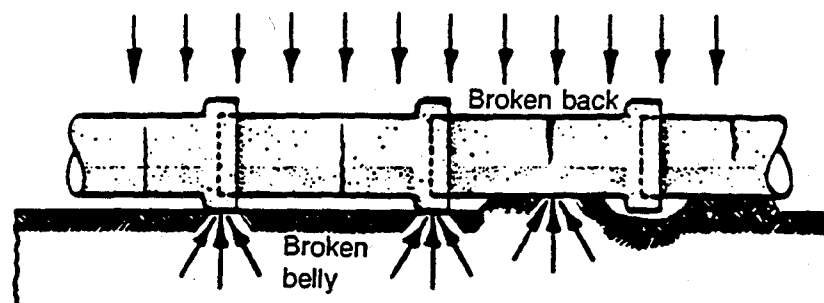
The bedding preparation is critical to both structural performance and service life. An important function of the bedding is to provide uniform support along the barrel of each pipe section. The bed should be placed to uniform grade and line to ensure good vertical alignment and to avoid excessive stresses at joints. The bed material should be free of rock formations, protruding stones, frozen lumps, roots, and other foreign matter that may cause unequal settlement. When a corrugated metal culvert is being placed, the corrugations should be firmly seated in the foundation material.

Transverse or circumferential cracks in rigid pipe may be caused by poor bedding. Cracks can occur across the bottom of the pipe (broken belly) when the pipe is only supported at the ends of each section. This is generally the result of poor installation practices such as not providing indentions (bell holes) in hard foundation material for the ends of bell and spigot-type pipe or not providing a sufficient depth of suitable bedding material. Cracks may occur across the top of pipe (broken back) when settlement occurs and rocks or other areas of hard foundation material near the midpoint of a pipe section are not adequately covered with suitable bedding material. Transverse cracking is illustrated in Figure 4-16.

The bedding distributes the load reaction around the lower periphery of the pipe. The required supporting strength of the pipe is directly related to this load distribution. Pipe set on a flat foundation without bedding results in high load, concentration at the bottom of the pipe and is likely to result in shear cracking of the pipe at the five o'clock and seven o'clock locations. Figure 4-17 illustrates how the distribution of the bedding over increasing percentages of the outside diameter can increase the supporting strength of the culvert. Any time a pipe is installed on a flat-bottom foundation, it is essential that the bedding material be uniformly compacted under the haunches of the culvert.



Properly prepared bedding evenly distributes loads.
Improperly prepared bedding may result in stress concentrations.



Improperly prepared bedding.

Figure 4-16 Transverse or circumferential cracks

- **Backfilling** - The backfill is made up of two areas that may require different material and separate compaction criteria. The first area extends from the bedding to approximately 300 mm (12 inches) above the culvert. The second area includes the remaining fill.

The load-carrying capacity of an installed culvert depends largely on the initial backfilling around the culvert. Since proper compaction of backfill material is so important, material and density criteria is often included in the bedding requirements.

For trench installations, where space is limited, tamping by pneumatic or mechanical impact tampers is usually the most effective means of compaction. Impact tampers are most effective for clay soils while granular soils are consolidated best by vibration. Backfill material should be placed in layers not exceeding 150 mm (6 inches) deep, deposited alternately on opposite sides of the culvert.

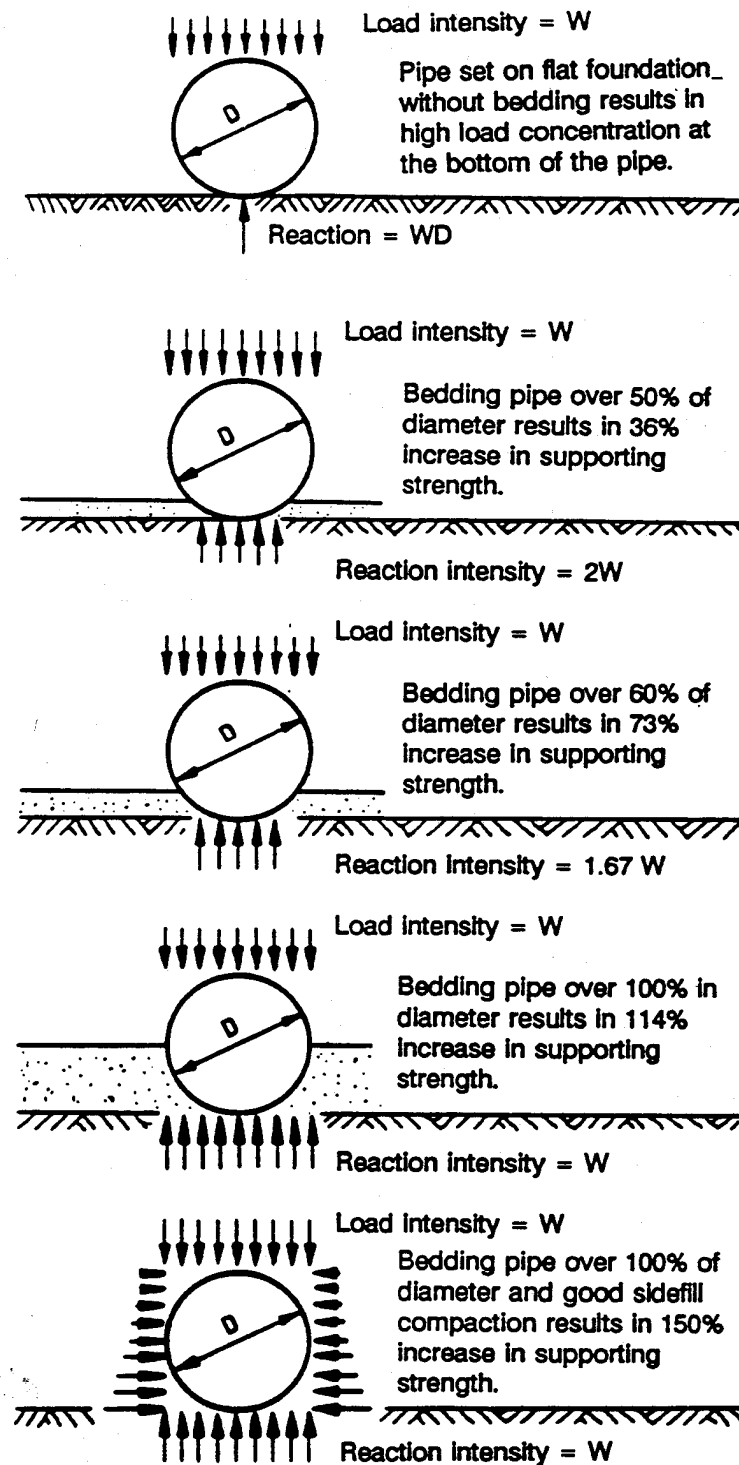


Figure 4-17 Correlation of bedding and supporting strength for rigid pipe

- **Embankments**

Culverts placed in an embankment are usually bedded in natural ground and are overlaid by a constructed embankment. The required supporting strength of a buried pipe is determined by the total load that is imposed upon the pipe. The magnitude of the load is influenced by the uniformity and stability of the support soil, as well as conditions around and over the pipe. However, the load-carrying capability of rigid culverts is essentially carried by the structural strength of the pipe itself since rigid pipe is stiffer than the surrounding soil. A well-compacted soil envelope is required to develop the lateral pressures necessary to maintain the shape of flexible culverts.

Embankment installations can be divided into three groups: positive projection, negative projection, and induced trench. The essential features of these types of installations are shown in Figure 4-18. Refer to ConnDOT Standard Sheets and Standard Specifications for trench and embankment pipe installation.

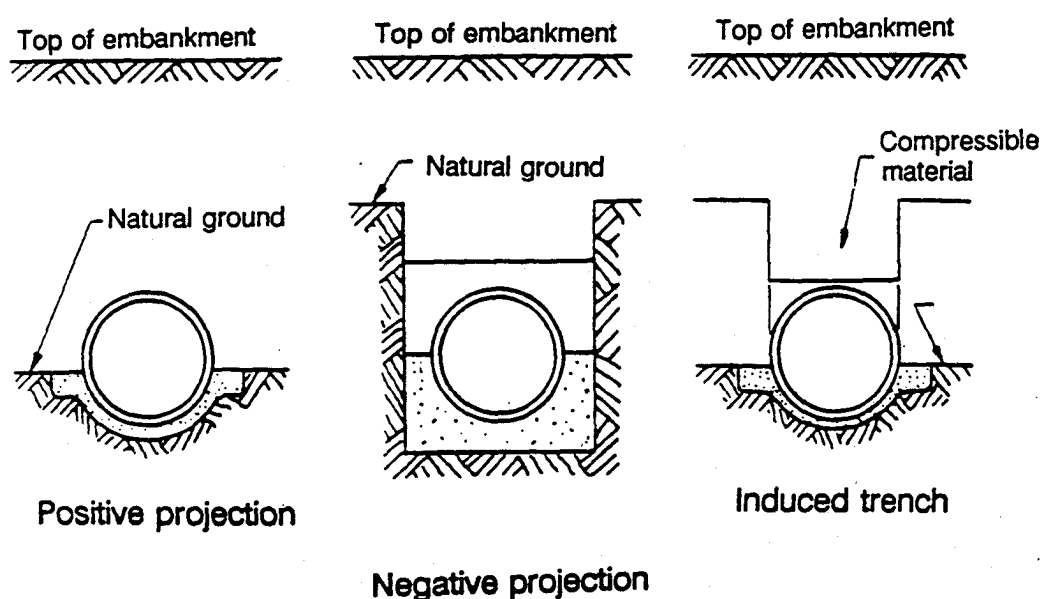


Figure 4-18 Essential features of various types of installation

Positive projection pipe is installed with the top of the pipe projecting above the surface of the natural ground, or compacted fill, and then covered with earth fill. Negative projection pipe is installed in relatively shallow trenches so that the top of the pipe is below the level of the natural ground or compacted fill. It is then covered with earth fill to the required depth. The induced trench pipe is usually installed as positive projection. However, when the fill has been placed to a depth of at least one pipe diameter over the proposed top of the pipe, a trench is excavated over the pipe and backfilled with a more compressible material.

- **Bored, Augured or Jacked**

The process of tunneling and jacking pipe culverts is used where deep installations are necessary or where conventional open excavation and backfill methods may not be practical.

The usual procedure in jacking pipe is to equip the leading edge with a cutter, or shoe, to protect the pipe. As succeeding lengths of pipe are added between the lead pipe and the jacks, and the pipe is jacked forward, soil is excavated and removed through the pipe. Material is trimmed so that the bore size slightly exceeds the outside diameter of the pipe and excavation does not precede the jacking operation more than is necessary. Such a procedure usually results in minimum disturbance to the natural soils adjacent to the pipe. A minimum 36" diameter is usually required for conventional jacking operations. A typical installation for jacking Pipe is shown in Figure 4-19.

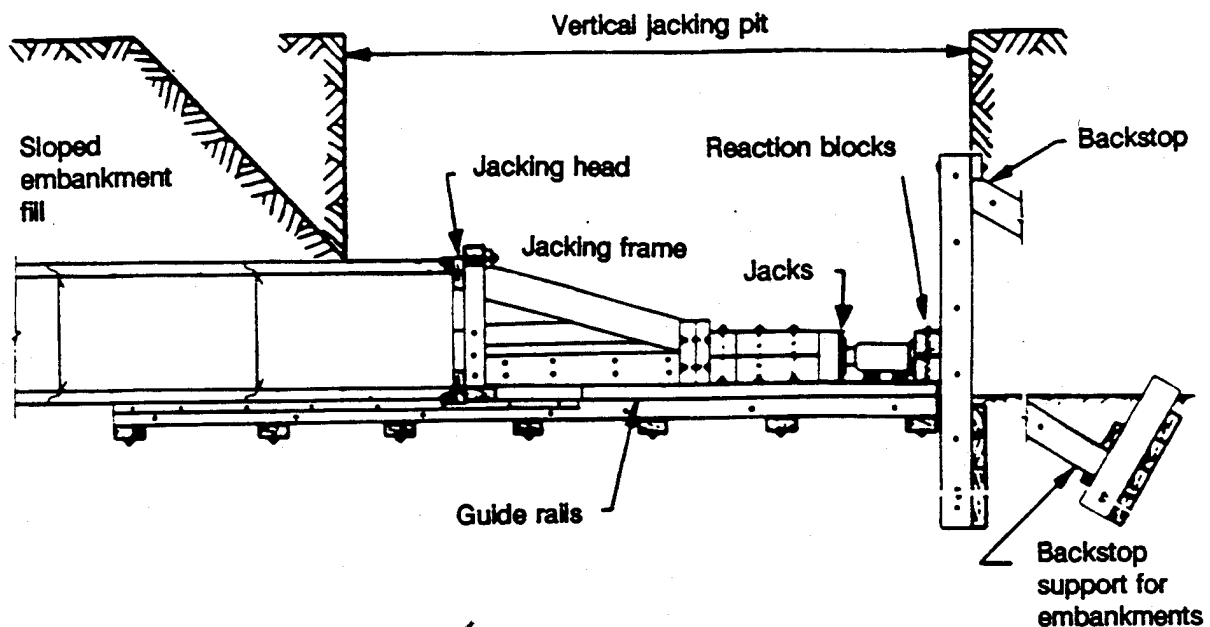


Figure 4-19 Typical jacking installation

A lubricant, such as a bentonite slurry, is sometimes pumped into the space between the tunnel bore and the outside of the pipe to reduce the frictional resistance. After the jacked pipe has reached its final position, grout is frequently pumped into the same space to, ensure continuous bearing with the surrounding soil.

Two types of loads are imposed upon concrete pipe installed by the jacking method: the axial load due to the jacking pressures applied during installation; and the earth loading due to the overburden, with some possible influence from live loadings, which generally become effective only after installation is completed.

There are several advantages to jacking pipe:

- Traffic is not interrupted on the overlying roadway.
- Depth of the installation is not a concern.
- Cutting and patching of the pavement can be avoided.
- Minimum disturbance to the natural soils is experienced.
- Loads on the pipe are less than loads in trenched installations.

The disadvantages are:

- Expensive equipment and skilled operators are required.
- There must be adequate room within the right-of-way to construct the jacking pit.

More detailed information can be found in the *Concrete Pipe Handbook* and *Design Data 13. Jacking Concrete Pipe*, both from the American Concrete Pipe Association.

- **Tunneling**

Tunneling techniques may be appropriate where a drainage structure must pass through an embankment or under a roadway or railway when open cutting is found to be too costly or too disruptive. Steel tunnel liner plates can be used to support an underground excavation. The design criteria set forth in AASHTO LRFD Bridge Design Specifications section 12.13 must be followed. Tunnel liner plates can also be used in the relining of existing structures when access is limited.

4.5.2 Culvert Installation Methods

The performance of culverts and their appurtenances is dependent on practices during installation. Items that require particular attention during design and construction of new culverts and repairs include:

- **Backfills and Fills**

Suitable backfill material and adequate compaction are of critical importance. A well-compacted soil envelope is needed to develop the lateral pressures required to maintain the shape of flexible culverts. Well-compacted backfill is also important to the performance of rigid culverts to prevent such things as settlement of the roadway and movement of water along the barrel. The design must specify material type and degree of compaction. Care should be taken that the backfill material does not contribute to corrosion of the culvert.

- **Trench Width**

Trench width can significantly affect the earth loads on rigid culverts. It is, therefore, important that trench widths be specified on the plans and that the specified width for rigid pipe not be exceeded without authorization from the design engineer. For flexible culverts a minimum trench width backfilled with premium backfill material is required to provide adequate side support. A narrower width of premium backfill for flexible pipe should not be provided without authorization from the design engineer.

- **Foundations and Bedding**

A foundation capable of providing uniform and stable support is important for both flexible and rigid culverts. Establishing a suitable foundation requires removal and replacement of any hard spots or soft spots. Bedding is needed to level out any irregularities in the foundation and to ensure uniform support. When using flexible culverts, bedding should be shaped to a sufficient width to permit compaction of the remainder of the backfill, and enough loose material should be placed on top of the bedding to fill the corrugations. When using rigid culverts, the bedding should conform to the bedding conditions specified in the plans and should be shaped to allow compaction and to provide clearance for the bell ends on bell and spigot-type rigid pipes. Adequate support is critical in rigid pipe installations, or shear stress may become a problem. The necessary details, when different from ConnDOT Standard Sheets, should be provided to the Soils and Foundations Section for approval.

- **Construction Loads**

Culverts are generally designed for the loads they must carry after construction is completed. Construction loads may exceed design loads. These heavy loads can cause damage if construction equipment crosses over the culvert installation before adequate fill has been placed or moves too close to the trench walls, creating unbalanced loadings. Additional protective fill or other measures may be needed for equipment crossing points.

- **Camber**

In high fills the center of the embankment may settle more than the areas under the embankment side slopes. In such cases it may be necessary to camber the foundation slightly, as shown in Figure 4-20. This should be accomplished by using a flat grade on the upstream half of the culvert and a steeper grade on the downstream half of the culvert. The initial grades should not cause water to pond or pocket. The method and dimensions for cambering should be coordinated with the Soils and Foundations Section.

- **Materials**

During construction, the materials delivered must be exactly as specified. Inadequate thickness, size, or quality of material can lead to maintenance problems or failure. During installation the materials must be handled properly to prevent defects and loss of intended shape, size, or quality.

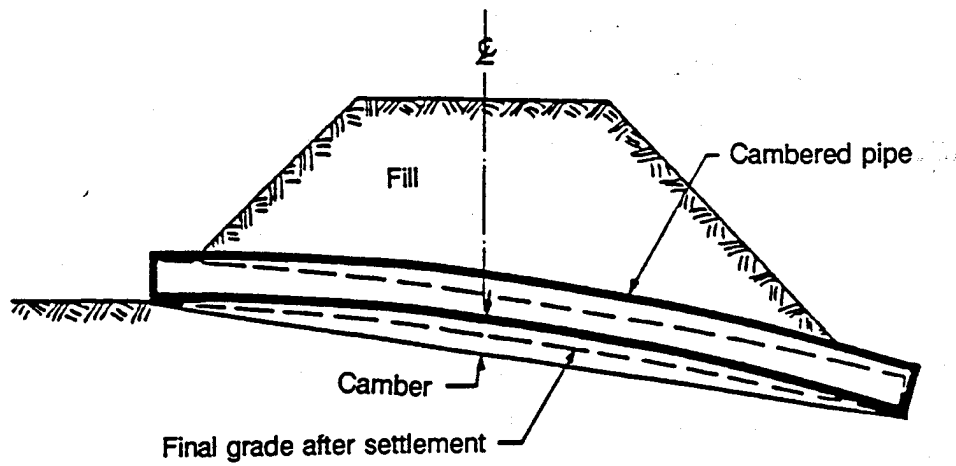


Figure 4-20 Camber allows for settlement of a culvert under a high fill

4.6 ConnDOT Culvert Material Requirements

4.6.1 Introduction

Conduits used to pass drainage under the highway or through the highway embankment shall be of the type which provides the necessary capacity at the lowest overall cost.

4.6.2 Reinforced Concrete Pipe (RCP)

Reinforced concrete pipe (RCP) should be considered for use in all locations except those subject to salt water and unstable foundations. This material has been used with great success on all highways since the inception of the Department. RCP, like all conduit materials, is limited to height of overfill and live loads, especially during construction. RCP shall meet the load-strength criteria as shown in the AASHTO LRFD Bridge Design Specifications, Section 12.10. Strength computations should be performed if the cover is over 15 feet or under 2 feet. It may be desirable to consider other than standard bedding in order to arrive at the most economical installation. If other bedding classes are used, details will have to be included in the plans. A unit weight of 2000 kg/m³ (125 lbs/cf) shall be used to determine the loading if the actual earth load is not known. When RCP is to be jacked through an embankment a special pipe is required and is usually Class V.

4.6.3 Corrugated Metal Pipe (CMP)

There are three types of metal pipe which are normally considered, they are aluminum, coated galvanized steel and aluminum coated steel. CMP shall meet the load strength criteria as shown in AASHTO LRFD Bridge design specifications, section 12.7. Strength computations should be performed for CMP at any cover.

- Aluminum pipes are predominantly used in salt water environments.
- Aluminum coated steel pipe has shown great success in durability compared to other types of coated galvanized steel pipe.
- Coated galvanized steel will normally be coated with a polymeric coating unless asphalt coating is specified. If asphalt coating is used it will be produced with a 40% paved invert in accordance with our specifications. Corrugated metal pipe will normally be specified for conduits installed essentially parallel to cut or fill slopes with cross slopes of 1:4 or greater. This pipe will normally be used where the foundation conditions are unstable. In such instances the culverts are installed with a camber, as specified by the Soils and Foundation unit or responsible consulting engineer, in anticipation of foundation settlement.
- A new type of corrugation (spiral rib) is available in both steel and aluminum. This corrugation provides a lower Mannings "n" value.

4.6.4 Corrugated Structural Plate Pipe (CSPP)

Structural plate pipes are manufactured from corrugated steel plates with 150mm X 50 mm (6 in X 2 in) corrugations or corrugated aluminum plates with 228mm X 64mm (9 X 2 ½ inches) corrugations which are bolted together at the site. These pipes are available starting at a diameter of 1500mm (60 in) and are able to tolerate higher heights of fill and also provide satisfactory results at locations of unstable foundations. Steel structural plates are to be coated with asphalt in accordance with the specifications.

4.6.5 Corrugated Polyethylene Pipe (CPP)

Corrugated polyethylene pipe is a product which has been used in the past as underdrain pipe in the smaller sizes. However, larger sizes of high density polyethylene pipe (HDPE) are available for storm drainage systems and culverts. This material can be considered for use in a salt water environment. Guidelines for the use of this product are found in Appendix C.

4.6.6 Polyvinyl Chloride (PVC)

Polyvinyl chloride plastic pipe (PVC) is a product for storm drainage systems and culverts which can also be considered for use in a salt water environment. The height of fill requirements for this product shall conform to the same requirements as for the use of HDPE.

4.6.7 Special Connections

Specially designed concrete junction boxes may be needed to connect larger pipes which will not fit into standard catch basins or manholes. If pipes are large enough, they can accommodate riser structures with "tees" for access or inlets. Elbows, tees and wyes may be considered for unusual situations but access must be available for debris removal.

4.6.8 Railroad Crossings

Conduits placed under railroad road-beds shall be designed to conform to the pertinent provisions of the current American Railway Engineering Association (AREA) Specifications and shall also conform to any applicable pipeline occupancy specifications for the type of conduit designed.

4.6.9 Airports

Materials to be considered for airport drainage shall comply with the current "Standards for Specifying Construction of Airports."

References

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

American Association of State Highway and Transportation Officials. LRFD Bridge Design Specifications

FHWA-RD-94-096 – Culvert Repair Manual-Volume 1 & 2, May 1995

James D. Arnoult, Culvert Inspection Manual, Federal Highway Administration, July 1996

“You Can Compare the Bottom Lines of Different Products”, Pipeline – Facts You Should Know About Corrugated Steel Pipe, National Corrugated Steel Pipe Association, Washington, D.C., Winter, 1988.

Thomas J. Wonsiewicz, Life Cycle Cost Analysis: Key Assumptions and Sensitivity of Results, National Corrugated Steel Pipe Association, Washington, D.C.

Life-Cycle Cost Analysis of Pavement, NCHRP Synthesis of Highway Practice 122, Transportation Research Board, National Research Council, Washington, DC 1985.

Carl E. Kurt and Garth W. McNichol, “Microcomputer-Based culvert Ranking System, Culvert and Pipelines: Design, Monitoring, Evaluation, and Repair, TRR 1315, Transportation Research Board, National Research Board, National Research Council, Washington, DC, 1991.

Bryan E. Little, Theodore A. Mize, and Robert J. Bailey, “Evaluation of Shear Plates and Grouted Sheat Key Joint Performance of a Three-Sided Precast Culvert,” Culverts and Pipelines: Design, Monitoring, Evaluation, and Repair, TRR 1315, Transportation Research Board, National Research Board, National Research Council, Washington, DC, 1991.

CON/SPAN Culvert Systems, Dayton, Ohio.

Introducing TECH/SPAN, the Optimal Precast Concrete Arch System, The Reinforced Company, McLean, Virginia.

“Culverts and Small Bridges Shotcreted Over Inflated Forms”, Concrete Construction, Concrete Construction Publications, Inc., Addison, Illinois, January 1987.

ConnDOT Bridge Inspection Manual, BITM 90 Section 13, 19 and FHWA Report #FHWA-IP-86-2, Culvert Inspection Manual

Appendix A Culvert Inspection Guideline

Routine Inspection:

The inspection of the culverts and appurtenant features should follow the rating guideline outlined in Section Nos. 6.7.8 and 6.7.9 of the ConnDOT Bridge Inspection Manual (ConnDOT BIM). These ratings should be assigned and documented for existing culverts to remain in use, with recommendations provided. The procedure outlined in the ConnDOT BIM shall be used as the format for the inspection. A report documenting the results of the culvert inspection shall be provided. Two hard copies of this document and one disk containing the pertinent NBIS data in electronic format shall be provided.

Approach Roadway and Embankment:

- Inspect the roadway for settlement due to culvert flattening, for evidence that the roadway has been patched or otherwise built-up, for cracks running parallel to the culvert centerline and for signs of erosion or failure of the embankment slope over the culvert. Look for signs of sink holes over the culvert that may be due to “piping” of the fill material through joints in the culvert.
- Evaluate the approach roadway for alignment, clearances, adequate shoulder profile, and safety features such as guide rails. These observations will not affect the condition code of the culvert, but should be included in the inspection report.
- Inspect the approach embankments for erosion protection such as slope paving or vegetation. Note the overall adequacy of this protection and note any vegetation near the culvert where root systems may damage the culvert.

Waterway:

- Note any observed changes in stream/culvert horizontal or vertical alignment that might affect the hydraulic adequacy of the culvert or cause scour.
- Note any significant changes in the ground cover or land use within the tributary watershed that might change the volume of water the culvert must pass.
- Note whether or not the stream bed fluctuates between high and low flow volumes. Document any changes in the amount, type or location of any channel erosion and stream bed aggradation/degradation.
- Document any observable high water mark relative to the culvert barrel.
- Measure and document the volumetric flow rate and velocity of the water flow upstream, downstream and through the culvert (if possible).

- Document any debris buildup or other obstructions in the waterway upstream, downstream, or within the culvert and any effects the obstruction may have upon the hydraulic adequacy. Note any maintenance requirements for post inspection follow up.

Culvert Barrel and End Treatments:

- Check the culvert headwall, wingwalls, cutoff walls, and slope protection for any deficiencies or deterioration, undermining, scour, piping, tipping, settlement or slope failure.
- Check the culvert barrel for deformations, settlement (either a smooth slope over the length of the culvert or an abrupt change at a joint), leaking or distressed joints and other deficiencies or signs of deterioration. Check for evidence that lateral earth pressure is causing bulging, flattening, peaking, sliding or rotation in the barrel or barrel components as outlined in the ConnDOT BIM, Appendix 6.5 and Appendix 6.7.
- Document any debris or sediment build-up within the barrel.
- Verify the minimum required measurements of the culvert as outlined in ConnDOT BIM, Appendix 6.5. Dimensions should be taken at the inlet, outlet, mid-length and at 8 m (26 ft.) intervals (maximum) as applicable, if access to the interior of the culvert is possible. Locations of sagging, bulging, flattening or peaking should also be measured.
- Check the barrel footings for scour and undermining (typically found in culverts that pass the natural stream bed through the culvert) and around the inlet and outlet of closed barrel sections.
- Probe all areas of scour, undermining and joint mortar loss and note the average depth at each location and maximum depth overall.
- Check to see if the culvert barrel has separated from the headwalls or cutoff walls.
- Check to make sure headwalls and wingwalls are adequately retaining the embankment material and protecting it against erosion.
- Where practical, the floors of metal pipe culverts should be sounded with a metal rod in an attempt to locate voids due to undermining.

Special Note About Corrugated Metal Culverts:

- Culverts with mitered ends (ends cut to match the embankment slope) and ends of skewed culverts (where the end is cut parallel to the centerline of the roadway) have reduced strength in these areas and have increased susceptibility to buckling. Therefore, even minor deformations are significant.
- Projections of culvert ends beyond the embankment lead to increased probability of erosion and piping around the culvert barrel.

Aprons, Energy Dissipaters and Flumes:

- Check the apron, which reduces erosion at the inlet and outlet (typically a concrete slab or riprap), for deterioration, missing stones, undermining of slab, movement of stones due to scour and deterioration of the joint between the apron slab and headwall.
- Check the energy dissipaters, which reduce outflow velocity and downstream erosion (typically riprap or concrete basin), for missing stones, movement of stones, scour, undermining, deterioration and overall effectiveness.
- Check the flumes (typically concrete, bituminous material or riprap) for deterioration, erosion, debris and signs of water bypass.

Documentation Required for a Routine Inspection:

- Document all observed deficiencies and deteriorations. Dimensions of deficient or damaged areas should include length, width, height, depth of material loss, orientation and location relative to a fixed, easily identifiable point.
- Estimate and record the depth of fill and pavement over the culvert.
- Draw a site plan that shows the orientation of all the culvert elements relative to the roadway. Sketch and dimension the boundaries of the waterway for the length necessary to show the alignment with the culvert (30 m (100 ft.) minimum). The sketch shall include both the inlet and outlet sides of the culvert, and dimension the typical width, depth, estimated velocity and direction of flow of the waterway, as well as all major changes in waterway cross section. All obstructions, deficiencies, changes in land use, changes in topography, sediment buildup, erosion, etc., shall be noted on the sketch (see Figure A-5-1 in ConnDOT BIM, Appendix 6.5 for an example sketch).
- Particular care shall be given to documenting any increase in quantity or size of deteriorations, erosion, sediment buildup, changes in waterway centerline, etc., that have changed since the last inspection. If the condition rating has changed since the last inspection, a photograph and detailed explanation of why the rating has changed shall be included in the report.
- Document any recent construction activity in the area that may affect the water flow through the culvert (i.e. changes in drainage area, land use, topography, etc.)
- Photographs taken shall include, but not be limited to the following:
 - View of the roadway above the culvert, taken from each roadway approach toward the culvert.
 - View looking at the culvert inlet and outlet taken from the waterway centerline or edge upstream and downstream of the culvert. The distance at which these photographs are taken shall be the distance sufficient to provide an overall view of the culvert and waterway at the culvert.

- View looking upstream from the culvert inlet and downstream from the culvert outlet.
- Typical views of all major components where deficiencies, erosion, sediment buildup, obstructions, etc., were noted. Include detailed photographs as necessary.

(Note: Pictures taken of the roadway from the approaches and detail photos of deficiencies, erosion, deterioration, etc., may be omitted if photos from previous inspections adequately reflect the current condition. However, all photos taken of the waterway approaches and culvert openings should be updated during each inspection so that lateral movement and changes in alignment over time can be documented.)

In-Depth Inspections:

- In addition to the requirements for routine inspections, all exposed surfaces of concrete culverts should be tapped with a hammer to locate delaminations.

Documentation of In-Depth Inspection:

- Same requirements as for routine inspection.

Report Review

- Cross reference the inspection report, inspection field notes and photographs to ensure they are mutually supportive of their documentation.
- The inspection reviewer should determine if a pattern of deterioration or progressive deterioration is taking place. Progression will be determined by comparing present and past inspection reports.
- Special attention shall be given to field note documentation indicating changes in the culvert shape, the presence of scour, piping or other problem that may warrant further investigation.
- Culverts are usually designed to use the vertical and horizontal earth pressures to maintain their stability and increase their live load capacity. Therefore, any change in the “As-Built” condition of the soil surrounding the culvert (addition or removal) should be noted and possibly analyzed for effects on the stability and capacity of the culvert.
- Changes in drainage area, land use and topography that increase the depth or velocity of flow within the waterway should be assessed in terms of how they may effect the long term waterway stability (i.e. lateral movement, aggradation, degradation, etc.) and hydraulic adequacy of the culvert.

Maintenance Concerns

- By the nature of their construction, culverts constrict the flow of water in the waterway. This constriction increases the potential for waterway blockage with debris and sediment, and increases the probability of scour around the culvert especially if high outlet velocities or turbulent flow conditions adjacent to the inlet are encountered. Therefore, any blockages noted should be removed as soon as possible (during the inspection if possible).

- Culverts typically have steep approach embankments, as well as large, abrupt drop-offs at the culvert crossings. Headwalls and wingwalls may be exposed to traffic presenting collision hazards to passable vehicles. Routine maintenance should include safety considerations (i.e. guide rails) as well as structural and hydraulic considerations.
- The addition of cut-off walls should be considered where “piping” or seepage around the outside of the culvert structure is found or suspected.
- Severely deteriorated culverts may be reviewed for repair by relining the culvert barrel.

Appendix B Drainage Facility Condition Surveys Guideline

Existing drainage facilities including pipes, catch basins, manholes, junction chambers, sedimentation/gross particle separators, cross culverts and ditches/swales which are scheduled to remain in use as part of the project should be inspected to verify their general condition early in the design process. A condition survey must be conducted for drainage systems that already have a service life of 10 years or more. Available previous condition reports should be reviewed prior to inspection to identify critical areas that need special attention.

Visual inspection should be performed to verify the existing drainage facilities' condition by referencing the AASHTO Highway Drainage Guidelines Volume XIV, CULVERT INSPECTION MANUAL (FHWA-IP-86-2), CULVERT REPAIR PRACTICES MANUAL Volume I (FHWA-RD-94-096), CULVERT REPAIR PRACTICES MANUAL, Volume II (FHWA-RD-95-089), and the Department's Bridge Inspection Manual. The inspection should be conducted in conformance with the Department's confined space program. Where siltation build-up hampers inspection, the drainage systems should be cleaned and visually inspected prior to recommending a video inspection. Normally video inspection of pipes should only be done if there is an indication that there may be evidence of distress such as roadway settlement, pavement patches, roadway build-up or embankment failure.

The condition survey should be documented in a report that includes the time and date of inspection; plans with sketches and measurements; itemized listing of the drainage facilities' location and condition; and photographic evidence of drainage facility/roadway section/embankment side slope erosion or failure. The report should also include expected service life and recommendations considering life cycle analysis; and specific recommendations on how the identified deficient drainage facilities can be remedied to avoid potential problems during construction. A copy of the report should also be provided in electronic format.

The designer should consult with the Drainage Engineer of the appropriate Departmental District for past problems, site conditions and proposed future improvements.

The following key elements where appropriate should be identified as part of the condition survey:

Structures

- Siltation, debris
- Crack, spall, settlement, etc.

Pipe / Culvert

- Corrosion, abrasion of pipes and bolt
- Water leakage
- Joint, seam defect and misalignment
- Cracking
- Visible waterline
- Deformation of pipe cross section

- Pipe material (concrete, steel, masonry stone, plastic, etc.)
- Siltation, debris

Inlet / Outlet

- Erosion
- Siltation, debris
- Condition of inverts (buried)
- Piping
- Condition of culvert ends / headwalls / wingwalls
- Undermining
- Scouring
- Outlet protection (type, limits and condition)
- Waterway condition

Roadway / Embankment Slope

- Settlement
- Cracking
- Patching
- Frequent overlays
- Erosion

Form 1: STORM SEWER SYSTEM – PIPES BETWEEN DRAINAGE STRUCTURES

Station, Offset: _____

Project No. _____

Type: _____

Route No. _____

Size: _____

Date _____

Length : _____

	Condition Rating *	Condition			
	(0-9)	Good	Fair	Poor	N/A
Siltation, Debris		☐	☐	☐	☐
Visible Waterline		☐	☐	☐	☐
Water Leakage		☐	☐	☐	☐
Alignment	☐				☐
Joints	☐				☐
Concrete	☐				☐
Shape	☐				☐
Horizontal Diameter	☐				☐
Seams	☐				☐
Corrosion and Rust	☐				☐

* - See Tables B-1 and B-2 for ratings. These tables may be used for various pipe materials (where appropriate).

Remarks/Findings:

Recommendations:

FORM 2: STORM SEWER SYSTEM – DRAINAGE STRUCTURES

Station, Offset: _____

Project No. _____

Type: _____

Route No. _____

Date _____

Condition

	Good	Fair	Poor	N/A
Cover	☐	☐	☐	☐
Grate	☐	☐	☐	☐
Top	☐	☐	☐	☐
Crack, Spall, Settlement	☐	☐	☐	☐
Siltation, Debris	☐	☐	☐	☐

Remarks/Findings:

Recommendations:

FORM 3: STORM SEWER SYSTEM – INLET/OUTLET FACILITIES TO/FROM CHANNEL, DITCHES, ETC...

Station, Offset: _____

Project No. _____

Type: _____

Route No. _____

Outlet Protection (type and limits): _____

Date _____

	Condition			
	Good	Fair	Poor	N/A
Erosion	☐	☐	☐	☐
Inverts	☐	☐	☐	☐
Piping	☐	☐	☐	☐
Siltation, Debris	☐	☐	☐	☐
Culvert Ends	☐	☐	☐	☐
Wingwalls	☐	☐	☐	☐
Headwalls	☐	☐	☐	☐
Undermining	☐	☐	☐	☐
Scour	☐	☐	☐	☐
Waterway Adequacy	☐	☐	☐	☐
Outlet Protection	☐	☐	☐	☐

Remarks/Findings:

Recommendations:

RATING GUIDELINES FOR ROUND CORRUGATED METAL PIPE BARRELS

Table B-1

RATING	CONDITION	RATING	CONDITION
9	New condition	4	<u>Shape</u> : marginal significant distortion throughout length of pipe, lower third may be kinked - <u>Horizontal diameter</u> : 10 percent to 15 percent greater than design <u>Seams or joints</u> : moderate cracking at bolt holes on one seam near top of pipe, deflection caused by loss of backfill through open joints <u>Metal</u> : - <u>Aluminum</u> : superficial corrosion, slight pitting - <u>Steel</u> : superficial rust, no pitting
8	<u>Shape</u> : good, smooth curvature in barrel - <u>Horizontal</u> : within 10 percent of design <u>Seams and joints</u> : tight, no openings <u>Metal</u> : - <u>Aluminum</u> : superficial corrosion, slight pitting - <u>Steel</u> : superficial rust, no pitting	3	<u>Shape</u> : poor with extreme deflection at isolated locations, flattening of crown, crown radius 20 to 30 feet - <u>Horizontal diameter</u> : in excess of 15 percent greater than design <u>Seams</u> : 3 in. long cracks at bolt holes on one seam <u>Metal</u> : - <u>Aluminum</u> : extensive corrosion, significant attack of core alloy - <u>Steel</u> : extensive heavy rust, deep pitting
7	<u>Shape</u> : generally good, top half of pipe smooth but minor flattening of bottom - <u>Horizontal diameter</u> : within 10 percent of design <u>Seams or joints</u> : minor cracking at a few bolt holes, minor joint or seam openings, potential for backfill infiltration <u>Metal</u> : - <u>Aluminum</u> : moderate corrosion, no attack of core alloy - <u>Steel</u> : moderate rust, slight pitting	2	<u>Shape</u> : critical, extreme distortion and deflection throughout pipe, flattening of crown, crown radius over 30 feet - <u>Horizontal diameter</u> : More than 20 percent greater than design <u>Seams</u> : Plate cracked from bolt to bolt on one seam <u>Metal</u> : - <u>Aluminum</u> : extensive perforations due to corrosion - <u>Steel</u> : extensive perforations due to rust
6	<u>Shape</u> : fair, top half has smooth curvature but bottom half has flattened significantly - <u>Horizontal diameter</u> : within 10 percent of design <u>Seams or joints</u> : minor cracking at bolt is prevalent in one seam in lower half of pipe. Evidence of backfill infiltration through seams or joints <u>Metal</u> : - <u>Aluminum</u> : significant corrosion, minor attack of core alloy - <u>Steel</u> : fairly heavy rust, moderate pitting	1	<u>Shape</u> : partially collapsed with crown in reverse curve <u>Seams</u> : failed <u>Road</u> : closed to traffic
5	<u>Shape</u> : generally fair, significant distortion at isolated locations in top half and extreme flattening of invert - <u>Horizontal diameter</u> : 10 percent to 15 percent greater than design <u>Seams or joints</u> : moderate cracking at bolt holes along one seam near bottom of pipe, deflection of pipe caused by backfill infiltration through seams or joints <u>Metal</u> : - <u>Aluminum</u> : significant corrosion, moderate attack of core alloy - <u>Steel</u> : scattered heavy rust, deep pitting	0	<u>Pipe</u> : totally failed <u>Road</u> : closed to traffic

RATING GUIDELINES FOR REINFORCED CONCRETE PIPE BARRELS

Table B-2

RATING	CONDITION	RATING	CONDITION
9	New condition	4	<u>Alignment</u> : marginal; significant settlement and misalignment of pipe; evidence of piping; end sections dislocated about to drop off <u>Joints</u> : differential movement and separation of joints, significant infiltration or exfiltration at joints <u>Concrete</u> : cracks open more than 0.12 in. with efflorescence and spalling at numerous locations; spalls have exposed rebars which are heavily corroded; extensive surface scaling on invert greater than 0.5 in.
8	<u>Alignment</u> : good, no settlement or misalignment <u>Joints</u> : tight, with no defects apparent <u>Concrete</u> : no cracking, spalling, or scaling present; surface in good condition	3	<u>Alignment</u> : poor with significant ponding of water due to sagging or misalignment of pipe; end section drop off has occurred <u>Joints</u> : significant openings; dislocated joints in several locations exposing fill material; infiltration or exfiltration causing misalignment of pipe and settlement or depressions in roadway <u>Concrete</u> : extensive cracking; spalling, and minor slabbing; invert scaling has exposed reinforcing steel
7	<u>Alignment</u> : generally good; minor misalignment at joints; no settlement <u>Joints</u> : minor openings, possible infiltration/exfiltration <u>Concrete</u> : minor hairline cracking at isolated locations; slight spalling or scaling present on invert	2	<u>Alignment</u> : critical; culvert not functioning due to alignment problems throughout <u>Concrete</u> : severe slabbing has occurred in culvert wall, invert concrete completely deteriorated in isolated locations
6	<u>Alignment</u> : fair, minor misalignment and settlement at isolated locations <u>Joints</u> : minor backfill infiltration due to slight opening at joints; minor cracking or spalling at joints allowing exfiltration <u>Concrete</u> : extensive hairline cracks, some with minor delaminations or spalling; invert scaling less than 0.25 in. deep or small spalls present	1	<u>Culvert</u> : partially collapsed <u>Road</u> : closed to traffic
5	<u>Alignment</u> : generally fair, minor misalignment or settlement throughout pipe; possible piping <u>Joints</u> : open and allowing backfill to infiltrate; significant cracking or joint spalling <u>Concrete</u> : cracking open greater than 0.12 in. with moderate delamination and moderate spalling exposing reinforcing steel at isolated locations; large areas of invert with surface scaling or spalls greater than 0.25 in. deep	0	<u>Culvert</u> : total failure of culvert and fill <u>Road</u> : closed to traffic

Appendix C High Density Polyethylene Pipe (HDPE) Guideline

HDPE is available in 300mm (12 in), 375mm (15 in), 450mm (18 in), 600mm (24 in), 750mm (30 in), 900mm (36 in), 1050mm (42 in), 1200mm (48 in) diameters.

Advantages of HDPE:

HDPE is lightweight; easy to handle; hydraulically efficient; easy to cut; unaffected by brackish water, chemicals, and corrosive elements found in soils; produced in 6m (20 ft.) lengths; and is somewhat flexible, thus permitting smooth curvilinear installation.

Design Considerations:

Only HDPE smooth interior (Type S or Type D) shall be utilized. This type is hydraulically efficient and reportedly easier to install, since it is more rigid than the corrugated interior version.

Designers must recognize that a buried plastic pipe is a composite structure made up of a plastic ring and the soil envelope, and that both materials play a vital part in the structural integrity of the plastic pipe. In contrast, a buried reinforced concrete element is less influenced by the soil envelope.

- The successful performance of HDPE depends upon proper bedding, backfill and care in installation.
- The initial cost should not be the only basis for culvert material selection. The most economical culvert is one which has the lowest total annual cost over the design life of the structure.
- HDPE can easily be damaged during excavation activities for items such as underground utilities. Although field repairs can be accomplished using repair couplers available from the manufacturer, the designer needs to assess the overall risk of damage associated with other excavation activities before HDPE is selected.
- Minimum cover shall be established by the engineer based on an evaluation of specific site conditions. In the absence of pipe strength calculations, the minimum cover above the pipe shall be at least 0.9m (3 ft) or one pipe diameter (whichever is larger). The minimum cover should be maintained before allowing vehicles or heavy construction equipment to traverse the pipe trench.
- Maximum cover should be limited to 2.5m (8 ft) (measured to top of pipe).
- Pipe strength calculations in accordance with AASHTO Standard Specification for Highways and Bridges, Section 18 (ASD) or AASHTO LRFD Bridge Design Specifications Section 12 are required for the following installations:
 1. Installations subject to vehicle loads
 2. Fills greater than 2.5m (8 ft) (measured to top of pipe)
 3. Fills less than 0.9m (3 ft) (measured to top of pipe)
 4. Adverse soil conditions
 5. High water table
- Because HDPE pipe is relatively lightweight, buoyancy forces, especially at the culvert inlet, may be a concern. Anchorage in the form of a headwall, slope paving or other stabilization methods may be necessary.

- HDPE is susceptible to fire damage, especially at outfall locations. Therefore, fire damage may result due to an adjacent grass fire. Overall, the likelihood of damage is considered low.
- Since proper bedding and backfill are vital to a successful installation, diligent construction and inspection is needed.
- Installation shall conform to Section 6.5.1 of the Department's Standard Specifications.
- The design or construction engineer may elect to specify a Type II bedding installation when native backfill material is judged to be inadequate for use.
- Vibratory compaction of backfill can cause HDPE to shift and therefore appropriate measures and monitoring during installation are necessary. Normally, visual inspections are adequate to confirm the installation is acceptable. However, a mandrel test may be requested by the engineer when it is necessary to confirm the acceptability of an installation.
- When specifying HDPE pipe, designers must consider loads from construction vehicles **as well as those experienced during construction staging operations.**

Applications:

- The use of HDPE can be considered in the following locations:
 1. Temporary installations
 2. Areas remote from the traveled portions of pavements
 3. Medians
 4. Parking lots, (where vehicular traffic is light to moderate and truck traffic is light)
 5. Longitudinal installations on local and collector routes within the shoulder areas
 6. Slope drains
 7. Areas with little or no underground utility involvement
 8. Where parallel underground utility work is not likely in the foreseeable future
(Where parallel underground utility work is likely, the designer must be aware that it is possible to disrupt the supporting soil envelope adjacent to the pipe, which in turn can compromise the structural integrity of the pipe/soil system)

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5.1 Overview

5.1.1 Introduction

The purpose of this chapter is to outline the types of data that are normally required for drainage analysis and design, possible sources, and other aspects of data collection.

It is necessary to identify the types of data that will be required prior to conducting an engineering analysis. The effort necessary for data collection and compilation should be tailored to the importance of the project. Not all of the data discussed in this chapter will be needed for every project. A well planned data collection program leads to a more orderly and effective analysis, and a design that is commensurate with project scope, project cost, the complexity of site hydraulics, and regulatory requirements.

Data collection for a specific project must be tailored to site conditions, scope of the engineering analysis, unique project requirements, and regulatory requirements.

The designer must compile the data that are specific to the subject site. Following are the major types of data that may be required:

- watershed characteristics including specific soil types
- stream reach data (especially in the vicinity of the facility)
- other physical data in the general vicinity of the facility such as utilities, easements, headwater elevation control, etc.
- hydrologic and meteorologic data (stream flow and rainfall data related to maximum or historical peak as well as low flow discharges and hydrographs applicable to the site)
- existing and proposed land use data in the subject drainage area and in the general vicinity of the facility
- anticipated changes in land use and/or watershed characteristics
- floodplain and environmental regulations
- surficial geology

Watershed, stream reach and site characteristic data can be obtained from a field reconnaissance of the site. Examination of available maps and aerial photographs is also an excellent means of defining physical characteristics of the watershed.

5.1.2 Drainage Reconnaissance

Make a complete field drainage reconnaissance of the site and its contributing watershed as part of the hydraulic analysis and design. Requirements for small drainage facilities are less extensive than those for major facilities such as bridges. However, the purpose of each evaluation is to provide an accurate picture of the conditions within the zone of hydraulic influence of the facility.

The following are the data that can be obtained or verified:

- contributing drainage area characteristics including land use
- stream reach data - cross sections and thalweg profile
- existing structures

- location and survey for development, existing structures, etc., that may affect the determination of allowable flood levels, capacity of proposed drainage facilities, or acceptable outlet velocities
- drift/debris characteristics
- general ecological information about the drainage area and adjacent lands
- highwater elevations including the date of occurrence

Much of the data can only be obtained from an on-site inspection. It is often much easier to interpret published sources of data after an on-site inspection. Only after a thorough study of the area and a complete collection of all required information should the designer proceed with the design of the hydraulic facility. All pertinent data and facts gathered through the evaluation shall be documented.

5.1.3 Sources of Data

Much of the data and information necessary for the design of highway drainage facilities may be obtained from some combination of the sources listed in Appendix A of this chapter. While the various government agencies and offices referred to in Appendix A are available to answer questions or direct the designer to other sources of information, it is the responsibility of the designer to independently research and collect the available data.

5.2 Watershed Characteristics

5.2.1 Area

The size of the contributing drainage area expressed in hectares (acres) or square kilometers (square miles) is determined from some or all of the following:

- Direct field surveys with conventional surveying instruments
- Use of USGS topographic maps together with field checks to determine any changes in the contributing drainage area
- Topographic maps or other survey data may be available in the Department's Central Surveys office
- Aerial maps or aerial photographs of the State are available from the ConnDEP and the Department's Bureau of Policy and Planning
- Mapping prepared by the municipality or regional agency
- Atlas of the Public Water Supply Sources and Drainage Basins of Connecticut (DEP Bulletin No. 4), June 1982

In determining the size of the contributing watershed, any areas outside the physical boundaries of the drainage area that have runoff diverted into the drainage area being analyzed shall be included in the total contributing drainage area. In addition, it must be determined if flood waters are naturally diverted out of the basin before reaching the site. Diversion to or from the watershed in question should be specifically documented in the hydrology section of the hydraulic or drainage design report.

5.2.2 Slopes

Determine the slope of the stream, the average slope of the watershed (basin slope), and land slope. Hydrologic and hydraulic procedures in other chapters of this manual are dependent on these slopes.

5.2.3 Land Use

- Define and document the present and expected future land use (where practicable), particularly the location, degree of anticipated urbanization and data source
- Information on existing use and future urbanization trends may be obtained from aerial photographs
- Zoning maps and master plans, USGS maps and municipal planning agencies provide land use data
- Specific information about particular tracts of land can often be obtained from owners, developers, realtors and local residents
- Existing land use data for small watersheds can best be determined or verified from a field investigation

Field surveys can be used to update information on maps and aerial photographs, especially in basins that have experienced changes in development since the maps or photos were prepared.

5.2.4 Streams, Rivers, Ponds, Lakes and Wetlands

At all streams, rivers, ponds, lakes and wetlands that will affect, or may be affected by, the proposed structure or construction, the following data should be secured. This is important in determining the expected hydrology and may be needed for regulatory permits.

- Outline of the boundary (perimeter) of the water body for the ordinary highwater
- Elevation of normal as well as highwater for various frequencies. (See Chapter 6, Hydrology.)
- Detailed description of any natural spillway, manmade spillway, or outlet works, including dimensions, elevations and operational characteristics
- Detailed description of any emergency spillway works including dimensions and elevations
- Description of adjustable gates, soil and water control devices
- Profile along the top of any dam and a typical cross section of the dam
- The use of the water resource (fish, recreation, power, irrigation, municipal or industrial water supply, etc.)
- The existing conditions of the stream, river, pond, lake or wetlands with respect to turbidity and silt.
- Riparian ownership(s) as well as any water rights

5.2.5 Environmental Considerations

The need for environmental data stems from the need to investigate and mitigate possible impacts due to specific design configurations. Information necessary to define the need for mitigation measures for fish passage should be coordinated with Conn D.E.P. Fisheries Unit and/or U.S. Fish and Wildlife.

5.2.6 Geological Considerations

All watersheds in the State of Connecticut have had the soils classified by the U.S. Department of Agriculture Natural Resources Conservation Service (NRCS), formerly Soil Conservation Service (SCS). The soils surveys are prepared for the counties within the State and can be obtained from the abutting states. These surveys indicate the types of soil and the areas where they are found. From these plans, soil groups can be developed which can then be used for:

- Determining runoff coefficients used with the Rational method. (See Chapter 6, Hydrology.)
- Determining curve numbers for use in computer models such as HEC-1 and NRCS methods. (See Chapter 6, Hydrology.)

The regional regression equations (see Chapter 6, Hydrology) require that the percentage of coarse-grained stratified drift within the watershed be determined. This information is available from the Hydraulics and Drainage Section. Include copies of overlays for U.S.G.S. 1:24000 scale quadrangle maps. The USGS is also a primary source of data related to surficial geology.

5.3 Site Characteristics

5.3.1 Introduction

A complete understanding of the physical nature of the natural channel or stream reach is of prime importance to a good hydraulic design. Any work being performed, proposed, or completed, that changes the hydraulic efficiency of a stream reach must be studied to determine its effect on the stream flow. The designer should be aware of plans for channel modifications, and any other changes which might effect the facility design. The stream may be classified as:

- rural or urban
- improved or unimproved
- narrow or wide
- rapid or sluggish flow
- stable, transitional, or unstable
- sinuous, straight, braided, alluvial, or incised

Geomorphological data are important in the analysis of channel stability and scour. Types of needed data are:

- sediment transport and related data
- stability of form over time (braided, meandering, etc.)
- scour history/evidence of scour
- bed and bank material identification

5.3.2 Roughness Coefficients

Roughness coefficients, ordinarily in the form of Manning's "n" values, shall be estimated for the entire flood limits of the stream. Report No. FHWA-TS-84-204, "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains," should be consulted for determining the appropriate "n" values. It is advisable to take still pictures of the flood plain to document the "n" values selected.

5.3.3 Stream Profile

Obtain stream bed profile data to determine the average slope and to encompass any proposed construction. These data should extend sufficiently upstream and downstream. Limits will be established by the hydraulic engineer. Identification of "headcuts" which could migrate to the site under consideration is particularly important.

5.3.4 Stream Cross-Sections

Stream cross-section data shall be obtained that will represent the typical conditions at the structure site as well as other locations where stage-discharge and related calculations will be necessary. The location and extent of cross-sections will be determined by the hydraulic engineer. See Chapter 9, Bridges, for additional guidance in determining the location of stream cross sections.

5.3.5 Existing Structures

Secure the location, size, description, condition, observed flood stages, and channel section relative to existing structures on the stream reach and near the site in order to determine their capacity and effect on the stream flow. Investigate structures, downstream or upstream within the area of influence, which may cause backwater or retard stream flow. Note the manner in which existing structures have been functioning with regard to such things as scour, overtopping, debris and ice passage, fish passage, etc. For bridges, include span lengths, type of piers, and substructure orientation which usually can be obtained from existing structure plans. Other necessary culvert data includes size, inlet and outlet geometry, slope, end treatment, culvert material, and flow line profile. Photographs and highwater profiles or marks of flood events at the structure and past flood scour data can be valuable in assessing the hydraulic performance of the existing facility.

5.3.6 Acceptable Flood Levels

Improvements, property use, and other developments adjacent to the proposed site both upstream and downstream may determine acceptable flood levels. Note incipient inundation elevations of these improvements or fixtures. In the absence of upstream development, acceptable flood levels may be based on freeboard requirements to the highway itself. In these instances, the presence of downstream development becomes particularly important as it relates to potential overflow points along the road grade.

5.3.7 Flood History

The history of past floods is a valuable tool for sizing structures. Historical flood data is also utilized in the development of flood hazard evaluation studies. Information may be obtained from newspaper accounts, local residents, flood marks or other positive evidence of the height of historical floods. Evaluate changes in channel and watershed conditions since the occurrence of the flood in relating historical floods to present conditions. (See Chapter 6 – Appendix E – Major Flood Events in Connecticut since 1927).

Recorded flood data may be available from agencies such as U.S. Army Corps of Engineers, U.S.G.S., ConnDEP, NRCS, and ConnDOT.

5.3.8 Debris and Ice

Investigate the potential quantity and size of debris, ice, etc. available for transport by a stream during flood events. The effect of backwater from debris and ice jams on recorded flood heights should be considered when using stream flow records.

5.3.9 Scour Potential

Scour potential is an important consideration relative to the stability of the structure over time. Scour potential is a function of a combination of the stability of the natural materials at the facility site, tractive shear force exerted by the stream, sediment transport characteristics of the stream and relative floodplain/channel conveyance characteristics. Data on natural materials may be obtained from the Department or by tests at the site.

Collect enough bed and bank material samples for classifying channel type, stability and gradations. A geotechnical study will be required to determine the substrata if scour studies are needed. Also, these data are needed to determine the presence of bed forms so that a reliable Manning's "n" and bed form scour can be estimated.

5.3.10 Controls Affecting Design

Many controls will affect the final design of drainage structures. These include allowable headwater level, allowable flood level, allowable velocities, and resulting scour and other site specific considerations. Data and information related to such controls can be obtained from Federal, State and local regulatory agencies, and site investigations to determine what natural or man-made controls should be considered in the design. In addition, there may be downstream and upstream controls which should be documented.

5.3.11 Downstream Control

Any ponds or reservoirs, along with their spillway elevations and design levels of operation, shall be noted. Also, channel constrictions or points of abrupt grade changes should be identified. Their effect on backwater and/or streambed aggradation may directly influence the proposed structure. Also, any downstream confluence of two or more streams shall be studied to determine the effects of backwater or streambed change resulting from that confluence.

5.3.12 Upstream Control

Upstream control of runoff in the watershed shall be noted. Conservation and/or flood control reservoirs in the watershed may effectively reduce peak discharges at the site and may also retain some of the watershed runoff. Obtain capacities and operation designs for these features. The NRCS, Corps of Engineers, Bureau of Reclamation, consulting engineers and other reservoir sponsors often have complete reports concerning the operation and design of proposed or existing conservation and/or flood control reservoirs.

The redirection of flood waters can significantly affect the hydraulic performance of a site. Some actions that redirect flows are debris jams, mud flows and highways or railroads.

5.3.13 Survey Requirements

Complete and accurate survey information is necessary to develop a design that will best serve the requirements of a site. The amount of survey data gathered shall be commensurate with the importance of the proposed structure and the expected flood hazard.

At many sites photogrammetry is an excellent method of securing the topographical components of drainage surveys. Planimetric and topographic data covering a wide area are available in many locations. A supplemental field survey may be required to provide data in areas obscured on the aerial photos (underwater, under trees, etc.).

Data collection shall be as complete as possible during the initial survey in order to avoid repeat visits. Thus, data needs must be identified and tailored to satisfy the requirements of the specific location and size of the project early in the project design phase. Coordination between the project engineer and surveyor before the initial field work is begun will help insure the acquisition of sufficient, but not excessive survey data.

5.3.14 Field Review

Field reviews shall be made in order for the designer to become familiar with the site. The most complete survey data cannot adequately depict all site conditions or substitute for personal inspection by someone experienced in drainage design. Visit the project site before any detailed drainage/hydraulic design is undertaken. Factors that most often need to be confirmed by field inspection are:

- identification of apparent environmental concerns
- selection of roughness coefficients
- evaluation of apparent flow direction and diversions
- flow concentration
- observation of land use and related flood hazards
- geomorphic relationships
- highwater marks or profiles and related frequencies
- adequacy or suitability of existing and proposed drainage outlets

Before making the field visit, determine what kind of equipment is needed and, most importantly, the critical items at the site. Take photographs. As a minimum, looking upstream and downstream from the site as well as along the contemplated highway centerline in both directions. Details of the stream bed and banks should also be photographed along with structures in the vicinity both upstream and downstream. Take close up photographs complete with a scale or grid to facilitate estimates of the stream bed gradation.

5.4 Data Evaluation

5.4.1 Objective

Once the needed data have been collected, the next step is to compile it into a usable format. The designer must ascertain whether the data contains inconsistencies or other unexplained anomalies which might lead to erroneous calculations or results. Analyze the data to draw all of the various pieces of collected information together, and to fit them into a comprehensive and accurate representation of the hydrologic and hydraulic characteristics of a particular site.

5.4.2 Evaluation

Experience, knowledge, and judgment are important parts of data evaluation. It is in this phase that reliable data is separated from that which is less reliable and historical data is combined with that obtained from measurements. The designer shall evaluate data for consistency and identify any changes from established patterns. The designer must review previous studies, old plans, etc., for types and sources of data, how the data was used and any indications of accuracy and reliability. Historical data shall be reviewed to determine whether significant changes have occurred in the watershed and whether the data can be used. Data acquired from the publications of established sources such as the USGS can usually be considered as valid and accurate.

Basic data, such as streamflow data derived from non-published sources, shall be evaluated and summarized before use. Maps, aerial photographs, and land use studies shall be compared with one another and with the results of the field survey. Any inconsistencies shall be resolved. General references shall be consulted to help define the hydrologic character of the site or region under study and to aid in the analysis and evaluation of data.

5.4.3 Sensitivity

Often sensitivity studies are useful in evaluating data and the importance of specific data items to the final design. They consist of conducting a design with a range of values for specific data items. The effect on the final design can then be established. This is useful in determining what specific data items have major effects on the final design and the importance of possible data errors. Time and effort can then be spent on the more sensitive data items making sure these data are as accurate as possible. This does not mean that inaccurate data are accepted for less sensitive data items, but it allows prioritization of the data collection process given a limited budget and time allocation.

The results of this type of data evaluation will provide a reliable description of the site with the limited time and resources committed to this effort. The effort of data collection and evaluation shall be commensurate with the importance and extent of the project and/or facility.

References

FHWA. Hydraulic Design Series HDS-2, Hydrology. 1996.

Appendix A - Sources Of Data

ConnDOT

Hydraulics and Drainage Section

- Hydraulic Reports
- Stream Channel Encroachment Line
Maps
Reports
- Federal Emergency Management Agency
F.I.S. Reports
F.I.S. Maps
- NRCS County Soil Maps
- Metropolitan District Commission Contour Maps 1:240
- U.S.G.S. Quadrangle Maps 1:24000
- Scour Evaluation Reports (Level II Studies)
- Coarse Grained Stratified Draft Maps
- Recent Project Information
- Comparative Scour Reports
- U.S.G.S. Bridge and Channel Assessment Reports
- Database for all bridges over water

Bridge Safety Unit

- Bridge Inspection Reports
- Bridge repair and maintenance information

Soils & Foundation Unit

- Boring Reports

Engineering Records & Reproductions

- Project Plans & Reports

Environmental Planning

- Aerial Photographs

Central Surveys

- Photogrammetric Mapping

ConnDEP

- Federal Emergency Management Agency
F.I.S. Reports
F.I.S. Maps
- Stream Channel Encroachment Line
Maps
Reports
- Aerial Photographs

- U.S.G.S. Quadrangle Maps 1:24000
- Information on dams regulated by the DEP

University of Connecticut

- Map and Geographic Information Center (MAGIC)
On-line library of digital geospatial data
<http://magic.lib.uconn.edu/>

USDA Natural Resources Conservation Services

- County Soil Reports
- Hydrologic Reports
- Hydraulic Reports
- Watershed Flood Control Reports

U.S. Geological Survey

- Gage Station Data
- Hydrologic Reports
- Flood Analysis

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6.1 Hydrologic Design Practice

6.1.1 Introduction

Following is a summary of practices which should be followed for hydrologic analysis. For a more detailed discussion refer to the publication, Highway Drainage Guidelines, published by the American Association of State Highway and Transportation Officials. Some of the recommended methods have been developed in the English system. When the agency responsible provides a metric conversion, this manual will be updated.

6.1.2 Surveys

Since hydrologic considerations can influence the selection of a highway corridor and the alternate routes within the corridor, studies and investigations, including consideration of the environmental and ecological impact of the project, should be undertaken. 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.

6.1.3 Flood Hazards

When not clearly identified on NFIP maps and FIS studies, a hydrologic analysis is a prerequisite to identifying flood hazard areas and determining those locations at which construction and maintenance will be unusually expensive or hazardous.

6.1.4 Coordination

Since many levels of government plan, design and construct highways and water resource projects which might have an affect 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 analysis.

6.1.5 Documentation

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

The following items shall be included in the documentation file (see Chapter 1, Section 1.6). The intent is not to limit data to only those items listed, but rather to establish a minimum requirement consistent with the design procedures as outlined in this chapter. If circumstances are such that the hydrology is prepared other than the normal procedures or is governed by factors other than hydrologic factors, a narrative summary detailing the design basis shall appear with the other data.

The following items used in the design or analysis shall 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)
- Appendix D of this chapter provides a format for the Hydrologic Design Report

6.1.6 Evaluation Of Runoff Factors

For all hydrologic analysis, 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, natural and artificial controls, channel modification, aggradation - degradation and possibilities for ice and debris to be a significant problem
- flood plain characteristics
- meteorological characteristics

6.1.7 Flood History

All hydrologic analyses should consider the flood history of the area and the effect 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. Appendix E has mapping showing major floods in Connecticut since 1927. USGS water resources data should also be referenced for floods of record and stream flow records.

6.1.8 Hydrologic Method

Many hydrologic methods are available. The methods to be used and the circumstances for their use are listed below. If possible the results should be compared to a gage station which has a watershed with similar characteristics.

6.1.9 Design Discharge

The design discharge is based on an independent hydrologic analysis as outlined in the approved methods below. This discharge may be different from the FEMA or SCEL discharge. However, in all cases where these discharges have been established, the hydraulic analysis shall also include these discharges.

6.1.10 Approved Methods

The following hydrologic methods are acceptable for use with conditions as noted.

- Rational Method - This method shall be used only for drainage areas less than 81 ha (200 acres).
- Stream Gage Data - The U.S. Geological Survey maintains a network of stream gaging stations throughout Connecticut. From long term periods of observation and through statistical analysis of the resultant data, peak flow rates for various return frequencies can be developed. This data shall be used wherever possible in the design of hydraulic facilities.
- U.S.G.S. Regression Equations (L.A. Weiss) shall be used for all routine designs at sites greater than 2.59 km² (1 mi²) in tributary area unless stream gage data is available or unique watershed characteristics, such as significant detention storage dictate otherwise.
- Computer Models for Hydrograph Generation - Many watersheds exhibit characteristics which require the use of a computer model to estimate design discharge quantities from run-off hydrographs. Additionally, hydrograph generation models are useful in the analysis of "before and after" conditions as related to proposed activities within a given basin. Where use of a computer model is indicated, the methods described in Section 6.14 shall be used.
- FEMA Flood Insurance Study Discharge Rates - Discharges specified in the appropriate flood insurance study shall be used for ANALYSIS where the regulatory floodway is adopted or where FEMA has established discharge for a watercourse. The DESIGN discharge may vary from the flood insurance study quantity if the results of an independent analysis so indicate.
- SCEL – Stream Channel Encroachment Discharge Rates – Discharges specified in the appropriate stream channel encroachment study shall be used for analysis where SCEL are established. The design, FEMA and SCEL discharges may vary, as they are all developed independently and use different methods.
- Tidal Hydrology – The maximum discharge associated with astronomical tides or storm surges at a tidal crossing can be determined using either Neill's procedure, the orifice equation or Chang's procedure as outlined in HEC-18. Alternately, the discharge can be determined using one of several unsteady flow models as discussed in Chapter 9, Section 9.4.5. The level of effort and accuracy of these methods varies greatly. The selected method will depend on site conditions and should be commensurate with the scope and cost of the project. Coordination with the Hydraulics and Drainage Section is recommended. See also Section 6.1.14.

6.1.11 Design Frequency

Design frequencies are selected commensurate with the facilities cost, amount of traffic, potential flood hazard to property, expected level of service, and budgetary constraints as well as the magnitude and risk associated with damages from larger flood events. (See Appendix A)

6.1.12 Economics

Hydrologic analysis should include the determination of several design flood frequencies for use in the hydraulic design. These frequencies are used to size different drainage facilities so as to allow for an optimum design, which considers both risk of damage and construction cost.

Consideration should be given to what frequency flood was used to design other structures along a highway corridor.

6.1.13 Review Frequency

All proposed culverts and bridges conveying watercourses designed using the designated return frequency shall be checked using a flood of greater magnitude.

6.1.14 Tidal Conditions

Unlike inland rivers where the design discharge is fixed by runoff and is virtually unaffected by the waterway provided, the size of the waterway opening of a tidal structure can modify the tidal regime and the associated tidal discharges.

Extreme events associated with riverine floods and tidal storm surges should be used to determine the hydraulic adequacy of a bridge. These events would have a return period based on the structure classifications as defined in the culvert and bridge chapters, Chapters 8 and 9, Sections 8.3 and 9.3. Difficulty arises in determining whether the storm surge, flood or the combination of the storm surge and flood should be considered controlling.

When inland flood discharges are small in relationship to the magnitude of the storm surge and are the result of the same event, then the flood discharge can be added to the discharge associated with the design tidal flow, or the volume of the runoff hydrograph can be added to the volume of the tidal prism. If the inland flood and the storm surge may result from different storm events, then a joint probability approach may be warranted to determine the magnitude of the design discharge.

In some cases there may be a lag time between the storm surge discharge and the stream flow discharge at the highway crossing. For this case, stream flow-routing methods such as the NRCS TR-20, USACOE HEC-1 or HEC-HMS model can be used to estimate the timing of the flood hydrograph derived from runoff of the watershed(s) draining into the sound or estuary.

The selection of the method to use to combine flood and tidal surge flows is a matter of judgement and must consider the characteristics of the site and the storm events.

6.2 Overview

6.2.1 Introduction

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.

6.2.2 Definition

Hydrology is generally defined as a science dealing with the properties, distribution and circulation of water on and under the earth and in the atmosphere. For the purpose of this manual, hydrology will deal with estimating flood magnitudes as the result of precipitation. In the design of highway drainage structures, floods are usually considered in terms of peak runoff or discharge in cubic meters per second, m^3/s (cubic feet per second, cfs) and hydrographs as discharge per time. For structures which are designed to control volume of runoff, like detention storage facilities, or where flood routing through culverts is used, then the entire discharge hydrograph will be of interest.

6.2.3 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 such things as:

- rainfall amount and storm distribution
- drainage area size, shape and orientation
- ground cover and soil type
- slopes of terrain and stream(s)
- antecedent moisture condition
- basin storage potential (overbank, ponds, wetlands, reservoirs, channel, etc.)
- watershed development potential
- type of precipitation (rain, snow, hail, or combinations thereof)

6.2.4 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 available and applicable to a particular analysis.

Suggested data sources for material related to hydrologic studies or analyses are as follows:

1. ConnDOT, Hydraulics and Drainage Section (FEMA studies, SCEL reports, design reports, scour evaluation reports)
2. ConnDEP, Inland Water Resources Division (FEMA studies, SCEL reports, Flood Control Project Information)
3. Municipality in which the project is proposed.
4. U.S. Geological Survey (stream gage data, floods of record)
5. Regional Planning Agencies
6. U.S.D.A., Natural Resources Conservation Service (Design reports)
7. U.S. Army Corps of Engineers (Flood Control Projects)

6.3 Symbols And Definitions

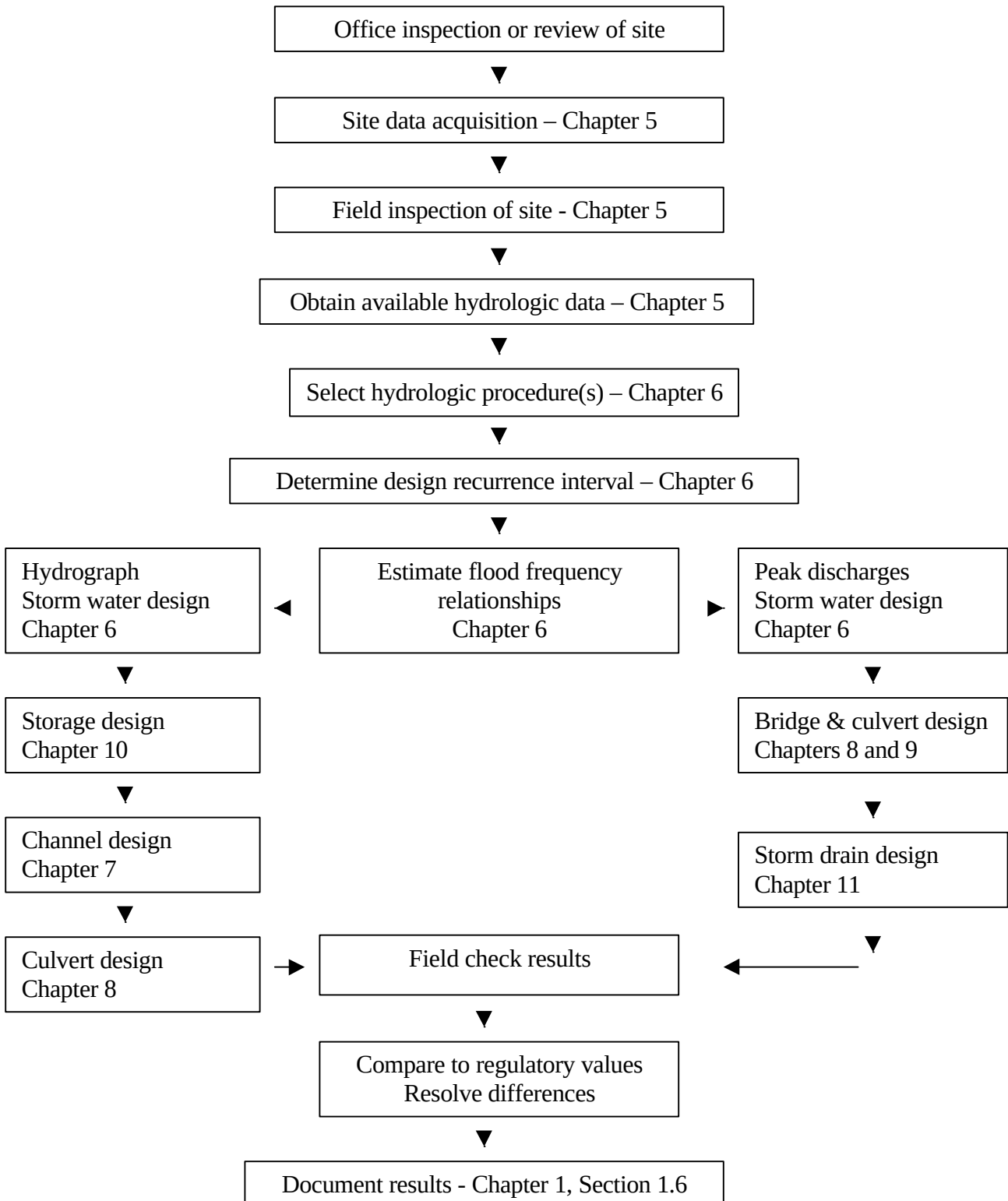
To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in hydrologic publications.

Table 6-1 Symbols And Definitions

<u>Symbol</u>	<u>Definition</u>	<u>Units</u>
A	Drainage area	ha, km ² (acres, mi ²)
BDF	Basin development factor	-
C	Runoff coefficient	-
C _f	Frequency factor	-
CN	NRCS-runoff curve number	-
C _b , C _p	Physiographic coefficients	-
d	Time interval	h
DH	Difference in elevation	m (ft)
I	Rainfall intensity	mm/h (in/hr)
IA	Percentage of impervious area	%
I _a	Initial abstraction from total rainfall	mm (in)
K	Frequency factor for a particular return period and skew	-
L	Lag	h
l	Length of mainstream to furthest divide	m (ft)
L _{ca}	Length along main channel to a point opposite the watershed centroid	km (mi)
M	Rank of a flood within a long record	-
n	Manning roughness coefficient	-
N	Number of years of flood record	years
NRCS	Natural Resources Conservation Service (formerly Soil Conservation Service)	-
P	Accumulated rainfall	mm (in)
Q	Rate of runoff	m ³ /s (cfs)
q	Storm runoff during a time interval	mm (in)
R	Hydraulic radius	m (ft)
RC	Regression constant	-
RQ	Equivalent rural peak runoff rate	m ³ /s (cfs)
S or Y	Ground slope	m/m, m/km (ft/ft, ft/mi) or %
S	Potential maximum retention storage	mm (in)
SL	Main channel slope	m/m (ft/ft)
S _L	Standard deviation of the logarithms of the peak annual floods	-
ST	Basin storage factor	%
T _B	Time base of unit hydrograph	h
t _c or T _c	Time of concentration	min or h
T _L	Lag time	h
T _r	Snyder's duration of excess rainfall	h
UQ	Urban peak runoff rate	m ³ /s (cfs)
V	Velocity	m/s (ft/s)
X	Logarithm of the annual peak	-

6.4 Hydrologic Analysis Procedure Flowchart

The hydrologic analysis procedure flowchart shows the steps needed for the hydrologic analysis and the designs that will use the hydrologic estimates.



6.5 Concept Definitions

Following are discussions of concepts which will be important in a hydrologic analysis. These concepts will be used throughout the remainder of this chapter in dealing with different aspects of hydrologic studies.

Antecedent Moisture Conditions - Antecedent moisture conditions are the soil moisture conditions of the watershed at the beginning of a storm. These conditions affect the volume of runoff generated by a particular storm event.

Depression Storage - Depression storage is the natural depressions within a watershed which store runoff. Generally, after the depression storage is filled, runoff from these low areas will commence.

Frequency - Frequency is the number of times a flood of a given magnitude can be expected to occur on an average over a stated period of time. Frequency analysis is then the estimation of peak discharges for various recurrence intervals. Another way to express frequency is with probability. Probability analysis seeks to define the flood flow with a probability of being equaled or exceeded in any year.

Hydraulic Roughness - Hydraulic roughness is a composite of the physical characteristics which influence the flow of water across the earth's surface, whether the conditions are natural or channelized. It affects both the time response of a watershed and drainage channel as well as the channel storage characteristics.

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

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

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

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

Lag Time - The lag time is defined as the time from the centroid of the excess rainfall to the peak of the hydrograph.

Peak Discharge - The peak discharge, sometimes called peak flow, is the maximum rate of flow of water passing a given point as a result of a rainfall event or the maximum discharge on a runoff hydrograph.

Rainfall Excess - The rainfall excess is the water available to runoff after interception, depression storage and infiltration have been satisfied.

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

Time Of Concentration - The time of concentration is the time it takes the drop of water falling on the hydraulically most remote point in the watershed to travel through the watershed to the point of analysis.

Unit Hydrograph - A unit hydrograph is the direct runoff hydrograph resulting from a rainfall event which has a specific temporal and spatial distribution, which lasts for a specific duration and has unit volume (or results from a unit depth of rainfall). The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is equal to one millimeter (inch) of runoff from the drainage area. When a unit hydrograph is shown with units of m^3/s (cfs), it is implied that the ordinates are m^3/s per millimeter (cfs per inch) of direct runoff. References - For a more complete discussion of these concepts and others related to hydrologic analysis, the reader is referred to – "Highway Hydrology" Federal Highway Administration Hydraulic Design Series No. 2, Second Edition, (FHWA-NHI-02-001, October 2002) and Guidelines for Hydrology - Volume II Highway Drainage Guidelines, prepared by the Task Force On Hydrology and Hydraulics, AASHTO Highway Subcommittee on Design.

6.6 Design Frequency

6.6.1 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 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 exceedence probability equals $100/RI$.

The engineer must recognize that flood discharges for larger watercourses and rivers where the 100 year frequency must be used are the result of statistical analysis. When considering the 100 year flood discharges, for example, it is often misconstrued as a flood which happens once in a hundred years. In reality, it has a one percent chance of occurring in any one year. However, it can occur several times in any one year, as it did in Connecticut during 1955 and the midwest in 1993. The 100 year flood can occur more than once during a 100 year period as is evident in Connecticut where the floods of 1938, 1955, 1982 and 1984 all exceeded the 100 year flood in some areas of the state. As future flood data is evaluated it can change the statistical discharge when added to the statistical analysis dependent on the length of records considered. This usually will result in increasing the discharge. However, discharges can also be decreased if flood control structures are implemented as is the case on the Connecticut, Quinebaug, Shetucket, Naugatuck and other rivers.

6.6.2 Frequency Table

Appendix A presents design frequencies to be used by the Department for the various drainage/hydraulic facilities.

6.6.3 Rainfall vs. Flood Frequency

Drainage systems and structures are designed based on a chosen 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, there may or may not be a direct relationship between rainfall and flood frequency.

6.6.4 Rainfall Curves

Rainfall data are available for many geographic areas. From these data, rainfall intensity-duration curves have been developed for the commonly used design frequencies. Appendix B at the end of this chapter contains the table recommended for use. Additionally, total rainfall-duration-frequency relationships are presented for use with hydrologic models. (Table B-1)

6.6.5 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 statistical estimates of peak discharges.
- Ungaged sites - the site is not near a gaging station and no streamflow record is available.

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

6.7 Hydrologic Procedure Selection

6.7.1 Overview

Streamflow measurements for determining a flood frequency relationship at a site are usually unavailable; in such cases, it is accepted practice to estimate peak runoff rates and hydrographs using statistical or empirical methods. In general results from using several methods should be compared and evaluated, not averaged. The Department practice shall be to use the discharge that best reflects local project conditions with the reasons documented. The use for each procedure is outlined with each hydrologic procedure given below.

6.7.2 Peak Flow Rates Or Hydrographs

A consideration of peak runoff rates for design conditions is generally adequate for conveyance systems such as storm drains or open channels. However, if the design must include flood routing (e.g., storage basins or complex conveyance networks), a flood hydrograph is usually required. The development of runoff hydrographs (typically more complex than estimating peak runoff rates) is usually accomplished using computer programs.

6.7.3 Hydrologic Procedures

The methods or procedures in the ensuing text are recommended for hydrologic studies performed in conjunction with the design of transportation facilities. In addition to the procedures noted, numerous proprietary software packages are available for hydrologic design. The hydraulic engineer should obtain approval from the Hydraulics and Drainage Section for the use of programs not specifically listed prior to their application in Department designs. The designer is referred to the Federal Highway Administration document entitled Hydraulic Design Series No. 2, "Highway Hydrology," Second Edition (FHWA-NHI-02-001, October 2002) for more detailed information on hydrologic methods. Additionally, computer program documentation, detailed application instructions, and related manuals must be referenced for background information.

All hydrologic studies shall be documented in accordance with the "Hydrologic Design Report Format" and all design discharge values for all bridges and culverts conveying watercourses shall be approved by the Department's Hydraulics and Drainage Section prior to the start of hydraulic design activities. (See Appendix D for report format).

6.8 Calibration

6.8.1 Definition

Calibration is a process of varying the parameters or coefficients of a hydrologic method so that it will estimate peak discharges and hydrographs consistent with local rainfall and streamflow data.

Figure 6-1 is an illustration of a hydrograph resulting from flow data as compared to a hydrograph resulting from using a non-calibrated and calibrated hydrologic procedure. It can be seen that the calibrated hydrograph, although not exactly duplicating the hydrograph from streamflow data, is a much better representation of the streamflow hydrograph than the non-calibrated hydrograph.

6.8.2 Hydrologic Accuracy

The accuracy of the hydrologic estimates will have a major affect on the design of drainage or flood control facilities. Although it might be argued that one hydrologic procedure is more accurate than another, practice has shown that all of the methods discussed in this chapter can, if calibrated, produce acceptable results consistent with observed or measured events. What should be emphasized is the need to calibrate the method for local conditions. This calibration process can result in much more accurate and consistent estimates of peak flows and hydrographs.

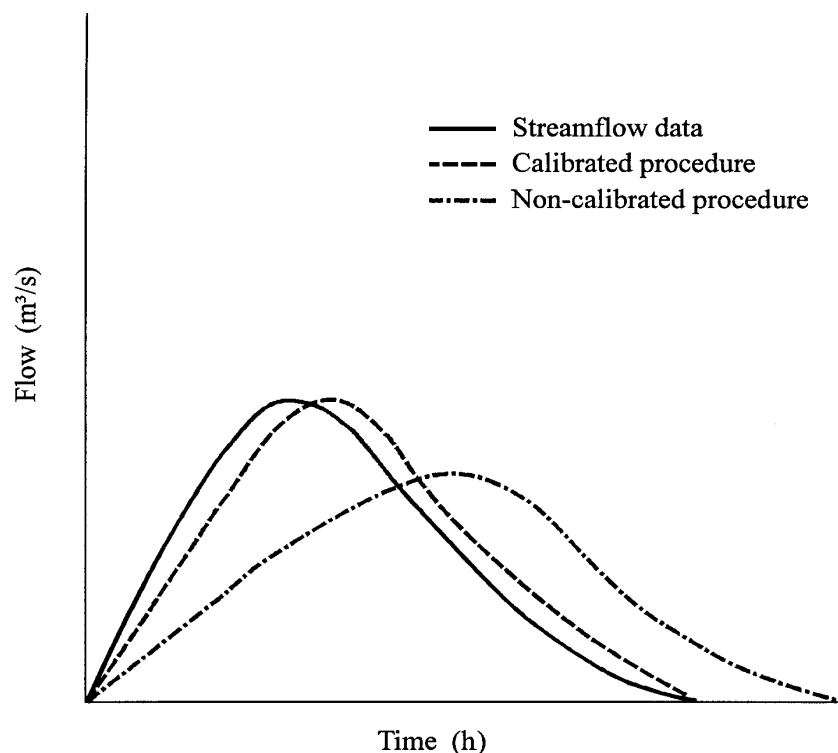


Figure 6-1 Calibrated Hydrograph

6.8.3 Calibration Process

The calibration process can vary depending on the data or information available for a local area.

1. If streamflow data are available for an area, the hydrologic procedures can be calibrated to these data. The process would involve generating peak discharges and hydrographs for different input conditions (e.g., slope, area, antecedent soil moisture conditions) and comparing these results to the gaged data. Changes in the model would then be made to improve the estimated values as compared to the measured values.
2. After changing the variables or parameters in the hydrologic procedure the results should be checked against another similar gaged stream or another portion of the streamflow data that were not used for calibration.
3. If some local agency has developed procedures or equations for an area based on streamflow data, general hydrologic procedures can be calibrated to these local procedures. In this way the general hydrologic procedures can be used for a greater range of conditions (e.g, land uses, size, slope).
4. The calibration process should only be undertaken by personnel highly qualified in hydrologic procedures and design.
5. Should it be necessary to use unreasonable values for variables in order for the model to produce reasonable results, then the model should be considered suspect and its use carefully considered (e.g., having to use terrain variables that are obviously dissimilar to the geographic area in order to calibrate to measured discharges or hydrographs).

6.9 Rational Method

6.9.1 Introduction

The rational method is recommended for estimating the design storm peak runoff for areas as large as 81 ha (200 ac). This method, while first introduced in 1889, is still used in many engineering offices in the United States. Even though it has frequently come under criticism for its simplistic approach, no other drainage design method has received such widespread use.

6.9.2 Application

Some precautions should be considered when applying the rational method.

- The first step in applying the rational method is 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 determine if the natural drainage divides have been altered.
- In determining the runoff coefficient C value for the drainage area, thought should be given to future changes in land use that might occur during the service life of the proposed facility that could result in an inadequate drainage system.
- The charts, graphs and tables included in this section are not intended to replace reasonable and prudent engineering judgment which should permeate each step in the design process.

6.9.3 Characteristics

Characteristics of the rational method which limit its use to 81 ha (200 ac) 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. For this reason, the rational method is inappropriate for watersheds greater than about 81 ha (200 ac).

- (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 (C) 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. Many guidelines and tables have been established, but seldom, if ever, have they been supported with empirical evidence.

(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. With only the peak rate of runoff, the rational method severely limits the evaluation of design alternatives available in urban and in some instances, rural drainage design.

6.9.4 Equation

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

$$Q = 0.00278 CIA \quad (Q = CIA) \quad (6.1)$$

where: Q = maximum rate of runoff, m^3/s (ft^3/s)

C = runoff coefficient representing a ratio of runoff to rainfall

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

A = drainage area tributary to the design location, ha (acres)

6.9.5 Infrequent Storm

The runoff coefficients given in Tables 6-3 through 6-5 are applicable for storms of 2-year to 10-year frequencies. Less frequent, higher intensity storms will require modification of the runoff 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 major storms can be made by multiplying the right side of the rational formula by a frequency factor C_f . The rational formula now becomes:

$$Q = 0.00278 CC_f IA \quad (Q = CC_f IA) \quad (6.2)$$

C_f values are listed in Table 6-2. The product of C_f times C shall not exceed 1.0.

Table 6-2 Frequency Factors For Rational Formula

<u>Recurrence Interval (years)</u>	<u>C_f</u>
25	1.1
50	1.2
100	1.25

6.9.6 Procedures

The results of using the rational formula to estimate peak discharges are very sensitive to the parameters that are used. The designer must use good engineering judgment in estimating values that are used in the method. Following is a discussion of the different variables used in the rational method.

Time Of Concentration

The time of concentration is the time required for water to flow from the hydraulically most remote point of 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).

Appendix C (Travel Time Estimation) at the end of this chapter describes the method based on the NRCS Technical Release No. 55 (2nd Edition). This method shall be used for the rational method. Note: under certain circumstances, where tributary areas are very small or completely paved, the computed time of concentration would be very short. For design purposes the minimum time of concentration for paved areas shall be 5 minutes and 10 minutes for grassed areas.

Common Errors

Two common errors should be avoided when calculating t_c . First, in some cases runoff from a portion of the drainage area which 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.

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.

Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate mm/h (in/h) for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be

determined from Rainfall-Intensity-Duration curves. The rainfall intensity can be determined from rainfall-intensity-duration Table B-2 which can be found in Appendix B.

Runoff Coefficient

The runoff coefficient C is the variable of the rational method least susceptible to precise determination and requires judgment and understanding on the part of the designer. While engineering judgment will always be required in the selection of runoff coefficients, a typical coefficient represents the integrated effects of many drainage basin parameters, the following discussion considers only the effects of soil groups, land use and average land slope.

Methods for determining the runoff coefficient are presented based on hydrologic soil groups and land slope (Table 6-3), land use (Table 6-4) and a composite coefficient for complex watersheds (Table 6-5).

Table 6-3 gives the recommended coefficient C of runoff for pervious surfaces by selected hydrologic soil groupings and slope ranges. From this table the C values for non-urban areas such as forest land, agricultural land, and open space can be determined. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Infiltration is the movement of water through the soil surface into the soil. Based on infiltration rates, the NRCS has divided soils into four hydrologic soil groups as follows:

- Group A Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well drained sands and gravels.
- Group B Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.
- Group C Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.
- Group D Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a claypan or clay layer at or near the surface and shallow soils over nearly impervious parent material.

The NRCS has developed detailed soil surveys for all counties within Connecticut. From these documents, the designer can determine the nature and relative percentages of the soils within a given watershed. It is important to note that the level of effort required in the determination of soil types is commensurate with the size of the watershed and the design objectives. Normally, in the computation of discharge quantities for gutter flow analysis and related storm drainage design, a detailed evaluation of soil types is not necessary, as contributing areas adjoining highways are usually relatively small. However, in the design of cross culverts, channels or interceptor ditches the determination of soil types will provide valuable assistance to the design engineer in the evaluation of the runoff potential from a particular watershed.

The second factor for consideration in the determination of a runoff coefficient is land use. As unimproved areas are developed, the potential for increased runoff becomes greater due to the loss of vegetative cover, the reduction in retention by surface depressions and the increase in impervious surface area. Table 6-4 lists recommended ranges for the runoff coefficient value classified with respect to the general character of the tributary area. **The potential for future watershed development should be considered by the designer.**

The final element to be factored into the determination of runoff coefficients is the land slope. As the slope of the drainage basin increases, the selected C value should also increase. This is caused by the fact that 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.

In summary, it should be reiterated that in assigning a value to the runoff coefficient for use in the rational method, the engineer must rely heavily on experience and judgement.

Table 6-3 Recommended Coefficient Of Runoff For Pervious Surfaces By Selected Hydrologic Soil Groupings And Slope Ranges

<u>Slope</u>	<u>A</u>	<u>B</u>	<u>C</u>	<u>D</u>
Flat (0 - 1%)	0.04-0.09	0.07-0.12	0.11-0.16	0.15-0.20
Average (2 - 6%)	0.09-0.14	0.12-0.17	0.16-0.21	0.20-0.25
Steep (Over 6%)	0.13-0.18	0.18-0.24	0.23-0.31	0.28-0.38

Source: Storm Drainage Design Manual, Erie and Niagara Counties Regional Planning Board.

Table 6-4 Recommended Coefficient Of Runoff Values For Various Selected Land Uses

<u>Description of Area</u>	<u>Runoff Coefficients</u>
Business: Downtown areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential: Single-family areas	0.30-0.50
Multi units, detached	0.40-0.60
Multi units, attached	0.60-0.75
Suburban	0.25-0.40
Residential (0.5 ha (1.2 ac) lots or more)	0.30-0.45
Apartment dwelling areas	0.50-0.70
Industrial: Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30

Table 6-5 Coefficients For Composite Runoff Analysis

<u>Surface</u>	<u>Runoff Coefficients</u>
Street: Asphalt	0.70-0.95
Concrete	0.80-0.95
Drives and walks	0.75-0.85
Roofs	0.75-0.95

6.10 Analysis Of Stream Gage Data

6.10.1 Introduction

Many gaging stations exist throughout the State 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 located on the gaged stream but not at the gaging site, transferring the known frequency discharge may be possible. (See Section 6.11)
- 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 Connecticut office of the USGS has developed regression equations for rural watersheds in Connecticut. (See Section 6.12.)

Two methods of estimating flood frequency curves from stream gage data are provided in this manual. The first method is a graphical procedure based on the Gumbel method. The second, a statistical method, which makes use of the Log Pearson Type III frequency distribution.

6.10.2 Application

The designer shall use the stream gage analysis findings for design when there is sufficient years of measured or synthesized stream gage record. The most current stream gage data shall be used for frequency analysis. The data should be obtained directly from the Connecticut office of the USGS. The preferred method for analyzing stream gage data is the Log Pearson Type III method with the Gumbel graphical method being used primarily as a check to insure errors are not made — particularly with the frequency estimates of larger floods. Where series discrepancies 20% +/- are encountered in the findings between the two methods, special studies may be required. These special studies shall consist of such things as comparison with regression equations, application of other flood-frequency methods, and the collection and analysis of historical data. Outliers shall be placed into perspective using the procedure found in Water Resources Council Bulletin 17B.

6.10.3 Graphical Method Procedure

In order to develop frequency-discharge information from stream gage data, a record of 10 years or more is required. Gage stations with records which include controlled watershed runoff should be avoided since the natural response to flood events may not have been recorded. The following steps may then be taken to develop a flood-frequency curve representing the data.

Step 1 The peak discharges for each water year are listed in chronological order. A water year extends from October 1 through September 30.

Step 2 After the discharges have been listed, they are then numbered in order of their magnitude; that is, the highest discharge for a particular gaging station is assigned the number 1, then next highest 2 and so on. The numbering system will indicate the relative distribution of floods during a given period of years.

Step 3 The probability of exceedence, also called the plotting position, for each annual peak flow is computed using the formula:

$$\text{plotting position} = M/(N+1) \quad (6.3)$$

Where: N = number of years of record

M = rank of a given flood beginning with 1

The reciprocal of the plotting position, $(N + 1)/M$, is the recurrence interval in years.

The recurrence interval vs. discharge for each year is plotted on commercially available log-probability paper, with the recurrence interval or plotting position as the x-axis and the magnitude of the associated discharge as the y-axis log scale. The data plots shown on the graph illustrate the frequency distribution of the floods for a given station.

Step 5 If Gumbel paper is used the points should theoretically tend to fall in a straight line. This special paper has been developed so that sample data will plot as a straight line if the data are distributed according to the formula:

$$F(Q) = e^{-x} \quad (6.4)$$

$$\text{Where: } x = e^{\alpha(Q - \beta)} \quad (6.5)$$

$$\alpha = 1.281/S \quad (6.6)$$

$$\beta = Q' - 0.450 S \quad (6.7)$$

Q = mean flow

Q' = mean peak

S = standard deviation

This paper is not available commercially, but most USGS offices have prepared forms of the paper on which the horizontal scale has been transformed by the double-logarithmic transform of Equation 6.4.

Step 6 The recurrence interval and corresponding discharge is believed reliable up to a point where the points fall appreciably out of line. Values read beyond this point are less reliable. Extrapolation beyond the limits of the plotted points is not recommended.

Step 7 This frequency curve applies only to the point on the stream at which the gage is located. The use of the flood frequency curve is as follows: enter with the desired frequency or RI and read up to the geometric mean line, then move across to the discharge scale for the design discharge. The reverse procedure is used when a discharge is known and information on frequency is desired.

Step 8 Flood frequency curves can also be plotted with stream stage rather than discharge as the ordinate. This is somewhat more convenient when checking high water elevations at a bridge site.

The analysis of gaged data permits an estimate of the peak discharge for the desired return period at a particular site. A best fit line can be drawn through the data points by eye, and the peak flow corresponding to the desired return period could be extracted from the curve. This is a very subjective method and each designer may derive different estimates from the same data set. Experience has shown that statistical frequency distributions may be more representative of naturally occurring floods and can be reliable when used for prediction. Although several different distributions are used for frequency analysis, experience has shown the log-Pearson Type III distribution to be one of the most useful. The log-Pearson III distribution and the process of fitting it to a particular data sample are described in detail in Water Resources Council Bulletin 17B, "Guidelines for Determining Flood Flow Frequency," 1981. The following abbreviated procedure is taken from that publication.

In the course of preparing a frequency analysis for a particular watershed, the designer will undoubtedly encounter situations where further adjustments to the data are necessary. Special handling of outliers, historical data, incomplete data, and zero flow years is covered in detail in Bulletin 17B.

The computer system HYDRAIN provides the Log Pearson III flood frequency analysis. The analysis follows the Bulletin 17B guidelines for the calculation of a log-Pearson frequency curve based on the mean, standard deviation and skewness of the logarithms of the recorded annual peak flows.

6.10.4 Statistical Method Procedure

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:

$$Q = (\Sigma X) / N \quad (\text{mean of logs}) \quad (6.8)$$

Where: N = number of observations and X is the logarithm of the annual peak.

The standard deviation of logs is:

$$S_L = \{[\Sigma X^2 - (\Sigma X)^2 / N] / [N - 1]\}^{1/2} \quad (6.9)$$

The coefficient of skew of logs is:

$$G = [N^2(\Sigma X^3) - 3N(\Sigma X)(\Sigma X^2) + 2(\Sigma X)^3] / [N(N - 1)(N - 2)S_L^3] \quad (6.10)$$

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

$$\log Q = Q_L + KS_L \quad (6.11)$$

Where: $\log Q$ = the logarithm of the flood magnitude.

Q_L = the mean of the logarithms of the peak annual floods.

K = a 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 = the 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.

6.10.5 Skew

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 using the above formula 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.

6.11 Transferring Gaged Data

6.11.1 Procedure

Gaged data can be transferred up or downstream on the gaged stream only. If the drainage area for the location of concern is $\geq 75\%$ and $\leq 125\%$ of the drainage area at the gage, then the gaged data can be transferred with equation 6.12.

6.11.2 Transfer Equation

The following equation shall be used to transfer gage data:

$$\frac{Q_1 / A_1}{Q_2 / A_2} = \frac{A_1^{[(0.894 / A_1^{0.048}) - 1]}}{A_2^{[(0.894 / A_2^{0.048}) - 1]}} \quad \text{(English only)} \quad (6.12)$$

Q_1 and A_1 represent the discharge rate and watershed area at one point in the watershed and Q_2 and A_2 represent the rate and area at the gage or known outlet which remain constant while Q_1 and A_1 are varied.

Q = discharge in cubic feet per second

A = drainage area in square miles

Source: Adopted from Mockus, V., SCS National Engineering Handbook, Section 4, Hydrology, 1972

6.12 USGS Regression Equations – Rural Areas (English Units) [Replaced by StreamStats]
CONSULTING ENGINEERS GENERAL MEMORANDUM 07-06 StreamStats

6.12.1 Introduction

The USGS Regression Equations for computing peak flows from ungaged basins in Connecticut were developed by Log-Pearson Type III multiple regression analysis from records of 105 stream gaging stations with 10 to 45 years of continuous records. Rainfall data was taken from 28 precipitation gaging stations of the National Weather Service with 10 to 40 years of records.

The regression equations relate a dependent variable (peak discharge) to independent variables associated with physical and climatic basin characteristics. These independent variables include drainage areas, streambed length and slope, 24 hour rainfall amounts and areas of coarse-grained stratified drift.

The USGS Regression Equations for Connecticut from Weiss, 1983, Conn. Water Resources Bulletin No. 36.

6.12.2 Limits

THE ENGINEER MUST ASSURE THAT THE FOLLOWING CRITERIA ARE MET IN ORDER TO APPLY THE EQUATIONS:

- 1) The drainage area must be greater than one (1) square mile and less than one thousand (1000) square miles.
- 2) The drainage basin must not be significantly affected by flood control impoundments or by storage facilities having 4.5 million cubic feet (103 AF) or more of storage per square mile.
- 3) The watershed must not exhibit a significant degree of development. If more than 30 percent of the drainage basin is urbanized, an adjustment to the rural peak discharge will be required, see Section 6.13 USGS REGRESSION/EQUATIONS-ADJUSTMENT for basin urbanization.

PROCEDURES FOR THE APPLICATION OF THE USGS EQUATIONS

- 1) Determine if drainage basins usable storage is less than 4.5 million cubic feet per square mile.
 - a) If the storage in the upper reach is greater than 4.5 million cubic feet per square mile, this area can be eliminated from the effective drainage area as above storage.
 - b) If the storage is located in the central portion or at the design point of the watershed and is greater than 4.5 million cubic feet per square mile, this procedure cannot be used.
- 2) Obtain the following drainage basin characteristics:
 - A = Drainage area in square miles from topographic mapping or an appropriate alternate source.
 - L = Stream length in miles from the design point to the extreme rim of the basin divide (from topographic mapping).
 - S_m = Streambed slope expressed in feet/mile between points 10 percent and 85 percent of the distance from the study site to the basin divide.
 - %A_{SD} = Percentage of drainage area underlain by coarse-grained stratified drift (from surficial geology maps copies available from Hydraulics and Drainage Section). This is expressed as a whole number.

I_x = 24-hour rainfall, in inches. (See Figures 6.2 – 6.6)

- 3) Apply the appropriate equation for the desired return frequency (See Table 6.6.)
- 4) Check degree of development in the watershed and adjust for urbanization, if required (Section 6.13).

6.12.3 Calibration

The results obtained from the application of USGS regression equations can be calibrated to stream gage information when the ungaged stream has watershed characteristics (drainage area, length, slope, coarse-grained stratified drift, urbanization, etc.) which are similar to a gaged watershed. A step by step calibration procedure is outlined below.

1. Compute the frequency-discharges at the site of interest using the USGS regression equations (Section 6.12).
2. Check USGS gage records to determine if a gaged watershed has similar characteristics (drainage area, length, slope, etc.) to the ungaged watershed. Table 3 of the publication entitled “Evaluation and Design of a Streamflow-Data Network for Connecticut” by L.A. Weiss, 1983 (see reference list), provides a listing of the basin characteristics for the gages used in the development of the regression equations. However, since considerable time has elapsed since the completion of the regression study, the USGS must be consulted regarding the most current streamflow information. If homogeneity of the watersheds is determined, obtain the necessary streamflow data. If discharge-frequency information has not been developed from the streamflow data, see Section 6.10.
3. Compute the frequency-discharges at the gage using the USGS regression equations.
4. Compare the computed discharges (Step 3) to the discharges determined from actual gage records (Step 2). An adjustment factor for a given storm frequency can be determined as the ratio of the computed discharge (Step 3) to the gage discharge (Step 2).
5. Multiply the frequency-discharges for the site (Step 1) by the corresponding adjustment factor as determined in Step 4. The resulting discharges represent the adjusted or calibrated discharges for the site based on a comparative gaged watershed.
6. The determination of similarity between watersheds and the use of the unadjusted or adjusted discharges requires considerable judgement of the engineer and should be clearly documented in the hydrologic report.

6.12.4 500 Year Flood

Use Frequency-Discharge Curve using a straight line approximation plotted on Log-Probability paper.

This is reversed. Should be ratio of gage discharge (Step 2) / computed discharge (Step 3)

Standard Error of Estimate

$$Q_2 = \frac{7.6(A)^{0.97} (I_2)^2}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.17} (\%A_{sd} + 1)^{0.2}}$$

+/- 36.7% **(6.13)**

$$Q_{10} = \frac{6.6(A)^{1.0} (I_{10})^{1.89}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.22} (\%A_{sd} + 1)^{0.19}}$$

+/- 39.2% **(6.14)**

$$Q_{25} = \frac{21.2(A)^{1.0} (I_{25})^{1.21}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.23} (\%A_{sd} + 1)^{0.2}}$$

+/- 42.2% **(6.15)**

$$Q_{50} = \frac{23.2(A)^{1.03} (I_{50})^{1.14}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.26} (\%A_{sd} + 1)^{0.2}}$$

+/- 44.2% **(6.16)**

$$Q_{100} = \frac{28.7(A)^{1.04} (I_{100})^{1.08}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.27} (\%A_{sd} + 1)^{0.22}}$$

+/- 46.8% **(6.17)**

Table 6-6 USGS Regression Equations For Connecticut

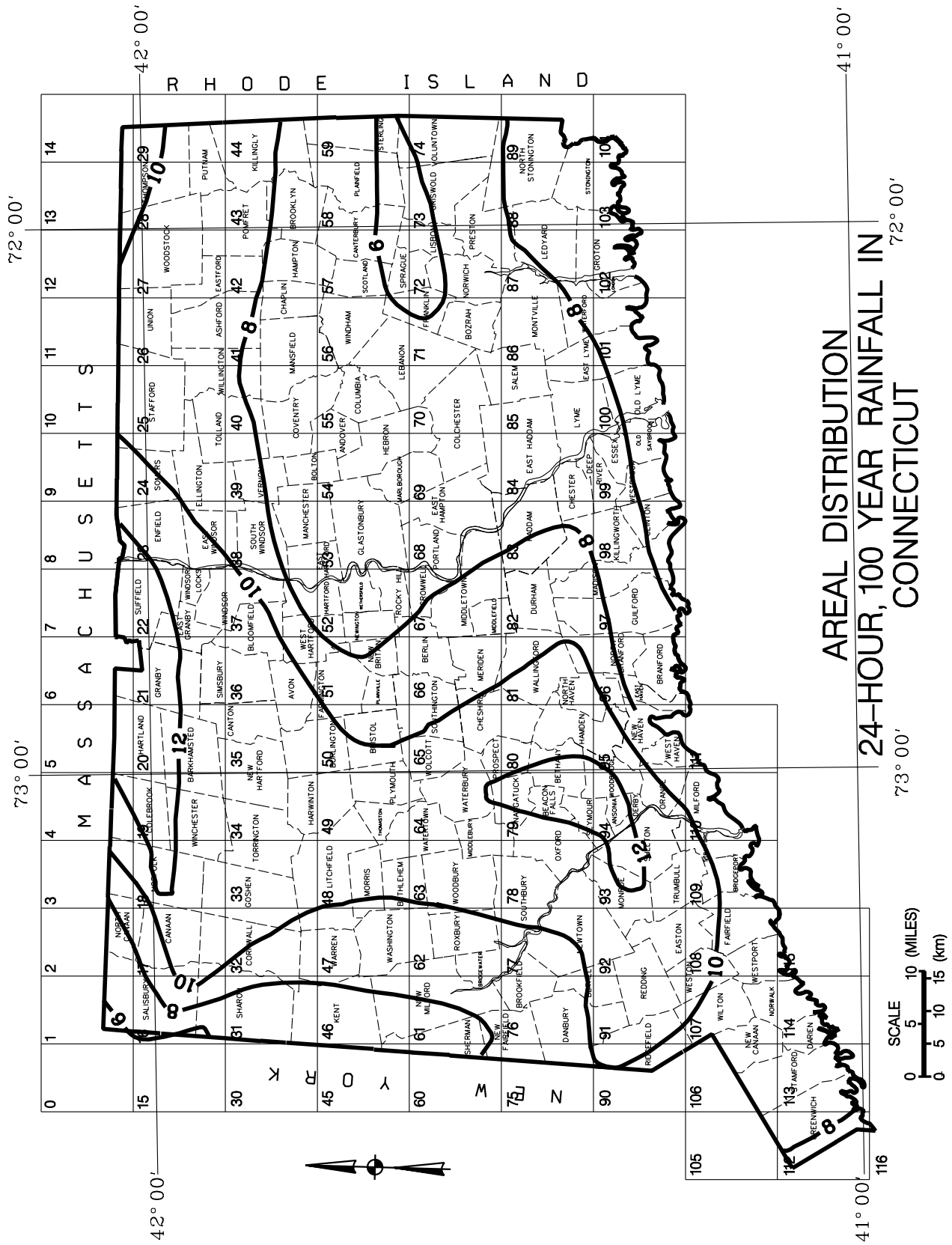


Figure 6-2

For use with USGS Regression Equations only.
Source: L. A. Weiss, U.S. Geological Survey, 1983.

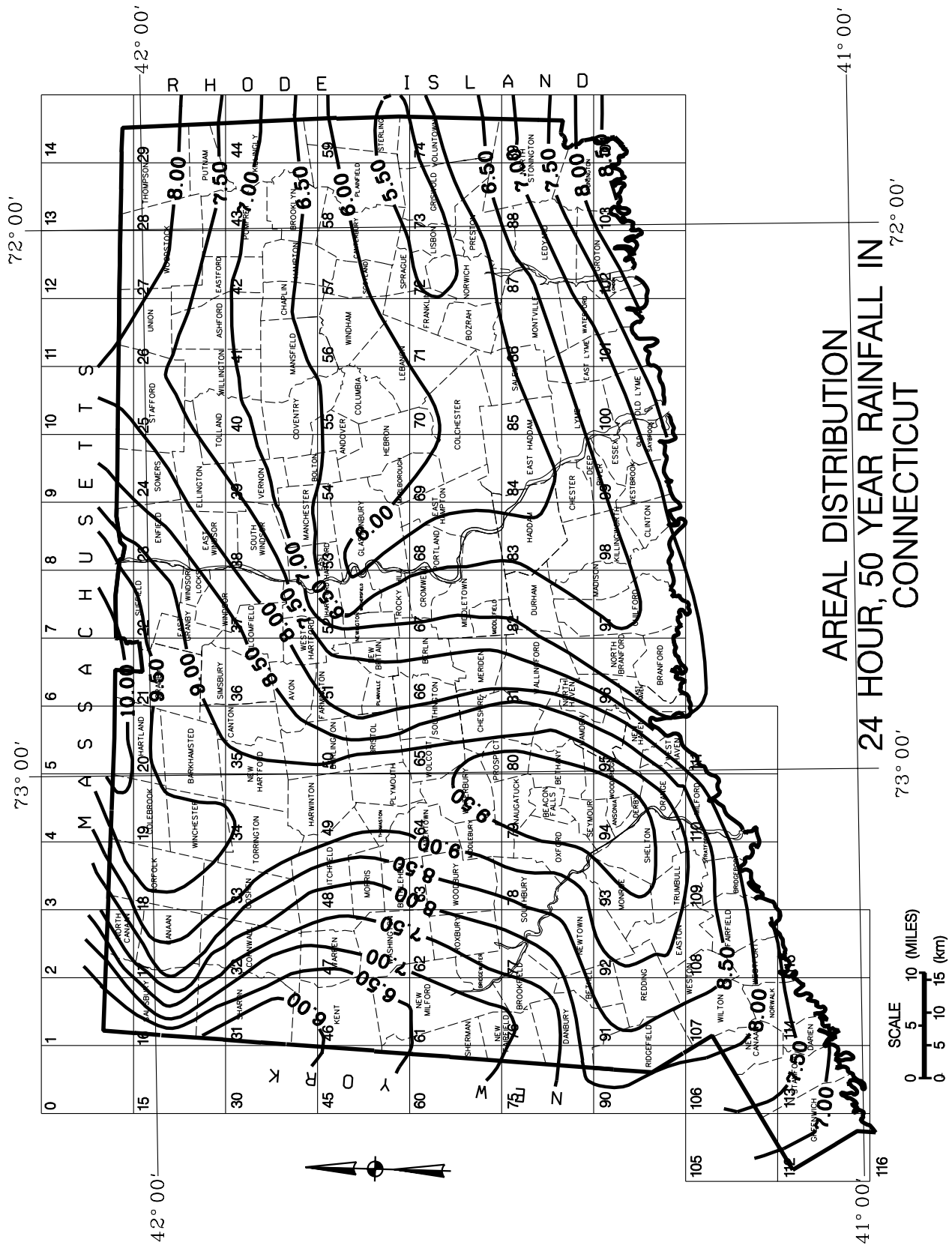


Figure 6-3

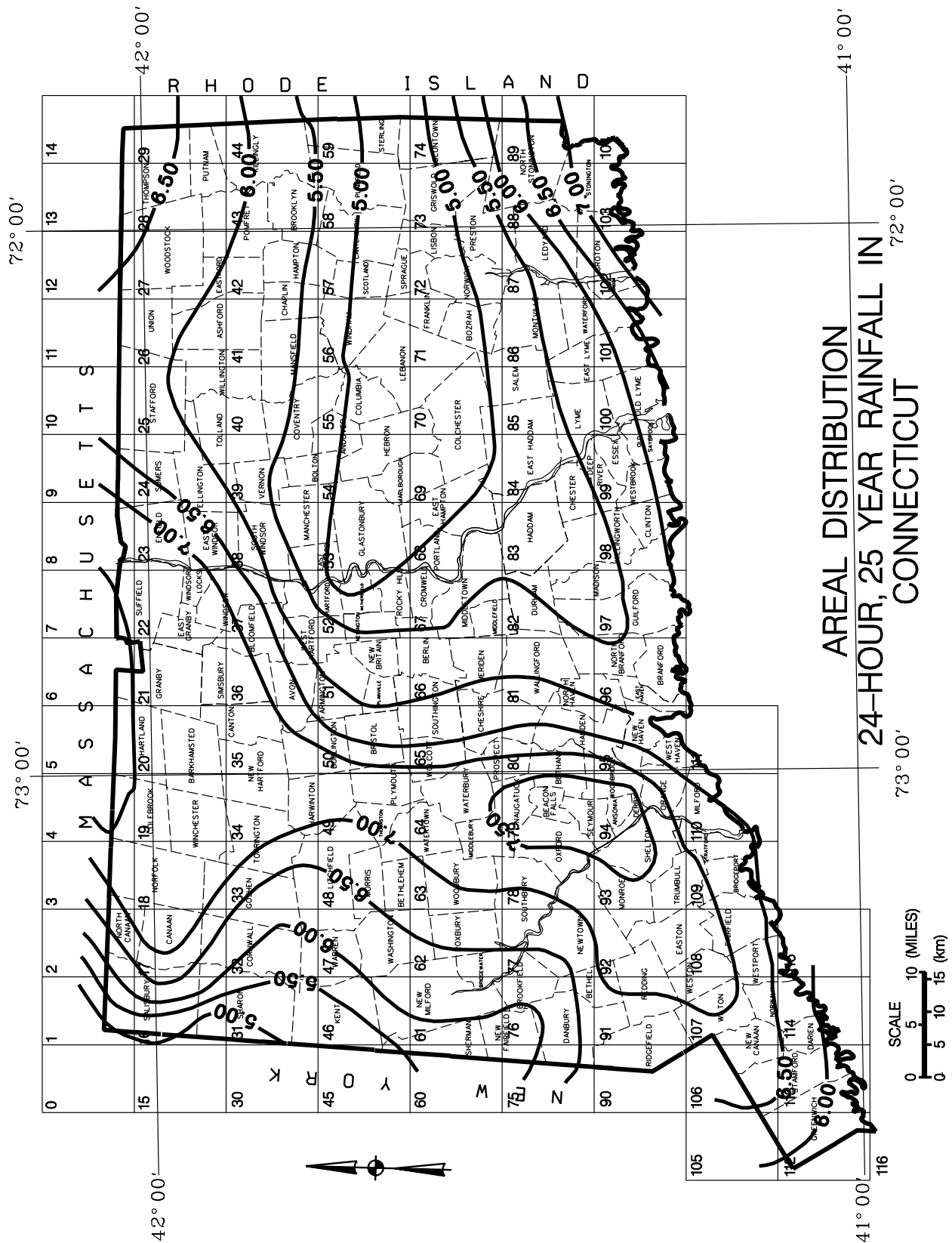


Figure 6-4

For use with USGS Regression Equations only.
Source: L. A. Weiss, U.S. Geological Survey, 1983.

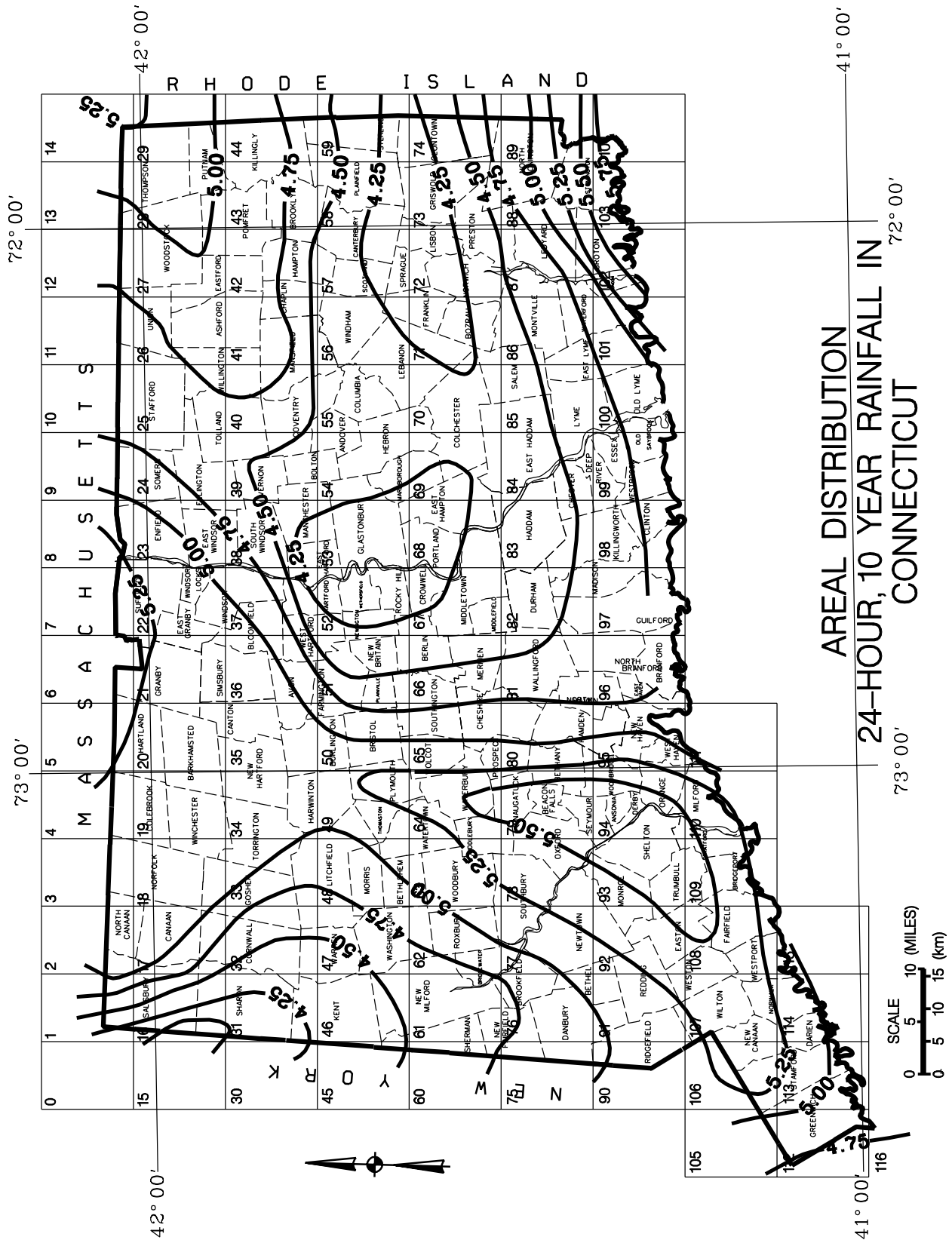


Figure 6-5

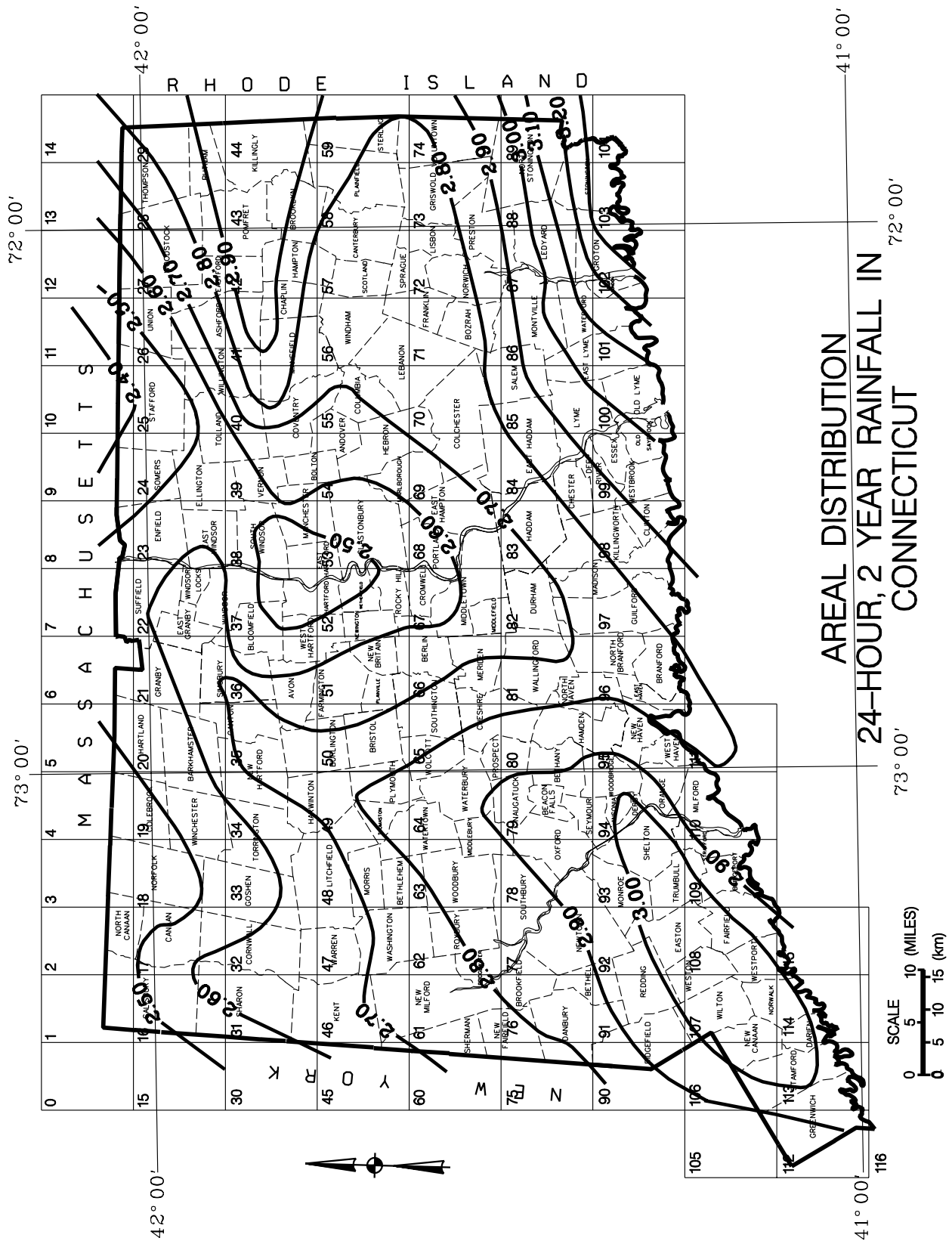


Figure 6-6

6.13 USGS Regression Equations – Adjustment for Basin Urbanization

6.13.1 Introduction

The United States Geological Survey has developed three sets of regression equations to estimate flood discharges for ungaged urban sites for various recurrence intervals. Two sets of equations are based on seven independent parameters and the third is based upon three independent parameters. The three parameter estimating equations, listed in Table 6.6, are recommended for use where applicable in the evaluation of urban peak discharges for ungaged sites with drainage areas between 1 and 100 square miles. For complete information regarding background data and research related to the development of these equations, the reader is referred to the publication entitled "Flood Characteristics of Urban Watersheds in the United States, U.S. Geological Survey Paper 2207," dated 1983.

The Basin Development Factor (BDF) has been identified by the USGS as the most important indicator of urbanization for use in the estimating equations. This factor, which provides a measure of the efficiency of the drainage systems within an urbanizing watershed, is determined as explained below. (See FHWA publication entitled "Highway Hydrology," HDS-2, Second Edition, October 2002.)

6.13.2 Characteristics

The basin is first divided into three sections as shown in Figure 6.7. Each section contains approximately a third of the drainage area of the watershed. Travel time is given consideration when drawing these boundaries so that the travel distances along two or more streams within a particular third are about equal. This does not mean that the travel distances of all three subareas are equal; only that within a particular subarea the travel distances are approximately equal.

Within each section of the basin, four aspects of the drainage system are evaluated and assigned a code, as follows:

- 1) Channel Improvements - If channel improvements such as straightening, enlarging, deepening and/or clearing are prevalent for the main drainage channel and principal tributaries (those that drain directly into the main channel), then a code of one (1) is assigned. Any one, or all of these improvements would qualify for a code of one (1). To be considered prevalent, at least 50 percent of the main drainage channel and principal tributaries must be improved to some extent over natural conditions. If channel improvements are not prevalent, a code of zero (0) should be assigned.
- 2) Channel Linings - If more than 50 percent of the main drainage channel and principal tributaries have been lined with an impervious material, such as concrete, then a code of one (1) is assigned. If less than 50 percent of these channels are lined, then a code of zero (0) is assigned. The presence of channel linings would probably indicate the presence of channel improvements as well. Therefore, this is an added factor and indicates a more highly developed drainage system.
- 3) Storm Drains - Storm drains are defined as enclosed drainage structures (usually pipes), frequently used on the secondary tributaries where the drainage is received directly from streets or parking lots. Quite often these drains empty into the main tributaries and channel which are either open channels, or in some basins may be enclosed as pipes or box culverts. When more than 50 percent of the secondary tributaries within a section consists of storm drains, then a code of one (1) is assigned. If less than 50 percent of the secondary tributaries

consist of storm drains, then a code of zero (0) is assigned. It should be noted that if more than 50 percent of the main drainage channels and principal tributaries are enclosed, then the aspects of channel improvements and channel linings would also be assigned a code of one (1).

- 4) Curb and Gutter Streets - If more than 50 percent of a subarea is urbanized (covered by residential, commercial and/or industrial development), and if more than 50 percent of the streets and highways in the subarea are constructed with curbs and gutters, then a code of one (1) should be assigned. Otherwise, a code of zero (0) is assigned. Frequently, drainage from curb and gutter streets will empty into storm drains.

The above guidelines for determining the various drainage system codes are not intended to be precise measurements. A certain amount of subjectivity is involved. It is recommended that field checking be performed to obtain the best estimate. The basin development factor (BDF) is computed as the sum of the assigned codes (SEE BDF calculation sheet Table 6-7). With three subareas per basin, and four drainage aspects to which codes are assigned in each subarea, the maximum value for a fully developed drainage system would be 12.

Conversely, if the drainage system has not been developed, then a BDF of zero (0) would result.

The BDF is a simple index to estimate for an existing urban basin. The 50 percent guideline is usually not difficult to evaluate because many urban areas tend to use the same design criteria throughout, and therefore the drainage aspects are similar throughout. Also, the BDF is convenient to use for projecting future development. Projections of full development, or intermediate stages of development, can usually be obtained from city engineers.

Equations

USGS Three Parameter Estimating Equations

Standard error %

$$UQ_2 = 13.2A^{0.21}(13 - BDF)^{-0.43}RQ_2^{0.73}$$

+/- 43 **(6.18)**

$$UQ_{10} = 9.51A^{0.16}(13 - BDF)^{-0.36}RQ_{10}^{0.79}$$

+/- 41 **(6.19)**

$$UQ_{25} = 8.68A^{0.15}(13 - BDF)^{-0.34}RQ_{25}^{0.80}$$

+/- 43 **(6.20)**

$$UQ_{50} = 8.04A^{0.15}(13 - BDF)^{-0.32}RQ_{50}^{0.81}$$

+/- 44 **(6.21)**

$$UQ_{100} = 7.70A^{0.15}(13 - BDF)^{-0.32}RQ_{100}^{0.82}$$

+/- 46 **(6.22)**

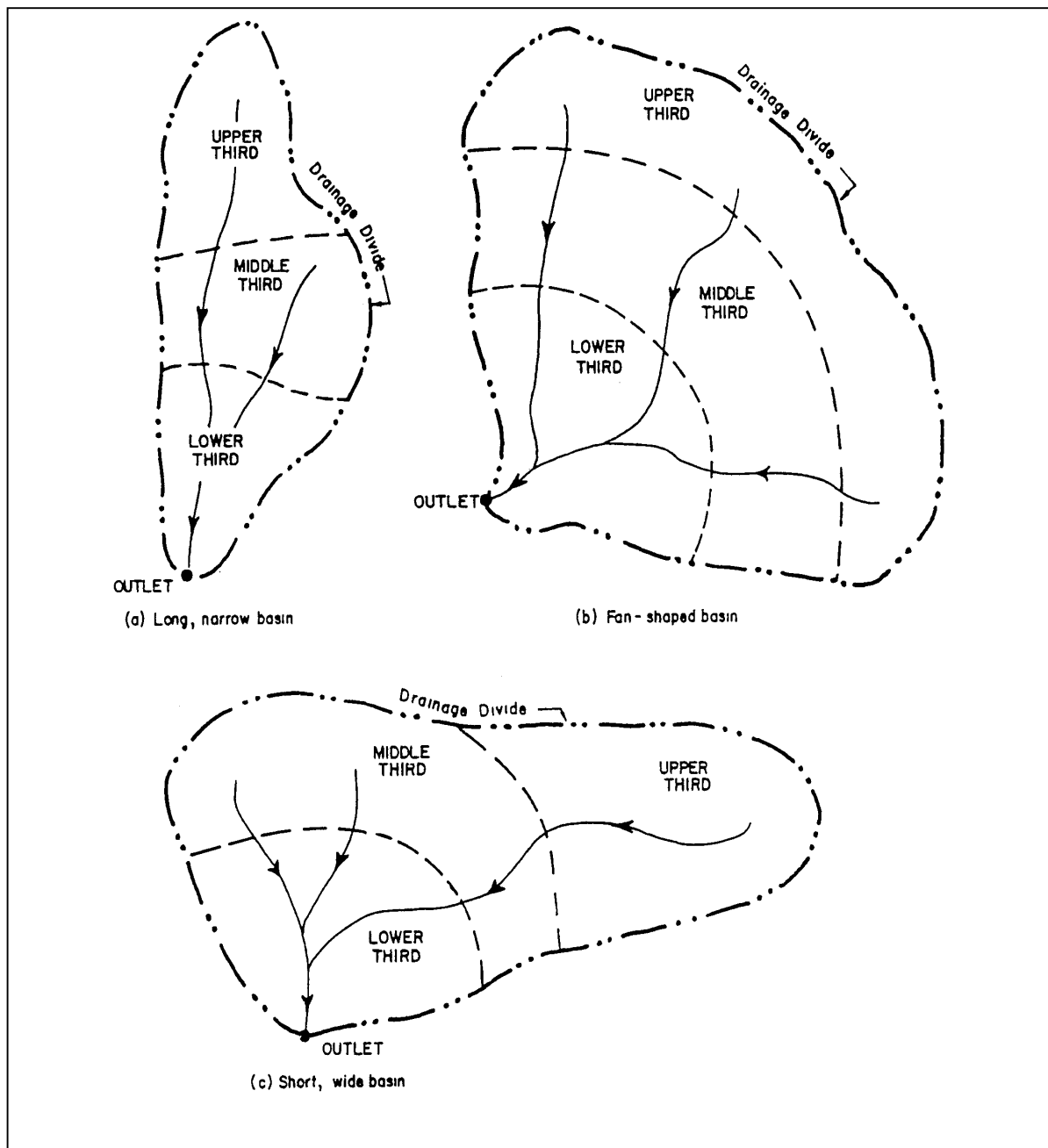
$$UQ_{500} = 7.47A^{0.16}(13 - BDF)^{-0.30}RQ_{500}^{0.82}$$

+/- 52 **(6.23)**

Where:

- UQ_r = the peak discharge of recurrence interval, r, for an urbanized condition in cfs
 A = the drainage area in square miles
 BDF = the Basin Development Factor
 RQ_r = the peak discharge of recurrence interval, r, for rural conditions

Table 6-6



Schematic of typical drainage basin shapes and subdivision into basin thirds. Note that stream-channel distances within any given third of a basin in the examples are approximately equal, but between basin thirds the distances are not equal, to compensate for relative basin width of the thirds.

Figure 6-7 Subdivision of Watersheds for Determination of Basin Development Factor

Table 6-7 BASIN DEVELOPMENT FACTOR CALCULATION SHEET

Basin Characteristic	Upper Third	Middle Third	Lower Third
Channel Improvements			
Channel Linings			
Storm Drains or Storm Sewers			
Curb and Gutter Streets			
Sub Totals			

_____ + _____ + _____ = BDF

6.14 Computer Programs for Hydrologic Modeling

6.14.1 Introduction HEC-1 and TR-20

The "HEC-1" or "HEC-HMS" computer program was developed by the U.S. Army Corps of Engineers, Hydrologic Engineering Center to simulate flood hydrology in complex river basins. The program entitled "Technical Release 20, Computer Program for Project Formulation, Hydrology", (TR20), was written by the Hydrology Branch of the Soil Conservation Service, (SCS), now NRCS, for use in the hydrologic evaluation of flood events for water resource projects. Both are single event models designed to estimate direct surface runoff resulting from synthetic or natural rainstorms.

6.14.2 Application

These computer programs are recommended for use under the following circumstances:

- 1) The watershed under study is ungaged and significant reservoir or valley storage is present, thereby preventing the use of regional regression equations.
- 2) A stormwater detention design or study is to be undertaken and reservoir routing computations are required.
- 3) The designer must construct a model to simulate an actual storm event and sufficient data is available to facilitate calibration of the results. (May be desirable or necessary in circumstances which involve litigation).
- 4) Pre/post development peak discharge rates must be determined in order to document the effects of a proposed activity.
- 5) A dam breach analysis is required.
- 6) May be used on any watershed, as is limited by individual computer program, and is recommended when watershed exceeds the limits established by other methods.

In addition to the circumstances noted above, the Department may require the use of these or similar programs for various site specific reasons.

Due to the complex nature of these programs, it is not intended that this chapter provide the user with comprehensive guidelines for program application. Rather, the ensuing text will provide a brief overview of program capabilities and recommendations for report documentation.

6.14.3 Description

Program Descriptions and Modeling Recommendations

The Hydrologic Modeling System HEC-HMS is designed to simulate the precipitation-runoff processes of dendritic watershed systems. It supercedes HEC-1 and provides a similar variety of options but represents a significant advancement in terms of both computer science and hydrologic engineering. In addition to unit hydrograph and hydrologic routing options, capabilities include a

linear quasi-distributed runoff transform (ModClark) for use with gridded precipitation, continuous simulation with either a one-layer or more complex five-layer soil moisture method, and a versatile parameter estimation option.

HEC-1 This computer program simulates a) rainfall and/or snowmelt runoff from subbasins within a watershed, b) flow through stream reaches, c) flows from urban areas, d) flow diversions, and e) flow through reservoirs including a breached dam condition with a varying (degrading) failure section. Effectively, there is no limitation related to basin size as the program is suitable for use in the analysis of watersheds which vary in size from small urban catchments to large, multi-basin river systems. In order to transform rainfall values to runoff, a unit hydrograph or kinematic wave approach is employed. The unit hydrograph, which is most commonly used, is the method recommended by the Department unless it can be demonstrated by the designer that basin characteristics require the use of the kinematic wave method.

Point rainfall amounts for synthetic storms shall be those quantities presented in Table B-1 of Appendix B of this chapter for the appropriate duration and return frequency. The model shall be constructed such that multiple flood analyses are generated and to include as a minimum the 2, 10, 25, 50, 100 and 500 year events. In studies which involve large dams, the 1/2 Probable Maximum Flood (PMF) or Probable Maximum Flood (PMF) may also be required as directed by the Department of Environmental Protection. The computer model shall be calibrated to a gage if possible.

Times of concentration and travel times for use in the HEC-1 program shall be developed using the methodology described in Appendix C of this chapter.

TR-20 Technical Release No.20 is a computer program which computes direct runoff from a rainstorm, generates flood hydrographs from surface runoff, and routes flow through channel reaches or reservoirs. Output data includes peak discharges with related times of occurrence and water surface elevations at designated cross sections or structures. With the exception of the channel routing routine (Modified Att-Kin Method), TR-20 is based upon the methodologies contained in the NRCS (formerly SCS) National Engineering Handbook, Section 4, Hydrology, commonly referred to as NEH 4.

Based on the assumption that rainfall depth is uniformly distributed throughout the watershed, the areal limitations for subcatchment or overall basin size range from approximately 0.25 km² (0.1 mi²) to 1035 km² (400 mi²).

As required with the HEC-1 program, the hydraulic engineer shall utilize the rainfall data presented in Table B-1 of Appendix B of this chapter as input for the desired storm event. The rainfall distribution for all watersheds in Connecticut shall be the 24 hour "Type III" storm as recommended by the NRCS, and Appendix C methodologies shall be used for time of concentration and travel time computations.

All hydrologic studies performed utilizing TR-20 for watersheds greater than 2.59 square kilometers (one square mile) shall include multiple flood analyses to include the 2, 10, 25, 50, 100 and 500 year events, except where dam studies require the analysis of probable maximum floods.

The computer model shall be calibrated to a gage station if possible.

TR-55 Urban Hydrology for Small Watersheds, Version 2.0. This Rainfall Runoff Model was developed by the US Soil Conservation Service, now known as the Natural Resources Conservation Service (NRCS). TR-55 presents simplified procedures to calculate storm runoff volume, peak rate of discharge, hydrographs, and storage volumes required for flood water reservoirs. These procedures are applicable in small watersheds, especially urbanizing watersheds, in the United

States. The primary functions of the program are for peak runoff computations using the Graphical Peak Discharge Method, the Tubular Peak Discharge Method and Temporary Storage. Support functions include the computation of the runoff curve number (CN), the Time of concentration (T_c) and travel time through a subarea (T_t). Limits: NRCS type distributions, 24-hour duration rainfall, 10 subwatersheds, minimum 0.1 hour and maximum 10-hour time of concentration.

6.14.4 Schematic

A watershed schematic representing the physical subbasin elements and their interrelationships shall be constructed for all cases in which a computer program is used to develop a hydrologic model.

6.14.5 Documentation

Input Data Documentation

All input parameters used in hydrologic computer models shall be fully documented for inclusion in a hydrologic design report. The designer must submit, as appropriate, watershed mapping which indicates overall basin limits and subbasin divides, T_c and T_t computations including flow paths for the various elements, curve number determinations, soil type determinations based on appropriate mapping, cover types or zoning information, stage/storage/discharge relationships for reservoir routing, structure routing, channel routing data and any other pertinent data.

6.15 Hydrology for Temporary Facilities

6.15.1 Introduction

Temporary hydraulic facilities include, among other things, all channels, culverts, bridges or channel constrictions such as cofferdams which are required for stage construction, haul roads, channel relocations, culvert installations, bridge construction, temporary roads, or detours. They are to be designed in the same level of care that is used for the primary facility.

These designs are to be included in the plans for the project. Appendix F of this chapter provides a procedure to determine the temporary design flood frequency based on the percent design risk and rating selection factors. Section 9.6.12 of Chapter 9, Bridges, discusses the hydraulic design of temporary facilities.

6.15.2 Detours And Temporary Roadways

Drainage systems for these are to be designed for a 2-year frequency if the roadway is required for a year or less and a 5-year frequency if required for longer than a year. All other temporary hydraulic facilities connected with these roads are to be designed for frequencies as determined by using Section 6.15.4.

6.15.3 Haul Roads

Hydraulic facilities for haul roads which cross or encroach into a watercourse are to be designed for a frequency as determined by using a Design Risk of 50% in step 3 of Appendix F. As a general rule, to avoid excess upstream flooding, the profile of the road should connect the tops of the channel embankments and the road designed to be overtopped by those events which exceed the design discharge. Sufficient cover must be provided over the temporary conduit to insure structural integrity. The structural analysis of the conduit is to be included with the design.

The plan is to include a warning to the contractor that this road is expected to be under water during certain rainfall events for undetermined lengths of time.

6.15.4 Other Hydraulic Facilities

The selection of a design flood frequency for the remaining temporary hydraulic facilities involves consideration of several factors. These factors are rated considering their severity as 1, 2, or 3 for low, medium or high conditions. Appendix F provides a step by step procedure for determining the design flood frequency.

6.16 Average Daily and Spring Flows

For gaged streams where the gage is within reasonable proximity to the project site, average daily and spring flows will be determined by review of the available gage records.

For ungaged streams the average daily and spring flows will be computed by the following equations:

Average Daily Flow:

$$Q_{AD}(\text{cms}) = [A (\text{km}^2)]^{0.98} * 0.0208 \quad (Q_{AD}(\text{cfs}) = [A (\text{sm})]^{0.98} * 1.87) \quad (6.24)$$

Average Spring Flow:

$$Q_{AS}(\text{cms}) = [A (\text{km}^2)]^{0.988} * 0.04 \quad (Q_{AS}(\text{cfs}) = [A (\text{sm})]^{0.988} * 3.62) \quad (6.25)$$

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Appendix A - Design Frequencies

Design frequencies for hydraulic facilities related to transportation projects shall be as follows:

<u>ITEM</u>	<u>DESIGN FREQUENCY</u>
• Curb inlet/storm drainage system	10 year (Note 1) also Chapter 11, Storm Drainage Systems
• Channels/ditches (Not conducting a watercourse)	10 year (Note 6) also Chapter 7, Channels
• Watercourse channels	50 year (minimum) also Chapter 7, Channels (Note 2)
• Culverts Watershed area < 2.59 km ² (1 mi ²) Watershed area > 2.59 km ² (1 mi ²)	50 year (Notes 3,4,5) also Chapter 8, Culverts 100 year (Notes 3,4) also Chapter 8, Culverts
• Bridges	100 year (Notes 3,4) Also see Chapter 9, Bridges
• Detention facilities	See Storage Chapter 10, Storage Facilities
• Pumping Stations	50 year - Expressways 25 year - Other roads

NOTES:

1. Depressed expressways will be analyzed for a fifty (50) year frequency and all other depressed roads for a twenty-five (25) year frequency. The floods from these storms will not be allowed to close the facility to traffic.

Depressed Expressways or Roadways are defined as sag vertical curves where water is removed only through a storm system. This does not apply to a sag vertical curve in a fill section where the water would escape over berms and curbs and down an embankment. The procedure is to design the drainage inlets and storm system for a 10 year frequency and then to impose the higher frequency storm on the inlets and storm system. If the higher frequency storm closes the facility to traffic then additional inlets and/or larger pipes will be required.

2. All watercourse channels shall be analyzed for the 2, 25, 50 and 100 year storms at a minimum. All perennial watercourse channels shall incorporate an inner channel to concentrate the 2 year design frequency and low flows. Certain activities, such as the construction of channel segments contiguous to flood improvement projects or channel construction adjacent to areas that must be protected from flooding could require a higher frequency for design. (See Chapter 7, Channels.)
3. All culvert/bridge designs generated using the designated frequencies shall be checked using a flood of greater magnitude to ensure that there are no unexpected flood hazards. For watershed > 2.59 km² (1 mi²) the 500 year flooding event will also be evaluated. For “critical activities” as defined in the Flood Management Regulations for State agencies (Section 25-68b-25-68h, CGS)

and for certain extremely large structures, the design storm frequency shall be the 500 year event. For watersheds $<2.59 \text{ km}^2$ (1 mi^2) which convey a watercourse the 100 year flooding event will be evaluated. (See Chapter 9, Bridges.)

4. Rating curves shall be developed to document the performance of the proposed facility over a full range of storm events including the highway overtopping discharge or frequency where practical.
5. If the cross culverts with watershed areas $<2.59 \text{ km}^2$ (1 mi^2) that do not convey a watercourse a 25 year frequency shall be used. (Also see Chapter 8, Culverts.)
6. The design frequency used for the design of bridges or culverts shall be used for any channel improvements associated with the design.

Appendix B**PRECIPITATION FREQUENCY ESTIMATES
FOR CONNECTICUT**

The Hydrometeorological Design Studies Center of the National Oceanic and Atmosphere Administration (NOAA) National Weather Service (NWS) released updated precipitation frequency estimates. Refer to [Engineering Bulletin EB-2015-2 dated November 3, 2015](#).

Appendix C - Time Of Concentration

6.C.1 Introduction

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

Following is a discussion of procedures and equations for calculating time of concentration and travel time.

6.C.2 Time Of Concentration

The time of concentration, which is denoted as T_c , is defined as the time required for a particle of runoff to flow from the hydraulically most distant point in the watershed to the outlet or design point. Factors that affect the time of concentration are the length of flow, the slope of the flow path, and the roughness of the flow path. For flow at the upper reaches of a watershed, rainfall characteristics, most notably the intensity, may also influence the velocity of the runoff.

The time of concentration equals the sum of the travel times on each segment of the principal flow path, accordingly, it is useful to describe the segments of flow paths. Sheet flow occurs in the upper reaches of a watershed. Such flow occurs over short distances and at shallow depths prior to the point where topography and surface characteristics cause the flow to concentrate in rills and swales. The depth of such flow is usually 20 to 30mm (3/4 in to 1 in) or less. Concentrated flow is runoff that occurs in rills and swales and has depths on the order to 40 to 100 mm (1.5 in to 4 in). Part of the principal flow path may include pipes or small streams. The travel time through these segments would be computed separately. Velocities in open channels are usually determined assuming bank-full depths.

The following equation represents the time of concentration which is the sum of the travel times (T_t) values for the various consecutive flow segments:

$$T_c = T_{t1} + T_{t2} + \dots T_{tm} \quad (6.C.1)$$

Where T_c = time of concentration, h
m = number of flow segments
 T_{tm} = travel time segment, h

6.C.3 Travel Time, T_t

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

6.C.4 Sheet-Flow Travel Time, T_t

Sheet flow is a shallow mass of runoff on a plane surface with the depth uniform across the sloping surface. **Typically flow depths will not exceed 30mm (1 in). Such flow occurs over relatively short distances, rarely more than about 91.4m (300 ft), but most likely less than 46m (150 ft).** Sheet flow rates are commonly estimated using the NRCS TR-55 (1986) variation of the kinematic wave equation:

$$T_t = \frac{0.091(nL)^{0.8}}{P_2^{0.5} S^{0.4}} \quad \left(T_t = \frac{0.007(nL)^{0.8}}{P_2^{0.5} S^{0.4}} \right) \quad (6.C.2)$$

Where T_t = travel time, h
 n = Manning's roughness coefficient (values of n can be obtained from Table C.1)
 L = flow length, m (ft)
 S = slope of the hydraulic grade line (land slope), m/m (ft/ft)
 P_2 = 2 year, 24 hour rainfall depth, mm (in) (See Table B-1.)

TR-55 recommends an upper limit of $L=91.4\text{m}$ (300 ft) for using Equation 6.C.2, although others have suggested that 91.4m (300 ft) is too long of a flow length for Connecticut so **engineering judgement should be used when selecting the flow length.**

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

$$T_t = L/(3600V) \quad (6.C.3)$$

Where: T_t = travel time, h
 L = flow length, m (ft)
 V = average velocity, m/s (ft/s)
 3600 = conversion factor from seconds to hours.

n	Surface Description
0.011	Smooth asphalt
0.012	Smooth concrete
0.05	Fallow (no residue)
	Cultivated soils
0.06	Residue cover = 20%
0.17	Residue cover >20%
0.13	Range (natural)
	Grass
0.15	Short grass prairie
0.24	Dense grasses
0.41	Bermuda grass
	Woods**
0.40	Light underbrush
0.80	Dense underbrush

Table C-1 Mannings's Roughness Coefficient (n) for Overland Sheet Flow*

* Values obtained from NRCS TR-55 (1986) and McCuen (1989).

** When selecting n for woody underbrush, consider cover to a height of about 25mm (1in). This is the only part of the plant cover that will obstruct sheet flow.

6.C.5 Shallow Concentrated Flow Travel Time

After a maximum of 91.4 m (300 ft), sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from equations 6.C.4 and 6.C.5, in which average velocity is a function of watercourse slope and type of channel.

$$\text{Unpaved} \quad V = 4.9178(s)^{0.5} \quad (V = 16.1345(s)^{0.5}) \quad (6.C.4)$$

$$\text{Paved} \quad V = 6.1961(s)^{0.5} \quad (V = 20.3284(s)^{0.5}) \quad (6.C.5)$$

Where: V = average velocity, m/s (ft/s)

s = slope of hydraulic grade line (watercourse slope), m/m (ft/ft)

These two equations are based on the solution of Manning's equation with different assumptions for n (Manning's roughness coefficient) and r (hydraulic radius, m (ft)). For unpaved areas, n is 0.05 and r is 0.12 m (0.4 ft); for paved areas, n is 0.025 and r is 0.06 m (0.2 ft).

After determining average velocity, use equation 6.C.3 to estimate travel time, T_t for the shallow concentrated flow segment.

6.C.6 Open Channel Flow Travel Time

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation but may change with respect to stream reach.

Manning's equation is

$$V = (r^{2/3} s^{1/2})/n \quad (V = (1.49r^{2/3} s^{1/2})/n) \quad (6.C.6)$$

where: V = average velocity, m/s (ft/s)

r = hydraulic radius, m (ft) (equal to a/p_w)

a = cross sectional flow area, m^2 (ft^2)

p_w = wetted perimeter, m (ft)

s = slope of the hydraulic grade line (watercourse slope), m/m (ft/ft)

n = Manning's roughness coefficient (see Appendix A of Chapter 8, Culverts and Table 7-1 of Chapter 7, Channels)

After average velocity is computed using equation 6.C.6, T_t for the channel segment can be estimated using equation 6.C.3.

6.C.7 Reservoir Or Lake Flow Travel Time

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

$$V_w = (gD_m)^{0.5} \quad (6.C.7)$$

Where: V_w = the wave velocity across the water, m/s (ft/s)

g = 9.81 m/s^2 (32.2 ft/s^2)

D_m = mean depth of lake or reservoir, m (ft)

Generally, V_w will be high 2.44 - 9.14 m/s (8-30 ft/s).

One must not overlook the fact that equation 6.C.7 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 generally much longer and 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 the Storage Chapter. Equation 6.C.7 can be used for swamps with much open water, but where the vegetation or debris is relatively thick (less than about 25% open water), Manning's equation is more appropriate. For additional discussion of equation 6.C.7 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.

After wave velocity is computed using equation 6.C.7, T_t for the reservoir or lake can be estimated using equation 6.C.3.

6.C.8 Limitations

- Manning's kinematic solution should not be used for sheet flow longer than 91.4 m (300 ft). Equation 6.C.2 was developed for use with the four standard rainfall intensity-duration relationships.
- In watersheds with storm drains, carefully identify the appropriate hydraulic flow path to estimate T_c . Storm drains generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or nonpressure flow.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert.

Time of Concentration Worksheet (Page 1 of 3)

Project No. _____ By _____ Date _____

Location _____ Checked _____ Date _____

Circle one: Present Developed

Circle one: metric units English units

NOTES: Space for as many as two segments per flow type can be used for each worksheet.
Must include a map showing travel time flow paths.

<u>Sheet flow (Applicable to T_c only)</u>		Segment ID			
1	Surface description (Table 6.C.1) _____				
2	Manning's roughness coeff., n (Table 6.C.1) _____				
3	Flow length, L m (ft) (See Section 6.C.4) _____				
4	Two-yr 24-hr rainfall, P_2 mm (in) (Table B-1) _____				
5	Land slope, s m/m (ft/ft) _____				
6	Travel Time, T_t h (Equation 6.C.2) _____		+		=

<u>Shallow concentrated flow</u>		Segment ID			
7	Surface description (paved or unpaved) _____				
8	Flow length, L m (ft) _____				
9	Watercourse slope, s m/m (ft/ft) _____				
10	Average velocity, V m/s (ft/s) (Equation 6.C.4 or 6.C.5) _____				
11	Travel Time, T_t h (Equation 6.C.3) _____		+		=

Time of Concentration Worksheet (Page 2 of 3)

<u>Channel flow</u>	Segment ID			
12 Cross sectional flow area, a	m ² (ft ²)			
13 Wetted perimeter, p _w	m (ft)			
14 Hydraulic radius, r = a/p _w	m (ft)			
15 Channel slope, s	m/m (ft/ft)			
16 Manning's roughness coeff., n (Table 7-1 & Append. A, Ch. 8)				
17 Average velocity, v (Equation 6.C.6)	m/s (ft/s)			
18 Flow length, L	m (ft)			
19 Travel Time, T _t (Equation 6.C.3)	h		+	=

Channel Section ID Schematic

Channel Section ID Schematic

Time of Concentration Worksheet (Page 3 of 3)

<u>Reservoir or lake flow</u>		Segment ID			
20	Mean depth of lake or reservoir, D_m	m (ft)			
21	Wave velocity across the water, V_w	m/s (ft/s)			
22	Travel time, T_t (Equation 6.C.3)	h		+	=
Total T_c = (add T_t in steps 6, 11, 19, and 22).....					

Appendix D – Hydrologic Design Report Format

6.D.1 Introduction

Hydrologic studies for all watersheds greater than 81 ha (200 acres) shall be documented using the following report formats. For smaller areas, the degree of report documentation shall be commensurate with the complexity of the associated hydraulic design.

6.D.2 Submittal

Preliminary hydrologic studies should be completed as early as possible during the design phase of a project. Studies completed should initially be documented in accordance with the report format and submitted for review by the Hydraulics and Drainage Section. Following this review, and upon incorporation of recommended revisions as necessary, the hydrologic study shall be completed and the design can start.

6.D.3 Hydrologic Design Report

The following shall be included in the draft and final reports:

1. Narrative to include stream name, general basin description, location of study site, hydrologic design method used, design frequency, etc.
2. Watershed area map showing overall basin boundary and subarea delineations. The scale and overall size of this map shall be as required for the watershed under study; the map may accompany the report as a separate item.
3. Complete listing of all input data with supporting computations and materials.
4. Tabulation of regression equation results or copy of computer model printout, as applicable. Computer printouts shall include complete input data and the resultant, unabbreviated output.
5. Flood frequency curves.
6. Information regarding the flood of record with associated high water marks, if available. Describe any unique circumstances which caused or contributed to the flood of record, such as an upstream dam break, high tailwater condition downstream, or debris blockage at the site.
7. Brief narrative summary of study results.
8. A copy of the computer disk(s).
9. The signature with Connecticut seal and professional engineer number of the approved hydraulic engineer who prepared or supervised the preparation of the report. The report shall also include the names identifying the responsible engineers who prepared and checked the work.

Appendix E – Major Flood Events in Connecticut Since 1927

Figure E-1 November 1927 Flood

Figure E-2 March 1936 Flood

Figure E-3 September 1938 Flood

Figure E-4 January 1949 Flood

Figure E-5 August 1955 Flood

Figure E-6 October 1955 Flood

Figure E-7 January 1978 Flood

Figure E-8 January 1979 Flood

Figure E-9 June 1982 Flood

Figure E-10 June 1984 Flood

Figure E-11 June 1992 Flood

Note:

These maps were developed from various sources of hydrologic data and are intended to provide a general overview of major flood events experienced in Connecticut. They are based on limited information and should be used as a tool to determine the magnitude of flood experienced in a particular region in the state.

It is strongly recommended that more current and detailed information be collected when studying a specific site along a stream reach. Stream flow data and floods of records are available through the USGS Water Resources Division.

Rainfall and/or gauge data identifying a flood event specific to a site may not be consistent with the general information presented in the flood maps. It is the more detailed, site specific information that should be used in lieu of the general information provided herein.

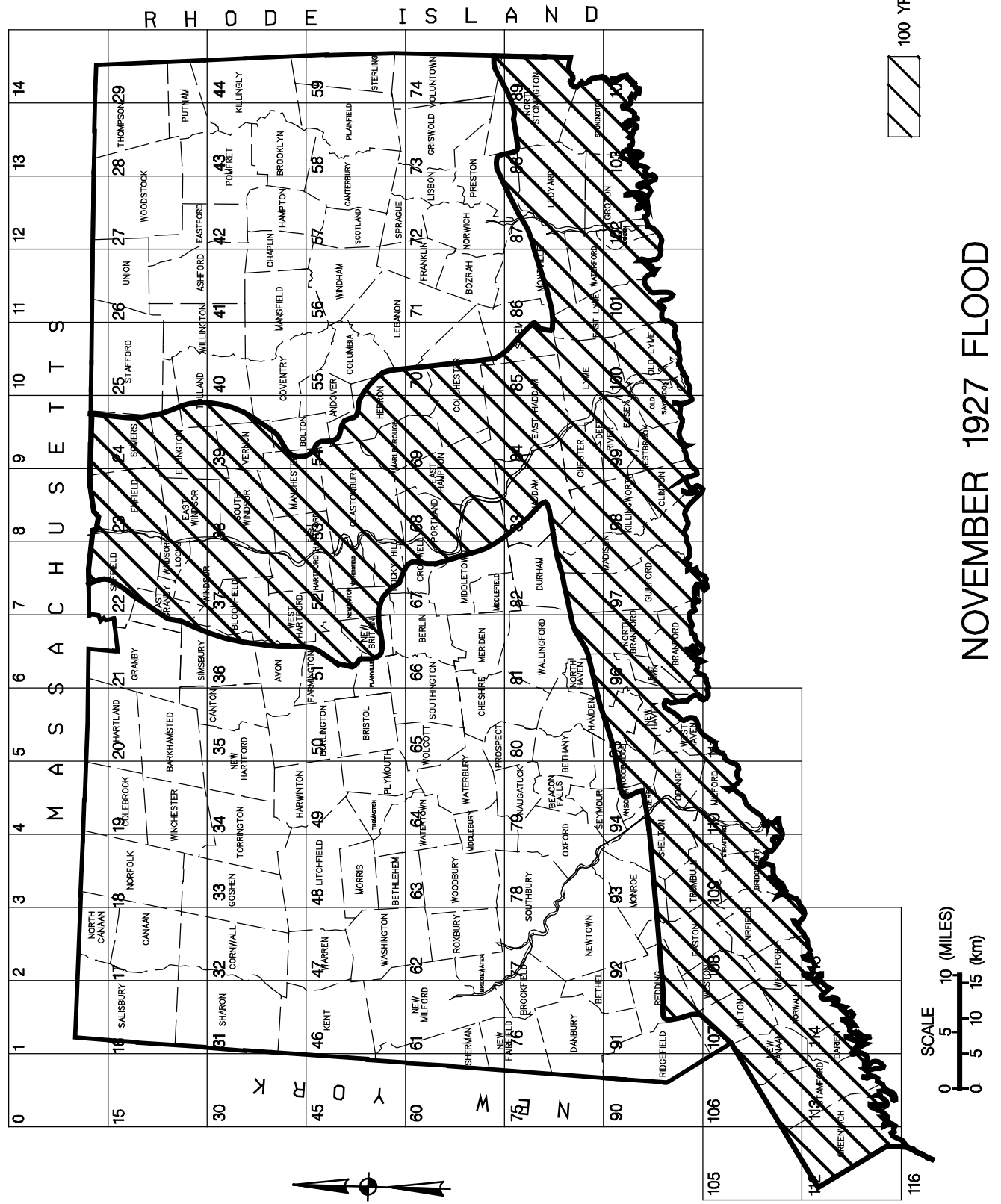


Figure E-1 November 1927 Flood

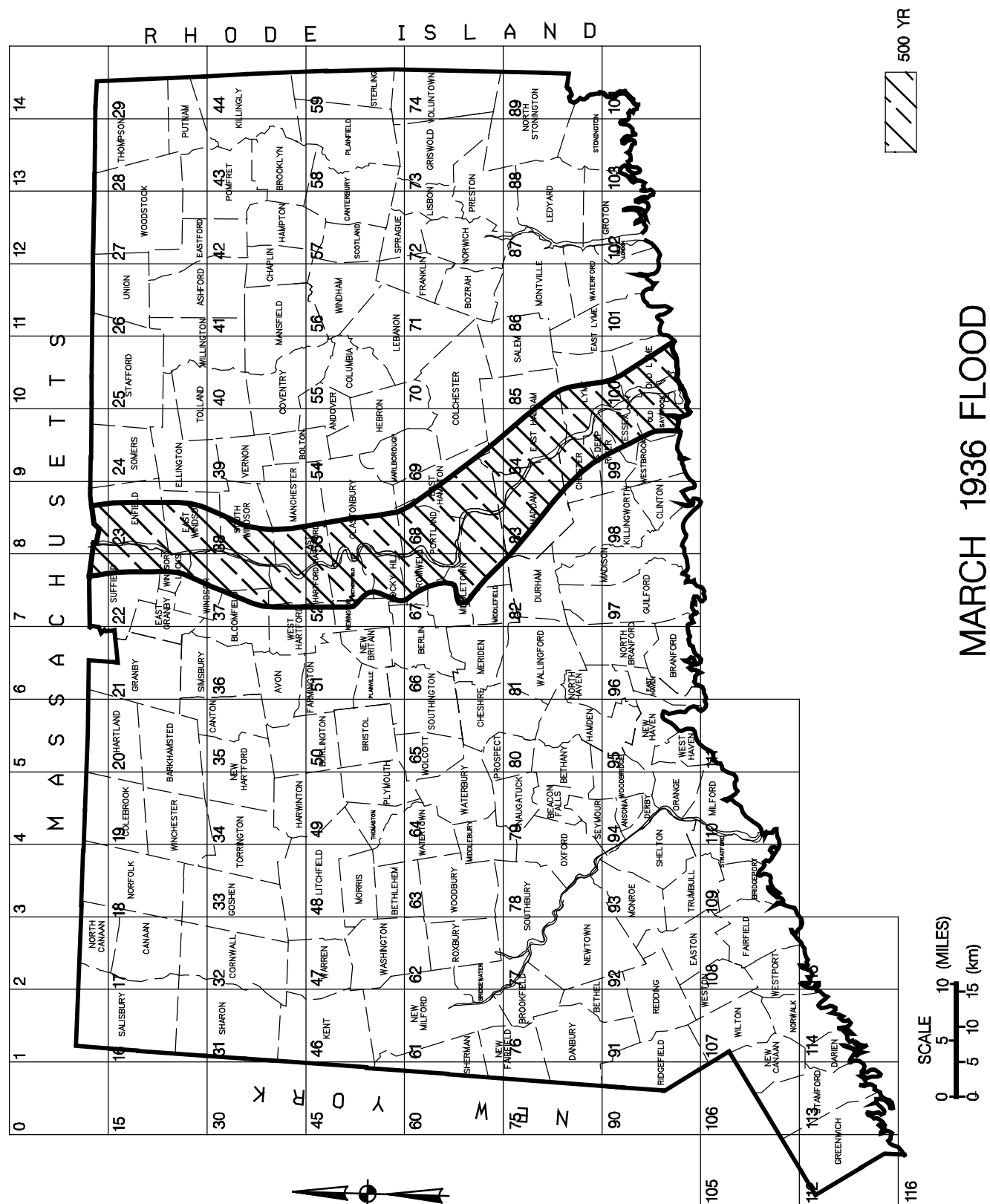


Figure E-2 March 1936 Flood

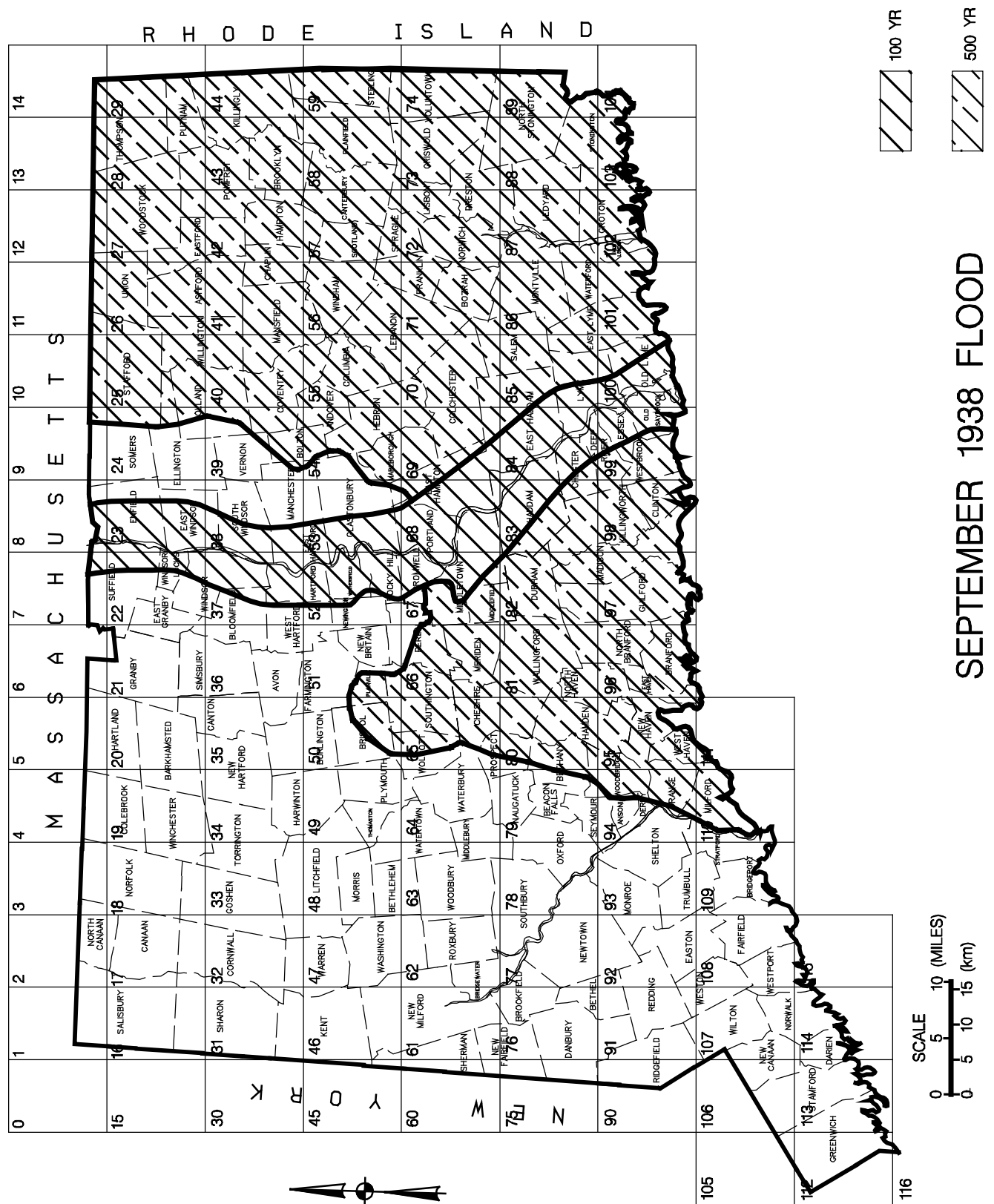
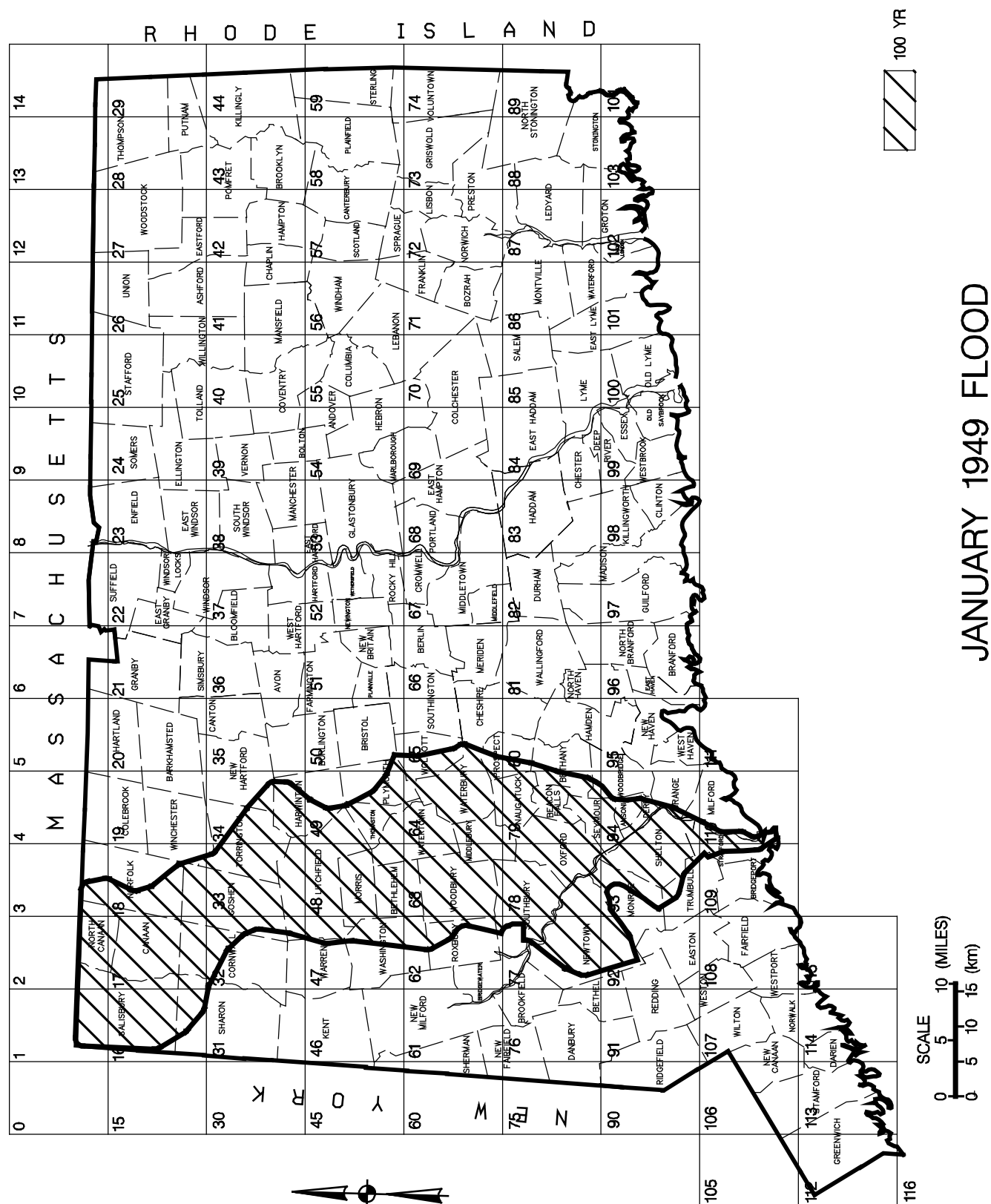


Figure E-3 September 1938 Flood





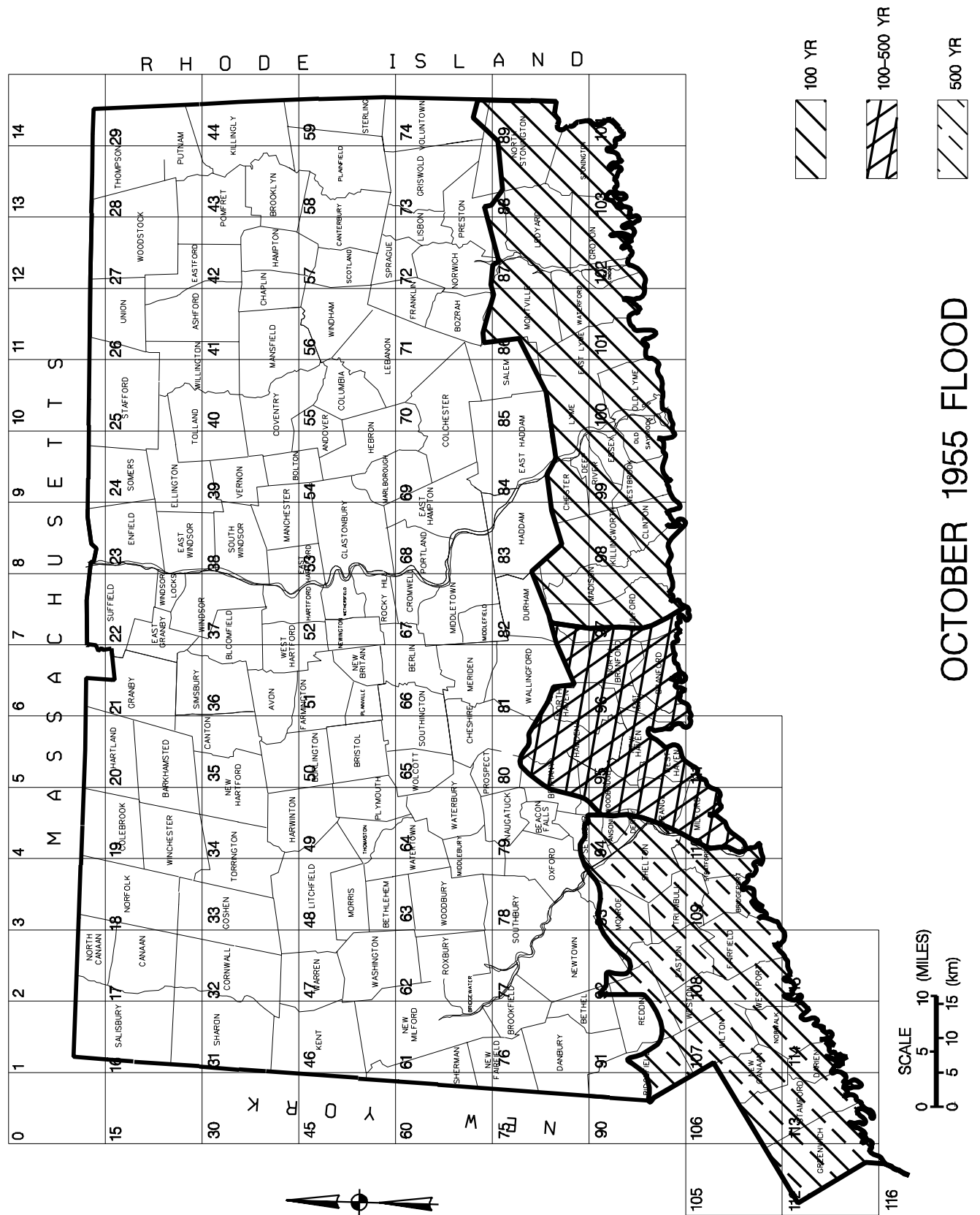


Figure E-6 October 1955 Flood

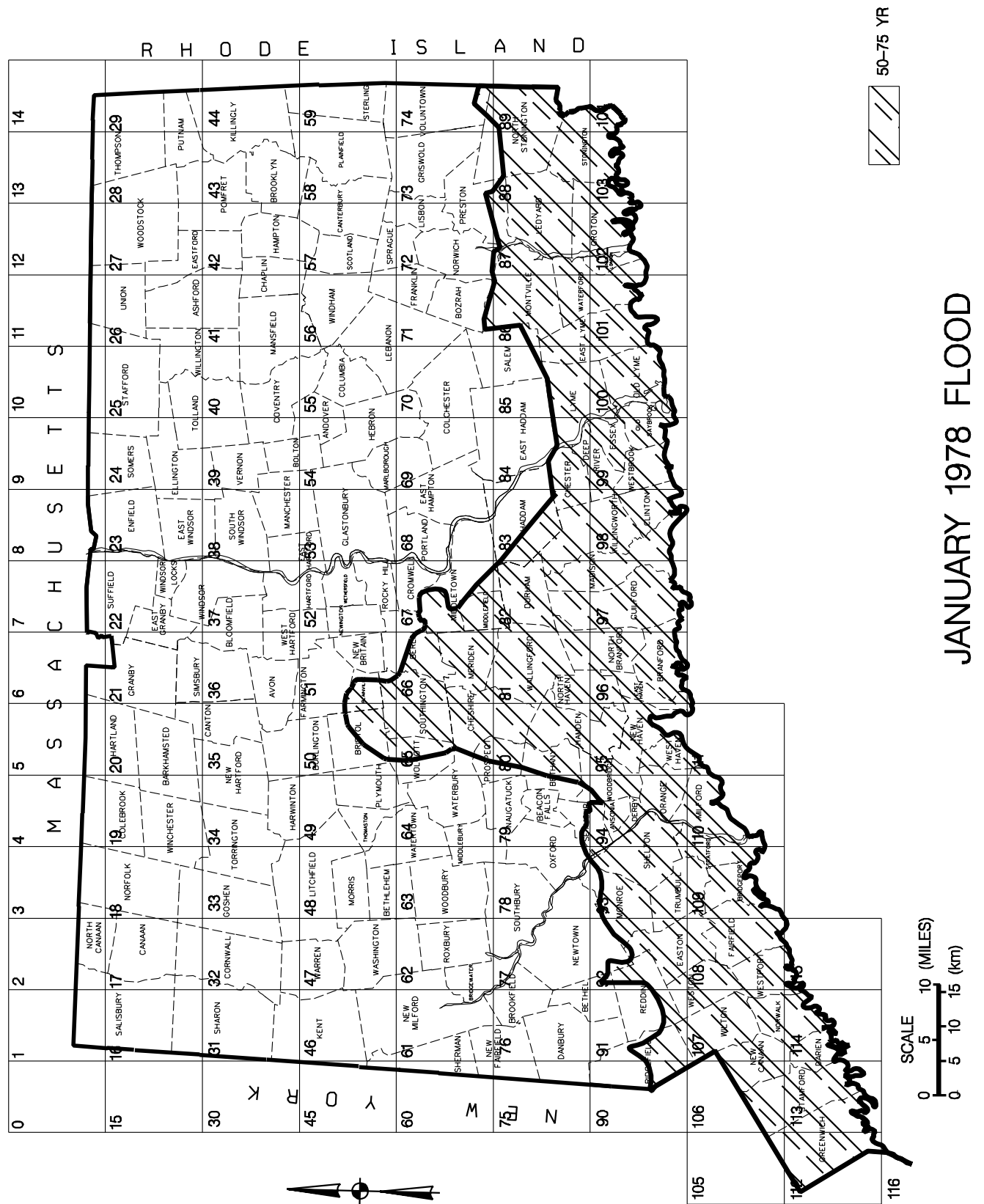


Figure E-7 January 1978 Flood

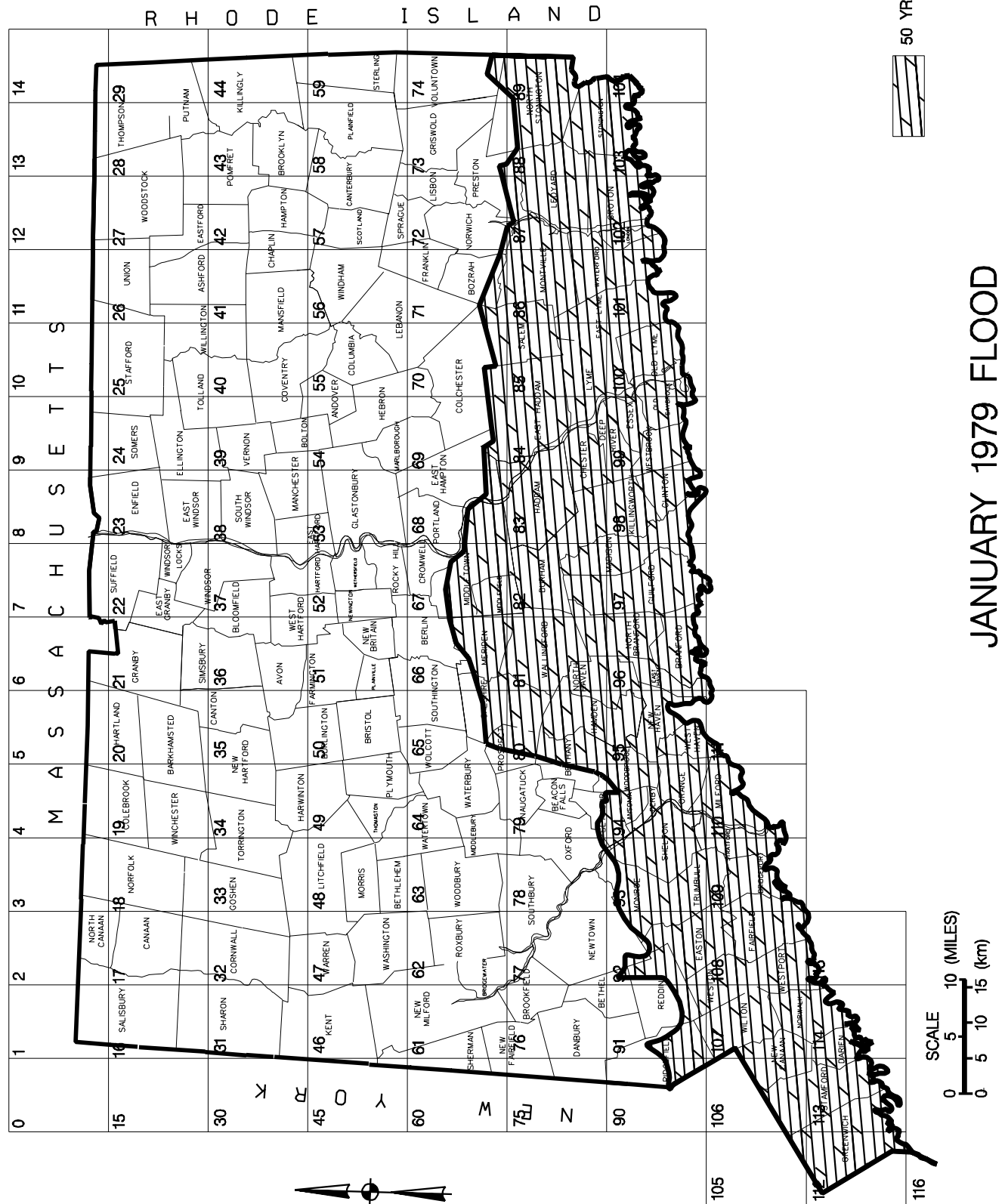


Figure E-8 January 1979 Flood



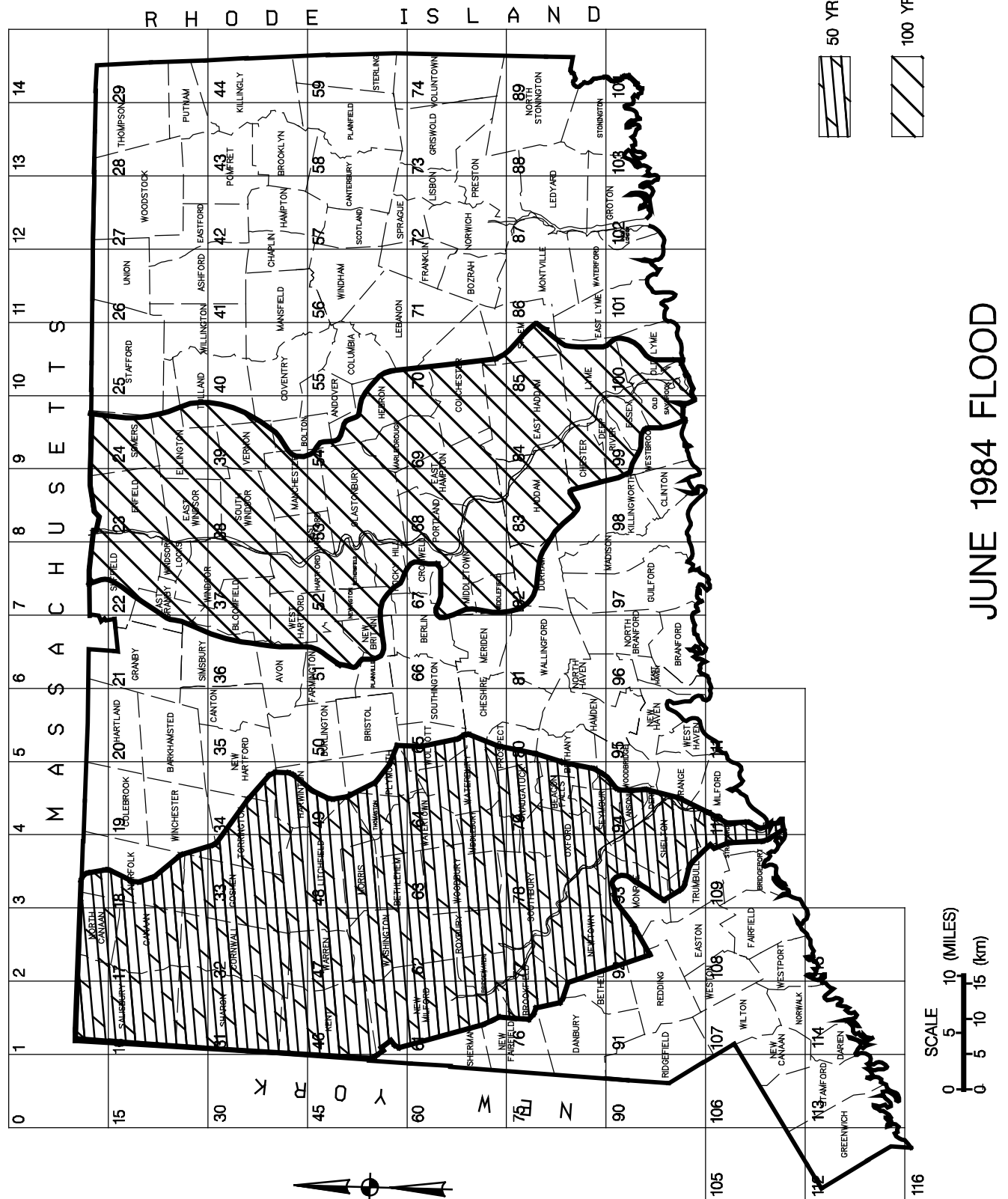


Figure E-10 June 1984 Flood

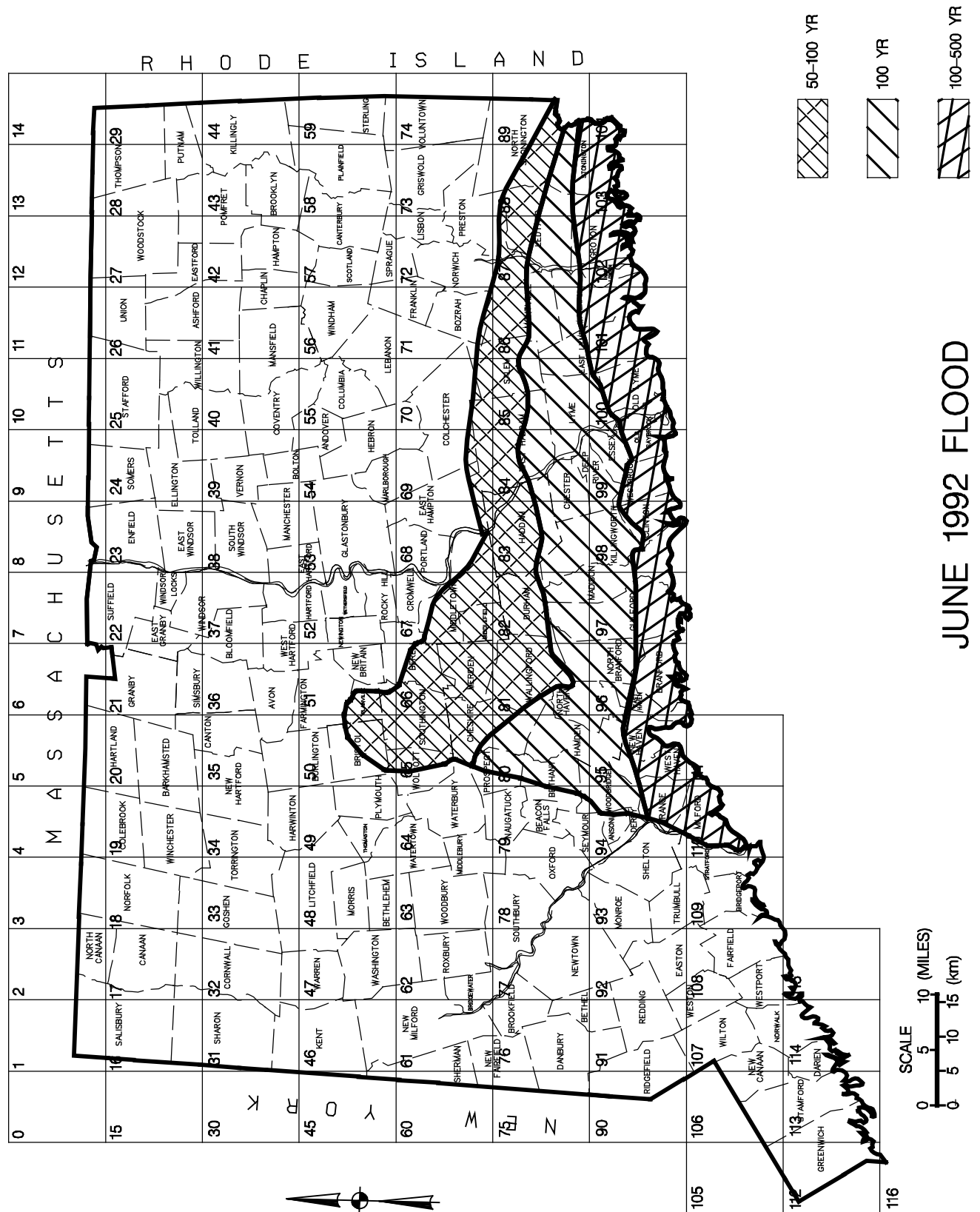


Figure E-11 June 1992 Flood

Appendix F – Hydrology for Temporary Facilities

Step 1: Determine Impact Ratings

The following selection factors are rated considering their severity as 1, 2, or 3 for low, medium or high conditions.

Potential Loss of Life - If inhabited structures, permanent or temporary, can be inundated or are in the path of a flood wave caused by an embankment failure, then this item will have a multiple of 15 applied. If no possibility of the above exists, then loss of life will be the same as the severity used for the A.D.T.

Property Damages - Private and public structures (houses, commercial, or manufacturing); appurtenances such as sewage treatment and water supply; utility structures either above or below ground, are to have a multiple of 10 applied. Active cropland, parking lots, recreational areas are to have a multiple of 5 applied. All other areas shall use the severity determined by site conditions.

Traffic Interruption - Includes consideration for emergency supplies and rescue; delays; alternate routes; busses; etc. Short duration flooding of a low volume roadway might be acceptable. If the duration of flooding is long (more than a day), and there is a nearby good quality alternate route, then the flooding of a higher volume highway might also be acceptable. The severity of this component is determined by the detour length multiplied by the average daily traffic projected for bi-directional travel.

Detour Length - The length in kilometers (miles) of an emergency detour by other roads should the temporary facility fail.

Height Above Streambed - The difference in elevation in meters (feet) between the traveled roadway and the bed of the waterway.

Drainage Area - The total area contributing runoff to the temporary facility, in km² (mi²).

Average Daily Traffic - The average amount of vehicles traveling bi-directional through the area in a 24-h period.

RATING SELECTION

<u>Factor</u>	<u>Rating</u>		
	1	2	3
Loss of Life	See Instructions		
Property Damage	See Instructions		
Traffic Interruptions	< 2000	2000-4000	> 4000
Detour Length, km (mi)	< 8 (< 5)	8-16 (5-10)	> 16 (> 10)
Height Above Streambed, m (ft)	< 3 (< 10)	3-6 (10-20)	> 6 (> 20)
Drainage Area, km ² (mi ²)	< 2.6 (< 1)	2.6-26.0 (1-10)	> 26.0 (> 10)
Rural ADT	< 400	400-1500	> 1500
Suburban ADT	< 750	750-1500	> 1500
Urban ADT	< 1500	1500-3000	> 3000

IMPACT RATING TABLE

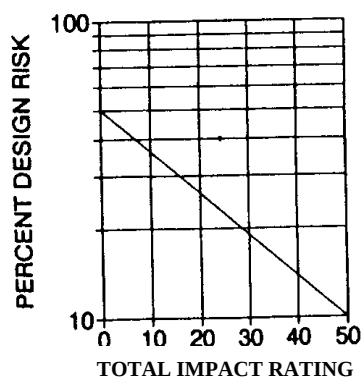
Loss of Life Rating (See Instructions)= _____
 Property Damage Rating (See Instructions) = _____
 Traffic Interruption Rating = _____
 Detour Length Rating = _____
 Height Above Streambed Rating = _____
 Drainage Area Rating = _____
 Average Daily Traffic Rating = _____

 Total Impact Rating = (sum of the above) = _____

Step 2: Determine risk percentage

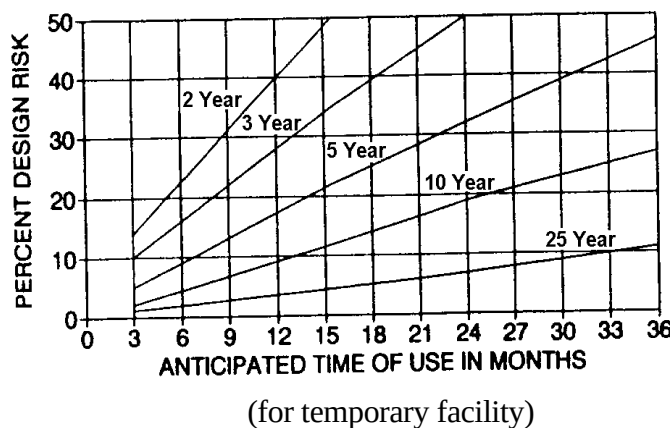
Step 3: Determine Temporary Design Frequency

DESIGN RISK VS. IMPACT RATING



Percent Design Risk = _____

DESIGN FREQUENCY (YEAR)



(for temporary facility)

Design Frequency = _____ years

Step 4: Determine Temporary Design Discharge

- A. If sufficient discharges have been developed either by the designer or a Flood Insurance Study, then the Temporary Design Discharge should be taken either directly or from a frequency curve plot of the data, based on the design frequency determined in Step 3. Enter the Temporary Design Discharge below. *If Discharge – Frequency information is unavailable, proceed to Step 4 B.*

 Temporary Design Discharge = _____ m^3/s (_____ cfs)

B. Use only when Discharge – Frequency information is unavailable

(1) Determine Multiplier Ratio

<u>Year</u>	<u>Multiplier</u>	<u>Year</u>	<u>Multiplier</u>
2.0	0.8	10.0	1.9
3.0	1.2	25.0	2.7
5.0	1.4		

(2) Compute the Temporary Design Discharge from the following equations

$$\text{Multiplier } \underline{\hspace{1cm}} \times 0.27 (Q_{50 \text{ yr. } \underline{\hspace{1cm}}}) = \underline{\hspace{1cm}} \text{ m}^3/\text{s} (\hspace{1cm} \text{cfs})$$

$$\text{Multiplier } \underline{\hspace{1cm}} \times 0.20 (Q_{100 \text{ yr. } \underline{\hspace{1cm}}}) = \underline{\hspace{1cm}} \text{ m}^3/\text{s} (\hspace{1cm} \text{cfs})$$

(3) Select the higher of the two discharges computed in Step 4B-(2). Enter discharge below.

Temporary Design Discharge = **m³/s (** **cfs)**

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7.1 Introduction

7.1.1 Purpose

The purpose of this chapter is to establish Department practice, specify design criteria, review design philosophy, and outline channel design procedures.

Hydraulic design associated with natural channels and roadway ditches is a process which selects and evaluates alternatives according to established criteria. These criteria are the standards established by the Connecticut Department of Transportation to insure that a highway facility meets its intended purpose without endangering the structural integrity of the facility itself and without undue adverse effects on the environment or the public welfare.

7.1.2 Definitions

- Open channels are a **natural** or **artificial** conveyance for water in which:
 - the water surface is exposed to the atmosphere
 - the gravity force component in the direction of motion is the driving force

There are **various types of open channels** encountered by the designer of transportation facilities:

- natural channel
- roadside channel or ditch
- drainage ditch or swale
- **Natural watercourse channels** are:
 - dynamic channels with their size and shape determined by natural forces
 - usually but not always are compound in cross section with a main channel for conveying low flows and a floodplain to transport flood flows
 - shaped geomorphologically by the long term history of sediment load and water discharge which they experience
- **Artificial channels** include roadside channels, drainage ditches and swales which:
 - have regular geometric cross sections (although this is not always the case for overbank areas) and
 - are unlined, or lined with artificial or natural material to protect against erosion

While the principles of open channel flow are the same regardless of the channel type, stream channels and artificial channels (primarily roadside channels) will be treated separately in this chapter as needed.

7.1.3 Significance

Channel analysis is necessary for the design of transportation drainage systems in order to assess:

- potential flooding caused by changes in water surface profiles
- disturbance of the river system upstream or downstream of the highway right-of-way
- changes in lateral flow distributions
- changes in velocity or direction of flow
- need for conveyance and disposal of excess runoff
- need for channel lining to prevent erosion

7.2 ConnDOT Design Practices

7.2.1 General

- Channel designs and/or designs of highway facilities that impact channels shall satisfy the policies of the Federal Highway Administration applicable to flood plain management if Federal funding is involved.
- Coordination with Federal, State, and local agencies concerned with water resources planning shall have high priority in the planning and design of highway facilities.
- Safety of the general public shall be an important consideration in the selection of cross-sectional geometry of artificial drainage channels.
- 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 stream bank stability shall be assessed.

7.2.2 ConnDEP Requirements

The following definitions and design criteria for open channels taken from Section 25-68h-3(d)(2) through (6) of the **Flood Management Statutes and Administrative Regulations** shall be incorporated into all ConnDOT projects:

- **Open Channels.**

The analysis and design of open channels shall be consistent with the type of channel and its intended purpose. Channels shall be classified as local drainage channels or as watercourse channels, depending on use, and shall be classified as alluvial or non-alluvial based upon their geologic characteristics.

- **Type A open channels** are local drainage channels with a primary purpose of conveying urban, parking lot and road runoff from small watersheds, frequently with intermittent flow and limited ecological value and are intended to convey their design flow within their banks. The following shall be included as required:

- Freeboard allowances shall be provided in proportion to the potential damages that could occur in the event of overtopping
- The use of impervious linings is discouraged except for very high velocity flow and steep slopes

- **Type B open channels** are natural perennial watercourses or man made channels planned to simulate a natural watercourse. The following shall be included where appropriate:

- Shall have minimum flow capacity of a flood equal to at least 25 year frequency flood.
- Shall have an inner channel to concentrate low flows with a capacity of a 2 year frequency flood.
- Shall have water surface profiles prepared for the 2, 25, and 100 year frequency floods.
- Shall consider the hydraulic capacity of floodplains.
- Shall have a sediment transport capacity similar to upstream and downstream channels.

- Shall be designed to minimize the use of artificial linings for flows in excess of the two year frequency flood.
 - Shall encourage ecological productivity and variety.
 - Shall be visually compatible with its surroundings.
 - The alignment and slope shall be compatible with natural channels in similar site conditions.
 - Variations in width, depth, invert elevations, and side slopes are encouraged for aquatic and visual diversity.
 - Straightening channels and decreasing their length is discouraged.
 - The cross sections used to define the channel and floodplain geometry for water surface profile computations shall be located upstream and downstream of hydraulic structures, at changes in bed slope or cross section shape, and generally at intervals of not more than ten times the width of the 100 year floodplain.
 - The friction coefficients used in the hydraulic analysis are to assume maximum seasonal vegetation conditions, and should be adjusted to the depth of flow.
- **Channel restoration** plans shall be prepared for all open channel work. The plan shall help restore and/or create an aquatic habitat suitable for fisheries, while maintaining or improving water quality, recreation, aesthetics and flow capacity. Coordination with the Fisheries and Wildlife Units of ConnDEP is recommended. The channel restoration plan shall include, as appropriate:
- Avoidance of barriers to fish movement
 - Formation of pools and riffles
 - Provision for areas of sheltered flow with use of deflectors, boulders, low check dams
 - Preservation of stream bank vegetation and establishment of new vegetation
 - Use of clean bed materials of a suitable size
 - Schedule work to minimize conflicts with spawning, stocking, and fishing seasons
 - Removal of excess debris
- **The design of rock riprap** in channels with uniform flow shall be based upon the tractive force methods defined in both this manual and the Connecticut Guidelines for Erosion and Sediment Control.
- The hydraulic analysis and modification of watercourses **prone to ice jams or floods due to ice** should be coordinated directly with the ConnDEP.
- **The water surface profiles of open channels in coastal areas** shall consider the potential combined occurrence of tides, storm surges, and peak runoff. The starting water elevation for the base flood in watersheds with times of concentration of over 6 hours shall be the ten year frequency tidal surge level.

7.3 Design Criteria

7.3.1 General

Design criteria establish the standards by which practices are placed into action. They form the basis for the selection of the final design configuration. Listed below are examples of design criteria which shall be considered for channel design.

7.3.2 Roadside Channels (Type A)

The following criteria apply to roadside channels:

- Channel side slopes shall not exceed the angle of repose of the soil and/or lining and shall be 1V:2H or flatter in the case of rock-riprap lining and 1V:3H in the case of vegetative lining.
- Flexible linings shall be designed according to the method of allowable tractive force.
- The design discharge for permanent roadside ditch linings shall have a 10-year frequency while temporary linings shall be designed for the 2-year frequency flow.
- Channel freeboard shall be 0.3 m (1 ft) or two velocity heads, whichever is larger.
- ConnDEP requirements for Type A channels shall be included in the design (see Section 7.2.2).

7.3.3 Natural Channels (Type B)

The following criteria applies to watercourse channels.

- If relocation of a natural channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, capacity, channel slope, and side slope shall conform to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.
- Streambank stabilization shall be provided, when appropriate, as a result of any stream disturbance or encroachment and shall include both upstream and downstream banks when required, as well as the local site.
- ConnDEP requirements for Type B channels shall be included in the design (see Section 7.2.2).

7.4 Open Channel Flow

7.4.1 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 principle 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 artificial channels.

7.4.2 Specific Energy

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

$$E = y + \alpha (V^2/2g) \quad (7.1)$$

Where: y = depth, m (ft)

α = velocity distribution coefficient (see equation 7.2)

V = mean velocity, m/s (ft/s)

g = gravitational acceleration, 9.81 m/s² (32.2 ft/s²)

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

7.4.3 Velocity Distribution Coefficient

Due to the presence of a free surface and also due to friction along the channel boundary, the velocities in a channel are not uniformly distributed in the channel section. As a result 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:

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

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, m² (ft²)

A_t = total cross-sectional area of section, m² (ft²)

n = number of subsections

7.4.4 Total Energy Head

The total energy head is the specific energy head plus the elevation of the channel bottom with respect to some datum. The locus of the energy head from one cross section to the next defines the energy grade line. See Figure 7-1 for a plot of the specific energy diagram.

7.4.5 Steady and Unsteady Flow

A steady flow is one in which the discharge passing a given cross section is constant with respect to time. The maintenance of steady flow in any reach requires that the rates of inflow and outflow be constant and equal. When the discharge varies with time, the flow is unsteady.

7.4.6 Uniform Flow and Nonuniform Flow

A nonuniform flow is one in which the velocity and depth vary in the direction of motion, while they remain constant in uniform flow. Uniform flow can only occur in a prismatic channel, which is a channel of constant cross section, roughness and slope in the flow direction; however, nonuniform flow can occur either in a prismatic channel or in a natural channel with variable properties.

7.4.7 Gradually-Varied and Rapidly-Varied

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

7.4.8 Froude Number

The Froude number, Fr , is an important dimensionless parameter in open channel flow. It represents the ratio of inertial forces to gravitational forces and is defined by:

$$Fr = V / [(gd/\alpha)^{0.5}] \quad (7.3)$$

Where: α = velocity distribution coefficient

V = mean velocity = Q/A , m/s (ft/s)

g = acceleration of gravity, 9.81 m/s^2 (32.2 ft/s^2)

d = hydraulic depth = A/T , m (ft)

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

T = channel topwidth at the water surface, m (ft)

Q = total discharge, m^3/s (ft^3/s)

This expression for Froude number applies to any single section channel. For rectangular channels the hydraulic depth is equal to the flow depth.

7.4.9 Critical Flow

Critical flow occurs when the specific energy is a minimum. 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 also the depth of maximum discharge when the specific energy is held constant. These relationships are illustrated in Figure 7-1. During critical flow the velocity head is equal to half the hydraulic depth. The general expression for flow at critical depth is:

$$\alpha Q^2/g = A^3/T \quad (7.4)$$

Where: α = velocity distribution coefficient

Q = total discharge, m^3/s (ft^3/s)

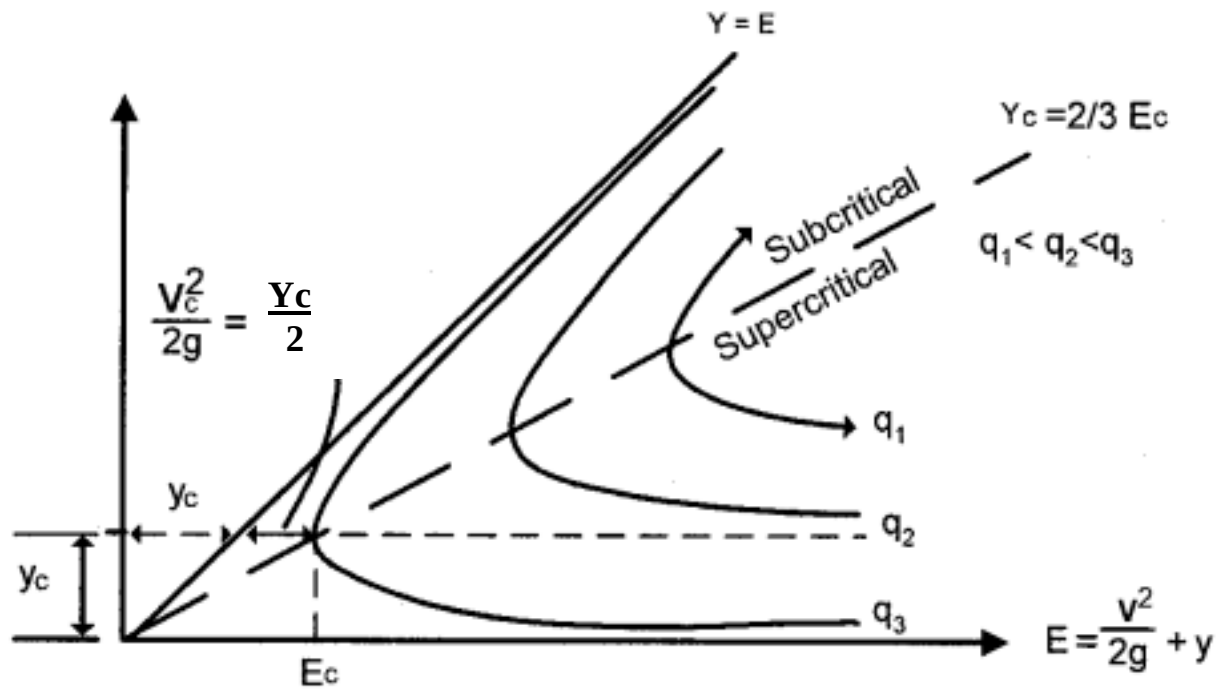
g = gravitational acceleration, $9.81 m/s^2$ ($32.2 ft/s^2$)

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

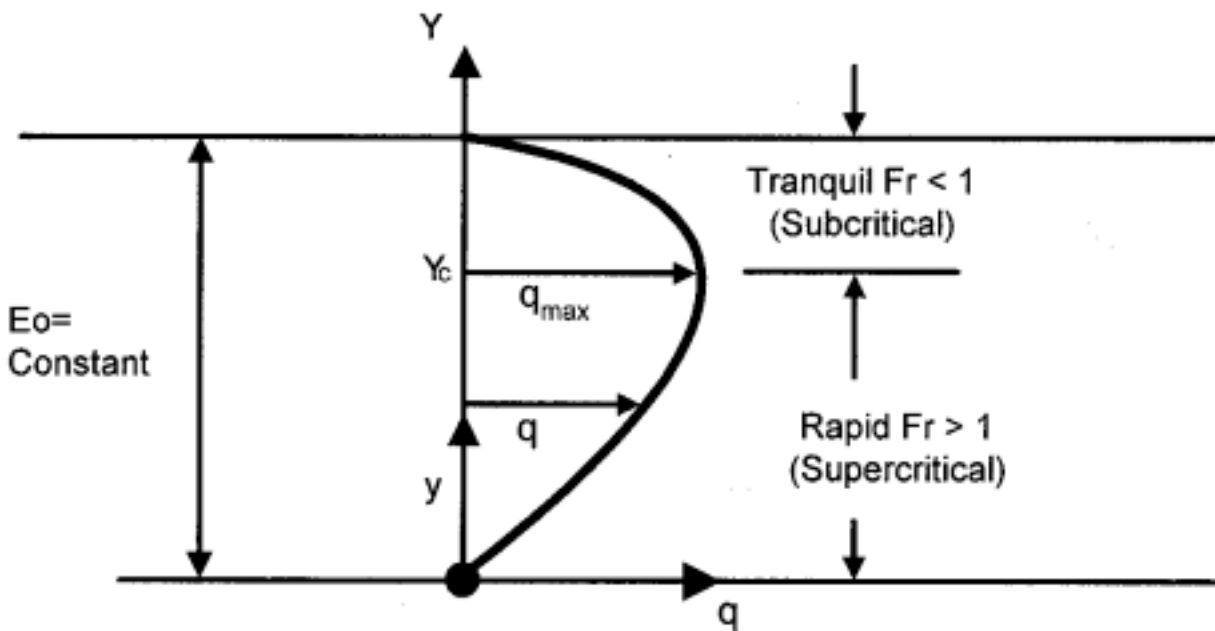
T = channel topwidth at the water surface, m (ft)

When flow is at critical depth, equation 7.4 must be satisfied, no matter what the shape of the channel.

- Subcritical Flow - Depths greater than critical occur in subcritical flow and the Froude number is less than one. In this state of flow, small water surface disturbances can travel both upstream and downstream, and the control is always located downstream.
- Supercritical Flow - Depths less than critical depth occur in supercritical flow and the Froude number is greater than one. Small water surface disturbances are always swept downstream in supercritical flow, and the location of the flow control is always upstream.
- Hydraulic Jump - A hydraulic jump occurs as an abrupt transition from supercritical to subcritical flow in the flow direction. There are significant changes in depth and velocity in the jump, and energy is dissipated. For this reason, **the hydraulic jump is often employed to dissipate energy and control erosion at highway drainage structures. The location of an induced hydraulic jump in a natural channel shall always be at a point which is remote from any bridge crossing.**



(a) Specific Energy Diagram



(b) Discharge Diagram

Figure 7-1 Specific Energy And Discharge Diagram For Rectangular Channels

(Adopted From Highways In The River Environment)

7.4.10 Flow Classification

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

- Steady Flow
 1. Uniform Flow
 2. Nonuniform Flow
 - a. Gradually Varied Flow
 - b. Rapidly Varied Flow
- Unsteady Flow
 1. Unsteady Uniform Flow (rare)
 2. Unsteady Nonuniform Flow
 - a. Gradually Varied Unsteady Flow
 - b. Rapidly Varied Unsteady Flow

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

7.4.11 Equations

The following equations are those most commonly used to analyze open channel flow. The use of these equations in analyzing open channel hydraulics is discussed in Section 7.7.

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

$$Q = A_1 V_1 = A_2 V_2 \quad (7.5)$$

Where: Q = discharge, m^3/s (ft^3/s)

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

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

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

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

$$V = (1/n)R^{2/3}S^{1/2} \quad (V = (1.49/n)R^{2/3}S^{1/2}) \quad (7.6)$$

Where: V = velocity, m/s (ft/s)

n = Manning's roughness coefficient

R = hydraulic radius = A/P , m (ft)

P = wetted perimeter, m (ft)

S = slope of the energy gradeline, m/m (ft/ft) (Note: For steady uniform flow, S = channel slope)

The selection of Manning's n is generally based on observation; however, considerable experience is essential in selecting appropriate n values. The range of n values for various types of channels and floodplains is given in Table 7-1.

The continuity equation can be combined with Manning's equation to obtain the steady, uniform flow discharge as:

$$Q = (1/n)AR^{2/3}S^{1/2} \quad (Q = (1.49/n)AR^{2/3}S^{1/2}) \quad (7.7)$$

For a given channel geometry, slope and roughness, and a specified value of discharge Q , a unique value of depth occurs in steady, uniform flow. It is called normal depth and 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 7.7.6 for a more detailed discussion of the computation of normal depth.

If the normal depth is greater than critical depth, the slope is classified as a mild slope, while on a steep slope, the normal depth is less than critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

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

$$K = (1/n)AR^{2/3} \quad (K = (1.49/n)AR^{2/3}) \quad (7.8)$$

and then equation 7.7 can be written as:

$$Q = KS^{1/2} \quad (7.9)$$

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 (see equation 7.2)

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

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

Where: h_1 and h_2 are the upstream and downstream stages, respectively, m (ft)

α = velocity distribution coefficient (equation 7.2)

V = mean velocity, m/s (ft/s)

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

The stage h is the sum of the elevation head z at the channel bottom 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 7-2. The energy equation states that the total energy head at an upstream cross section is equal to the energy head at a downstream section plus the intervening energy head loss. The energy equation can only be applied between two cross sections at which the streamlines are nearly straight and parallel so that vertical accelerations can be neglected.

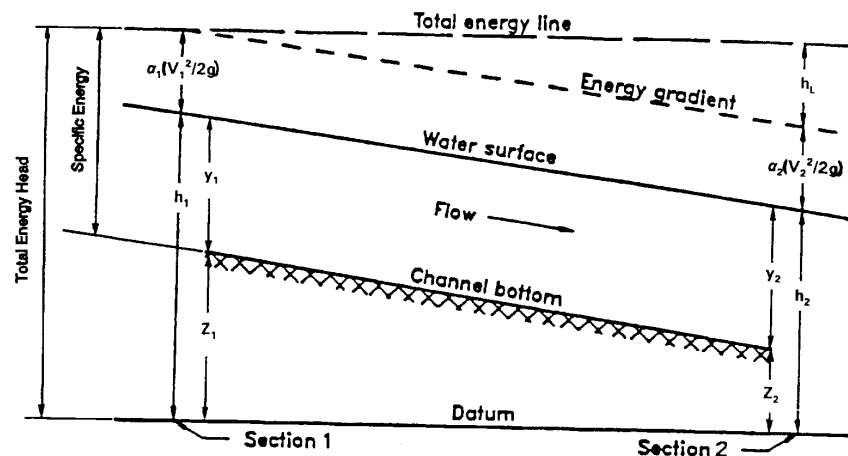


Figure 7-2 Terms In The Energy Equation

Source: FHWA, 1990

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

Type Of Channel And Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.025	0.030	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAMS			
1. Minor streams (top width at flood stage < 30 m)			
a. Streams on Plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones/weeds	0.030	0.035	0.040
3. Clean, winding, some pools/shoals	0.033	0.040	0.045
4. Same as above, but some weeds/stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150

Table 7-1 Values of Roughness Coefficient n (Uniform Flow) (continued)

Type Of Channel and Description	Minimum	Normal	Maximum
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
2. Flood Plains			
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	0.070	0.100	0.160
d. Trees			
1. Dense Willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of 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 > 30 m). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	—	0.060
b. Irregular and rough section	0.035	—	0.100

Source: Chow, V.T.

7.5 Stream Morphology

7.5.1 Introduction

The form assumed by a natural stream, which includes its cross-sectional shape as well as its planform, 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 bedforms 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 planform 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.

Equilibrium sediment load calculations can be made by a variety of techniques and compared from reach to reach to detect an imbalance in sediment inflow and outflow and thus identify an aggradation/degradation problem. The BRI-STARS model (see Section 7.7.8) is recommended as a tool. The proposed methodology to quantify the expected scour and/or sedimentation of potential problem locations should be approved by the Hydraulics and Drainage Section.

The natural stream channel will assume a geomorphological form which will be compatible with the sediment load and discharge history which it has experienced over time. To the extent that a highway structure disturbs this delicate balance by encroaching on the natural channel, the consequences of flooding, erosion and deposition can be significant and widespread. The hydraulic analysis of a proposed highway structure should include a consideration of the extent of these consequences.

7.5.2 Levels Of Assessment

The analysis and design of a stream channel will usually require an assessment of the existing channel and the potential for problems as a result of the proposed action. The detail of studies necessary should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream. Observation is the best means of identifying potential locations for channel bank erosion and subsequent channel stabilization. Analytical methods for the evaluation of channel stability can be classified as either hydraulic or geomorphic, and it is important to recognize that these analytical tools should only be used to substantiate the erosion potential indicated through observation. A brief description of the three levels of assessment are as follows.

Level 1

Qualitative assessment involving the application of geomorphic concepts to identify potential problems and alternative solutions. Data needed may include historic information, current site conditions, aerial photographs, old maps and survey notes, bridge design files, maintenance records and interviews with long-time residents.

Level 2

Quantitative analysis combined with a more detailed qualitative assessment of geomorphic factors. Generally includes water surface profile and scour calculations. This level of analysis will be adequate for most locations if the problems are resolved and relationships between different factors affecting stability are adequately explained. Data needed will include Level 1 data in addition to the information needed to establish the hydrology and hydraulics of the stream.

Level 3

Complex quantitative analysis based on detailed mathematical modeling and possibly physical hydraulic modeling. Necessary only for high risk locations, extraordinarily complex problems and possibly after the fact analysis where losses and liability costs are high. This level of analysis may require professionals experienced with mathematical modeling techniques for sediment routing (see Section 7.7.8) and/or physical modeling. Data needed will require Level 1 and 2 data as well as field data on bed load and suspended load transport rates and properties of bed and bank materials such as size, shape, gradation, fall velocity, cohesion, density and angle of repose.

7.5.3 Factors That Affect Stream Stability

Factors that affect stream stability and, potentially, bridge and highway stability at stream crossings, can be classified as geomorphic factors and hydraulic factors.

Geomorphic Factors

- Stream size
- Valley setting
- Natural levees
- Sinuosity
- Width variability
- Bar development
- Bed material
- Flow habit
- Floodplains
- Apparent incision
- Channel boundaries
- Degree of braiding
- Degree of anabranching
- Tree cover on banks

Figure 7-3 depicts examples of the various geomorphic factors.

Hydraulic Factors

- Magnitude, frequency and duration of floods.
- Bed configuration.
- Resistance to flow.
- Water surface profiles.

Figure 7-4 depicts the changes in channel classification and relative stability as related to hydraulic factors.

Rapid and unexpected changes may occur in streams in response to man's activities in the watershed such as alteration of vegetative cover. Changes in perviousness can alter the hydrology of a stream, sediment yield and channel geometry. Channelization, stream channel straightening,

stream levees and dikes, bridges and culverts, reservoirs and changes in land use can have major effects on stream flow, sediment transport and channel geometry and location. Knowing that man's activities can influence stream stability can help the designer anticipate some of the problems that can occur.

Natural disturbances such as floods, drought, earthquakes, landslides, and forest fires can also cause large changes in sediment load and thus major changes in the stream channel. Although difficult to plan for such disturbances, it is important to recognize that when natural disturbances do occur, it is likely that changes will also occur to the stream channel.

7.5.4 Stream Response To Change

The major complicating factors in river mechanics are: 1) the large number of interrelated variables that can simultaneously respond to natural or imposed changes in a stream system; and 2) the continual evolution of stream channel patterns, channel geometry, bars and forms of bed roughness with changing water and sediment discharge. In order to better understand the responses of a stream to the actions of man and nature, a few simple hydraulic and geomorphic concepts are presented herein.

The dependence of stream form on slope, which may be imposed independently of other stream characteristics, is illustrated schematically in Figure 7-5.

Any natural or artificial change which alters channel slope can result in modifications to the existing stream pattern. For example, a cutoff of a meander loop decreases channel sinuosity and increases channel slope. Referring to Figure 7-5, this shift in the plotting position to the right could result in a shift from a relatively tranquil, meandering pattern toward a braided pattern that varies rapidly with time, has high velocities, is subdivided by sandbars and carries relatively large quantities of sediment. Conversely, it is possible that a slight decrease in slope could change an unstable braided stream into a meandering one.

The different channel dimensions, shapes and patterns associated with different quantities of discharge and amounts of sediment load indicate that as these independent variables change, major adjustments of channel morphology can be anticipated. Further, a change in hydrology may cause changes in stream sinuosity, meander wave length, and channel width and depth. A long period of channel instability with considerable bank erosion and lateral shifting of the channel may be required for the stream to compensate for the hydrologic change.

Figure 7-6 illustrates the dependence of river form on channel slope and discharge, showing when $SQ^{0.25} \leq 0.00070$ in a sandbed channel, the stream will meander. Similarly, when $SQ^{0.25} \geq 0.0041$, the stream is braided.

In these equations, S is the channel slope in meters per meter (ft per ft) and Q is the mean discharge in cubic meters per second (cubic feet per second). Between these values of $SQ^{0.25}$ is the transitional range.

Many U.S. rivers plot in this zone between the limiting curves defining meandering and braided streams. If a stream is meandering but its discharge and slope border on a boundary of the transitional zone, a relatively small increase in channel slope may cause it to change, in time, to a transitional or braided stream.

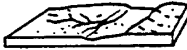
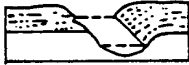
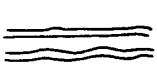
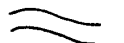
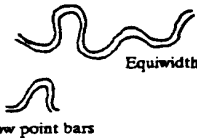
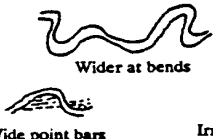
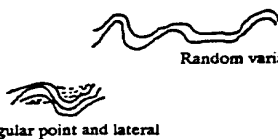
STREAM SIZE	Small (< 30 m wide)		Medium (30-150 m)		Wide (> 150 m)		
FLOW HABIT	Ephemeral (Intermittent)		Perennial but flashy		Perennial		
BED MATERIAL	Silt-clay	Silt	Sand	Gravel	Cobble or boulder		
VALLEY SETTING	 No valley; alluvial fan		 Low relief valley (< 30 m deep)		 Moderate relief (30-300 m)	 High relief (> 300 m)	
FLOOD PLAINS	 Little or none (< 2X channel width)		 Narrow (2-10 channel width)		 Wide (> 10X channel width)		
NATURAL LEVEES	 Little or None		 Mainly on Concave		 Well Developed on Both Banks		
APPARENT INCISION	 Not Incised		 Probably Incised				
CHANNEL BOUNDARIES	 Alluvial		 Semi-alluvial		 Non-alluvial		
TREE COVER ON BANKS	< 50 percent of bankline		50-90 percent		> 90 percent		
SINUOSITY	 Straight Sinuosity 1-1.05		 Sinuous (1.06-1.25)		 Meandering (1.25-2.0)		 Highly meandering (> 2)
BRAIDED STREAMS	 Not braided (< 5 percent)		 Locally braided (5-35 percent)		 Generally braided (> 35 percent)		
ANABRANCHED STREAMS	 Not anabranching (< 5 percent)		 Locally anabranching (5-35 percent)		 Generally anabranching (> 35 percent)		
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS	 Narrow point bars		 Wide point bars		 Irregular point and lateral bars		 Random variation

Figure 7-3 Geomorphic Factors That Affect Stream Stability

Source: Adapted From Brice and Blodgett, 1978

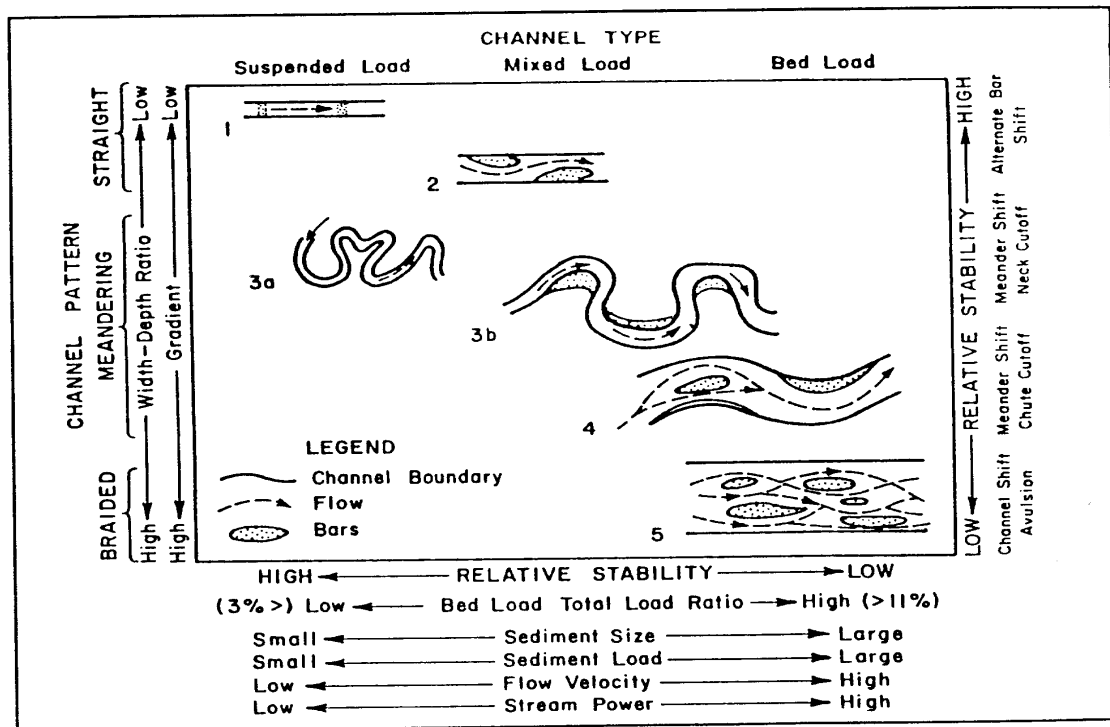


Figure 7-4 Channel Classification And Relative Stability As Hydraulic Factors Are Varied
 Source: After Shen et al., 1981

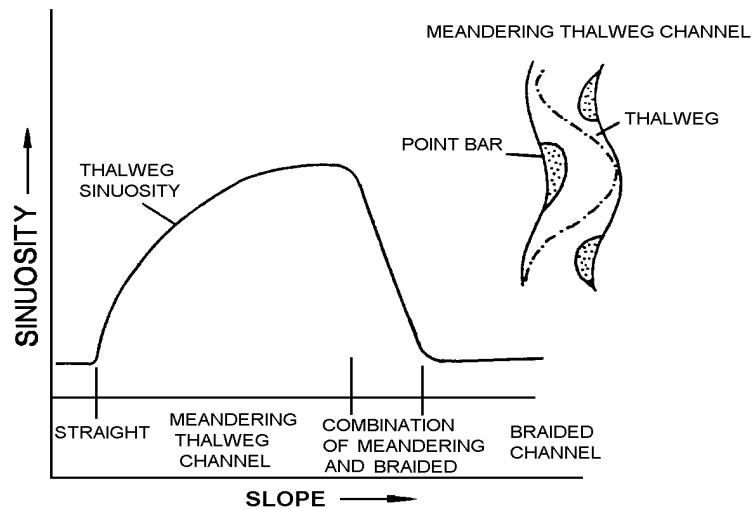


Figure 7-5 Sinuosity Versus Slope With Constant Discharge

Source: After Richardson et al., 1988

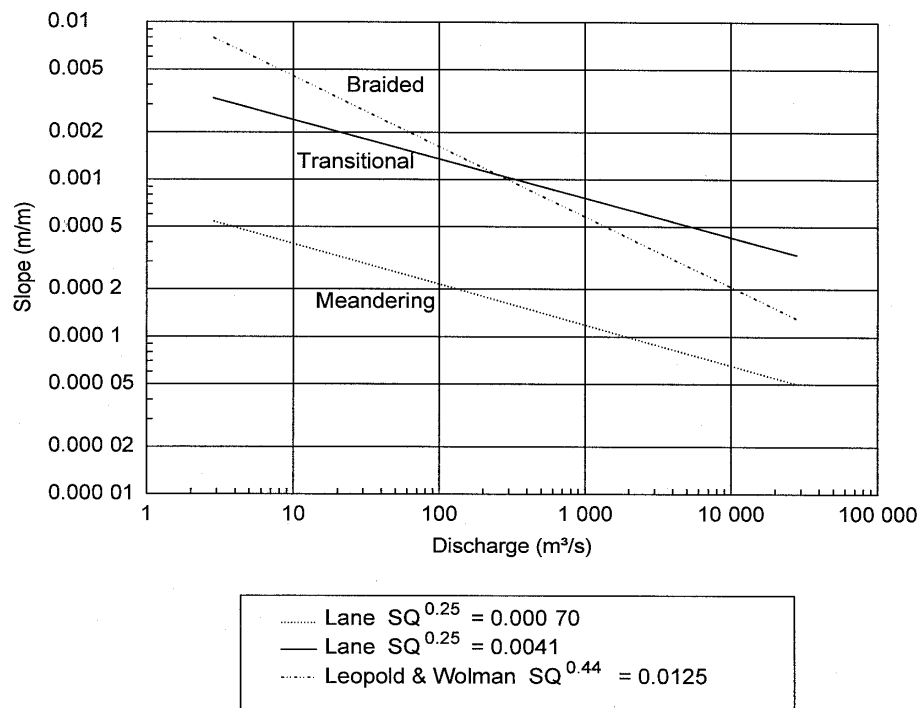


Figure 7-6 Slope-Discharge For Braiding Or Meandering Bed Streams

Source: After Lane, 1957

7.5.5 Countermeasures

A countermeasure is defined as a measure incorporated into a highway crossing of a stream to control, inhibit, change, delay, or minimize stream and bridge stability problems. They may be installed at the time of highway construction or retrofitted to resolve stability problems at existing crossings.

Retrofitting is good economics and good engineering practice in many locations because the magnitude, location and nature of potential stability problems are not always discernible at the design stage and indeed, may take a period of several years to develop.

The selection of an appropriate countermeasure for a specific bank erosion problem is dependent on factors such as the erosion mechanism, stream characteristics, construction and maintenance requirements, potential for vandalism and costs.

Below is a brief discussion of possible countermeasures for some common river stability problems.

Note: The reader is encouraged to consult with the references listed at the end of this chapter for detailed information on the design and construction of the countermeasures.

Meander Migration

The best countermeasure against meander migration is a crossing location on a relatively straight reach of stream between bends. Other countermeasures include the protection of an existing bank line, the establishment of a new flow line or alignment and the control and constriction of channel flow. Countermeasures identified for bank stabilization and bend control are bank revetments, spurs, retardance structures, longitudinal dikes, vane dikes, bulkheads and channel relocations. Measures may be used individually or a combination of two or more measures may be used to combat meander migration at a site (HEC-20, 1995).

Channel Braiding

Countermeasures used at braided streams are usually intended to confine the multiple channels to one channel. This tends to increase sediment transport capacity in the principal channel and encourage deposition in secondary channels.

The measures usually consist of dikes constructed from the limits of the multiple channels to the channel over which the bridge is constructed. Spur dikes at bridge ends used in combination with revetment on highway fill slopes, riprap on highway fill slopes only and spurs arranged in the stream channels to constrict flow to one channel have also been used successfully.

Degradation

Degradation in streams can cause the loss of bridge piers in stream channels, and piers and abutments in caving banks. A check dam, which is a low dam or weir constructed across a channel, is one of the most successful techniques for halting degradation on small to medium streams.

Longitudinal stone dikes placed at the toe of channel banks can be effective counter measures for bank caving in degrading streams. Precautions to prevent outflanking, such as tiebacks to the banks, may be necessary where installations are limited to the vicinity of the highway stream crossing. In general, channel lining alone is not a successful counter measure against degradation problems (HEC-20).

Aggradation

Current measures in use to alleviate aggradation problems at highways include channelization, bridge modification, continued maintenance, or combinations of these.

Channelization may include excavating and cleaning channels, constructing cutoffs to increase the local slope, constructing flow control structures to reduce and control of the local channel width, and constructing relief channels to improve flow capacity at the crossing. Except for relief channels, these measures are intended to increase the sediment transport capacity of the channel, thus reducing or eliminating problems with aggradation.

Another technique which shows promise is the submerged vane technique developed by the University of Iowa. The studies suggest that the submerged vane structure may be an effective, economic, low-maintenance, and environmentally acceptable sediment-control structure with a wide range of applications (HEC-20, Odgaard and others, 1986).

7.6 Roadside Channel (Type A) Design Procedure

7.6.1 General Design Considerations

The design of Type A channels shall consider flow capacity, economics, traffic safety, aesthetics, erosion control and maintenance. These considerations are usually interrelated to such an extent that optimum conditions cannot be met for one without compromising one or more of the others; the ideal being to achieve a reasonable balance.

The location, linings and cross slope of banks are important factors in safety considerations. Motorists' safety generally improves with increasing distance between the travelway and the channel. This distance may increase the cost of property acquisition which may be offset in part by a reduction in the cost of traffic protection. The channel may be located closer to the roadway if an errant vehicle can negotiate the lining and cross slope and recover.

All channels are to be designed so as to minimize erosion. Erosive velocities can be reduced by flattening channel grades where uniform flow conditions exist, otherwise an appropriate channel lining is to be used. To prevent erosion, all channels, ditches or swales will be lined as soon as they are excavated.

A roadside channel is defined as an open channel usually paralleling the highway embankment and within the limits of the highway right-of-way. It is normally trapezoidal or V-shaped in cross section and lined with grass or a special protective lining.

The primary function of roadside channels is to collect surface runoff from the highway and areas which drain to the right-of-way and convey the accumulated runoff to acceptable outlet points.

A secondary function of a roadside channel 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 channels is usually constrained to a large extent by the geometric and safety standards applicable to the project. These channels should accommodate the design runoff in a manner which assures the safety of motorists and minimizes future maintenance, damage to adjacent properties, and adverse environmental or aesthetic effects.

7.6.2 Vertical Alignment

Grade affects both the size of the channel and the flow velocity. The flows should be kept subcritical wherever possible so as to avoid adverse characteristics of supercritical flow.

The approximate grade of a channel is usually determined by the topography of the site. If the terrain is flat then deposition of sediment is unavoidable, the channel should be designed so that the deposition will occur at a location accessible to maintenance.

"Vertical grade drops" or check dams, constructed of concrete walls, stones, gabions, concrete cribbing, metal cribbing or treated timbers, are very useful in maintaining grades which produce acceptable velocities downstream and reduce the costs of lining a channel. These should only be used in channels not accessible to vehicles.

7.6.3 Horizontal Alignment

A straight alignment of a channel permits a simplified hydraulic design. A straight channel does not provide obstacles to the flow and normally will not pick up materials for later deposition at some point downgrade. It is not usually practical to design a straight channel and have it compatible with the terrain or existing streams. Changes in alignment should be as gradual as the right-of-way

and terrain permit. Whenever practicable alignment changes should be made in sections with a flat gradient, particularly if flows will become supercritical on a steeper slope. This practice will reduce the force of the water against the banks and allow the use of more effective erosion controls. When horizontal curvature is utilized, the effects of increased water surface on the outsides of curves should be considered.

7.6.4 Swales

Swales are shallow depressed areas used to drain medians, shelves with negative backslopes, and other areas where ditches are not feasible for either safety or aesthetic considerations. Grass-lined swales for the roadways generally have a longitudinal slope which conforms to the roadway grade except on flat grades, where the swale grade may be steeper than the roadway grade.

7.6.5 Interceptor Channels

These channels are used to intercept runoff from adjacent areas, to collect runoff from within the project, and to convey the runoff to suitable outlets, preferably watercourses. Interceptor channels or ditches are divided into three categories: Top-of-slope Channels, Toe-of-Slope Channels and Outlet Channels.

The layout of Interceptor Channels should be made on a topographical map which contains the project, location of storm drainage structures, contours, and drainage area limits.

For the purposes of payment and in accordance with the Department's Standard Specifications, a channel shall be interpreted to mean a natural or artificial watercourse having an average width at the bottom, after excavation, of 1.2m (4 ft) or more. A drainage ditch shall be interpreted to mean an unpaved, artificially constructed open depression having an average width of less than 1.2m (4 ft) at the bottom, after excavation, constructed for the purpose of carrying surface water.

- **Top-of-Slope channels** or ditches are located at the top of cut slopes for the purpose of intercepting runoff from natural slopes inclined towards the project. They serve to reduce erosion of the cut slope face and to prevent debris and sediment from washing onto the project.

The following should be considered before use of a Top-of-Slope Channel:

- The runoff from the contributing area can flow down the cut slope if it will not affect its stability. The Soils and Foundations Section should be consulted to determine this. If the slope will not be stable, then a top-of-slope channel is required.
- If the cost of additional drainage resulting from the absence of a channel exceeds the cost of the channel, then the channel may be warranted. The fact that top-of-slope channels are far removed from the travelway and are difficult to maintain should also be considered.
- These channels are to have a trapezoidal section with 3 horizontal to 1 vertical side slopes for grass linings; and 2 horizontal to 1 vertical side slopes for riprap or rigid linings.
- The channel grade should be such that ponding will not occur thereby causing saturation or overflow at the top of the slope which could result in slope failure. In areas of unstable slopes it may be necessary to intercept, and accelerate the removal of runoff with pipes. Channels crossing highly permeable slopes may require lining with impermeable material.
- To prevent slope failure, the top of the channel nearest the slope should be located no closer than five feet from the outer limit of the rounding created at the top of cut slopes.
- Excessively deep channel cuts to maintain constant grades should be avoided.

- **Toe-of-slope channels** are located near the toe of the embankment when it is necessary to convey water collected by storm drainage systems, swales, or runoff from terrain inclined toward the projects, or to keep flow within the ROW until a suitable outlet is available.

These channels are to have the same cross section geometry as top-of-slope channels if their location will not be hazardous to vehicular traffic. If it is determined that a trapezoidal channel will be hazardous, then the use of guide railing must be considered. The preferred design, for both safety and aesthetics, would be a wide parabolic section with vertical curves to round all angles. The reduction of flow velocities through use of side, shallow channels or swales will minimize erosion and may be possible without an appreciable increase in cost. This type of channel may be found to be more economical after considering the cost of the guide rail, lining and rights of way required for alternatives.

If trapezoidal channels are used they should be located 1.5m (5 feet) from the normal toe rounding of the embankment to prevent the channel from being obstructed by any natural sloughing and erosion of the adjacent embankment. The 5 foot shelf shall be on a slope of 1V:12H between the toe and the top of the channel unless rights of way requirements dictate otherwise.

- **Outlet channels**

Outlet channels are used to convey flow from storm drainage systems and swales to a watercourse or to an area not subject to erosion.

They are usually designed with side slopes of 1V:2H. For vehicular safety, it may be necessary to use flatter side slopes and wider trapezoidal channels, especially where outletting swales from roadway cut sections to the toes of adjoining embankments.

Outlet channels should be located beyond the area required for vehicular recovery. They should be accessible for maintenance since they may trap sediment and debris.

7.6.6 Type A Channel Design Concepts

HEC-15 provides a detailed presentation of stable channel design concepts related to the design of roadside and median channels which convey a design discharge less than **1.42 m³/s or 50 cfs**. This section provides a brief summary of significant concepts.

Stable channel design concepts provide a means of evaluating and defining channel configurations that will perform within acceptable limits of stability. For most highway drainage channels, bank instability and lateral migration can not be tolerated. Stability is achieved when the material forming the channel boundary effectively resists the erosive forces of the flow. Principles of rigid boundary hydraulics can be applied to evaluate this type of system.

Both velocity and tractive force methods have been applied to the determination of channel stability. Permissible velocity procedures are empirical in nature, and have been used to design numerous channels in the United States and throughout the world. However, tractive force methods consider actual physical processes occurring at the channel boundary and represent a more realistic model of the detachment and erosion processes.

The hydrodynamic force created by water flowing in a channel causes a shear stress on the channel bottom. The bed material, in turn, resists this shear stress by developing a tractive force. Tractive force theory states that the flow-induced shear stress should not produce a force greater than the tractive resisting force of the bed material. This tractive resisting force of the bed material creates the permissible or critical shear stress of the bed material. In a uniform flow, the shear stress is equal to the effective component of the gravitational force acting on the body of water parallel to the channel bottom. The average shear stress is equal to:

$$\tau = \gamma R S \quad (7.11)$$

where: τ = average shear stress, Pa (lb/ft²)
 γ = unit weight of water, 9810 N/m³ (62.4 lb/ft³) (at 15.6 °C (60 °F))
 R = hydraulic radius, m (ft)
 S = average bed slope or energy slope, m/m (ft/ft)

The maximum shear stress for a straight channel occurs on the channel bed and is less than or equal to the shear stress at maximum depth. The maximum shear stress is computed as follows:

$$\tau_d = \gamma d S \quad (7.12)$$

where: τ_d = maximum shear stress, Pa (lb/ft²)
 d = maximum depth of flow, m (ft)

Shear stress in channels is not uniformly distributed along the wetted perimeter of a channel. A typical distribution of shear stress in a trapezoidal channel tends toward zero at the corners with a maximum on the bed of the channel at its centerline, and the maximum for the side slopes occurs around the lower third of the slope as illustrated in Figure 7-7.

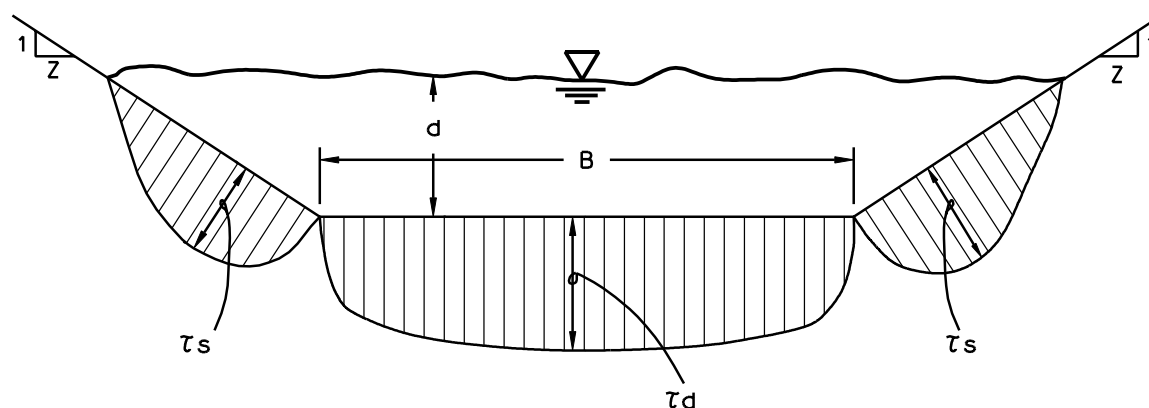


Figure 7-7 Distribution of shear stress

For trapezoidal channels lined with gravel or riprap having side slopes steeper than 3 horizontal to 1 vertical, side slope stability must also be considered. This analysis is performed by comparing the tractive force ratio between side slopes and channel bottom with the ratio of shear stresses exerted on the channel sides and bottom. The ratio of shear stresses on the sides and bottom of a trapezoidal channel, K_1 , is given in Figure 7-9 and the tractive force ratio, K_2 , is given in Figure 7-10. The angle of repose, θ , for different rock shapes and sizes is provided in Figure 7-11. The required rock size for the side slopes is found using the following equation:

$$(D_{50})_{\text{sides}} = \frac{K_1}{K_2} (D_{50})_{\text{bottom}} \quad (7.13)$$

where: D_{50} = the mean riprap size, m (ft)
 K_1 = ratio of shear stresses on the sides and bottom of a trapezoidal channel (see Figure 7-9).
 K_2 = ratio of tractive force on the sides and bottom of a trapezoidal channel (see Figure 7-10).

Flow around a bend in an open channel induces centrifugal forces because of the change in flow direction. This results in a superelevation of the water surface at the outside of bends and can cause the flow to splash over the side of the channel if adequate freeboard is not provided. This superelevation can be estimated by the following equation.

$$\Delta d = \frac{V^2 T}{g R_c} \quad (7.14)$$

where: Δd = difference in water surface elevation between the inner and outer banks of the channel in the bend, m (ft)
 V = average velocity, m/s (ft/s)
 T = surface width of the channel, m (ft)
 g = gravitational acceleration, 9.8 m/s^2 (32.2 ft/s^2)
 R_c = radius to the centerline of the channel, m (ft)

Equation 7.14 is valid for subcritical flow conditions. The elevation of the water surface at the outer channel bank will be $\Delta d/2$ higher than the centerline water surface elevation (the average water surface elevation immediately before the bend) and the elevation of the water surface at the inner channel bank will be $\Delta d/2$ lower than the centerline water surface elevation.

Flow around bends also creates secondary currents which impose higher shear stresses on the channel sides and bottom compared to straight reaches. Areas of high shear stress in bends are illustrated in Figure 7-8. The maximum shear stress in a bend is a function of the ratio of channel curvature to bottom width. This ratio increases as the bend becomes sharper and the maximum shear stress in the bend increases. The bend shear stress can be computed using the following relationship:

$$\tau_b = K_b \tau_d \quad (7.15)$$

where: τ_b = bend shear stress, Pa (lb/ft^2)
 K_b = function of R_c/B (see Figure 7-12)
 R_c = radius to the centerline of the channel, m (ft)
 B = bottom width of channel, m (ft)
 τ_d = maximum channel shear stress, Pa (lb/ft^2)

The increased shear stress produced by the bend persists downstream of the bend a distance L_p , as shown in Figure 7-8. This distance can be computed using the following relationship:

$$L_p = \frac{0.736 R^{7/6}}{n_b} \quad \left(L_p = \frac{0.603 R^{7/6}}{n_b} \right) \quad (7.16)$$

where: L_p = length of protection (length of increased shear stress due to the bend) downstream of the point of tangency, m (ft)
 n_b = Manning's roughness in the channel bend
 R = hydraulic radius, m (ft)

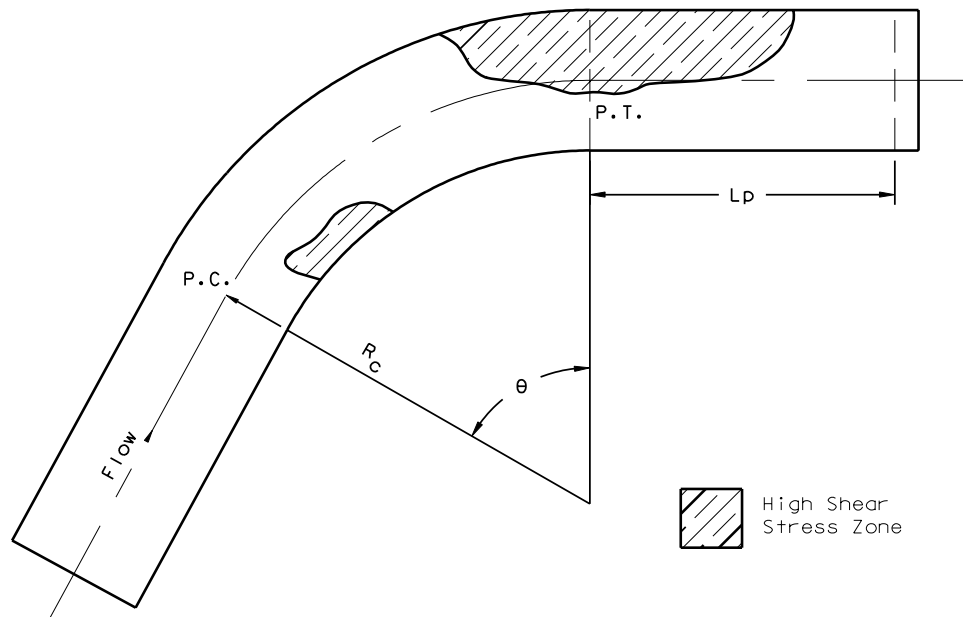


Figure 7-8 Shear stress distribution in channel bends

7.6.7 Design Parameters

Parameters required for the design of roadside and median channels include discharge frequency, channel geometry, channel slope, vegetation type, freeboard, and shear stress. This section provides criteria relative to the selection or computation of these design elements.

- **Discharge Frequency**

Roadside and median drainage channels are designed to carry the 10 year design flow. A 2-year return period is used for the design of temporary linings.

Channels will be designed to convey the discharge of any connecting facility, with consideration for possible additional flow from adjacent areas which may increase the discharge and require adjustments in the design. The preliminary layout of the channel should be made on a topographic map which contains the location of the project and has adequate contours to determine the contributing drainage areas. From this map, the discharge can be estimated at appropriate points along the channel. For watercourses, it is usually not necessary to compute more than one discharge unless there are extenuating circumstances. Additional information on design discharge can be found in other sections of this chapter.

- **Channel Geometry**

Channel side slopes for triangular or trapezoidal channels cannot exceed the angle of repose of the soil and/or lining material, and should generally be 3 horizontal to 1 vertical or flatter. In areas where traffic safety may be of concern, channel side slopes should be 4 horizontal to 1 vertical or flatter.

Design of roadside and median channels should be integrated with the highway geometric and pavement design to insure proper consideration of safety and pavement drainage needs.

- **Channel Slope**

Channel bottom slopes are generally dictated by the road profile or other constraints. However, if channel stability conditions warrant, it may be feasible to adjust the channel gradient slightly to achieve a more stable condition. Channel gradients greater than two percent may require the use of flexible linings to maintain stability. Most flexible lining materials are suitable for protecting channel gradients of up to 10 percent, with the exception of some grasses. Linings, such as riprap and wire-enclosed riprap are more suitable for protecting very steep channels with gradients in excess of 10 percent. Rigid linings, such as concrete paving, are highly susceptible to failure from structural instability due to such occurrences as overtopping, freeze thaw cycles, swelling, and excessive soil pore water pressure.

- **Roughness Coefficient (Manning's n values)**

The selection of an appropriate Manning's n value for design purposes is often based on observation and experience. Manning's n values are also known to vary with flow depth. Table 7-2 provides a tabulation of Manning's n values for various channel lining materials. Manning's roughness coefficient for vegetative and other linings vary significantly depending on the amount of submergence. The classification of vegetal covers by degree of retardance is provided in

HEC-15. Table 7-3 provides a list of Manning's n relationships for five classes of vegetation defined by their degree of retardance.

• Freeboard

The freeboard of a channel is the vertical distance from the water surface to the top of the channel banks. The importance of this factor depends on the consequence of overflow of the channel bank. At a minimum the freeboard should be sufficient to prevent waves, superelevation changes, or fluctuations in water surface from overflowing the sides. In a permanent roadside or median channel, 0.3m (1 ft) of freeboard is generally considered adequate. For temporary channels no freeboard is necessary. However, a steep gradient channel should have a freeboard height equal to the flow depth to compensate for the large variations in flow caused by waves, splashing, and surging.

• Shear Stress

The permissible or critical shear stress in a channel defines the force required to initiate movement of the channel bed or lining material. Table 7-4 presents permissible shear stress values for manufactured, vegetative, and riprap channel lining. The permissible shear stress for non-cohesive soils is a function of mean diameter of the channel material as shown in Figure 7-13. For larger stone sizes not shown in Figure 7-13 and rock riprap, the permissible shear stress is given by the following equation:

$$\tau_p = K_p D_{50} \quad (7.17)$$

where: τ_p = permissible shear stress, Pa (lb/ft²)
 D_{50} = mean riprap size, m (ft)
 K_p = 628 (4.0)

For cohesive materials the plasticity index provides a good guide for determining the permissible shear stress as illustrated in Figure 7-14.

Table 7-2. Manning's roughness coefficients for Roadside and Median Channels. *

Lining Category	ConnDOT Designations	Lining Type	n - Value for Given Depth Ranges		
			0-0.15 m (0-0.5 ft)	0.15-0.60 m (0.5-2.0 ft)	>0.60 m (>2.0 ft)
Rigid		Concrete	0.015	0.013	0.013
		Grouted Riprap	0.040	0.030	0.028
		Stone Masonry	0.042	0.032	0.030
		Soil Cement	0.025	0.022	0.020
		Asphalt	0.018	0.016	0.016
Unlined		Bare Soil	0.023	0.020	0.020
		Rock Cut	0.045	0.035	0.025
Erosion Control Matting** (Temporary or Permanent)	Type E	Woven Paper Net	0.016	0.015	0.015
		Jute Net	0.028	0.022	0.019
	Type F	Fiberglass Roving	0.028	0.021	0.019
	Type G	Straw with Net	0.065	0.033	0.025
		Curled Wood Mat	0.066	0.035	0.028
	Type H	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	Special	25 mm (1 in) D ₅₀	0.044	0.033	0.030
		50 mm (2 in) D ₅₀	0.066	0.041	0.034
Rock Riprap	Modified	125 mm (5 in) D ₅₀	0.095	0.062	0.035
		150 mm (6 in) D ₅₀	0.104	0.069	0.035
	Intermediate	200 mm (8 in) D ₅₀	--	0.072	0.037
		300 mm (12 in) D ₅₀	--	0.078	0.040
	Standard	380 mm (15 in) D ₅₀	--	--	0.042

NOTE: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary with the flow depth.

* Table reproduced from HEC-15

** See Section 7.6.8

Table 7-3. Manning's n relationships for vegetal degree of retardance.

Retardance Class***	Manning's n Equation*
A	$1.22 R^{1/6} / [30.2 + 19.97 \log (R^{1.4} S_o^{0.4})] (R^{1/6} / [15.8 + 19.97 \log (R^{1.4} S_o^{0.4})])$ (7-18)
B	$1.22 R^{1/6} / [37.4 + 19.97 \log (R^{1.4} S_o^{0.4})] (R^{1/6} / [23.0 + 19.97 \log (R^{1.4} S_o^{0.4})])$ (7-19)
C**	$1.22 R^{1/6} / [44.6 + 19.97 \log (R^{1.4} S_o^{0.4})] (R^{1/6} / [30.2 + 19.97 \log (R^{1.4} S_o^{0.4})])$ (7-20)
D	$1.22 R^{1/6} / [49.0 + 19.97 \log (R^{1.4} S_o^{0.4})] (R^{1/6} / [34.6 + 19.97 \log (R^{1.4} S_o^{0.4})])$ (7-21)
E	$1.22 R^{1/6} / [52.1 + 19.97 \log (R^{1.4} S_o^{0.4})] (R^{1/6} / [37.7 + 19.97 \log (R^{1.4} S_o^{0.4})])$ (7-22)

* Equations are valid for flows less than 1.42 m³/s (50 ft³/s).

Nomograph solutions for these equations are contained in HEC-15.

** Use on ConnDOT projects when calling for turf establishment

*** See HEC-15 for definition of retardance classes

Table 7-4 Permissible Shear Stresses for Lining Materials*

Lining Category	ConnDOT Designations	Lining Type	Permissible Unit Shear Stress	
			Pa	lb/ft ²
Erosion Control Matting**	Type E	Woven Paper Net	7.2	0.15
		Jute Net	21.5	0.45
	Type F	Fiberglass Roving:		
		Single	28.7	0.60
		Double	40.7	0.85
	Type G	Straw with Net	69.4	1.45
Vegetative	Type H	Curled Wood Mat	74.2	1.55
		Synthetic Mat	95.8	2.00
		Class A	177.2	3.70
		Class B	100.6	2.10
	Turf Establishment	Class C-Use for DOT projects	47.9	1.00
		Class D	28.7	0.60
Gravel Riprap	Special	Class E	16.8	0.35
		25 mm (1 in)	15.8	0.33
		50 mm (2 in)	31.6	0.67
Rock Riprap	Modified	125 mm (5 in)	79.8	1.68
		150 mm (6 in)	95.8	2.00
	Intermediate	200 mm (8 in)	127.7	2.68
		300 mm (12 in)	191.5	4.00
	Standard	380 mm (15 in)	239.3	5.00
Bare Soil		Non-cohesive	Figure 7-13	
		Cohesive	Figure 7-14	

* Reproduced from HEC-15

** See Section 7.6.8

7.6.8 Erosion Control Matting

Erosion control matting is evaluated by the Department for use in eight Types (A-H), grouped into two Classes. Types A through D are included in Class 1 and are designated as Slope Protection. This classification is based upon steepness of the slope and soil type. The purpose of Class 1 matting is to protect the seedbed from loss of soil, and promote the establishment of a warm-season, perennial vegetative cover.

Types E through H are included in Class 2 and are designated as Flexible Channel Liner Protection. This classification is based upon the permissible shear stress of the material. The purpose of Class 2 matting is to protect the geometry of a channel from loss of soil, and to promote the establishment of a warm-season, perennial vegetative cover. Class 2 Types are designated according to the shear stress limits shown in Table 7-5. The shear stress ranges are based on values published in HEC-15.

Table 7-5 Erosion Control Matting (Class 2) Shear Stress Ranges

<u>Matting Type</u>	<u>Permissible Shear Stress – Pa (lb/ft²)</u>
E	< 25 (<0.5)
F	25 to < 50 (0.5 to <1.0)
G	50 to < 100 (1.0 to <2.0)
H	≥ 100 (≥2.0)

Products approved by the Department for use as erosion control matting on State projects are placed on an Approved Products List. This list is updated annually and distributed by the Department's Office of Research and Materials. Products listed by brand or trade name on the Approved Products List have demonstrated their ability to meet or exceed the Department's minimum performance standards by independent testing conducted at facilities such as the Texas Department of Transportation/Texas Transportation Institute Hydraulics and Erosion Control Laboratory.

The drainage designer is primarily concerned with Class 2 matting materials for use in the construction of roadside channels. Erosion control matting in channel lining applications can be temporary (bio- or photodegradable) or permanent (non-bio- or non-photodegradable). Types E, F and G matting are generally temporary and include but are not limited to the following materials: (E) Woven Paper Net, Jute Net and (F) Single or Double Fiberglass Roving and (G) Straw with Net, Curled Wood Mat. Type H matting is generally composed of synthetic material, is permanent and is commonly used for turf reinforcement. **If the channel design requires the use of a permanent matting, the matting should be noted as permanent on the project plans.**

The designer should be aware that it is the contractor's option as to which product he or she actually uses on a project, provided that the product used is on the Approved Products List for that application. Appropriate judgement should be reflected in the engineering calculations when selecting the Type of Matting (E, F, G or H) to be specified on the project plans.

See Appendix B for additional guidance regarding the use of erosion control matting in vegetative (grass) lined swales.

7.6.9 Design Procedure for Roadside Ditches, Swales, and Channels (Type A)

This section presents a generalized procedure for the design of roadside and median channels. Although each project will be unique, the design steps outlined below will normally be applicable.

Step 1. Establish a Preliminary Drainage Plan

For proposed median or roadside channels, the following preliminary action should be taken:

- A. Prepare existing and proposed plan and profile of the proposed channels. Include any constraints on design such as highway and road locations, culverts, utilities, etc.
- B. Determine and plot on the plan the locations of natural basin divides and channel outlet points.
- C. Collect any available site data such as soil types and topographic information.

Step 2. Obtain or Establish Cross Section Data

Establish preliminary cross section geometric parameters and controlling physical features considering the following guides:

- A. Adequate channel depth should be provided to drain the pavement subbase and minimize freeze-thaw.
- B. Channel side slopes based on geometric design criteria including safety, economics, soil, aesthetics, and access should be chosen.

Step 3. Determine Initial Channel Grades

Plot initial grades on the plan and profile. Note that slopes on roadside channels in cuts are usually controlled by highway grades. Use the following guides when establishing initial grades:

- A. Provide a channel slope with a minimum 0.5% grade to minimize ponding and sediment accumulation.
- B. Where possible, avoid features which may influence or restrict grade, such as utility structures.

Step 4. Check flow Capacities and Adjust Sections as Necessary

- A. Compute the design discharge at the downstream end of channel segments.
- B. Set preliminary values for channel size, roughness, and slope, based on long term conditions and considering maintenance.
- C. Determine the maximum allowable depth of channel including freeboard.

- D. Check the flow capacity using Manning's equation
- E. If the capacity is not adequate, possible considerations for increasing the capacity are provided below.
 - 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

Step 5. Determine Channel Protection Needed

- A. Select a lining and determine the permissible shear stress from Table 7-4. For detailed information related to lining performance, see HEC-15.
- B. Estimate the flow depth and choose an initial Manning's n value from Table 7-2 or Table 7-3.
- C. Calculate the normal flow depth at design discharge using Manning's equation and compare with the estimated depth. If the flow depth is acceptable, continue with the design procedure. If the depth is not acceptable, repeat steps 5B and 5C.
- D. Compute the maximum shear stress at normal depth using equation 7.12.
- E. If the maximum shear stress (step 5D) is less than the permissible shear stress (step 5A), then the lining is acceptable. Otherwise consider the following options:
 - choose a more resistant lining
 - use concrete, gabions, or other more rigid lining either as full lining or composite (keeping in consideration the possible deficiencies of rigid linings)
 - decrease channel slope
 - decrease slope in combination with drop structures
 - increase channel width and/or flatten side slopes

If the maximum shear stress is excessively less than the permissible shear stress, the lining material may be redesigned to provide a more comparable lining material.

- F. For trapezoidal channels lined with gravel or riprap having side slopes steeper than 3 horizontal to 1 vertical, use equation 7-13 and Figures 7-9 and 7-10 to evaluate side slope stability.
- G. For flow around bends, use equation 7.15 and Figure 7-12 to evaluate lining stability.
- H. When channel gradients approach 10 percent, compare results obtained above with steep slope procedures in HEC-15.

- I. For composite linings use procedures in HEC-15.
- J. If a grass lining is designated without a permanent erosion control matting, then a temporary protective lining will be designed using a 2 year frequency storm repeating the above steps. See Appendix B for additional guidance regarding the use of erosion control matting in vegetative (grass) lined swales.

Step 6. Check Channel Transitions and End of Reach Conditions

Channel transition include locations where there are changes in cross section, slope, discharge, and/or roughness. At these locations, the gradually varying flow assumption may be violated, and a more detailed hydraulic evaluation may be required.

- A. Identify transition locations.
- B. Review hydraulic conditions upstream and downstream of the transition (flow area, depth, and velocity). If significant changes in these parameters are observed, perform additional hydraulic evaluations to determine flow conditions in the vicinity of the transition. See HEC-15.
- C. Provide for gradual channel transitions to minimize the possibility for sudden changes in hydraulic conditions at channel transitions.

Step 7. Analyze Outlet Points and Downstream Effects

- A. Identify any adverse impacts to downstream properties which may result from one of the following at the channel outlets:
 - increase or decrease in peak discharge
 - increase in flow velocity
 - confinement of sheet flow
 - change in outlet water quality
 - diversion of flow from another watershed
- B. Mitigate any adverse impacts identified in 7A. Possibilities in order relative to above impacts include:
 - enlarge outlet channel and/or install control structures to provide detention of increased runoff in channel
 - install velocity control or energy dissipation structure
 - increase capacity and/or improve lining of downstream channel
 - install sophisticated weirs or other outlet devices to redistribute concentrated channel flow
 - eliminate diversions which result in downstream damage and which cannot be mitigated in a less expensive fashion

To obtain the optimum roadside channel system design, it may be necessary to make several trials of the above procedure before a final design is achieved.

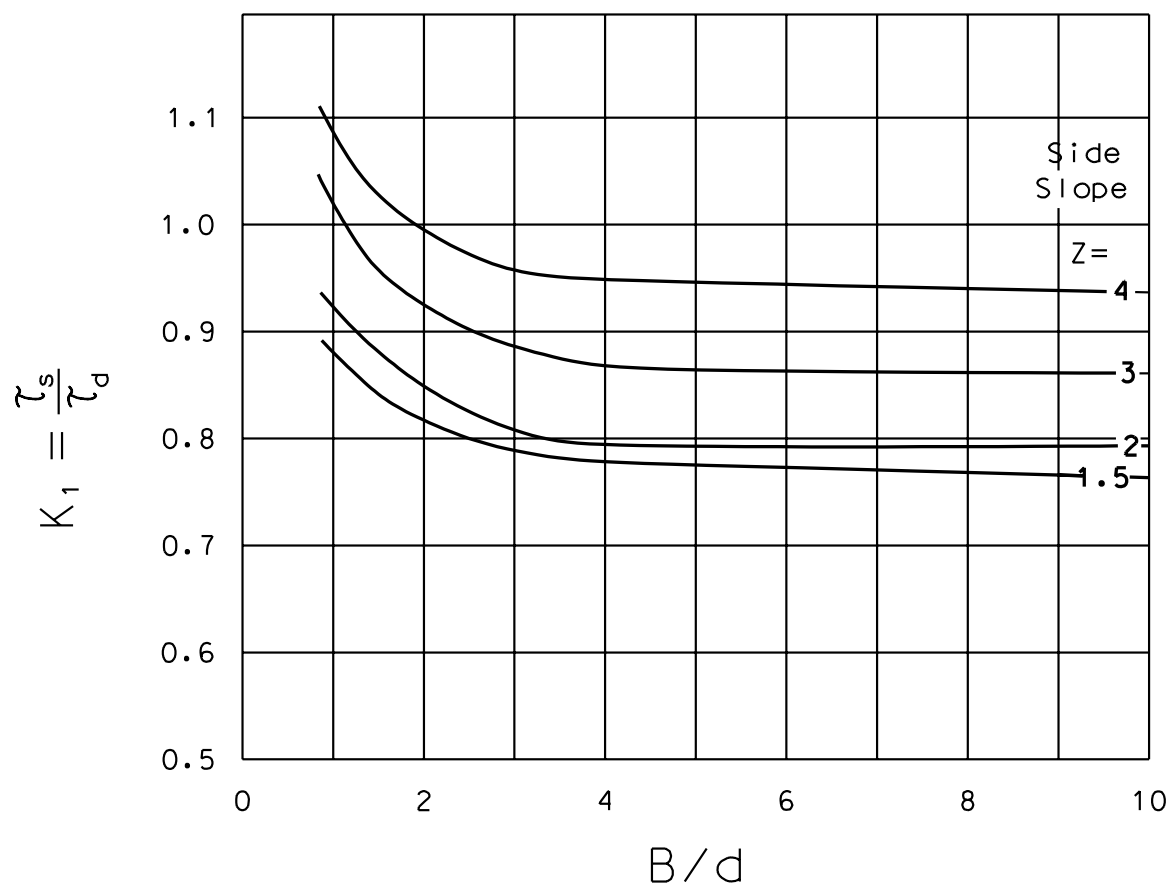


Figure 7-9 Channel Side Shear Stress to Bottom Shear Stress Ratio, K_1 .

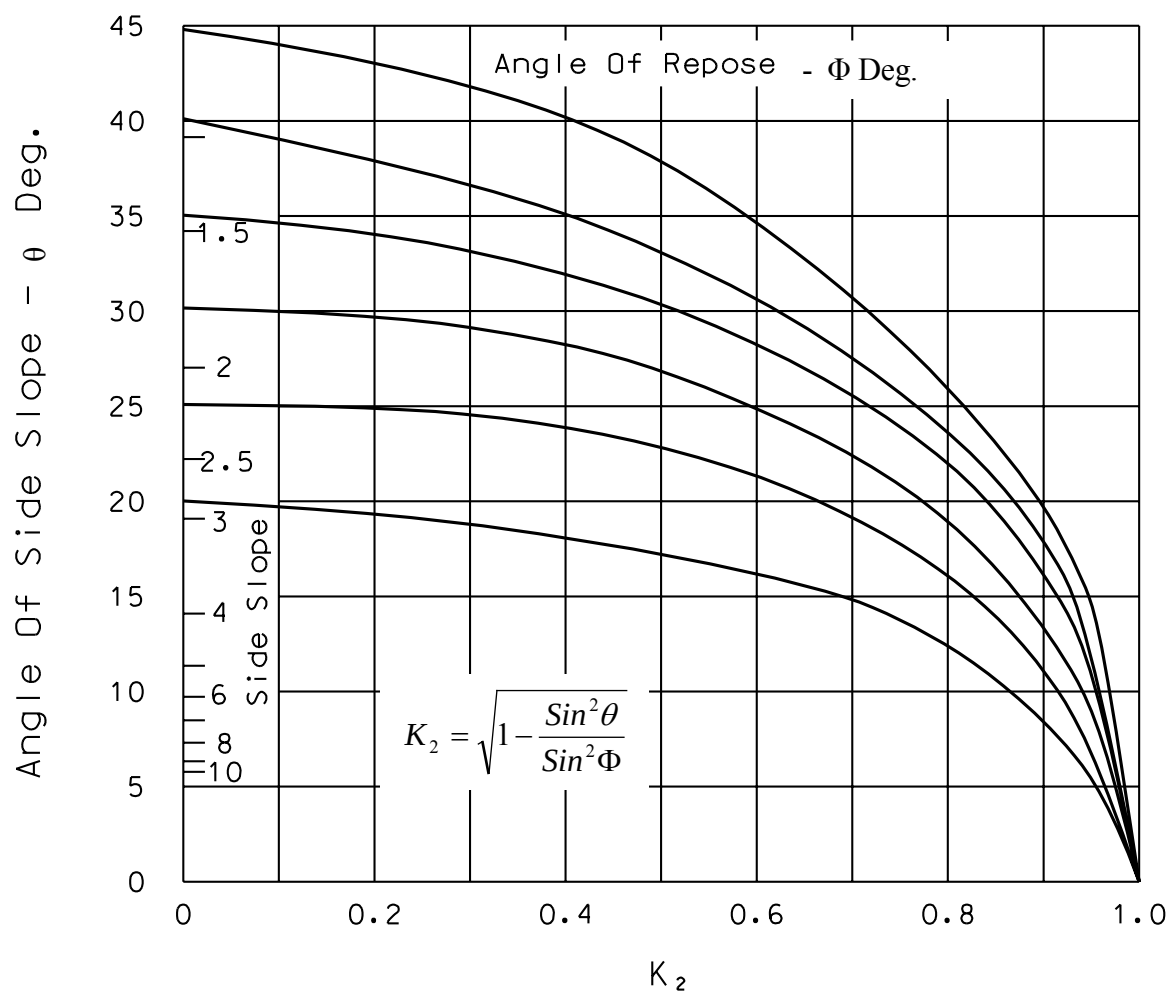


Figure 7-10 Tractive Force Ratio, K_2

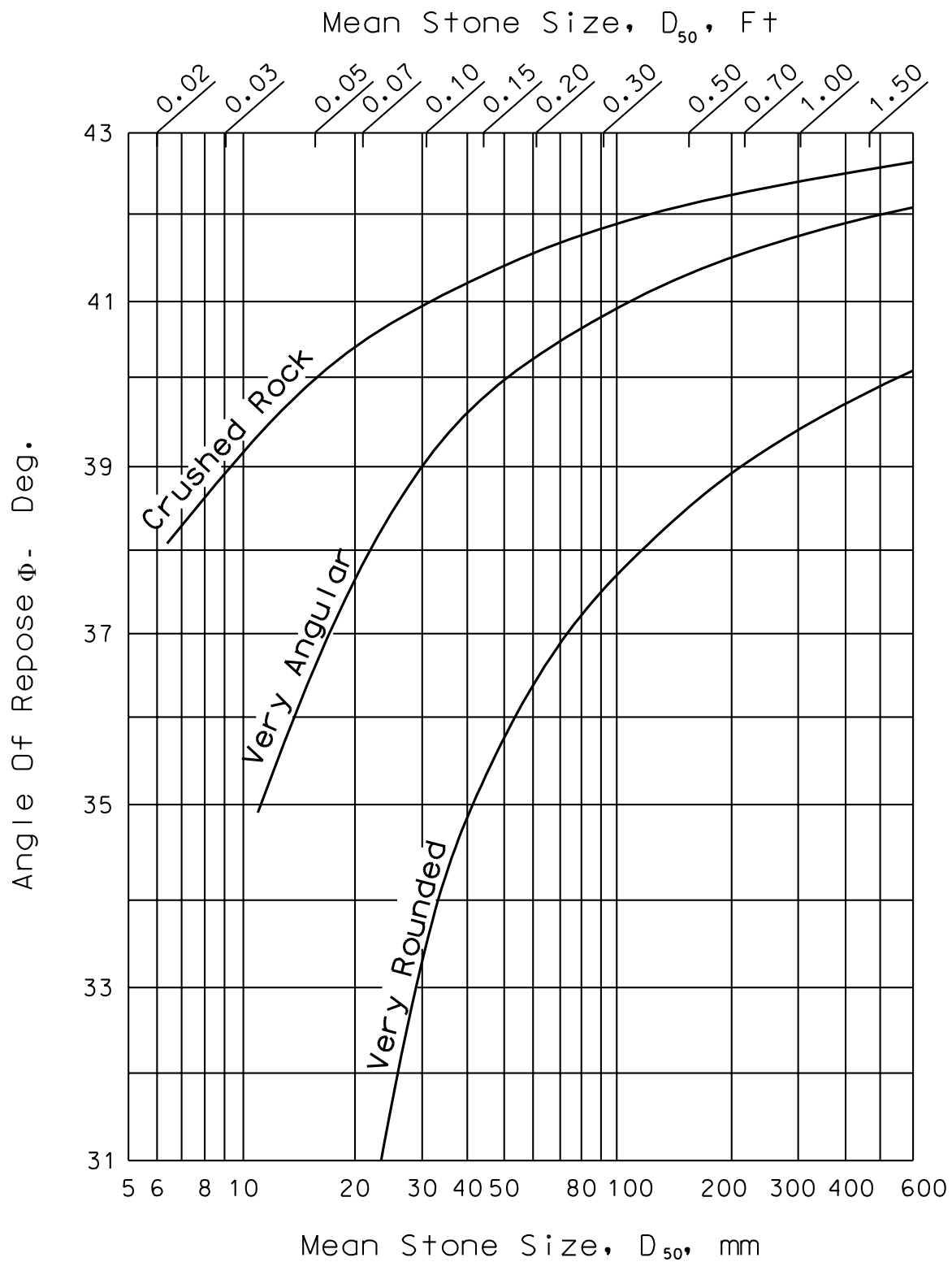


Figure 7-11 Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone

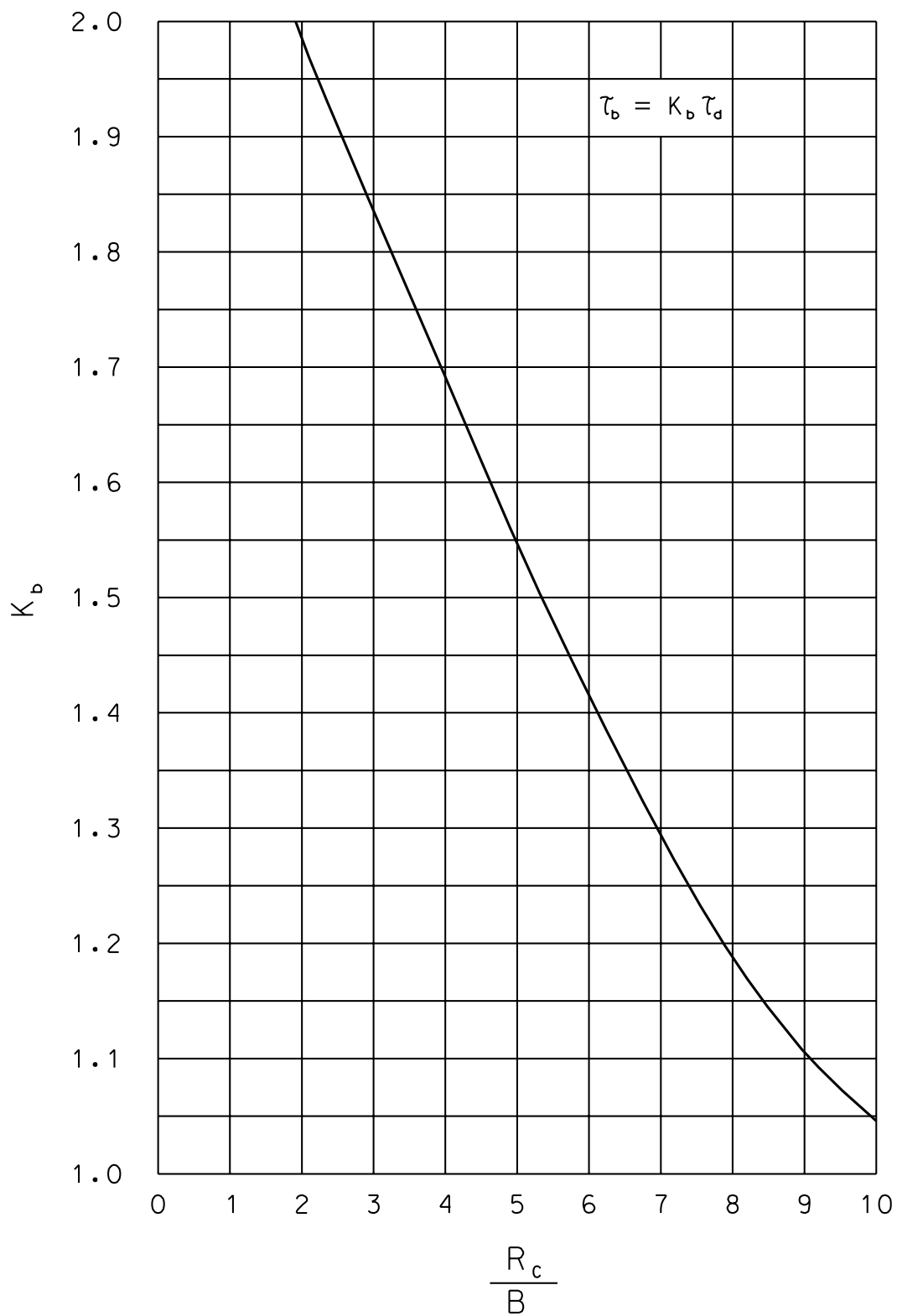
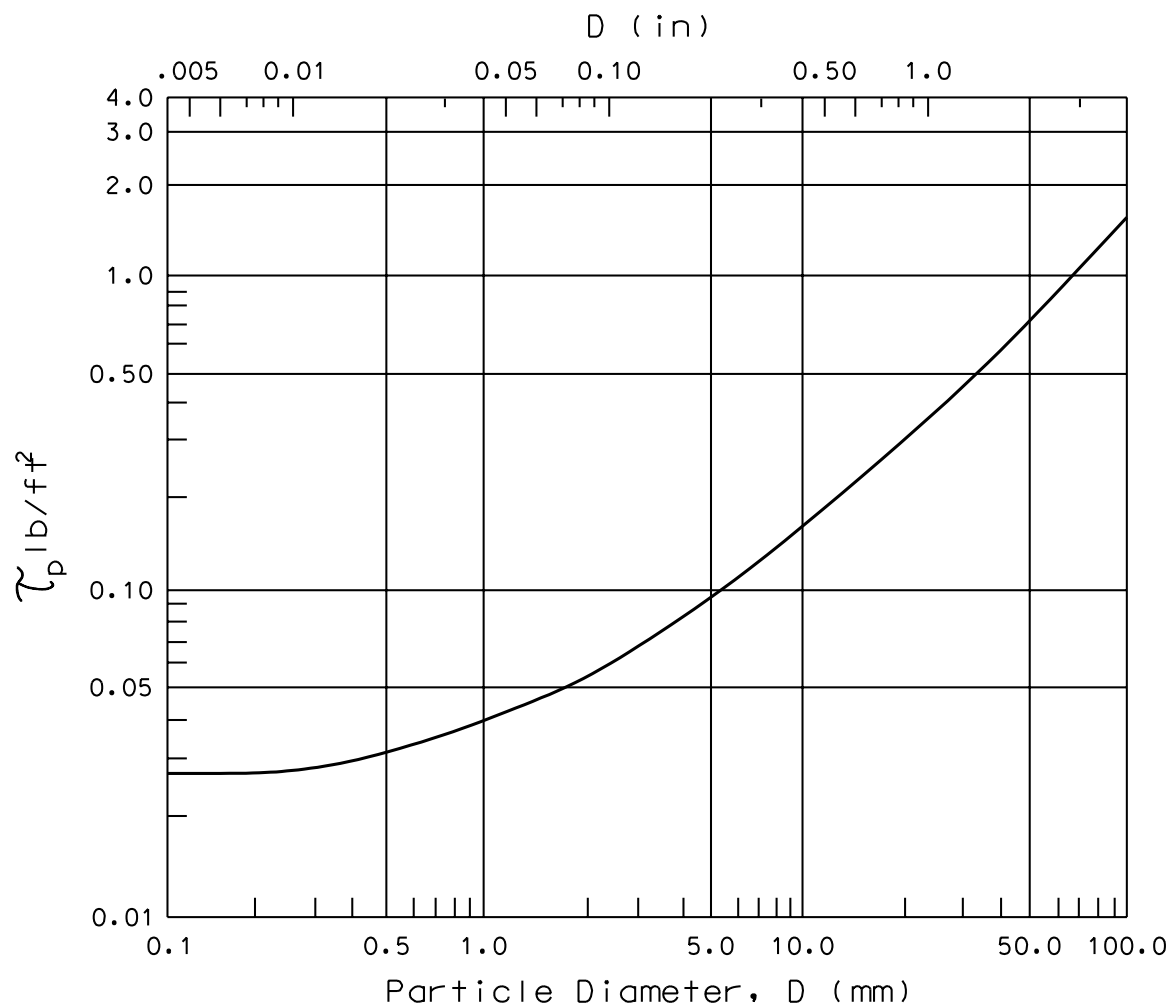


Figure 7-12 K_b Factor for Maximum Shear Stress on Channel Bends.



$$1 \text{ lb/ft}^2 = 47.88 \text{ N/m}^2 \text{ (Pascal)}$$

Figure 7-13 Permissible Shear Stress for Non-cohesive Soils

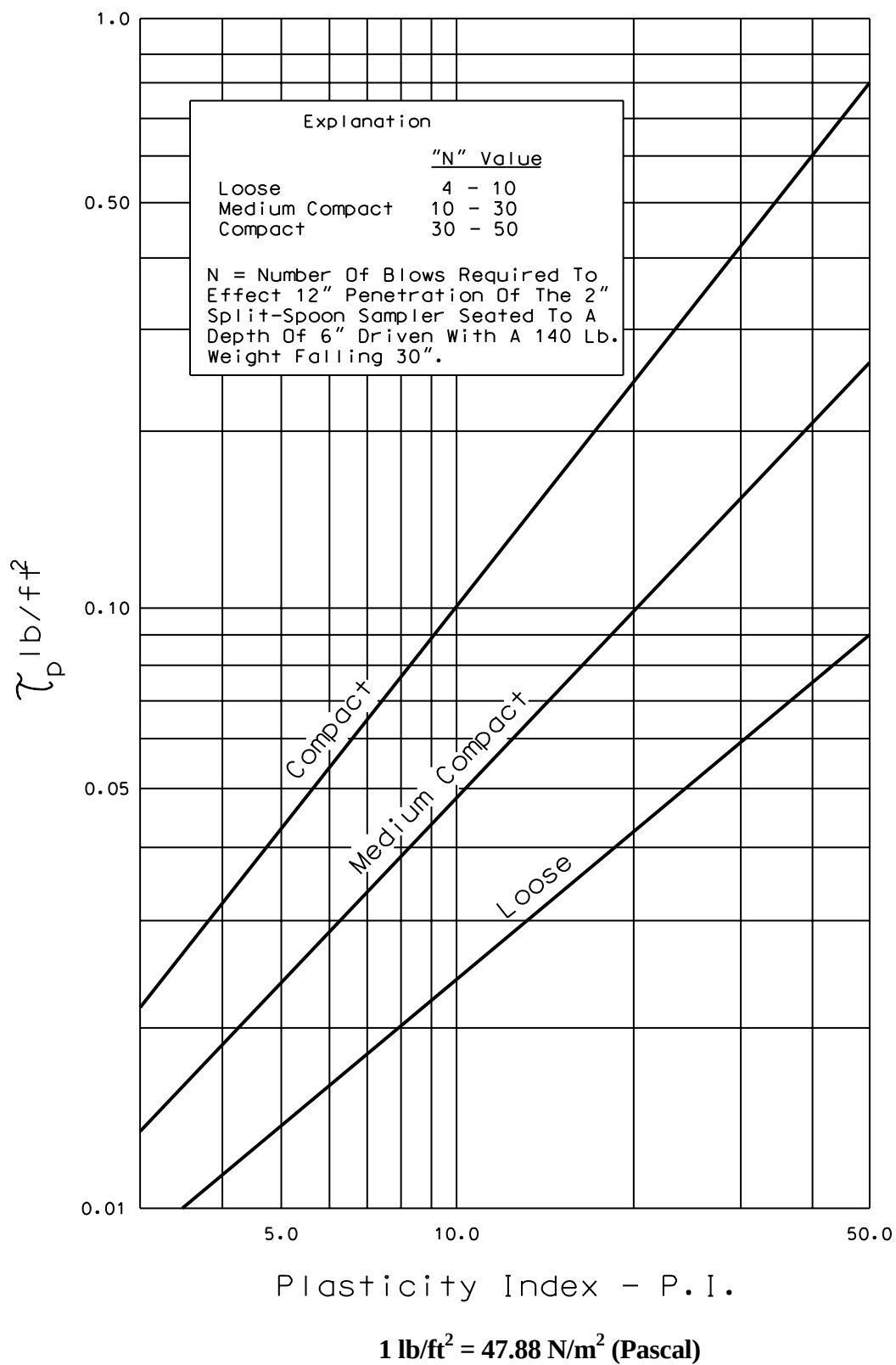


Figure 7-14 Permissible Shear Stress for Cohesive Soils

7.6.10 Documentation (Type A, Channel)

The following items shall be included in the documentation file (see Chapter 1, Section 1.6). The intent is not to limit data to only those items listed, but rather establish a minimum requirement consistent with the channel design procedures as outlined in this chapter. If circumstances are such that the design is prepared other than the normal procedures or is governed by factors other than hydrologic/hydraulic factors, then a narrative summary detailing the design basis shall appear with the other data:

The following items shall be included in the documentation file:

- Computations based on Section 7.6.9
- Channel configuration
- Complete watershed map
- Prepare report and file with background information

7.7 Hydraulic Analysis (Type B Channels)

7.7.1 General

A natural perennial watercourse or man-made channel planned to simulate a natural watercourse requires a hydraulic analysis. 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 exist. Manning's equation can be used to estimate highwater elevations for bridges that do not constrict the flow. 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.

The single-section method will generally yield less reliable results because it requires more judgment and assumptions than the step-backwater method. In many situations, however, the single-section method is all that is justified, e.g., a standard roadway ditch, culverts, storm drain outfalls, etc. Open channel flow analysis occasionally justifies a more detailed method of analysis than the single-section method or the computation of a water surface profile using the step-backwater method. Special analysis techniques include two-dimensional analysis, water and sediment routing, and unsteady flow analysis.

7.7.2 Cross Sections

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

Cross sections should be located to be representative of the subreaches between them. Stream locations with major breaks in bed profile, abrupt changes 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 change 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 velocity distribution (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 7-15.

- **Manning's n Value Selection**

Manning's n is affected by many factors and its selection in natural channels depends heavily on engineering experience. Pictures of channels and floodplains for which the discharge has been measured and Manning's n has been calculated are very useful (see Arcement and Schneider, 1984; Barnes, 1978). For situations lying outside the engineer's experience, a more regimented approach is presented in 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 streamflow data.**

Manning's n values for artificial channels are more easily defined than for natural stream channels. See Table 7-1 in Section 7.4.11 for typical n values of both artificial channels and natural stream channels.

- **Calibration**

The results of equations should be calibrated with historical highwater marks and/or gaged streamflow data where possible to ensure that they accurately represent local channel conditions. The following parameters, in order of preference, should be used for calibrations: Manning's n , slope, discharge and cross section. Proper calibration is essential if accurate results are to be obtained.

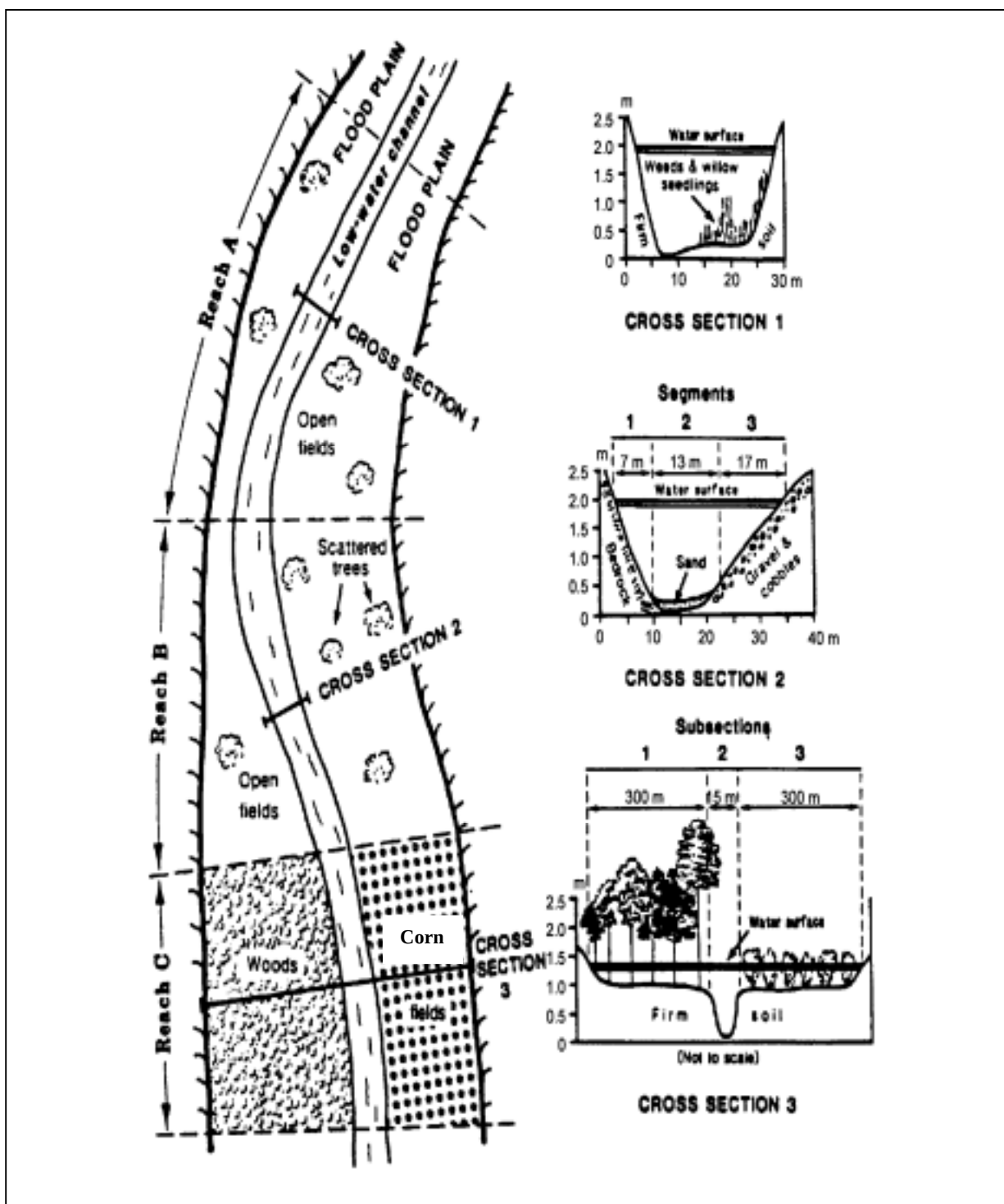
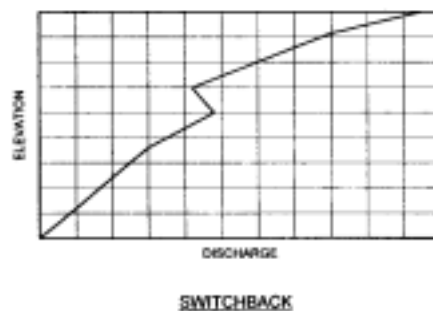


Figure 7-15 Hypothetical Cross Section Showing Reaches, Segments And Subsections Used In Assigning n Values

Source: FHWA, 1984

- **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 it had for a lesser water depth. With the combination of the lower hydraulic radius and the slightly larger cross-sectional area, a discharge is computed which is lower than the discharge based upon the lower water depth. **More subdivisions within such cross sections should be used in order to avoid the switchback.**



This phenomenon can occur in any type of conveyance computation, including the step-backwater method. Computer logic can be seriously confused if a switchback were to occur in any cross section being used in a step backwater program. For this reason, the cross section should always be subdivided with respect to both vegetation and geometric changes. Note that the actual n-value, itself, may be the same in adjacent subsections.

7.7.3 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 cross section properties including geometry, slope and roughness. It implicitly assumes the existence of steady, uniform flow; however, uniform flow rarely exists in either artificial or stream channels. Nevertheless, the single-section method is often used to design artificial channels for uniform flow as a first approximation, and to develop a stage-discharge rating curve in a stream channel for tailwater determination at a culvert or storm drain outlet.

A stage-discharge curve is a graphical relationship of streamflow depth or elevation to discharge at a specific point on a stream. This relationship should cover a range of discharges up to at least the base (100-year) flood. The stage-discharge curve can be determined as follows.

- Select the typical cross section at or near the location where the stage-discharge curve is needed.
- Subdivide cross section and assign n -values to subsections as described in Section 7.7.2
- Estimate water-surface slope. Since uniform flow is assumed, the average slope of the streambed can usually be used.
- Apply a range of incremental water surface elevations to the cross section.
- Calculate the discharge using Manning's equation for each incremental elevation. Total discharge at each elevation is the sum of the discharges from each subsection at that elevation. In determining hydraulic radius, the wetted perimeter should be measured only along the solid boundary of the cross section and not along the vertical water interface between subsections.
- After the discharge has been calculated at several incremental elevations, a plot of stage versus discharge should be made. This plot is the stage-discharge curve and it can be used to determine the water-surface elevation corresponding to the design discharge or other discharge of interest.

Alternatively, a graphical technique such as that given in Figure 7-16 or a nomograph as in Figure 7-17 can be used for trapezoidal and prismatic channels. The best approach, especially in the case of stream channels, is to use a computer program such as WSPRO, HEC-RAS, or HEC-2 to obtain the normal depth.

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

Alluvial channels present a more difficult problem in establishing stage-discharge relations by the single-section method because the bed itself is deformable and may generate bed forms such as ripples and dunes in lower regime flows. These bed forms are highly variable with the addition of form resistance, and selection of a value of Manning's n is not straightforward. Instead, several methods outlined in (Vanoni, 1977) have been developed for this case (Einstein-Barbarossa;

Kennedy-Alam-Lovera; and Engelund) and should be followed unless it is possible to obtain a measured stage-discharge relation.

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

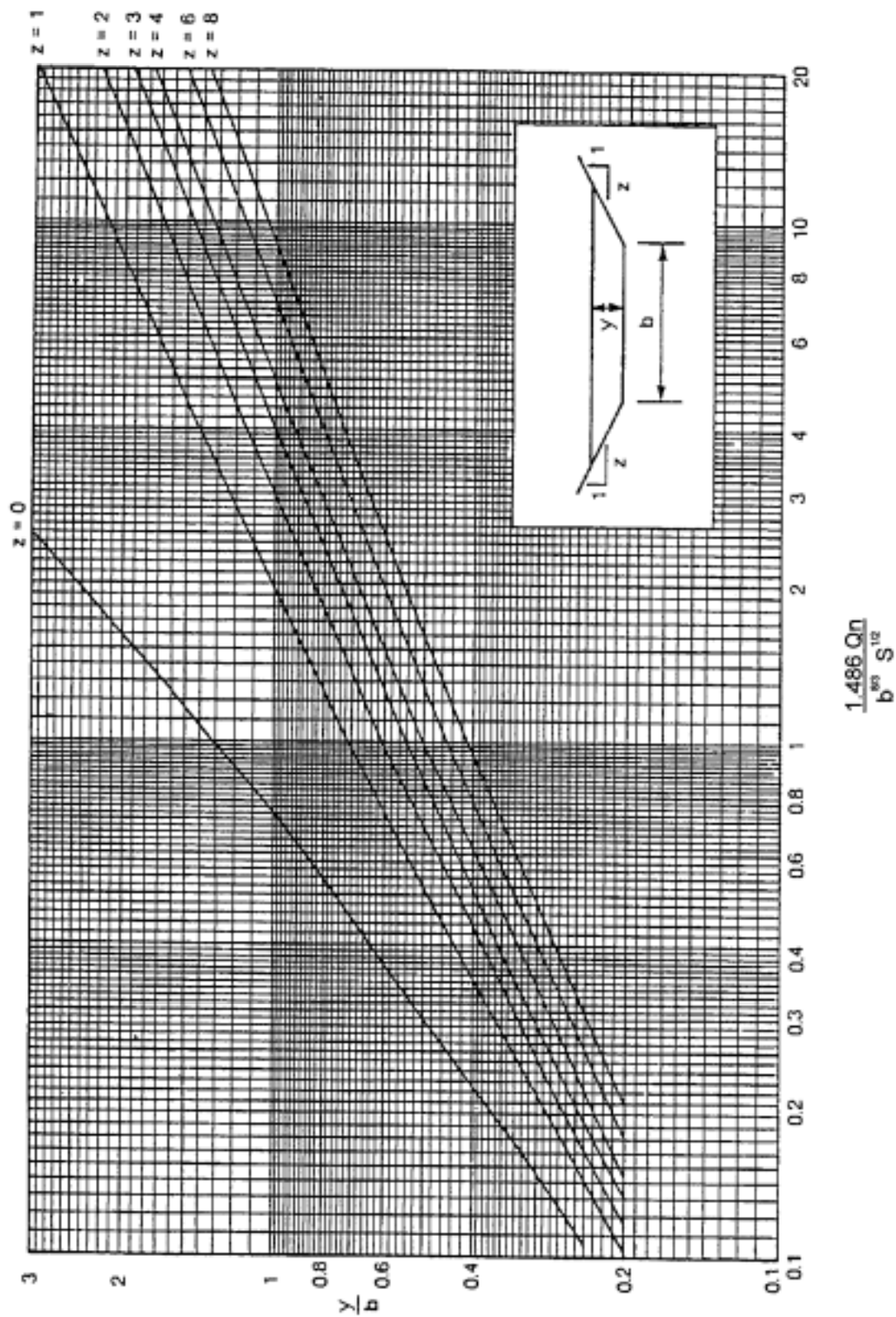


Figure 7-16 Trapezoidal Channel Capacity Chart

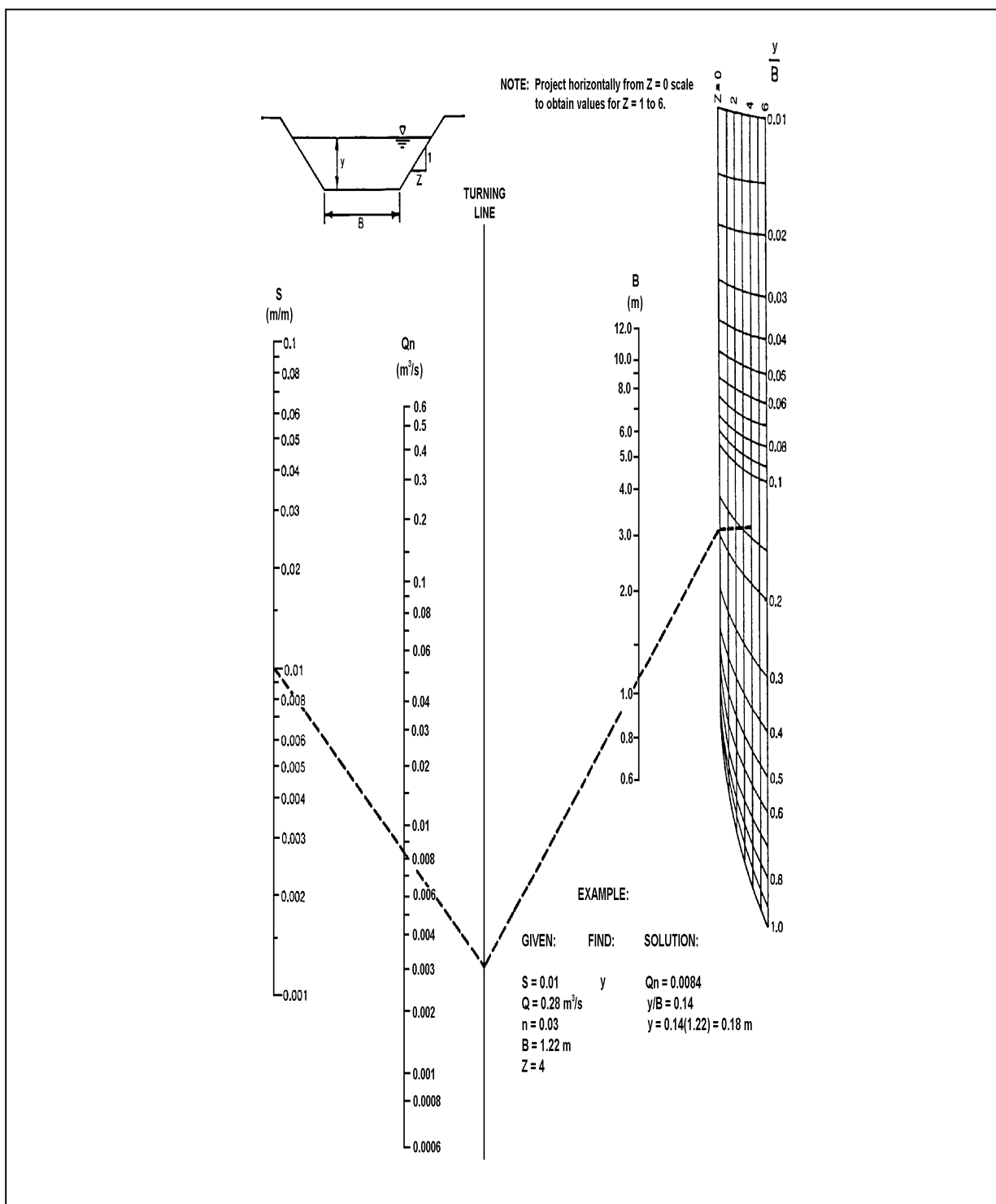


Figure 7-17 Nomograph For Normal Depth – metric units

Source: HEC-15

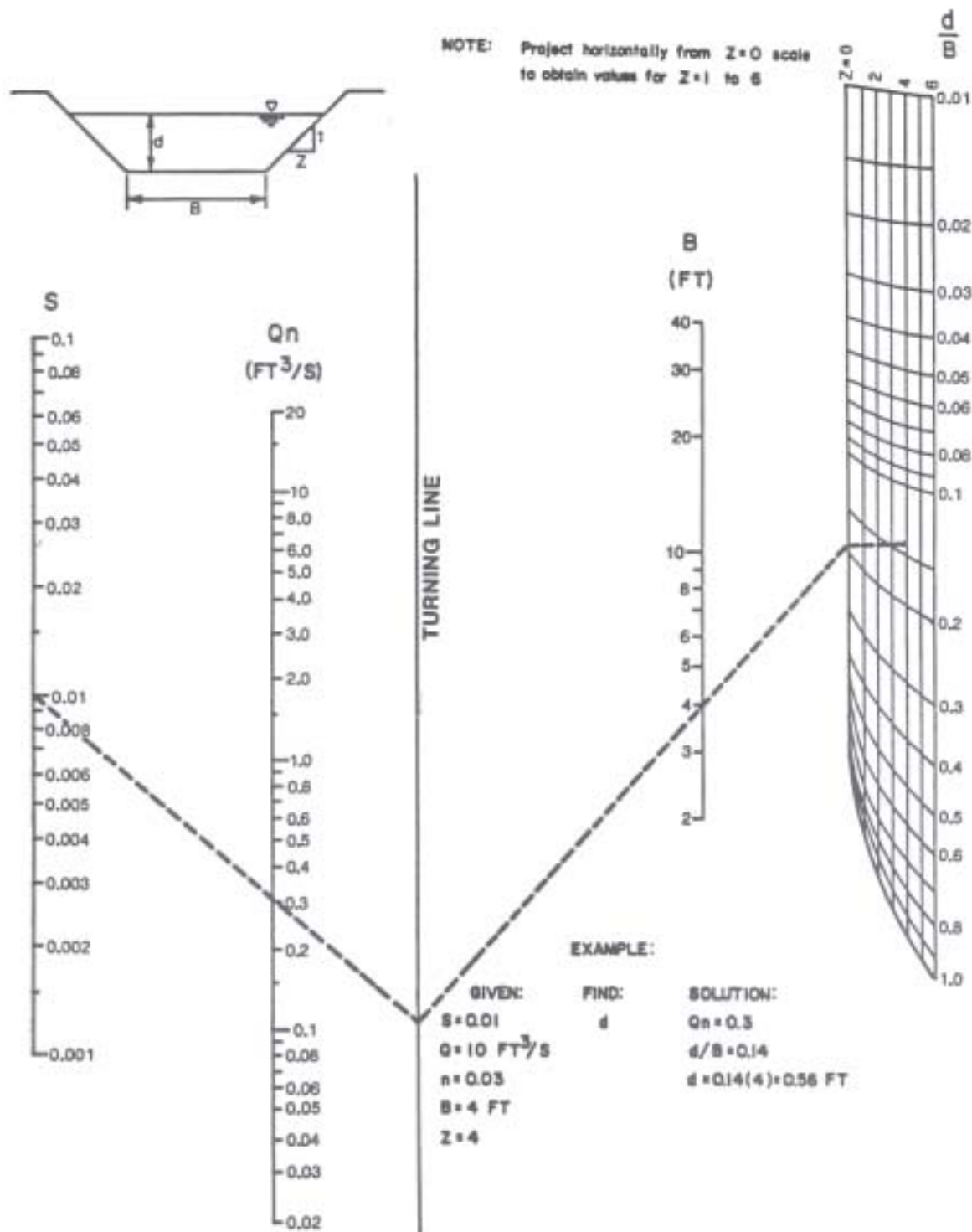


Figure 7-17.1 Nomograph For Normal Depth – English units

7.7.4 Step-Backwater Analysis

Step-backwater analysis is useful for determining unrestricted water surface profiles where a highway crossing is planned, and for analyzing how far upstream the water surface elevations are affected by a culvert or bridge. Because the calculations involved in this analysis are tedious and repetitive, it is recommended that a computer program such as the USGS/FHWA program Corps of Engineers HEC-2 or HEC-RAS or WSPRO be used.

These models are widely used for calculating water surface profiles for steady gradually varied flow in natural or constructed channels. Both subcritical and supercritical flow profiles can be calculated. The effects of bridges, culverts, weirs and structures in the floodplain may be also considered in the computations. The HEC-2 and HEC-RAS are also designed for application in floodplain management and flood insurance studies. These are steady flow, fixed-bed, one-dimensional models. Special analysis techniques (see Section 7.7.8) should be considered for complex situations where a step-backwater analysis might not give the desired level of accuracy.

7.7.5 Step-Backwater Models Methodology

The computation of water surface profiles by HEC-RAS, HEC-2 and WSPRO is based on the standard step method in which the stream reach of interest is divided into a number of subreaches by cross sections spaced such that the flow is gradually-varied in each subreach. The energy equation is then solved in a step-wise fashion for the stage at one cross section based on the stage at the previous cross section.

The method requires definition of the geometry and roughness of each cross section as discussed in Section 7.7.2. Manning's n values can vary both horizontally across the section as well as vertically. 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.

To amplify on the methodology, the energy equation is repeated from Section 7.4.11:

$$h_1 + \alpha_1(V_1^2/2g) = h_2 + \alpha_2(V_2^2/2g) + h_L \quad (7.23)$$

Where: h_1 and h_2 are the upstream and downstream stages, respectively, m (ft)

α = velocity distribution coefficient

V = mean velocity for the upstream and downstream stages, respectively, m/s (ft/s)

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

The stage h is the sum of the elevation head z at the channel bottom and the pressure head, or depth of flow y , i.e., $h = z + y$. The energy equation is solved between successive stream reaches with nearly uniform roughness, slope and cross-sectional properties.

The total head loss is calculated from:

$$h_L = K_m[(\alpha_1 V_1^2/2g) - (\alpha_2 V_2^2/2g)] + \underline{S}_f L \quad (7.24)$$

Where: K_m = expansion or contraction loss coefficient

$\underline{S}_f$ = friction slope - the mean slope of the energy grade line evaluated from Manning's equation and a selected averaging technique, m/m (ft/ft)

L = discharge-weighted or conveyance-weighted reach length, m (ft)

These equations are solved numerically in a step-by-step procedure called the Standard Step Method from one cross section to the next.

The default values of the minor loss coefficient K_m are zero and 0.1 for contractions and 0.5 and 0.3 for expansions in WSPRO and HEC-2, respectively. HEC-RAS has the same default values as HEC-2. The range of these coefficients, from ideal transitions to abrupt changes, is 0.0 to 1.0 for expansions and 0.0 to 0.5 for contractions. Guidance on the selection of expansion and contraction loss coefficients for bridges can be found in the HEC-RAS Hydraulic Reference Manual (USCE).

WSPRO calculates a conveyance-weighted reach length, L , as:

$$L = [(L_{lob}K_{lob} + L_{ch}K_{ch} + L_{rob}K_{rob})/(K_{lob} + K_{ch} + K_{rob})] \quad (7.25)$$

Where: L_{lob} , L_{ch} , L_{rob} = flow distance between cross sections in the left overbank, main channel and right overbank, respectively, m (ft)

K_{lob} , K_{ch} , K_{rob} = conveyance in the left overbank, main channel and right overbank, respectively, of the cross section with the unknown water surface elevation

HEC-2 and HEC-RAS calculate a discharge-weighted reach length, L , as:

$$L = [(L_{lob}Q_{lob} + L_{ch}Q_{ch} + L_{rob}Q_{rob})/(Q_{lob} + Q_{ch} + Q_{rob})] \quad (7.26)$$

Where: L_{lob} , L_{ch} , L_{rob} = flow distance between cross sections in the left overbank, main channel and right overbank, respectively, m (ft)

Q_{lob} , Q_{ch} , Q_{rob} = arithmetic average of flows between cross section for the left overbank, main channel and right overbank, respectively, m^3/s (ft^3/s)

WSPRO, HEC-RAS and HEC-2 allow the user the following options for determining the friction slope, S_f :

- Average conveyance equation

$$S_f = [(Q_u + Q_d)/(K_u + K_d)]^2 \quad (7.27)$$

- Average friction slope equation

$$S_f = (S_{fu} + S_{fd})/2 \quad (7.28)$$

- Geometric mean friction slope equation

$$S_f = (S_{fu}S_{fd})^{1/2} \quad (7.29)$$

- Harmonic mean friction slope equation

$$S_f = (2S_{fu}S_{fd})/(S_{fu} + S_{fd}) \quad (7.30)$$

Where: Q_u, Q_d = discharge at the upstream and downstream cross sections, respectively, m^3/s (ft^3/s)
 K_u, K_d = conveyance at the upstream and downstream cross sections, respectively
 S_{fu}, S_{fd} = friction slope at the upstream and downstream cross sections, respectively,
 m/m (ft/ft)

The default option is the geometric mean friction slope equation in WSPRO and the average conveyance equation in HEC-2 and HEC-RAS.

7.7.6 Water Surface Profile Computation

Water surface profile computation requires a beginning value of elevation or depth (boundary condition) and proceeds upstream for subcritical flow and downstream for supercritical flow. In the case of supercritical flow, critical depth is often the boundary condition at the control section, but in subcritical flow, uniform flow and normal depth may be the boundary condition. The starting depth in this case can either be found by the single-section method (slope-area method) or by computing the water surface profile upstream to the desired location for several starting depths and the same discharge. These profiles should converge toward the desired normal depth at the control section to establish one point on the stage-discharge relation. If the several profiles do not converge, then the stream reach may need to be extended downstream, or a shorter cross section interval should be used, or the range of starting water-surface elevations should be adjusted. In any case, a plot of the convergence profiles can be a very useful tool in such an analysis (see Figure 7-18).

Given a long enough stream reach, the water surface profile computed by step-backwater will converge to normal depth at some point upstream for subcritical flow. Establishment of the upstream and downstream boundaries of the stream reach is required to define limits of data collection and subsequent analysis. Calculations must begin sufficiently far downstream to assure accurate results at the structure site, and continued a sufficient distance upstream to accurately determine the impact of the structure on upstream water surface profiles (see Figure 7-19).

The Corps of Engineers (USCE, 1986) developed equations for determining upstream and downstream reach lengths as follows:

$$L_{dn} = 1.2 (HD^{0.8}/S) \quad (L_{dn} = 1.5 (HD^{0.8}/S)) \quad (7.31)$$

$$L_u = 2.1 [(HD^{0.6})(HL^{0.5})]/S \quad (7.32)$$

Where: L_{dn} = downstream study length (along main channel), m (ft) (for normal depth starting conditions)

L_u = estimated upstream study length (along main channel), m (ft) (required for convergence of the modified profile to within 0.03 m of the base profile). This length may be extended as necessary to satisfy regulatory requirements for profile convergence.

HD = average hydraulic depth (1% chance event flow area divided by the top width), m (ft)

S = average reach slope, m/m (ft/ft)

HL = headloss ranging between 0.15 and 1.5 m (0.05 and 5 ft) at the channel crossing structure for the 1% chance flood, m(ft)

References (Davidian, 1984 and USCE, 1986) are very valuable sources of additional guidance on the practical application of the step-backwater method to highway drainage problems involving

open-channels. These references contain more specific guidance on cross section determination, location and spacing and stream reach determination. Reference (USCE, 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.

7.7.7 Water Surface Profile Computation Procedure

A sample procedure is taken from "Hydrologic Engineering Methods For Water Resources Development - Volume 6, Water Surface Profiles," The Hydrologic Engineering Center, Corps of Engineers, U.S. Army, Davis, California.

A convenient form for use in calculating water surface profiles is shown in Figure 7-20. In summary, columns 2 and 4 through 12 are devoted to solving Manning's equation to obtain the energy loss due to friction, columns 13 and 14 contain calculations for the velocity distribution across the section, columns 15 through 17 contain the average kinetic energy, column 18 contains calculations for "other losses" (expansion and contraction losses due to interchanges between kinetic and potential energies as the water flows), and column 19 contains the computed change in water surface elevation. Conservation of energy is accounted for by proceeding from section to section down the computation form.

Column 1 - CROSS SECTION NO., is the cross section identification number. Kilometers upstream from the mouth are recommended.

Column 2 - ASSUMED, is the assumed water surface elevation which must agree with the resulting computed water surface elevation within ± 0.015 m (0.05 ft), or some allowable tolerance, for trial calculations to be successful.

Column 3 - COMPUTED, is the rating curve value for the first section, but thereafter, is the value calculated by adding ΔWS to the computed water surface elevation for the previous cross section.

Column 4 - A , is the cross section area. If the section is complex and has been subdivided into several parts (e.g., left overbank, channel and right overbank) use one line of the form for each sub-section and sum to get A_t , the total area of cross section.

Column 5 - R , is the hydraulic radius. Use the same procedure as for column 4 if section is complex, but do not sum subsection values.

Column 6 - $R^{2/3}$, is $2/3$ power of hydraulic radius.

Column 7 - n , is Manning roughness coefficient.

Column 8 - K , is conveyance and is defined as $(C_m AR^{2/3}/n)$ where C_m , is 1 for metric units (1.49 ft). If the cross section is complex, sum subsection K values to get K_t .

Column 9 - $\bar{K}_t$, is average conveyance for the reach, and is calculated by $0.5(K_{td} + K_{tu})$ where subscripts D and U refer to downstream and upstream ends of the reach, respectively.

Column 10 - $\bar{S}_f$, is the average friction slope through the reach determined by $(Q/\bar{K}_t)^2$.

Column 11 - L , is the discharge-weighted or conveyance-weighted reach length.

Column 12 - h_f , is energy loss due to friction through the reach and is calculated by $h_f = (Q/\bar{K}_t)^2 L = \bar{S}_f L$.

Column 13 - $\Sigma(K^3/A^2)$, is part of the expression relating distributed flow velocity to an average value. If the section is complex, calculate one of these values for each subsection and sum all subsection values to get a total. If one subsection is used, Column 13 is not needed and Column 14 equals one.

Column 14 - α , is the velocity distribution coefficient and is calculated by $\Sigma(K^3/A^2)/(K_t^3/A_t^2)$ where the numerator is the sum of values in Column 13 and the denominator is calculated from K_t and A_t .

Column 15 - V , is the average velocity and is calculated by Q/A_t .

Column 16 - $\alpha V^2/2g$, is the average velocity head corrected for flow distribution.

Column 17 - $\Delta(\alpha V^2/2g)$, is the difference between velocity heads at the downstream and upstream sections. A positive value indicates velocity is increasing, therefore, use a contraction coefficient for "other losses." A negative value indicates the expansion coefficient should be used in calculating "other losses."

Column 18 - h_o , is "other losses," and is calculated by multiplying either the expansion or contraction coefficient, K_m , times the absolute value of column 17.

Column 19 - ΔWS , is the change in water surface elevation from the previous cross section. It is the algebraic sum of columns 12, 17 and 18.

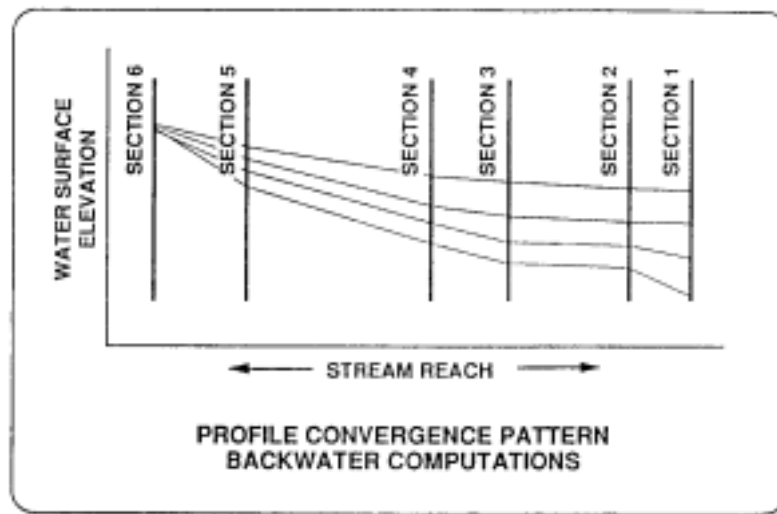


Figure 7-18 Profile Convergence Pattern Backwater Computation

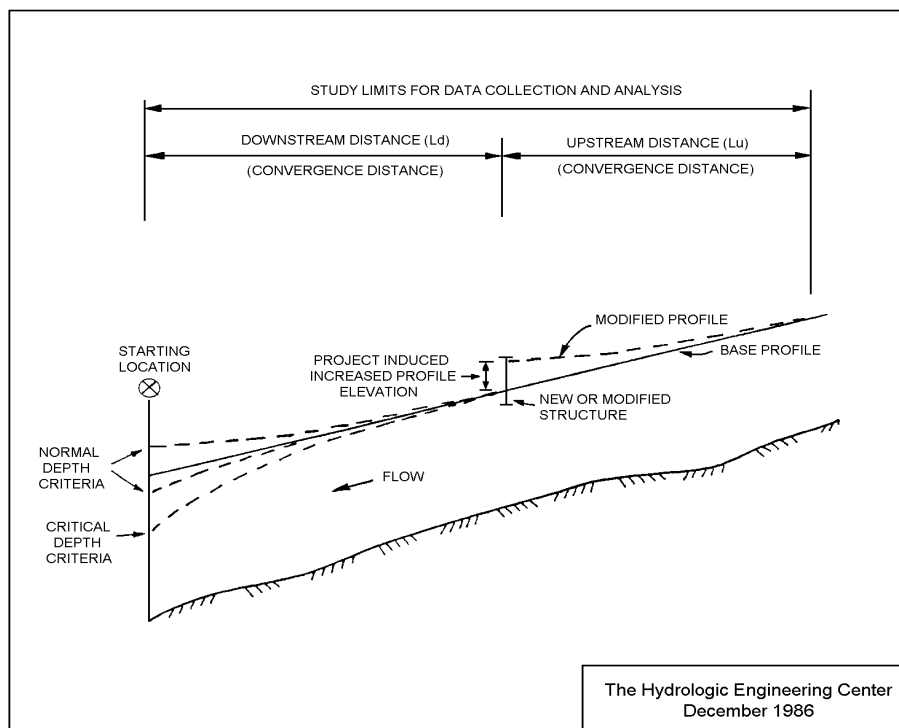


Figure 7-19 Profile Study Limits

Source: USCE, 1986

[illegible]

Figure 7-20 Water Surface Profile Form

7.7.8 Water And Sediment Routing

The BRI-STARS (Bridge Stream Tube Model for Sediment Routing Alluvial River Simulation) Model was developed by the National Cooperative Highway Research Program and FHWA. It is based on utilizing the stream tube method of calculation which allows the lateral and longitudinal variation of hydraulic conditions as well as sediment activity at various cross sections along the study reach. Both energy and momentum functions are used in the BRI-STARS model so the water surface profile computation can be carried out through combinations of subcritical and supercritical flows without interruption. The stream tube concept is used for hydraulic computations in a semi-two-dimensional way. Once the hydraulic parameters in each stream tube are computed, the scour or deposition in each stream tube determined by sediment routing will give the variation of channel geometry in the vertical direction.

The BRI-STARS model contains a rule-based expert system program for classifying streams by size, bed and bank material stability, planform geometry and other hydrologic and morphological features. Due to the complexities of a single classification system that utilizes all parameters, no universally acceptable stream classification method presently exists. Consequently this model does not contain a single methodology for classifying all streams. Instead, methodologies were first classified according to the channel sediment sizes they were derived for, then within each size group, one or more classification schemes have been included to cover a wider range of environments. The stream classification information can be used to assist in the selection of model parameters and algorithms (See Section 7.5).

Applications of the BRI-STARS can be summarized as follows:

- Fixed bed model to compute water surface profiles for subcritical, supercritical, or the combination of both flow conditions involving hydraulic jumps
- Movable bed model to route water and sediment through alluvial channels
- Use of stream tubes to allow the model to compute the variation of hydraulic conditions and sediment activity in the longitudinal as well as the lateral direction
- The armoring option allows simulation of longer term riverbed changes
- The minimization procedure option allows the model to simulate channel widening and narrowing processes
- The local bridge scour option allows the computation of pier and abutment scour
- The bridge routines for fixed geometry mode from WSPRO are available as an option in the program

7.7.9 Design Procedure for Natural Channels (Type B)

The analysis of a natural channel in most cases is in conjunction with the design of a highway hydraulic structure such as a culvert or bridge. In general, the objective is to convey the water along or under the highway in such a manner that will 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 detail of studies necessary should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream and adjoining floodplain as outlined in this section.

Although the following step-by-step procedure may not be appropriate for all possible applications, it does outline a process which will usually apply.

Step 1 Assemble site data and project file**A. Data Collection (see Data Collection, Chapter 5)**

- Topographic, site and location maps
- Roadway profile
- Photographs
- Field reviews
- Design data at nearby structures on the same watercourse
- Gaging records
- Historic flood data and local knowledge
- Bridge Scour Evaluations at site or at nearby bridges

B. Studies by other agencies

- Flood insurance studies
- Floodplain studies
- Watershed studies
- SCEL studies

C. Environmental constraints

- Floodplain encroachment
- Floodway designation
- Fish and wildlife habitat
- Commitments made during project development

D Design criteria

- See Section 7.3

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

- Qualitative assessment
- Single-section analysis
- Step-backwater analysis
- Special analysis techniques

C. Determine additional survey information

- 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 Hydrology, Chapter 6

Step 4 Perform hydraulic analysis

- A. Single-section analysis (7.7.3)
- Select representative cross section (7.7.2)
 - Select appropriate n values (Table 7-1)
 - Compute stage-discharge relationship
- B. Step-backwater analysis (7.7.4)
- C. Calibrate with known high water

Step 5 Perform stability analysis

- A. Geomorphic factors
- B. Hydraulic factors
- C. Stream response to change
- D. Scour Evaluation
- E. Determine velocities

Step 6 Design countermeasures

- A. Criteria for selection
- Erosion mechanism
 - Stream characteristics
 - Construction and maintenance requirements
 - Vandalism considerations
 - 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
- See Reference List
- HEC-18

Step 7 Documentation

- Prepare report and file with background information
- See Section 7.7.10

7.7.10 Documentation (Type B, Channel)

The following items shall be included in the documentation file (see Chapter 1, Section 1.6). The intent is not to limit data to only those items listed, but rather establish a minimum requirement consistent with the channel design procedures as outlined in this chapter. If circumstances are such that the design is prepared other than the normal procedures or is governed by factors other than hydrologic/hydraulic factors, then a narrative summary detailing the design basis shall appear with the other data.

The following items shall be included in the documentation file:

- stage discharge curves for the design, 100-year 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, 100-year and any historical floods
- energy dissipation calculations and designs
- copies of all computer analyses

7.8 Bank Protection (Type B Channels)

7.8.1 Purpose

One of the hazards of placing a highway near a river or stream channel or other water body is the potential for erosion of the highway embankment by moving water. If erosion of the highway embankment is to be prevented, bank protection must be anticipated and the proper type and amount of protection must be provided in the right locations.

Four methods of protecting a highway embankment from bank erosion are available to the designer. These are listed in descending order of desirability:

- relocating the highway away from the stream or water body
- protecting the embankment from erosion
- changing the direction of the current with training works
- moving the water body away from the highway (channel change)

This section provides procedures for the design of revetments to be used as channel bank protection and channel linings on larger streams and rivers (**i.e., having design discharges generally greater than 1.4 m³/s (50 cfs)**). Procedures are also presented for riprap protection at bridge piers and abutments. For small discharges, procedures presented in the roadside channel section should be used. Emphasis in this section has been placed on rock riprap revetments due to their costs, environmental considerations, flexible characteristics and widespread usage. Other channel stabilization methods such as spurs, guide banks retard structures, longitudinal dikes and bulkheads are discussed in "Stream Stability at Highway Structures," Hydraulic Engineering Circular No. 20.

7.8.2 Erosion Potential

Channel and bank stabilization is essential to the design of any structure affected by the water environment. The identification of the potential for bank erosion, and the subsequent need for stabilization, is best accomplished through observation. A three level analysis procedure is provided in FHWA Hydraulic Engineering Circular No. 20. This procedure is described in Section 7.5.2. The three level analysis provides a rigorous procedure for determining the geomorphological characteristics, evaluating the existing conditions through field observations and determining the hydraulic and sediment transport properties of the stream. If sufficient information is obtained at any level of the analysis to solve the problem, the procedure may be stopped without going on to the other levels.

Observations provide the most positive indication of erosion potential. Observation comparison can be based on historic information, or current site conditions. Aerial photographs, old maps and surveying notes and bridge design files and river survey data that are available as described in the Data Collection chapter, as well as gaging station records and interviews of long-time residents can provide documentation of any recent and potentially current channel movement or bank instabilities.

In addition, current site conditions can be used to evaluate stability. Even when historic information indicates that a bank has been relatively stable in the past, local conditions may indicate more recent instabilities. Local site conditions which are indicative of instabilities may include tipping and falling of vegetation along the bank, cracks along the bank surface, the presence of slump blocks, fresh vegetation laying in the channel near the channel banks, deflection of channel

flows in the direction of the bank due to some recently deposited obstruction or channel course change, fresh vertical face cuts along the bank, locally high velocities along the bank, new bar formation downstream from an eroding bank, local headcuts, pending or recent cutoffs, etc. It is also important to recognize that the presence of any one of these conditions does not in itself indicate an erosion problem; some bank erosion is common in all channels even when the channel is stable.

7.8.3 Symbols And Definitions

To provide consistency within this section as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in many bank and shore protection publications. Where the same symbol is used for more than one definition, the symbol will be defined where it is used.

Table 7-6 Symbols And Definitions

<u>Symbol</u>	<u>Definition</u>	<u>Units</u>
AOS	Apparent opening size in filter cloth	mm (inches)
A	Coefficient used to determine the apparent opening size	
C	Coefficient, relates free vortex motion to velocity streamlines for unequal radius of curvature	-
C _u	Uniformity coefficient	
D ₅₀	The median bed material size	m or mm (ft or inches)
D ₁₅	The 15% finer particle size	m or mm (ft or inches)
D ₈₅	The 85% finer particle size	m or mm (ft or inches)
d _{avg}	Average flow depth in the main flow channel	m (ft)
d _s	Estimated probable maximum depth of scour	m(ft)
g	Gravitational acceleration (9.81 m/s ²)	m/s ² (ft/s ²)
H	Wave height	m (ft)
k	Permeability	cm/s or mm/s (inches/s)
K ₁	Correction term reflecting bank angle	-
n	Manning's roughness coefficient	-
O ₉₅	Opening size in the geotextile material for which 95% of the openings are smaller	mm (inches)
Q _{mc}	Discharge in the zone of main channel flow	m ³ /s (ft ³ /s)
R	Hydraulic radius	m (ft)
R	Wave runup	m (ft)
R _o	Mean radius of the channel centerline at the bend	m (ft)
S _f , S	Friction slope or energy grade line slope	m/m (ft/ft)
SF	Stability factor	-
S _s , s	Specific gravity of the riprap material	-
T	Topwidth of the channel between its banks	m (ft)
V	Velocity	m/s (ft/s)
V _a	Mean channel velocity	m/s (ft/s)
W ₅₀	Weight of the median particle	kg (lb)
Z	Superelevation of the water surface	m (ft)
γ	Unit weight of the riprap material	kg/m ³ (lb/ft ³)
θ	Bank angle with the horizontal	degrees
Φ	Riprap material's angle of repose	degrees

7.8.4 Design Criteria

To provide an acceptable standard level of service, the Department traditionally employs widely-used pre-established design frequencies which are based on the importance of the transportation facility to the system and the allowable risk for that facility. This would be true of revetment protection. In addition, design standards of other agencies that have control or jurisdiction over the waterway or facility concerned should be incorporated or addressed in the design.

The criteria for the design of channels is found in the hydrology chapter and Section 7.3.3.

7.8.5 Bank And Lining Failure Modes

Potential Failures

Prior to designing a bank stabilization scheme, it is important to be aware of common erosion mechanisms and revetment failure modes, and the causes or driving forces behind bank erosion processes. Inadequate recognition of potential erosion processes at a particular site may lead to failure of the revetment system.

Many causes of bank erosion and revetment failure have been identified. Some of the more common include abrasion, debris flows, water flow, eddy action, flow acceleration, unsteady flow, freeze/thaw, human actions on the bank, ice, precipitation, waves, toe erosion and subsurface flows. However, it is most often a combination of mechanisms which cause bank and revetment failure and the actual mechanism or cause is usually difficult to determine. Failures are better classified by failure mode including:

- particle erosion
- translational slide
- modified slump
- slump

Particle Erosion

Particle erosion is the most commonly considered erosion mechanism. Particle erosion results when the tractive force exerted by the flowing water exceeds the bank material's ability to resist movement. In addition, if displaced stones are not transported from the eroded area, a mound of displaced rock will develop on the channel bed. This mound has been observed to cause flow concentration along the bank, resulting in further bank erosion.

A special type of particle erosion results in loss of the underlying material resulting in undermining and eventual collapse of the revetment protection. Usually the underlying material is lost through the revetment or piped under the toe of the revetment protection. This failure is very common in and extremely damaging to rigid types of protective linings. Providing a suitable filter, either natural or fabrics in conjunction with hydrostatic relief features will prevent this failure.

Another frequent type of particle erosion failure occurs at the edges of the protective feature. The interface creates turbulence which in turn increases the tractive stresses placed on the protective layer, underlying layers, and the natural bank material beyond the revetment. This failure area needs to receive special attention since extension of the protective feature usually only moves, not eliminates, the failure.

Translation Slide

A translational slide is a failure of riprap caused by the downslope movement of a mass of stones, with the fault line on a horizontal plane. The initial phases of a translational slide are indicated by cracks in the upper part of the riprap bank that extend parallel to the channel. As the slide progresses, the lower part of riprap separates from the upper part, and moves downslope as a homogeneous body. A resulting bulge may appear at the base of the bank if the channel bed is not scoured.

Modified Slump

The failure of riprap referred to as modified slump is the mass movement of material along an internal slip surface within the riprap blanket; the underlying material supporting the riprap does not fail. This type of failure is similar in many respects to the translational slide, but the geometry of the damaged riprap is similar in shape to initial stages of failure caused by particle erosion.

Slump

Slump is a rotational-gravitational movement of material along a surface of rupture that has a concave upward curve. The cause of slump failures is related to shear failure of the underlying base material that supports the riprap revetment. The primary feature of a slump failure is the localized displacement of base material along a slip surface, which is usually caused by excess pore pressure that reduces friction along a fault line in the base material.

7.8.6 Revetment Types

Common Types

The types of slope protection or revetment commonly used for bank and shore protection and stabilization include:

- riprap
- gabions
- concrete block slope protection
- concrete slope pavement
- articulating block mat systems

Riprap

Riprap has been described as a layer or facing of rock, dumped or hand-placed to prevent erosion, scour, or sloughing of a structure or embankment. Materials other than rock are also referred to as riprap; for example, rubble, broken concrete slabs and preformed concrete shapes (slabs, blocks, rectangular prisms, etc.). These materials are similar to rock in that they can be hand-placed or dumped onto an embankment to form a flexible revetment.

Gabions

Wire-enclosed rock, or gabion, revetments consist of rectangular wire mesh baskets filled with rock. These revetments are formed by filling pre-assembled wire baskets with rock, and anchoring to the channel bottom or bank. Wire-enclosed rock revetments are generally of two types distinguished by shape: rock and wire mattresses, or blocks. In mattress designs, the individual wire mesh units are laid end to end and side to side to form a mattress layer on the channel bed or bank. The gabion baskets comprising the mattress generally have a depth dimension which is much smaller than their width or length. Block gabions, on the other hand, are more equal-dimensional, having depths that are approximately the same as their widths, and of the same order of magnitude as their lengths. They are typically rectangular or trapezoidal in shape. Block gabion revetments are formed by stacking the individual gabion blocks in a stepped fashion.

Concrete Block Slope Protection

Pre-cast concrete block revetments are a recent development. The pre-formed sections which comprise the revetment systems are butted together or joined in some fashion; as such, they form a continuous blanket or mat. The concrete blocks which make up the mats differ in shape and method of articulation, but share certain common features. These features include flexibility, rapid installation and provisions for establishment of vegetation within the revetment. The permeable nature of these revetments permits free draining of the bank materials; the flexibility, although limited, allows the mattress to conform to minor changes in the bank geometry. Their limited flexibility, however, makes them subject to undermining in environments characterized by large and relatively rapid fluctuations in the surface elevation of the channel bed and/or bank. Unlike wire-enclosed rock, the open nature of the pre-cast concrete blocks does promote volunteering of vegetation within the revetment.

Concrete Slope Pavement

Concrete pavement revetments (concrete slope pavement) are cast in place on a prepared slope to provide the necessary bank protection. Like grouted rock, concrete pavement is a rigid revetment which does not conform to changes in bank geometry due to a removal of foundation support by subsidence, undermining, outward displacement by hydrostatic pressure, slide action, or erosion of the supporting embankment at its ends. The loss of even small sections of the supporting embankment can cause complete failure of the revetment system. Concrete pavement revetments are also among the most expensive streambank protection designs. In the past, concrete pavement has been best utilized as a subaqueous revetment (on the bank below the water surface) with vegetation or some other less expensive upper-bank treatment.

7.8.7 Flow Resistance

The hydraulic analysis performed as a part of the riprap design process requires the estimation of Manning's roughness coefficient. Physical characteristics upon which the resistance equations are based include the channel base material, surface irregularities, variations in section geometry, bed form, obstructions, vegetation, channel meandering, flow depth and channel slope. In addition, seasonal changes in these factors must also be considered. See Section 7.4.11, for a discussion of the selection of Manning's n values.

7.8.8 Extent Of Protection

Extent of protection refers to the longitudinal and vertical extent of protection required to adequately protect the channel bank.

Longitudinal Extent

The longitudinal extent of protection required for a particular bank protection scheme is highly dependent on local site conditions. In general, the revetment should be continuous for a distance greater than the length that is impacted by channel-flow forces severe enough to cause dislodging and/or transport of bank material. Although this is a vague criteria, it demands serious consideration. Review of existing bank protection sites has revealed that a common misconception in streambank protection is to provide protection too far upstream and not far enough downstream.

One criteria for establishing the longitudinal limits of protection required is illustrated in Figure 7-21. As illustrated, the minimum distances recommended for bank protection are an upstream distance of 1.0 channel width and a downstream distance of 1.5 channel widths from corresponding reference lines. All reference lines pass through tangents to the bend at the bend entrance or exit. This criteria is based on analysis of flow conditions in symmetric channel bends under ideal laboratory conditions. Real-world conditions are rarely as simplistic.

In actuality, many site-specific factors have a bearing on the actual length of bank that should be protected. A designer will find the above criteria difficult to apply on mildly curving bends or on channels having irregular, non-symmetric bends. Also, other channel controls (such as bridge abutments) might already be producing a stabilizing effect on the bend so that only a part of the channel bend needs to be stabilized. In addition, the magnitude or nature of the flow event might only cause erosion problems in a very localized portion of the bend, requiring that only a short channel length be stabilized. Therefore, the above criteria should only be used as a starting point. Additional analysis of site-specific factors is necessary to define the actual extent of protection required.

Field reconnaissance is a useful tool for the evaluation of the longitudinal extent of protection required, particularly if the channel is actively eroding. In straight channel reaches, scars on the channel bank may be useful to help identify the limits required for channel bank protection. In this case, it is recommended that upstream and downstream limits of the protection scheme be extended a minimum of one channel width beyond the observed erosion limits.

In curved channel reaches, the scars on the channel bank can be used to establish the upstream limit of erosion. Here a minimum of one channel width should be added to the observed upstream limit to define the limit of protection. The downstream limit of protection required in curved channel reaches is not as easy to define. Since the natural progression of bank erosion is in the downstream direction, the present visual limit of erosion might not define the ultimate downstream limit. Additional analysis based on consideration of flow patterns in the channel bend may be required.

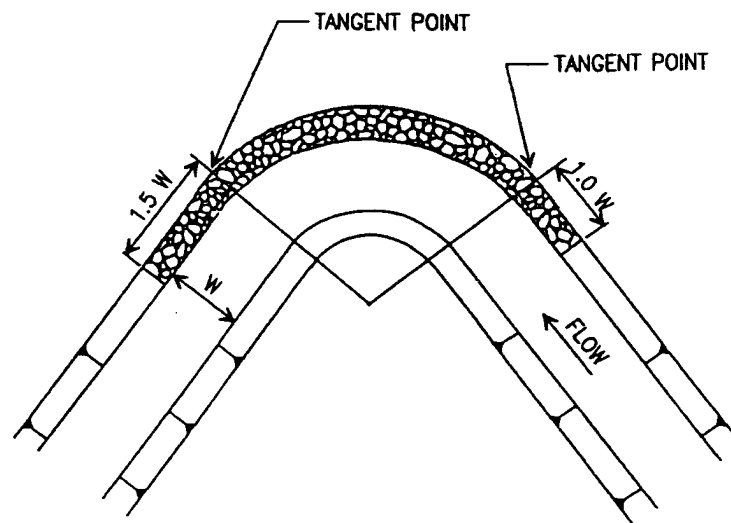


Figure 7-21 Longitudinal Extent Of Revetment Protection

Vertical Extent

The vertical extent of protection required of a revetment includes design height and foundation or toe depth.

Design Height

The design height of a riprap installation should be equal to the design highwater elevation plus an allowance for freeboard, generally 0.3 – 0.6m (1-2 ft). Freeboard is provided to ensure that the desired degree of protection will not be reduced by unaccounted factors. Some such factors include:

- wave action (from wind or boat traffic)
- superelevation in channel bends
- hydraulic jumps
- flow irregularities due to piers, transitions and flow junctions

In addition, erratic phenomena such as unforeseen embankment settlement, the accumulation of silt, trash and debris in the channel, aquatic or other growth in the channels and ice flows should be considered when setting freeboard heights. Also, wave run-up on the bank must be considered.

Wave Action

The prediction of wave heights from wind and boat generated waves is not as straightforward as other wave sources. Figure 7-22 provides a definition sketch for the wave height discussion to follow. The height of boat generated waves must be estimated from observations. The height of wind generated waves is discussed in Appendix A of this chapter.

It is necessary to estimate the magnitude of wave runup which results when waves impact the bank. Wave runup is a function of the design wave height, the wave period, bank angle and the bank surface characteristics (as represented by different revetment materials). For wave heights less than 0.6 m (2 ft), wave runup can be computed using Figure 7-23 and Table 7-7. The runup height (R) given in Figure 7-23 is for concrete pavement. Correction factors are provided in Table 7-7 for reducing the runup magnitude for other revetment materials. The correction factor is multiplied times the wave height to get the resulting wave runup (R).

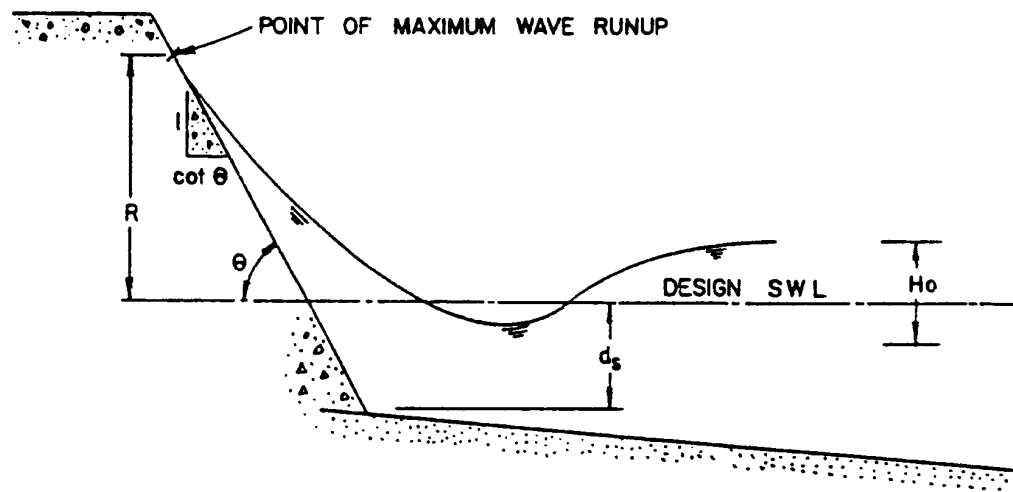


Figure 7-22 Wave Height Definition Sketch

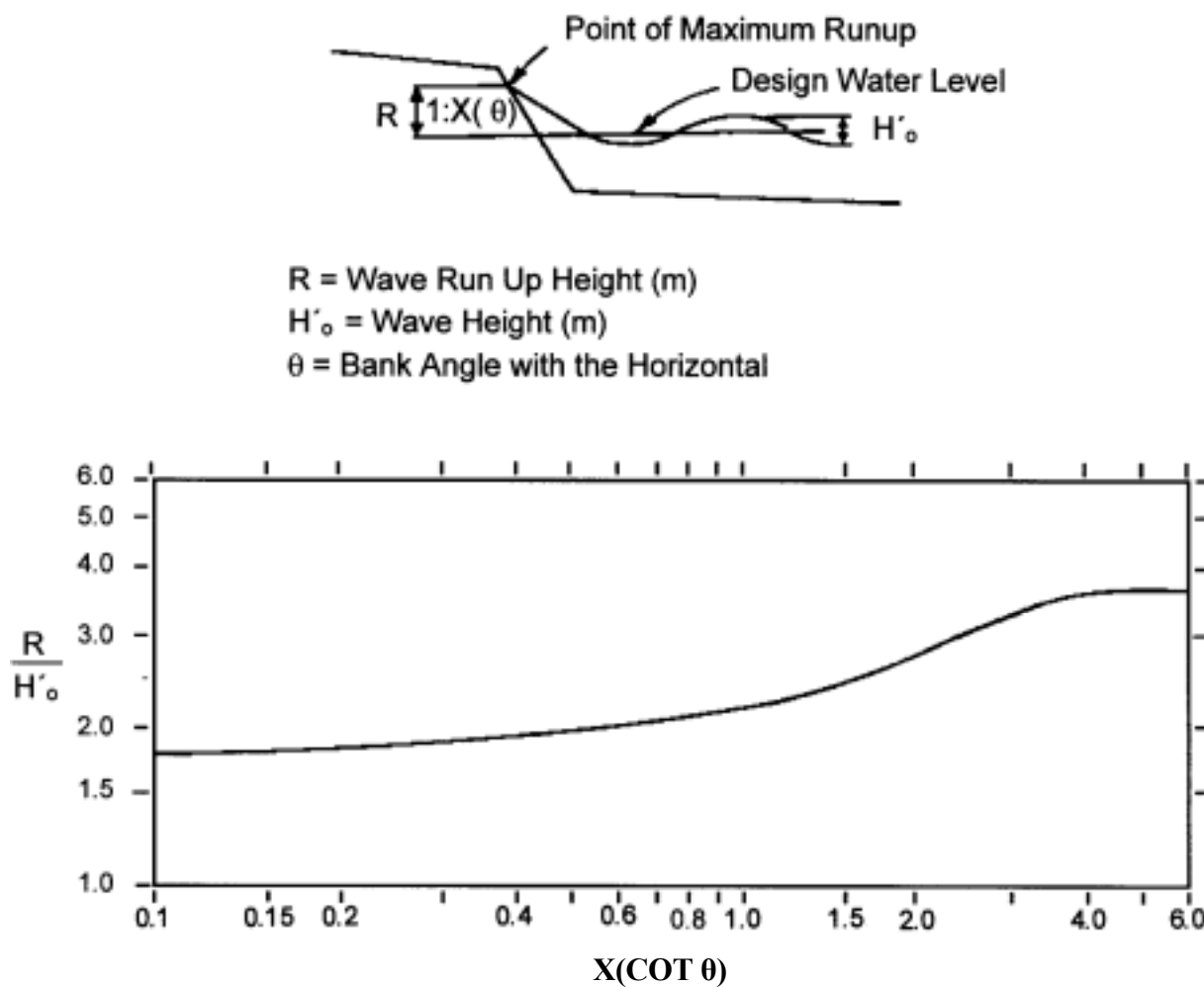


Figure 7-23 Wave Run-Up On Smooth, Impermeable Slopes

Table 7.7 Correction Factors For Wave Run-Up

Slope Surface Characteristics	Placement Method	Correction Factor
Concrete pavement	—	1.00
Concrete blocks (voids < 20%)	fitted	0.90
Concrete blocks (20% < voids > 40%)	fitted	0.70
Concrete blocks (40% < voids > 60%)	fitted	0.50
Grass	—	0.85 - 0.90
Rock riprap (angular)	random	0.60
Rock riprap (round)	random	0.70
Rock riprap (hand placed or keyed)	keyed	0.80
Grouted rock	—	0.90
Wire enclosed rocks/gabions	—	0.80

Flow In Channel Bends (Superelevation of Flow)

Flow conditions in channel bends are complicated by the distortion of flow patterns in the vicinity of the bend. In long, relatively straight channels, the flow conditions are uniform and symmetrical about the center line of the channel. However, in channel bends, the centrifugal forces and secondary currents produced lead to non-uniform and non-symmetrical flow conditions.

Special consideration must be given to the increased velocities and shear stresses that are generated as a result of non-uniform flow in bends.

Superelevation of flow in channel bends is another important consideration in the design of revetments. Although the magnitude of superelevation is generally small when compared with the overall flow depth in the bend (usually less than 0.3 m) it should be considered when establishing freeboard limits for bank protection schemes on sharp bends. The magnitude of superelevation at a channel bend may be estimated for subcritical flow by the following equation:

$$Z = C [(V_a^2 T)/(g R_o)] \quad (7.33)$$

where: Z = superelevation of the water surface, m (ft)

C = coefficient that relates free vortex motion to velocity streamlines for unequal radius of curvature,

V_a = mean channel velocity, m/s (ft/s)

T = water-surface width at section, m (ft)

g = gravitational acceleration - 9.81 m/s² (32.2 ft/s²)

R_o = the mean radius of the channel centerline at the bend, m (ft)

The coefficient C has been evaluated by the U.S. Geological Survey (USGS) and ranges between 0.5 and 3.0, with an average value of 1.5.

Freeboard

As indicated, there are many factors which must be considered in the selection of an appropriate freeboard height. As a minimum, it is recommended that a freeboard elevation of 0.3 (1 ft) be used in unconstricted reaches, and 0.6m (2 ft) in constricted reaches. When computational procedures indicate that additional freeboard may be required, the greater height should be used. In addition, it is recommended that the designer observe wave and flow conditions during various seasons of the year (if possible), consult existing records, and interview persons who have knowledge of past conditions when establishing the necessary vertical extent of protection required for a particular revetment installation.

Toe Depth

The undermining of revetment toe protection has been identified as one of the primary mechanisms of revetment failure. In the design of bank protection, estimates of the depth of scour are needed so that the protective layer is placed sufficiently low in the streambed to prevent undermining. The ultimate depth of scour must consider channel degradation as well as natural scour and fill processes.

The relationships presented in Equation 7.34 and 7.35 can be used to estimate the probable maximum depth of scour due to natural scour and fill phenomenon in straight channels, and in channels having mild bends. In application, the depth of scour, d_s , should be measured from the lowest elevation in the cross section. It should be assumed that the low point in the cross section may eventually move adjacent to the protection (even if this is not the case in the current survey).

$$d_s = 3.66 \text{ m (12 ft)} \quad \text{for } D_{50} < 1.5 \text{ mm (0.06 in)} \quad (7.34)$$

$$d_s = 1.738 D_{50}^{-0.11} \text{ m} \quad (d_s = 3.72 D_{50}^{-0.11} \text{ (ft)}) \quad \text{for } D_{50} > 1.5 \text{ mm (0.06 in)} \quad (7.35)$$

Where: d_s = estimated probable maximum depth of scour, m (ft)

D_{50} = median diameter of bed material, mm (inch)

The depth of scour predicted by Equations 7-34 and 7.35 must be added to the magnitude of predicted degradation and local scour (if any) to arrive at the total required toe depth.

7.9 Design Guidelines – Rock Riprap

7.9.1 Rock Riprap

This section contains design guidelines for the design of rock riprap. Guidelines are provided for bank slope, rock size, rock gradation, riprap layer thickness, filter design, edge treatment and construction considerations. In addition, typical construction details are illustrated. In most cases, the guidelines presented apply equally to rock and rubble riprap. Guidelines for other types are presented in HEC-11.

7.9.2 Bank Slope

A primary consideration in the design of stable riprap bank protection schemes is the slope of the channel bank. For riprap installations, normally the maximum recommended face slope is 1V:2H.

7.9.3 Rock Size

The stability of a particular riprap particle is a function of its size, expressed either in terms of its weight or equivalent diameter. In the following sections, relationships are presented for evaluating the riprap size required to resist particle and wave erosion forces.

7.9.4 Particle Erosion

Two methods or approaches have been used historically to evaluate a material's resistance to particle erosion. These methods are the permissible velocity approach and the permissible tractive force (shear stress) approach. Under the permissible velocity approach the channel is assumed stable if the computed mean velocity is lower than the maximum permissible velocity. The tractive force (boundary shear stress) approach focuses on stresses developed at the interface between flowing water and materials forming the channel boundary.

7.9.5 Design Relationship

A riprap design relationship that is based on tractive force theory yet has velocity as its primary design parameter is presented in Equation 7.36. The design relationship in Equation 7.36 is based on the assumption of uniform, gradually varying flow. Figure 7-25.1 presents a graphical solution to Equation 7.36. Equation 7.37 can be solved using Figures 7-26 and 7-27.

$$D_{50} = 0.00594 V_a^3 / (d_{avg}^{0.5} K_1^{1.5}) \quad (D_{50} = 0.001 V_a^3 / (d_{avg}^{0.5} K_1^{1.5})) \quad (7.36)$$

Where: D_{50} = the median riprap particle size, m (ft)
 C = correction factor (described below)
 V_a = the average velocity in the main channel, m/s (ft/s)
 d_{avg} = the average flow depth in the main flow channel, m (ft)

K_1 is defined as:

$$K_1 = [1 - (\sin^2 \theta / \sin^2 \Phi)]^{0.5} \quad (7.37)$$

Where: θ = the bank angle with the horizontal
 Φ = the riprap material's angle of repose

The average flow depth and velocity used in Equation 7.36 are main channel values. The main channel is defined as the area between the channel banks (see Figure 7-24 below).

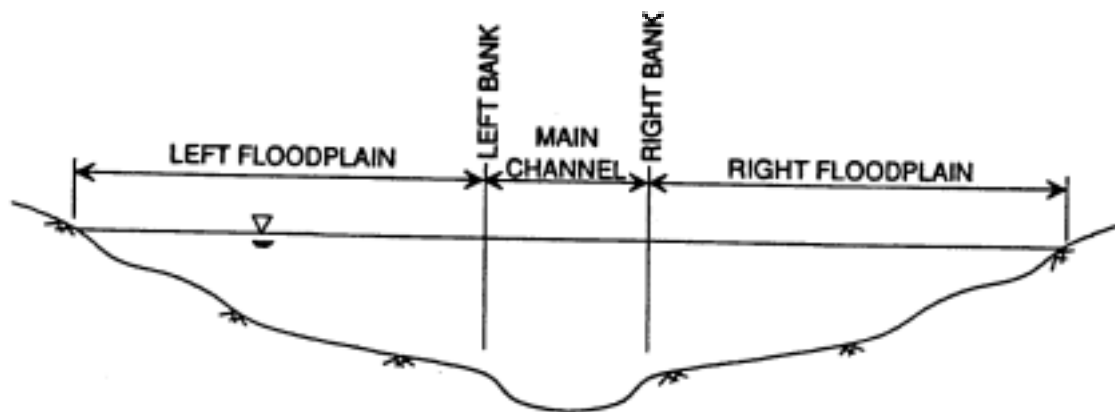


Figure 7-24 Definition Sketch; Channel Flow Distribution

Equation 7.36 is based on a rock riprap specific gravity of 2.65, and a stability factor of 1.2. Equations 7.38 and 7.39 present correction factors for other specific gravities and stability factors.

$$C_{sg} = 2.12/(S_s - 1)^{1.5} \quad (7.38)$$

Where: S_s = the specific gravity of the rock riprap

$$C_{sf} = (SF/1.2)^{1.5} \quad (7.39)$$

Where: SF = the stability factor to be applied.

The correction factors computed using Equations 7.38 and 7.39 are multiplied together to form a single correction factor C. This correction factor, C, is then multiplied by the riprap size computed from Equation 7.35 to arrive at a stable riprap size. Figure 7-28 provides a solution to Equations 7.39 and 7.38 using correction factor C.

The stability factor, SF, used in Equations 7.38 and 7.39 requires additional explanation. The stability factor is defined as the ratio of the average tractive force exerted by the flow field and the riprap material's critical shear stress. As long as the stability factor is greater than 1, the critical shear stress of the material is greater than the flow induced tractive stress, the riprap is considered to be stable. As mentioned above, a stability factor of 1.2 was used in the development of Equation 7.36.

The stability factor is used to reflect the level of uncertainty in the hydraulic conditions at a particular site. Equation 7.36 is based on the assumption of uniform or gradually varying flow. In many instances, this assumption is violated or other uncertainties come to bear. For example, debris and/or ice impacts, or the cumulative effect of high shear stresses and forces from wind and/or boat generated waves. The stability factor is used to increase the design rock size when these conditions must be considered. Table 7-7 presents guidelines for the selection of an appropriate value for the stability factor.

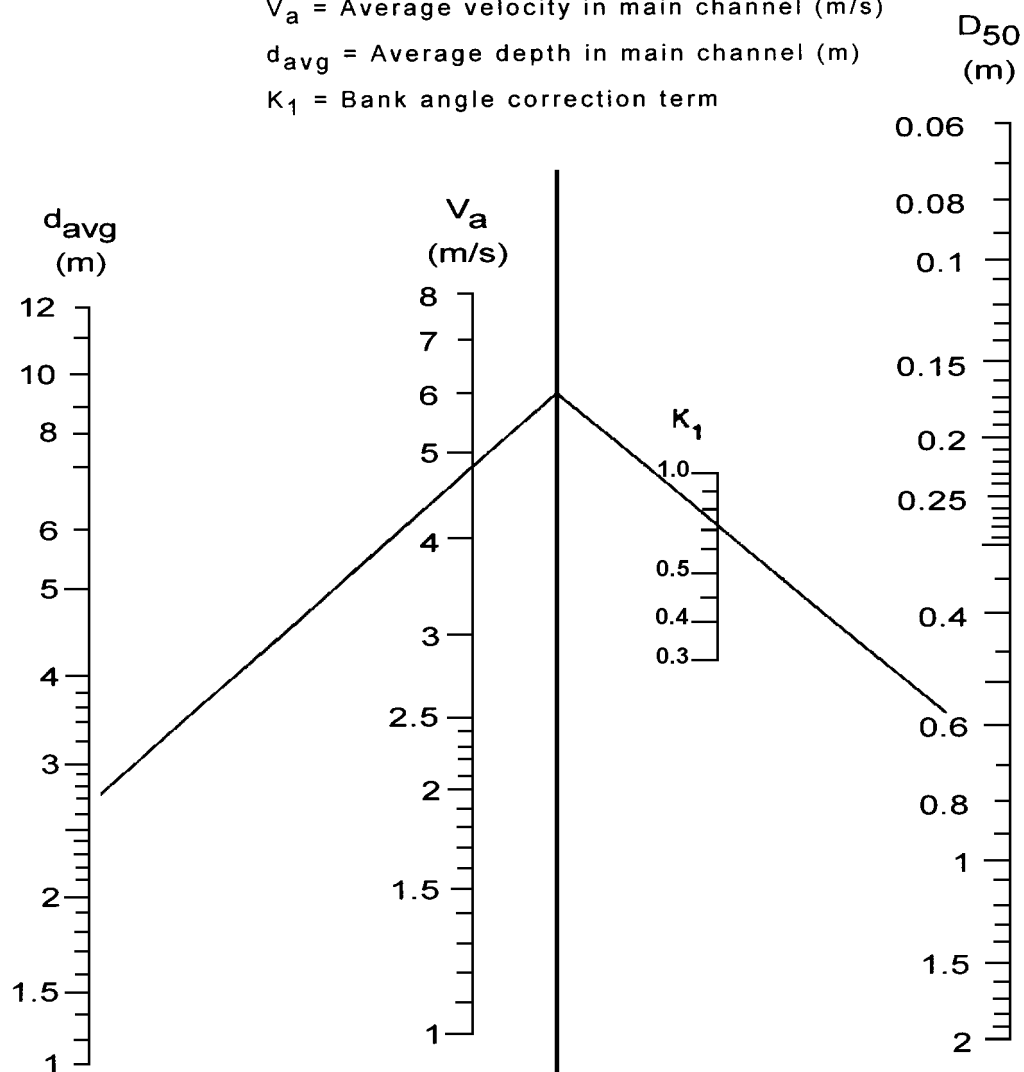
$$D_{50} = 0.00594 V_a^3 / (d_{avg}^{1/2} K_1^{3/2})$$

D_{50} = Median Riprap Size (m)

V_a = Average velocity in main channel (m/s)

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

K_1 = Bank angle correction term



Example

Given:

$V_a = 4.9$ m/s

$d_{avg} = 2.75$ m

$K_1 = 0.72$

Find:

D_{50}

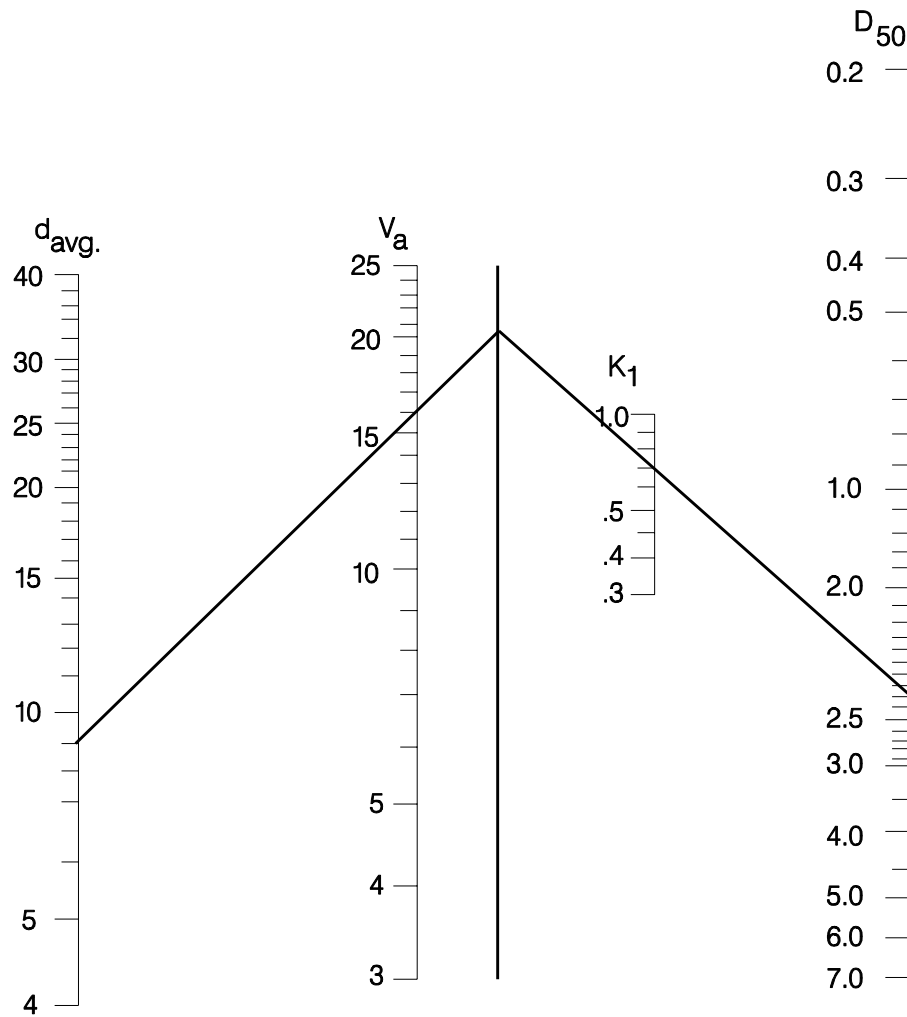
Solution:

$D_{50} = 0.69$ m

Figure 7-25 Riprap Size Relationship (metric units)

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

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:

$$\begin{aligned}
 V_a &= 16 \text{ ft./sec.} \\
 d_{avg.} &= 9 \text{ ft.} \\
 k_1 &= 0.72
 \end{aligned}$$

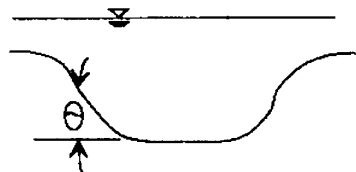
Find:

$$D_{50}$$

Solution:

$$D_{50} = 2.25$$

Figure 7-25.1 Riprap Size Relationship (English units)



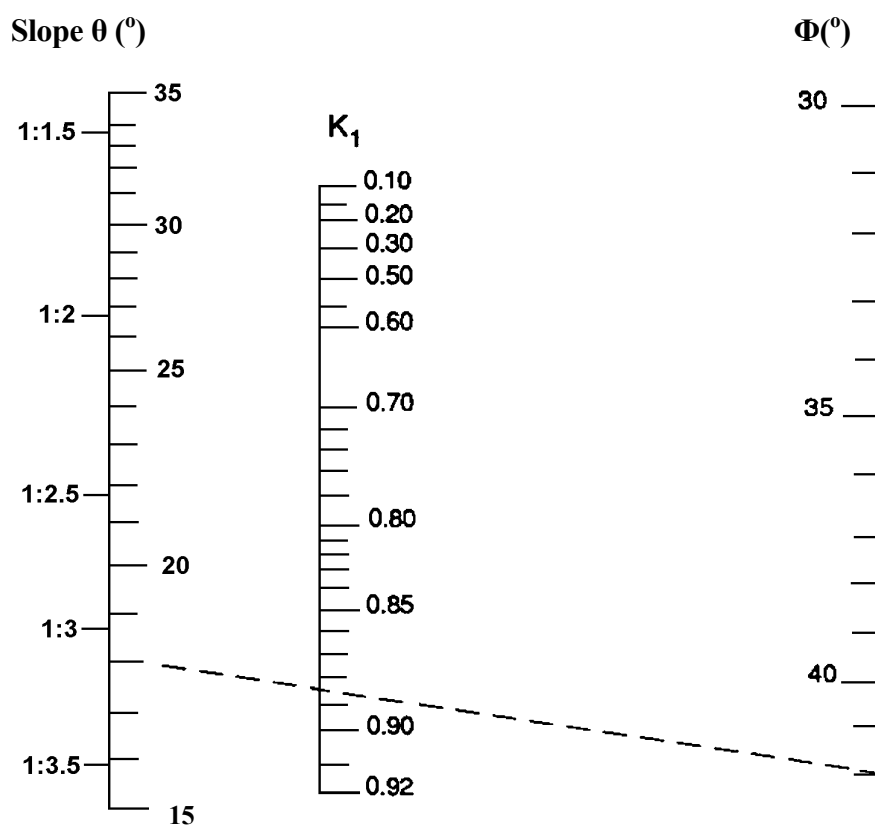
$$K_1 = \left[1 - \frac{\sin^2 \theta}{\sin^2 \Phi} \right]^{0.5}$$

θ = Bank angle with horizontal

θ = Bank angle with horizontal

Φ = Material angle of repose

See Figure 7-27 or 7-27.1



Example

Given:
 $\theta = 18^\circ$
 Very angular
 $D_{50} = 457 \text{ mm (1.5 ft)}$

Find:
 K_1

Solution:
 $\Phi = 42^\circ$
 $K_1 = 0.885$

Figure 7-26 Bank Angle Correction Factor (K_1) Nomograph

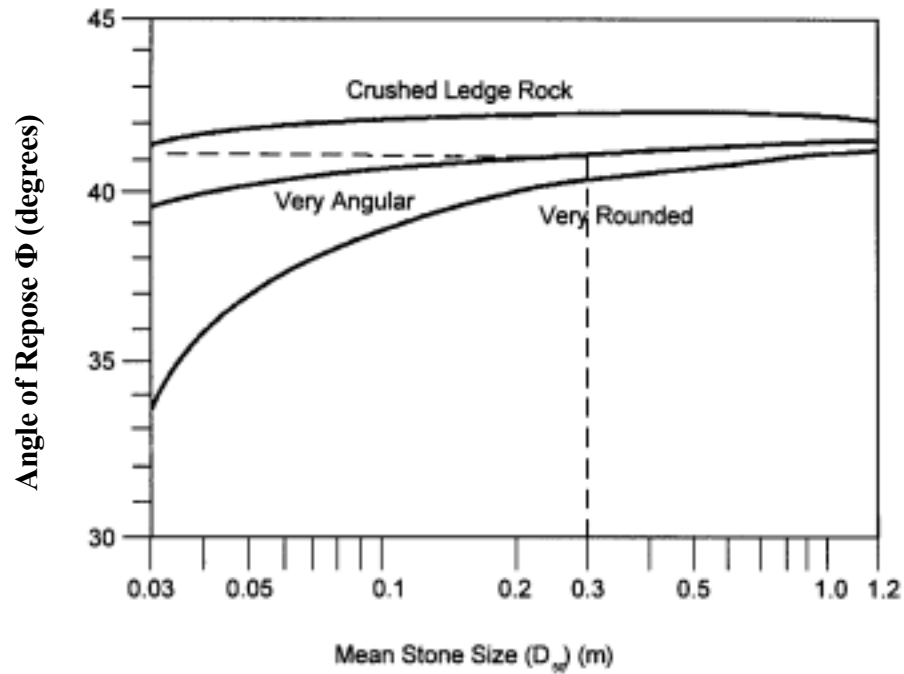


Figure 7-27 Angle Of Repose Of Riprap In Terms Of Mean Size And Shape Of Stone (metric units)

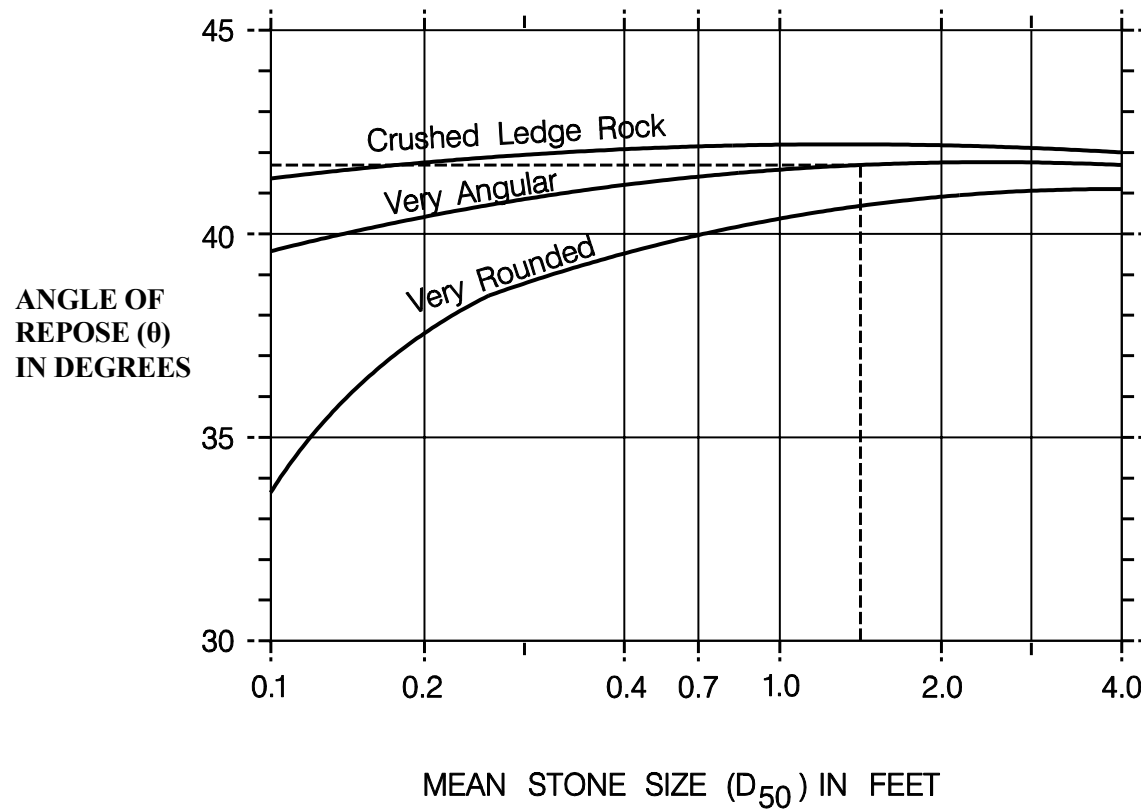


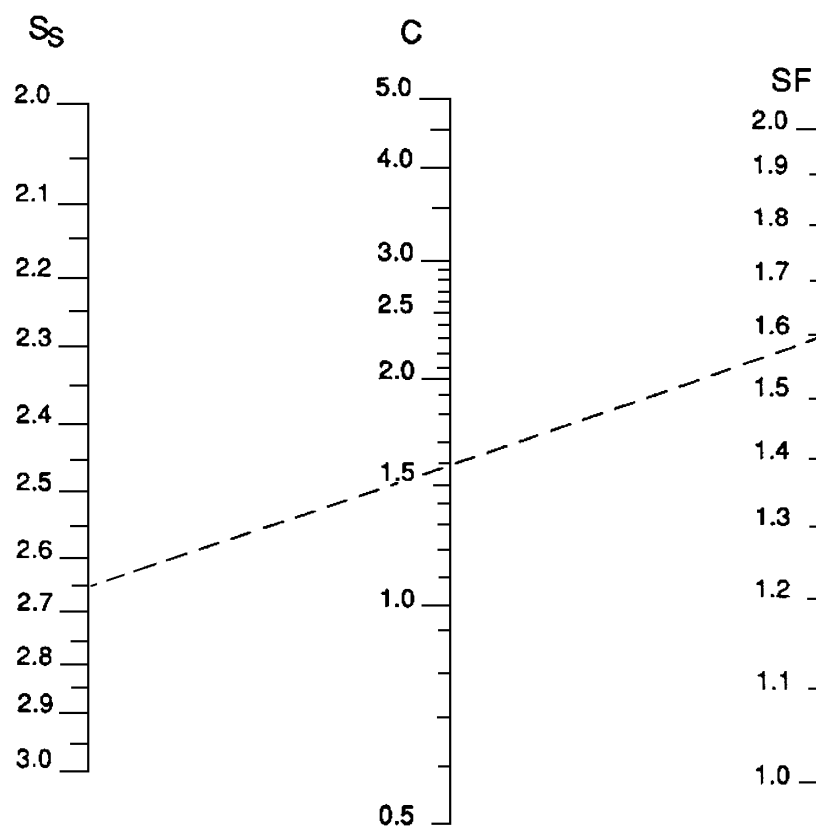
Figure 7-27.1 Angle Of Repose Of Riprap in Terms Of Mean Size and Shape Of Stone (English units)

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

CORR= D_{50} CORRECTION FACTOR

SF = STABILITY FACTOR

S_s = SPECIFIC GRAVITY OF ROCK



Example:

Given:
 S_s = 2.65
 SF = 1.60

Find:
 C

Solution:
 C = 1.59

Figure 7-28 Correction Factor For Riprap Size (metric or English)

Table 7-8 Guidelines For The Selection Of Stability Factors

<u>Condition</u>	<u>Stability Factor Range</u>
Uniform flow; Straight or mildly curving reach (curve radius/channel width > 30); Impact from wave action and floating debris is minimal; Little or no uncertainty in design parameters.	1.0 - 1.2
Gradually varying flow; Moderate bend curvature ($30 > \text{curve radius/channel width} > 10$); Impact from waves or floating debris moderate.	1.21 - 1.6
Approaching rapidly varying flow; Sharp bend curvature ($10 > \text{curve radius/channel width}$); Significant impact potential from floating debris and/or ice; Significant wind and/or boat generated waves (0.3 - 0.6 m (1-2 ft)); High flow turbulence; Turbulently mixing flow at bridge abutments; Significant uncertainty in design parameters.	1.61 - 2.0

7.9.6 Application

Application of the relationship in Equation 7.36 is limited to uniform or gradually varying flow conditions that are in straight or mildly curving channel reaches of relatively uniform cross section. However, design needs dictate that the relationship also be applicable in nonuniform, rapidly varying flow conditions often exhibited in natural channels with sharp bends and steep slopes, and in the vicinity of bridge piers and abutments.

To fill the need for a design relationship that can be applied at sharp bends and on steep slopes in natural channels, and at bridge abutments, it is recommended that Equation 7.36 be used with appropriate adjustments in velocity and/or stability factor as outlined in the following sections.

Wave Erosion

Waves generated by wind or boat traffic have also been observed to cause bank erosion on inland waterways. The most widely used measure of riprap's resistance to wave is that developed by R. Y. Hudson "Laboratory Investigations of Rubble-Mound Breakwaters," 1959. The so-called Hudson relationship is given by the following equation:

$$W_{50} = (\gamma_s H^3) / (2.20 [S_s - 1]^3 \cot \theta) \quad (7.40)$$

Where: W_{50} = weight of the median particle, kg (lb)
 γ_s = unit weight of riprap (solid) material, kg/m³ (lb/ft³)
 H = the wave height, m (ft)
 S_s = specific gravity of riprap material
 θ = bank angle with the horizontal

Assuming:

$S_s = 2.65$ and $\gamma_s = 2643 \text{ kg/m}^3$ (165 lb/ft³), Equation 7.40 can be reduced to:

$$W_{50} = 267.4 H^3 / \cot \theta \quad (W_{50} = 16.7 H^3 / \cot \theta) \quad (7.41)$$

In terms of an equivalent diameter Equation 7.41 can be reduced to:

$$D_{50} = 0.57 H / \cot^{1/3} \theta \quad (D_{50} = 0.75 H / \cot^{1/3} \theta) \quad (7.42)$$

Where: D_{50} = median riprap size, m

Methods for estimating a design wave height are presented in Appendix A of this chapter. Equation 7.42 is presented in nomograph form in Figure 7-29. Equations 7.41 and 7.42 can be used for preliminary or final design when H is less than 1.5 m (5 ft), and there is no major overtopping of the embankment.

7.9.7 Steep Slopes

Flow conditions in steep sloped channels are rarely uniform, and are characterized by high flow velocities and significant flow turbulence. In applying Equation 7.36 to steep slope channels, care must be exercised in the determination of an appropriate velocity. When determining the flow velocity in steep sloped channels, it is recommended that Equation 7.43 be used to determine the channel roughness coefficient. It is also important to thoughtfully consider the guidelines for selection of stability factors as presented in Table 7.8.

On high gradient streams it is extremely difficult to obtain a good estimate of the median bed material size. For high gradient streams with slopes greater than 0.002 m/m (ft/ft) and bed material larger than 0.06 m (0.2 ft) (gravel, cobble, or boulder size material), it is recommended that the relationship given in the following equation be used to evaluate the base Manning's n.

$$n = 0.32 S_f^{0.38} R^{-0.16} \quad (n = 0.39 S_f^{0.38} R^{-0.16}) \quad (7.43)$$

Where: S_f = friction slope, m/m (ft/ft)
 R = hydraulic radius, m (ft)

7.9.8 Bridge Piers

For recommendations, see Chapter 9, Bridges.

7.9.9 Ice Damage

Ice can affect riprap linings in a number of ways. Moving surface ice can cause crushing and bending forces as well as large impact loadings. The tangential flow of ice along a riprap lined channel bank can also cause excessive shearing forces. Quantitative criteria for evaluating the impact ice has on channel protection schemes are unavailable. However, historic observations of ice flows in New England rivers indicate that riprap sized to resist design flow events will also resist ice forces.

For design, consideration of ice forces should be evaluated on a case by case basis. In most instances, ice flows are not of sufficient magnitude to warrant detailed analysis. Where ice flows have historically caused problems, a stability factor of 1.2 to 1.5 should be used to increase the design rock size. Please note that the selection of an appropriate stability factor to account for ice generated erosive problems should be based on local experience.

7.9.10 Rock Gradation

The gradation of stones in riprap revetment affects the riprap's resistance to erosion. The stone should be reasonably well graded throughout the riprap layer thickness. Table 7-9 presents the median particle size of three types of riprap which have gradations defined in the ConnDOT standard specifications. All designs should consider using the ConnDOT standard gradations, however if a design requires a non-standard median particle size, then the AASHTO guidelines for rock gradations as presented in HEC-11 should be used.

Table 7-9 D₅₀ of Available Riprap

<u>Riprap Type</u>	<u>D₅₀ mm</u>
Modified	125 (5 inches)
Intermediate	200 (8 inches)
Standard	380 (15 inches)

7.9.11 Layer Thickness

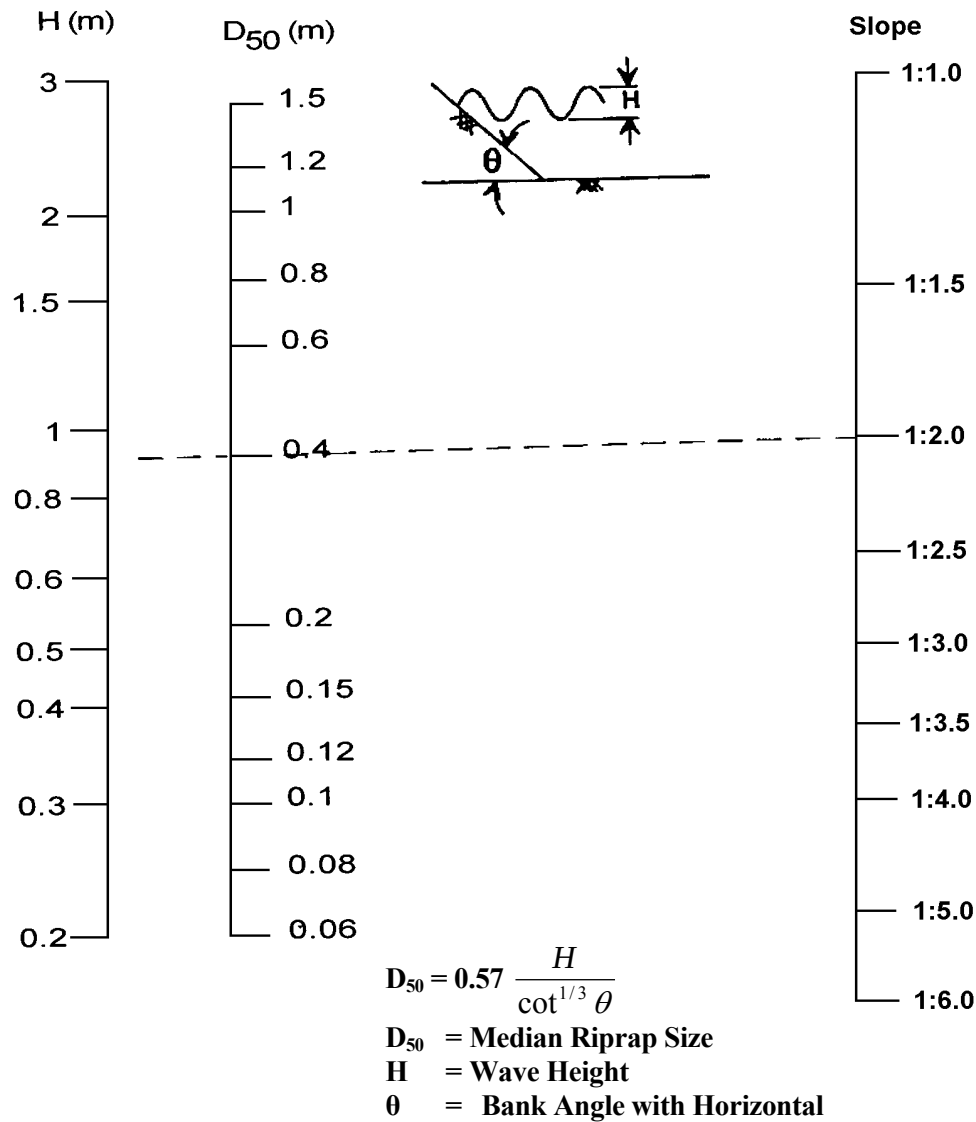
All stones should be contained reasonably well within the riprap layer thickness to provide maximum resistance against erosion. Oversize stones, even in isolated spots, may cause riprap failure by precluding mutual support between individual stones, providing large voids that expose filter and bedding materials, and creating excessive local turbulence that removes smaller stones. Small amounts of oversize stone should be removed individually and replaced with proper size stones. The following criteria apply to the riprap layer thickness.

1. It should not be less than the spherical diameter of the D₁₀₀ stone, or less than 2.0 times the spherical diameter of the D₅₀ stone, whichever results in the greater thickness.
2. It should not be less than 300 mm (12 in) for practical placement.
3. The thickness determined by either of the above criteria should be increased by 50% when the riprap is placed underwater to provide for uncertainties associated with this type of placement.
4. An increase in thickness, accompanied by an appropriate increase in stone sizes, should be provided where riprap revetment will be subject to attack by floating debris or ice, or by waves from boat wakes, wind, or bedforms.

The typical layer thickness for riprap (ConnDOT gradations) revetment is shown in Table 7-10.

Table 7-10 Riprap Layer Thickness

<u>Riprap Type</u>	<u>Riprap Layer Thickness mm</u>
Modified	300 (12 inches)
Intermediate	450 (18 inches)
Standard	900 (36 inches)



Example

Given:

Slope = 1V:2H

H = 0.91 m

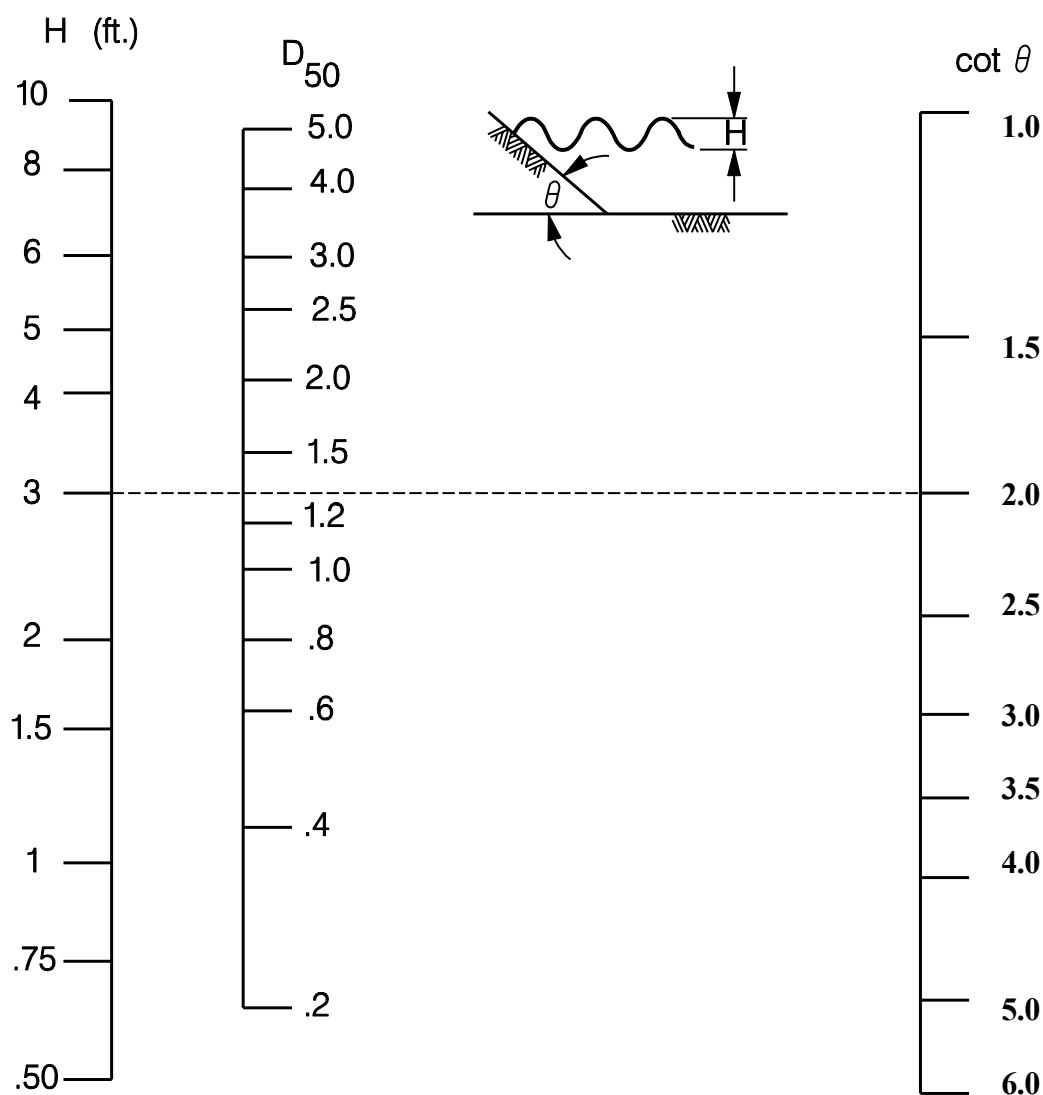
Find:

D₅₀

Solution:

D₅₀ = 0.4 m

Figure 7-29 Hudson Relationship For Riprap Size Required To Resist Wave Erosion – metric units



$$D_{50} = 0.57 \frac{H}{\cot^{1/3} \theta}$$

D₅₀ = Median Riprap Size

H = Wave Height

θ = Bank Angle with Horizontal

Example:

Given:

cot θ = 2:1

H = 3 ft.

Find:

D₅₀

Solution:

D₅₀ = 1.33 ft.

Figure 7-29.1 Hudson Relationship For Riprap Size Required To Resist Wave Erosion – English units

7.9.12 Filter Design

A filter is a transitional layer of gravel, small stone, or fabric placed between the underlying soil and the structure. The filter prevents the migration of the fine soil particles through voids in the structure, distributes the weight of the armor units to provide more uniform settlement and permits relief of hydrostatic pressures within the soils. A filter should be used whenever the riprap is placed on noncohesive material subject to significant subsurface drainage (such as in areas where water surface levels fluctuate frequently and in areas of high groundwater levels).

Granular Filters

Typical riprap installations use granular filters as a transitional layer between the riprap and underlying soil. This layer shall consist of the item “Granular Fill” which is found in the standard specifications. The required thickness of the granular fill layer based on the ConnDOT riprap type is shown in Table 7-11.

Table 7-11 Granular Fill Filter Thickness

Riprap Type	Granular Fill Filter Thickness (mm)
Modified	150 (6 inches)
Intermediate	150 (6 inches)
Standard	300 (12 inches)

In some cases further analysis of the granular filter layer may be warranted. Guidelines for granular filter design are found in HEC-11.

Fabric Filters

Synthetic fabric filters have found considerable use as alternatives to granular filters. The following list of advantages relevant to using fabric filters have been identified:

- Installation is generally quick and labor-efficient
- Fabric filters are more economical than granular filters
- Fabric filters have consistent and more reliable material quality
- Fabric filters have good inherent tensile strength
- Local availability of suitable granular filter material is no longer a design consideration when using fabric filters

Disadvantages include the following.

- Filter fabrics can be difficult to lay underwater
- Installation of some fabrics must be undertaken with care to prevent undue ultraviolet light exposure
- Bacterial activity within the soil or upon the filter can control the hydraulic responses of a fabric filter system

- Experimental evidence indicates that when channel banks are subjected to wave action, non-cohesive bank material has a tendency to migrate downslope beneath fabric filters; this tendency was not observed with granular filters
- Fabric filters may induce translational or modified slump failures when used under rock riprap installed on steep slopes if not properly keyed into the top of slope

Filter Fabric Design

The filter cloth (geotextile) design should consider the following performance areas.

- Soil Retention (Piping Resistance)
- Permeability
- Clogging
- Survivability

It is extremely desirable that individual site requirements be used to establish the necessary requirements. Generalized geotextile requirements should be used only on very small or non-critical/non-severe installations where a detailed analysis is not warranted. A discussion of the above special considerations is found in Hydraulic Engineering circular No. 11.

The design process generally consists of the following design steps:

- Step 1 - Evaluate the application site.
- Step 2 - Obtain and test soil samples
- Step 3 - Evaluate possible armor choices
- Step 4 - Calculate flow through geotextile
- Step 5 - Determine geotextile requirements:

- a. Soil Retention
- b. Permeability
- c. Clogging
- d. Survivability

Filter Fabric Placement

To provide good performance, a properly selected cloth should be installed with due regard for the following precautions:

- Heavy riprap may stretch the cloth as it settles, eventually causing bursting of the fabric in tension. A 150mm (6 inch) gravel bedding layer should be placed beneath standard riprap.
- The filter cloth should not extend into the channel beyond the riprap layer; rather, it should be wrapped around the toe material as illustrated in Figure 7-30.
- Adequate overlaps must be provided between individual fabric sheets.
- The filter should be installed loosely to allow for any stretching under settlement.
- Securing pins with washers are recommended at 0.6m to 1.5m (2 to 5 ft) intervals along the midpoint of the overlaps.

- Proper stone placement on the filter begins at the toe and proceeding up the slope. Dropping stone from heights greater than 0.6m (2 ft) can rupture fabrics (greater drop heights are allowable under water).
- “Drop height” of riprap must be limited to avoid tearing the fabric

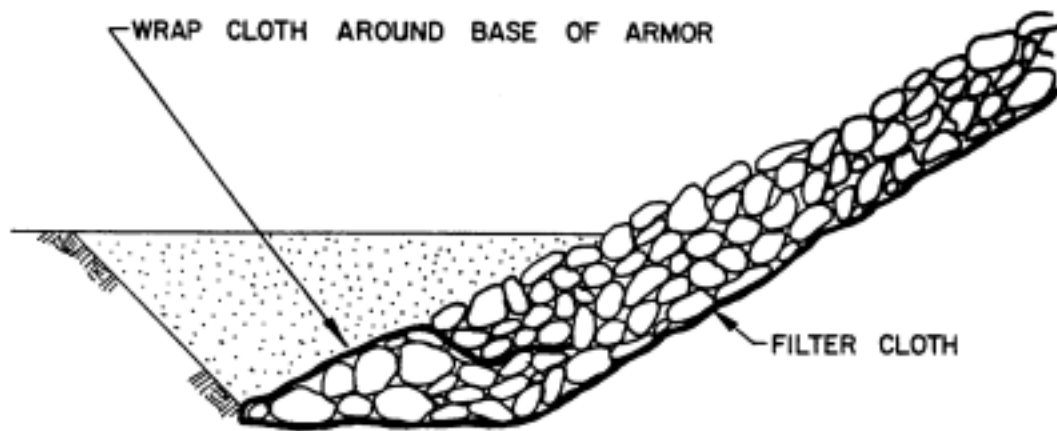


Figure 7-30 Filter Fabric Placement

7.9.13 Edge Treatment

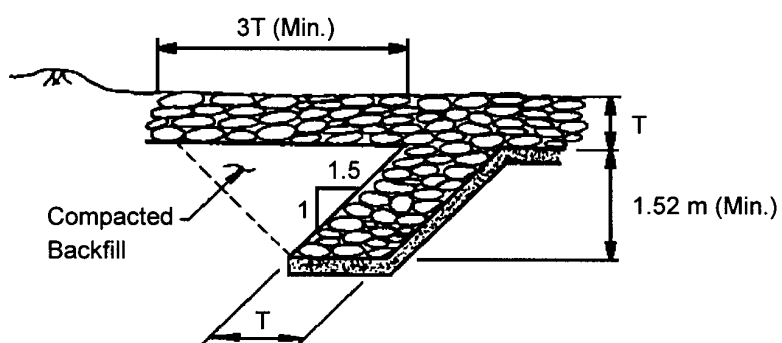
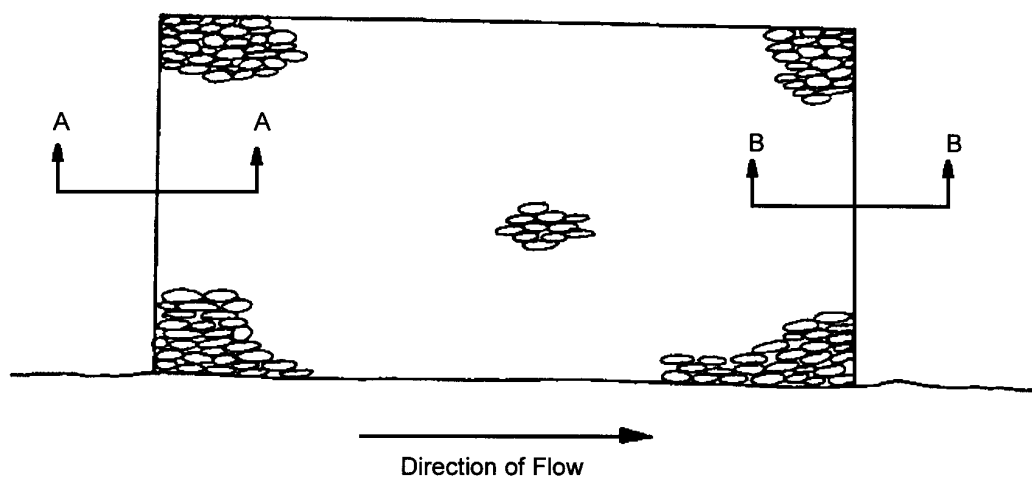
The edges of riprap revetments (flanks, toe and head) require special treatment to prevent undermining. The flanks of the revetment should be designed as illustrated in Figure 7-31. The upstream flank is illustrated in Section A-A and the downstream flank in Section B-B of this figure. A more constructible flank section uses riprap rather than compacted fill.

Undermining of the revetment toe is one of the primary mechanisms of riprap failure. The toe of the riprap should be designed as illustrated in Figure 7-32. The toe material should be placed in a toe trench along the entire length of the riprap blanket.

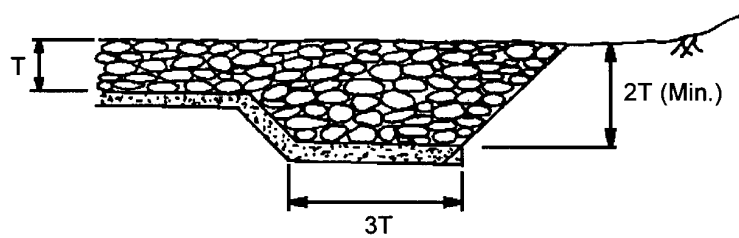
Where a toe trench cannot be dug, the riprap blanket should terminate in a thick, stone toe at the level of the streambed (see alternate design in Figure 7-32). Care must be taken during the placement of the stone to ensure that the toe material does not mound and form a low dike; a low dike along the toe could result in flow concentration along the revetment face which could stress the revetment to failure. In addition, care must be exercised to ensure that the channel's design capability is not impaired by too much riprap in a toe mound.

The size of the toe trench or the alternate stone toe is controlled by the anticipated depth of scour along the revetment. As scour occurs (and in most cases it will) the stone in the toe will launch into the eroded area as illustrated in Figure 7-33. Observation of the performance of these types of rock toe designs indicates that the riprap will launch to a final slope of approximately 1V:2H.

The volume of rock required for the toe must be equal to or exceed one and one-half times the volume of rock required to extend the riprap blanket (at its design thickness and on a slope of 1V:2H) to the anticipated depth of scour. Dimensions should be based on the required volume using the thickness and depth determined by the scour evaluation. The alternate location can be used when the amount of rock required would not constrain the channel. Establishing a design scour depth is covered in Section 7.8.8.



SECTION: A-A



SECTION: B-B

Figure 7-31 Typical Riprap Installation: Plan and Flank Details

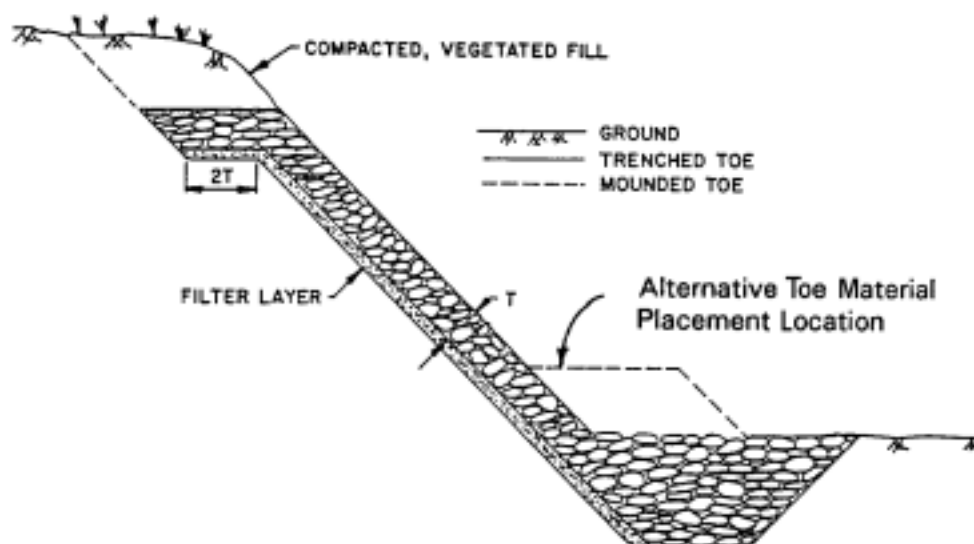


Figure 7-32 Typical Riprap Installation: End View (Bank Protection Only)

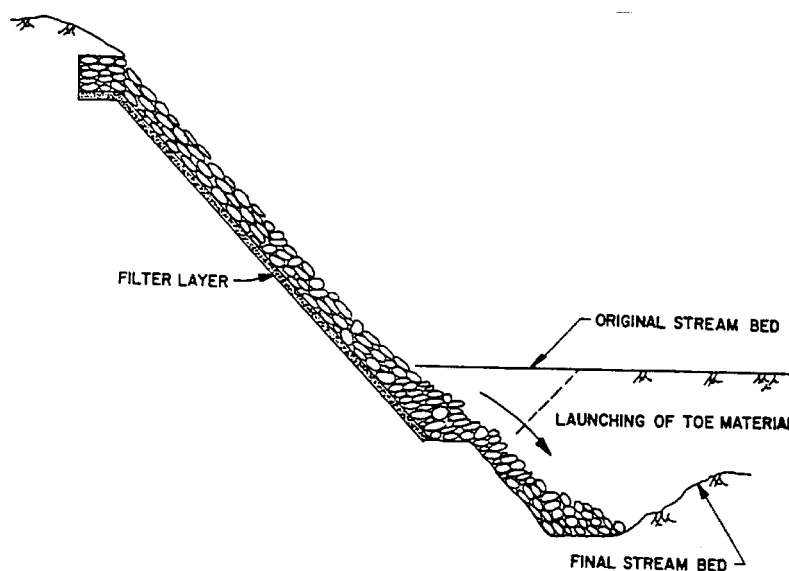


Figure 7-33 Launching Of Riprap Toe Material

7.9.14 Rock Riprap Design Procedure

Rock riprap design procedure outlined in the following sections is comprised of three primary sections: preliminary data analysis, rock sizing and revetment detail design. The individual steps in the procedure are numbered consecutively throughout each of the sections. Figure 7-34 provides a useful format for recording data at each step of the analysis.

Preliminary Data

- Step 1 Compile all necessary field data including (channel cross section surveys, soils data, aerial photographs, history of problems at site, etc.).
- Step 2 Determine design discharge.
- Step 3 Develop design cross section(s). Note: The rock sizing procedures described in the following steps are designed to prevent riprap failure from particle erosion.
- Step 4 Compute design water surface.
- A. When evaluating the design water surface, Manning's n should be estimated. If a riprap lining is being designed for the entire channel perimeter, an estimate of the rock size may be required to determine the roughness coefficient.
 - B. If the design section is a regular trapezoidal shape, and flow can be assumed to be uniform, design procedures in Section 7.7.3 can be used.
 - C. If the design section is irregular or flow is not uniform, backwater procedures must be used to determine the design water surface.
 - D. Any backwater analysis conducted must be based on conveyance weighing of flows in the main channel, right bank and left bank.
- Step 5 Determine design average velocity and depth.
- A. Average velocity and depth should be determined for the design section in conjunction with the computations of step 4. In general, the average depth and velocity in the main flow channel should be used.
 - B. If riprap is being designed to protect channel banks, abutments, or piers located in the floodplain, average floodplain depths and velocities should be used.
- Step 6 Compute the bank angle correction factor K_1 (Equation 7.37, Figures 7-26 and 7-27).

Rock Sizing

- Step 7 Determine riprap size required to resist particle erosion (Equation 7.36, Figure 7-25).
- A. Initially assume no corrections.

- B. Evaluate correction factor for rock riprap specific gravity and stability factor ($C = C_{sg}C_{sf}$).
 - C. If designing riprap for piers or abutments see Chapter 9, Bridges.
- Step 8 If entire channel perimeter is being stabilized, and an assumed D_{50} was used in determination of Manning's n for backwater computations, return to step 4 and repeat steps 4 through 7 with the revised D_{50} .
- Step 9 If surface waves are to be evaluated:
- A. Determine significant wave height.
 - B. Use Figure 7-29 to determine rock size required to resist wave action and Table 7-7 for correction factor).
- Step 10 Select final riprap.

Revetment Detail Design

- Step 11 Determine longitudinal extent of protection required (section 7.8.8).
- Step 12 Determine appropriate vertical extent of revetment (section 7.8.8).
- Step 13 Design filter layer (section 7.9.12).
- A. Determine appropriate filter material size, and gradation.
 - B. Determine layer thickness.
- Step 14 Design edge details (flanks and toe) (section 7.9.13).

PROJECT DESCRIPTION _____ _____ _____		Prepared by/Date: _____ Checked by/Date: _____ Sheet _____ of _____	

Definition Sketch:

Soil Characteristics:

Q TOTAL _____
 Q MC _____
 Q 15 _____
 Q 50 _____
 Q 95 _____

DEPTH OR W.S. (m)	A (m ²)	V _g (m/s)	d _n (m)	K ₁	D ₅₀ (mm)	S _f	S _b	C	C _{FFA}	D ₁₅ (mm)	NOTES

DEPTH OR W.S. (m)	A (m ²)	V _g (m/s)	d _n (m)	K ₁	D ₅₀ (mm)	S _f	S _b	C	C _{FFA}	D ₁₅ (mm)	NOTES

Design Sketch:				RIPRAP CHARACTERISTICS:				FABRIC CHARACTERISTICS:			
1 Water surface elevation	2 Main channel flow area	3 Main channel average velocity	4 Main channel average depth	5 Bank angle	6 Riprap angle of repose (°)	7 Bank angle correction	8 Riprap size	9 Stability factor	10 Riprap specific gravity	11 Riprap size correction factor ()	12 Perforation correction ()

RIPRAP CHARACTERISTICS:				FABRIC CHARACTERISTICS:			
Size (mm)	Class	AASHTO Gradation	Use	Granular Size (mm)	Fabric	AOCS	Perm.

Figure 7-34 Riprap Size Particle Erosion – metric units

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Appendix A Simplified Methods For Estimating Wave Height

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 and
- 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 Corp of Engineer's "Shore Protection Manual."

Predicting Wind Generated Waves

The height of wind generated waves is a function of:

- fetch length,
- windspeed,
- wind duration and
- the depth of water.

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.

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, 120 km (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, windspeed 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" (SPM) 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 Figure 7-A-1. If the estimated wave height from the nomograph is greater than 0.6 m (2 ft) it is recommended that the Army Corps of Engineers procedures be used to refine the input parameters.

Breaking Waves

Waves generated in deeper water and shoaling as they approach the embankment will be a maximum size wave that will reach it 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 stillwater 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 which 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 Figure 7-A-2.

Prediction Procedure

Following is an outline of a wave prediction procedure.

Windspeed Estimation

To estimate windspeed the following information is needed:

- actual wind records from the site,
- general wind statistics and
- best alternative source of wind information.

Site Maximization Procedure

Using the method presented in the Army Corp of Engineer's "Shore Protection Manual," (SPM) the site maximization procedure consists of the following steps.

- Adjust wind information to 10 m (33 ft) above water surface.
- Determine fetch limitations.
- Adjust wind information for over water conditions.
- Develop and plot a windspeed-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 (Figures 7-A-1 or 7-A-1.1).
- 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.

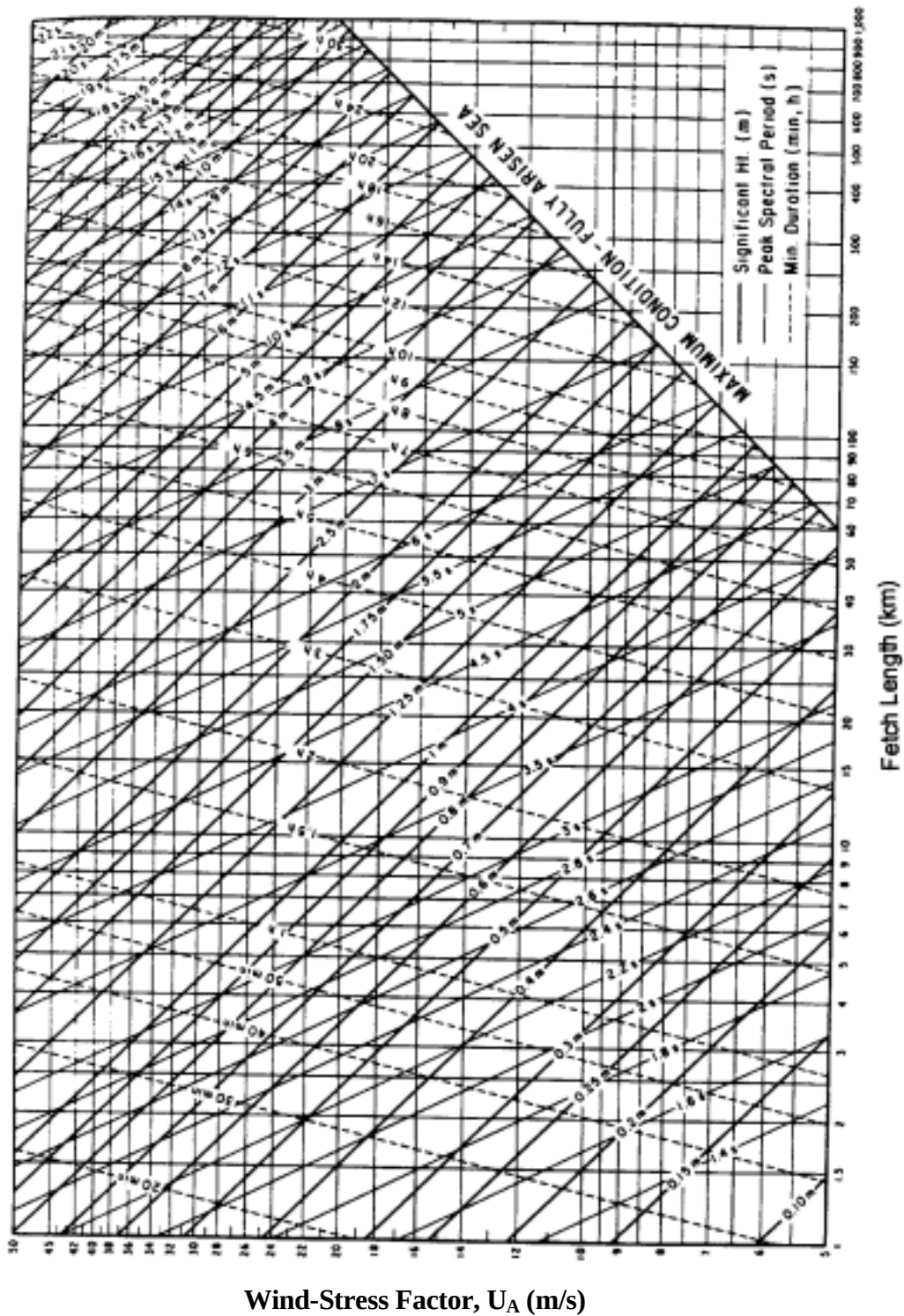


Figure 7-A-1 Nomographs Of Significant Wave Height Prediction Curves As Functions Of Windspeed, Fetch Length and Wind Duration (Metric units)

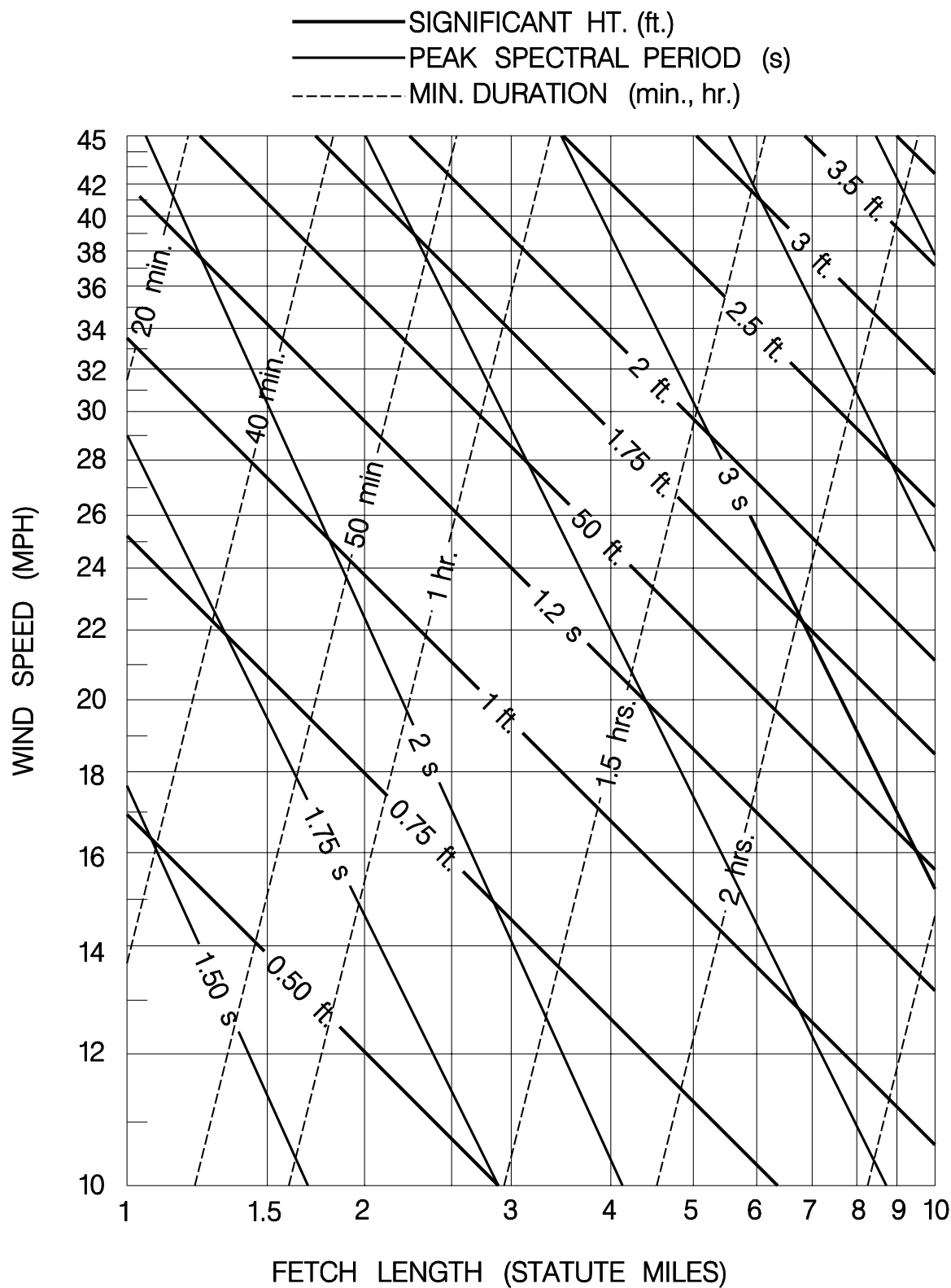


Figure 7-A-1.1 Nomograph of deep water significant wave height prediction curves (English units)

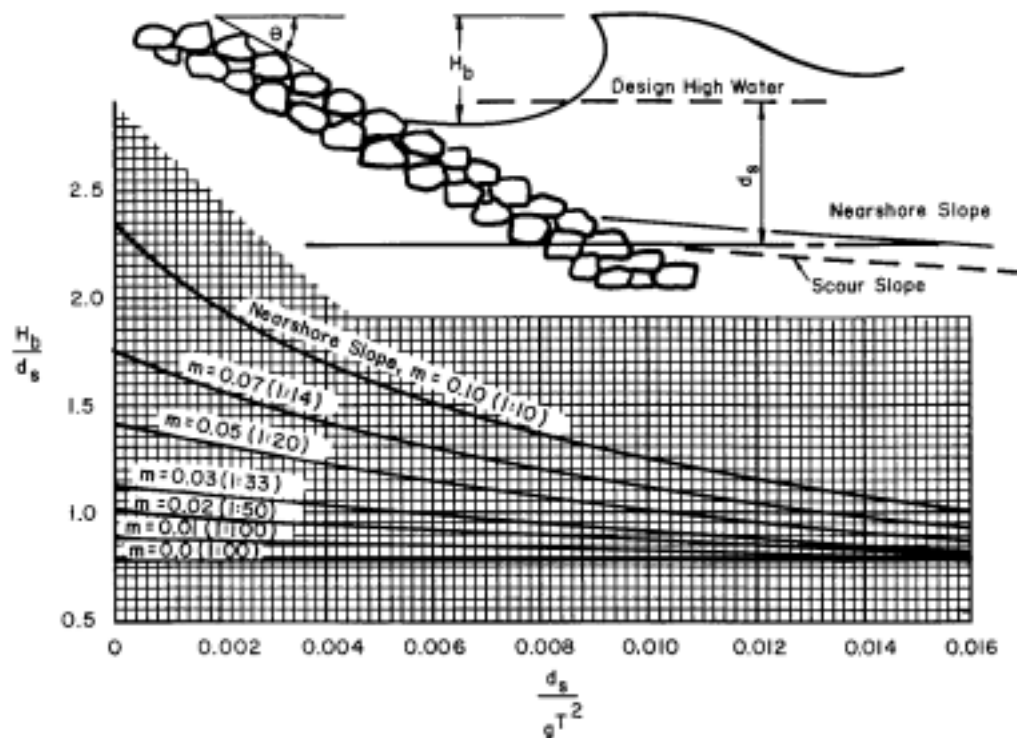


Figure 7-A-2 Design Breaker Wave

Design Breaker Wave

The following example illustrates how to use Figure 7-A-2 to estimate the maximum breaker wave height.

Example

By using hindcast methods, the significant wave height (H_s) has been estimated at 1.2 m (4 ft) with a 3-s period. Find the design wave height (H_b) for the slope protection if the depth of water (d_s) is only 0.6 m (2 ft) and the near-shore slope m (ft) is 1V:10H.

Solution

$$(d_s/gT^2) = 0.6 \text{ m} / [(9.81 \text{ m/s}^2) * (3 \text{ s})^2] = 0.007$$

From Figure 7-A-2, $H_b/d_s = 1.4$, and $H_b = 0.84 \text{ m}$

Answer

Since the maximum breaker wave height, H_b , is smaller than the significant deepwater wave height, H_s the design wave height is 0.84 m.

Appendix B Vegetative (Grass) Swales and Erosion Control Matting

Grass Adequate?

Vegetative (grass) lined swales are preferred for storm water quality (treatment) purposes. Therefore the designer should check, with computations, whether grass is an adequate lining for the proposed swale. The 10-year discharge and grass "Retardance Class C" (HEC-15) is used. Include this computation in the drainage report. **Please note that "Retardance Class C" is a nomenclature for certain types of grasses used in the HEC-15 design manual and is not a contract item and should not be called out on the plans.**

YES

Where grass is determined to be **adequate** for the design conditions, a temporary (biodegradable) lining is required to convey runoff until grass (turf) can be established in the swale. Erosion Control Matting, Class 2, Flexible Channel Liners (ConnDOT Approved Products List) is the item to be used for temporary linings. The types and criteria for erosion control matting are found on pages 7.6-11 and 7.6-12 of the ConnDOT Drainage Manual. The temporary lining is evaluated with the 2-year discharge. The computed width of flow in the swale determines the width of matting to be called for on the plans. Include this computation in the drainage report.

The limits of the erosion control matting should be shown on the plans and the width and type of matting should be called out. For example, an appropriate call out that is consistent with the ConnDOT payment item would be: "2m – Erosion Control Matting – Type F" or "2m – Erosion Control Matting – Type H (Permanent)." *It is generally not necessary to call out "grass or vegetative swale" on the plans where the swale is formed incidental to construction or reconstruction of the roadway in a cut section. First, grass is covered under the payment item "Turf Establishment", which is called out on the typical cross sections and often the limits are shown on the plans. Second, the grading of the swale is included in the payment items Earth Excavation or Rock Excavation.*

NO

Where grass is determined to be **inadequate** for the design conditions, a permanent, (non-biodegradable) synthetic erosion control matting can be used as turf reinforcement in lieu of using riprap for the swale lining. As turf is established, the roots of the grass become entangled in the matting and the permissible shear stress of this combination is greater than the permissible shear stress of the grass or the mat by itself. Since the matting is needed for both the permanent and the temporary condition, the matting is designed for the 10-year discharge and the permissible shear stress of the mat. Include this computation in the drainage report. Note that if a synthetic mat is adequate, the computed flow depth and width in the swale for the grass lining determines the width of the matting to be called out on the plans (the roughness or manning's n-value of the grass-mat combination is greater than the unvegetated mat) and is used to determine if the capacity of the swale section is adequate. A temporary lining is not required, as the mat has already been designed for the permanent use (10-year discharge) and will therefore be adequate for temporary erosion control (2-year discharge). Permanent, (non-biodegradable) synthetic erosion control matting is designated on the Department's Approved Products List as "Type H" matting. When a permanent matting is required it should be so noted on the plans.

The steps in Section 7.6.9 of the ConnDOT Drainage Manual should be followed for drainage swale design.

Swale Inlets Typically swale inlets are Type "C-L" catch basins with paved aprons and are designed for full interception even when they are located on grade (not at a low point) as they are depressed 127-mm (5 inches) below the finished grade. See Chapter 11, Appendix B, for grading at Type "C-L" catch basins. The capacity of the inlet is based on the maximum head (127-mm) over the grate without bypassing the inlet after an allowance of an appropriate clogging factor. The number and spacing of inlets required in a proposed swale is based on the inlet capacity noted above, an allowable flow depth considering freeboard and a reasonable width of flow.

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8.1 Introduction

8.1.1 Purpose

This chapter provides design procedures for the hydraulic design of highway culverts which are based on FHWA Hydraulic Design Series Number 5 (HDS5), Hydraulic Design of Highway Culverts. A culvert is defined as the following:

- A structure which is usually designed hydraulically to take advantage of submergence to increase hydraulic capacity
- A structure used to convey surface runoff or a watercourse through an embankment
- 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
- Requires a structural design. In addition to its hydraulic function, it must also carry construction and highway traffic and earth loads. (See Chapter 4, Culvert Repair, Materials, and Structural Design.)

8.1.2 Concepts

Following are discussions of concepts which are important in culvert design.

- **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.
- **Crown:** The crown (soffit) is the inside top of the culvert.
- **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 decreases the flow contraction at the inlet and thus increases the capacity of culverts. These inlets are referred to as either side- (walls tapered) or slope-tapered (increased flow-line slope at the entrance).
- **Invert:** The invert is the flowline of the culvert (inside bottom). However, if the culvert is buried to accommodate fish passage, then the invert should be considered the streambed.
- **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:** Usually a steep slope is present where critical depth is greater than normal depth and a mild slope is present 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 and a submerged **inlet** occurs when the headwater is greater than $1.2D$ where D is the culvert diameter or barrel height.

8.1.3 Symbols

To provide consistency within this chapter as well as throughout this manual the symbols given in Table 8-1 will be used. These symbols were selected because of their wide use in culvert publications.

Table 8-1 Symbols And Definitions

<u>Symbol</u>	<u>Definition</u>	<u>Units</u>
A	Area of cross section of flow	m^2 (ft ²)
AHW	Allowable HW	m (ft)
B	Barrel width	mm or m (in or ft)
D	Culvert diameter or barrel height	mm or m (in or ft)
d	Depth of flow	m (ft)
d_c	Critical depth of flow	m (ft)
g	Acceleration due to gravity	m/s^2 (ft/s ²)
H	Sum of $H_E + H_f + H_o$	m (ft)
H_b	Bend headloss	m (ft)
H_E	Entrance headloss	m (ft)
H_f	Friction headloss	m (ft)
H_L	Total energy losses	m (ft)
H_o	Outlet or exit headloss	m (ft)
H_v	Velocity head	m (ft)
h_o	Hydraulic grade line height above outlet invert	m (ft)
HW	Headwater depth	m (ft)
K_E	Entrance loss coefficient	-
L	Length of culvert	m (ft)
n	Manning's roughness coefficient	-
P	Wetted perimeter	m (ft)
Q	Rate of discharge	m^3/s (ft ³ /s)
R	Hydraulic radius (A/P)	m (ft)
S	Slope of culvert	m/m (ft/ft)
TW	Tailwater depth above invert of culvert	m (ft)
V	Mean velocity of flow with barrel full	m/s (ft/s)
V_d	Mean velocity in downstream channel	m/s (ft/s)
V_o	Mean velocity of flow at culvert outlet	m/s (ft/s)
V_u	Mean velocity in upstream channel	m/s (ft/s)
γ	Unit weight of water	N/m^3 (lb/ft ³)
τ	Tractive force	Pa (lb/ft ²)

8.2 Design Practices

8.2.1 ConnDOT

The following design practices are specific to culverts:

- All culverts shall be hydraulically designed.
- The overtopping flood selected shall be consistent with the class of highway and commensurate with the risk at the site.
- Data collection shall include topographic features, channel characteristics, aquatic life, highwater information, existing structures and other related site specific information.
- Culvert location in both plan and profile shall be investigated to avoid sediment build-up in culvert barrels unless specifically designed to have sediment for aquatic life.
- The cost savings of multiple use (utilities, livestock and wildlife passage, land access and fish passage) shall be weighed against the advantages of separate facilities.
- Culverts shall be designed to accommodate debris or proper provisions shall be made for debris maintenance.
- Material selection shall include consideration of service life which includes abrasion and corrosion. (See Chapter 4, Culvert Repair, Materials, and Structural Design.)
- Culverts shall be located and designed to present a minimum hazard to traffic and people.
- The detail of documentation for each culvert site shall be commensurate with the risk and importance of the structure. Design data and calculations shall be assembled in an orderly fashion and retained for future reference as provided for in Section 8.3.12.
- Where practicable, some means shall be provided for personnel and equipment access to facilitate maintenance.
- Adequate cover must be provided over the crown for structural requirements. (See Chapter 4, Culvert Repair, Materials, and Structural Design.)

8.2.2 ConnDEP Requirement

See Bridge chapter, Section 9.2.3.

8.2.3 Temporary Hydraulic Facilities

See Bridge chapter, Section 9.6.12.

8.3 Design Criteria

8.3.1 Structural Alternatives

A myriad of structure alternatives are available for use in a highway-stream crossing system when all of the possible combinations of bridge lengths, spans, pier types and orientation, geometries, parapet designs, and superstructure designs are considered. In addition, at many crossings, multiple bridges or a single bridge may be viable alternatives or large culverts may be used in lieu of one or more bridges.

The hydraulics of the highway-stream crossing system should be given considerable study in choosing the preferred design from the long list of available alternatives.

8.3.2 Bridge or Culvert

Occasionally, the waterway opening(s) for a highway-stream crossing can be provided for by either culvert(s) or bridge(s). Estimates of costs and risks associated with each will indicate which structure alternative should be selected on the basis of cost, depth of cover, constructability, utility impact, environmental concerns, aesthetics, etc. It should be pointed out that the considerations outlined in Table 8-2 are not necessarily advantages or disadvantages but determine when it is appropriate to select a bridge or culvert. Other considerations which may influence structure-type selection are discussed in Section 8.5 and Chapter 9.

8.3.3 Allowable Headwater

Allowable headwater is the depth of water that can be ponded at the upstream end of the culvert during the design flood which will be limited by one or more of the following:

- non-damaging to upstream property
- 0.3 m (1 ft) below the established hydraulic control
- equal to an HW/D no greater than 1.5
- the elevation where flow is diverted from the area tributary to the culvert

8.3.4 Review Headwater

The culvert should be analyzed for a storm of greater frequency to ensure the level of inundation is tolerable to upstream property and the roadway (see Table 8-4).

Table 8-2

BRIDGE OR CULVERT

<u>Bridges</u>	
Advantages	Disadvantages
Less susceptible to clogging with drift, ice and debris Waterway generally increases with rising water surface until water begins to submerge superstructure Flowline is flexible Minimal impact on aquatic environment and wetlands Widening does not usually affect hydraulic capacity	Require more structural maintenance than culverts Fill slopes susceptible to erosion and scour damage Piers and abutments must be designed not to fail due to scour Susceptible to ice and frost formation on deck Bridge railing and parapets hazardous as compared to recovery areas when it is possible to extend the culvert beyond the safe recovery area Deck drainage may require frequent maintenance cleanout Buoyant, drag and impact forces are hazards to bridges Susceptible to damage from stream meander migration
<u>Culverts</u>	
Advantages	Disadvantages
Provides an uninterrupted view of the road Roadside recovery area can be provided Grade raises and widening projects sometimes can be accommodated by extending culvert ends Require less structural maintenance than bridges Frost and ice usually do not form before other areas experience the same problems Capacity increases with stage Capacity can sometimes be increased by installing improved inlets Usually easier and quicker to build than bridges Scour is localized, more predictable and easier to control Can be used to arrest headcutting Storage can be utilized to reduce peak discharge	Silting in multiple barrel culverts may require periodic cleanout No increase in waterway as stage rises above soffit as the bottom is fixed May clog with drift, debris or ice Possible barrier to fish passage Susceptible to erosion of fill slopes and scour at outlets Susceptible to abrasion and corrosion damage Extension may reduce hydraulic capacity Inlets of flexible culverts susceptible to failure by buoyancy Rigid culverts susceptible to separation at joints Susceptible to failure by piping and/or infiltration

8.3.5 Tailwater Relationship - Channel

- Evaluate the hydraulic conditions of the downstream channel to determine a tailwater depth for a range of discharges which includes the review or design discharge.
- Calculate backwater curves at sensitive locations or use a single cross section analysis.
- Use the critical depth and equivalent hydraulic grade line if the culvert outlet is operating with a free outfall.
- Use the headwater elevation of any nearby, downstream culvert if it is greater than the channel depth.

8.3.6 Tailwater Relationship - Confluence with a Main Stream or Large Water Body

The analysis of tributary crossings located at or within the influence of the confluence with a main stream or large water body shall utilize the following applicable tailwater conditions.

- Use the high water elevation that has the same frequency as the design flood if events are known to occur concurrently (statistically dependent).
- If statistically independent, evaluate the joint probability of flood magnitudes and use a likely combination resulting in the greater tailwater depth. For example, a main stream (receiving waters) and tributary (culvert outfall) have a drainage area ratio of 100 to 1 and a 50 year design for the culvert. Table 8-3 indicates that:
 1. when a 50 year storm is applied to the tributary, the highwater of the main stream should be determined for a 10 year storm frequency and be used as the design criteria.
 2. a 50 year highwater on the main stream should be applied to a 10 year storm frequency on the tributary. The analyses should include any additional drainage area that the outfall receives between the outlet and the receiving waters. This analysis is intended to be used as a check similar to the check frequency for the structure (Table 8-4 and 9-2). Modification to the design may be appropriate if there is likelihood of danger to persons, extensive property damage, etc. It should be noted that it may be impractical to provide an improved design due to the backwater of the main tributary.
- If tidal conditions are present, use the main stream high tide or normal depth in the downstream channel, whichever is greater.
- The worst case bridge scour event occurs when low tailwater conditions exist. When evaluating/analyzing scour, careful consideration is required when selecting the reasonable lowest tailwater conditions.
- Rating curves should be developed based on the same design/tailwater combinations.

Table 8-3 Joint Probability Analysis

AREA RATIO	FREQUENCIES FOR COINCIDENTAL OCCURRENCE							
	10-Year Design		25-Year Design		50-Year Design		100-Year Design	
	Main Stream	Tributary	Main Stream	Tributary	Main Stream	Tributary	Main Stream	Tributary
10,000 TO 1	2	10	2	25	2	50	2	100
	10	2	25	2	50	2	100	2
1,000 TO 1	2	10	2	25	5	50	10	100
	10	2	25	2	50	5	100	10
100 TO 1	5	10	5	25	10	50	25	100
	10	5	25	5	50	10	100	25
10 TO 1	10	10	10	25	25	50	50	100
	10	10	25	10	50	25	100	50
1 TO 1	10	10	25	25	50	50	100	100
	10	10	25	25	50	50	100	100

Notes: Shaded values denote design combination for coincidental frequency occurrence.
Non-shaded values denote check combination for coincidental frequency occurrence.

8.3.7 Minimum Culvert Size

Culverts providing for passage of storm runoff from one side of the highway to the other shall not be smaller than 600mm (24 in) for interstate systems and 450mm (18 in) for other systems.

8.3.8 Maximum Velocity

The maximum velocity at the culvert outlet shall be consistent with the velocity in the natural channel or shall be mitigated with outlet protection measures, energy dissipation and if required, channel stabilization. (See Section 8.7 and Chapter 7.)

8.3.9 Minimum Velocity

The minimum velocity in the culvert barrel shall result in a tractive force ($\tau = \gamma d S$) greater than critical τ of the transported streambed material at low flow rates, unless material is required to aid in fish passage. See Section 7.6.6 for a detailed discussion on tractive force.

8.3.10 Storage - Temporary or Permanent

If storage is being assumed upstream of the culvert, consideration shall be given to:

- limiting the total area of flooding
- limiting the average time that bankfull stage is exceeded for the design flood to 48 hr in rural areas or 6 hr in urban areas
- ensuring that the storage area will remain available for the life of the culvert through the purchase of right-of-way or easement

8.3.11 Flood Frequency

The flood frequency used to design or review culverts shall be based on:

- the level of risk associated with failure of the crossing, increasing backwater, or redirection of the floodwaters
- an economic assessment or analysis to justify the flood frequencies greater or lesser than the minimum flood frequencies listed herein
- location of FEMA and SCEL mapped floodplains
- ConnDOT design criteria (see Table 8-4)

Table 8-4

SUMMARY OF HYDRAULIC DESIGN CRITERIA FOR CULVERTS

CONNDOT STRUCTURE CLASS ***	DRAINAGE AREA km ² (mi ²)	DESIGN FREQUENCY (year)	CHECK FREQUENCY (year)	BACKWATER m (ft)	MINIMUM ** FREEBOARD m (ft)
Minor	< 2.59 (1) (no established watercourse)	25	-	-	0.3 (1)
Small	< 2.59 (1)	50	100	-	0.3 (1)
Intermediate *	≥ 2.59(1) < 25.9(10)	100	500	≤ 0.3 (1)	0.3 (1)
Large *	≥ 25.9(10) < 2590(1000)	100	500	0.3 (1)	0.6 (2)

* The designer shall also consider bridge alternative (see 8.3.2 and Chapter 9) for this class when area > 2.59 km (1 mi²)

** Freeboard is defined as the vertical distance between the design water surface and the upstream control such as the low point of the roadway edge, sill of a building or other controlling element.

*** See Section 9.3.3 through 9.3.6 for detailed discussions on structure classification.

8.3.12 Documentation

The following items shall be included in the documentation file (See Chapter 1, Section 1.6). The intent is not to limit data to only those items listed, but rather establish a minimum requirement consistent with the culvert design procedures as outlined in this chapter. If circumstances are such that the hydraulic design procedure is prepared other than the normal procedures or is governed by factors other than hydrologic or hydraulic factors, a narrative summary detailing the design basis shall appear with the other data.

The following items shall be included in the documentation file:

- design discharge values and discharge rating curves
- culvert performance curves
- allowable headwater elevation and basis for its selection
- type of culvert entrance conditions
- culvert outlet velocities and energy dissipation calculations and designs
- cross section(s) used in the design water surface determinations
- roughness coefficient assignments (“n” values)
- observed highwater marks with associated dates and discharges
- natural, existing and proposed conditions water surface profiles for intermediate structures or larger
- velocity measurements and locations or velocity estimates. Include both the through-bridge and channel
- copies of all computer analyses and standard computation sheets given in this chapter
- magnitude and frequency of overtopping flood if less than the design
- tailwater depth or starting water surface elevation
- roadway geometry plan and profile
- potential flood hazard to adjacent properties
- completed applicable forms (see Appendix C through F of this chapter)
- for culverts conveying drainage areas $> 2.5 \text{ km}^2$ (1 mi^2) use the Hydraulic Data form in Appendix A of Chapter 9, Bridges.
- scour evaluation/analysis for bottomless structures for design 100 yr. flood and 500 yr. flood

8.4 Design Features

8.4.1 Culvert Sizes and Shape

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

The following minimum sizes shall be used to avoid maintenance problems and clogging:

- 600 mm (24 in) for Interstate Systems, or equivalent for non-circular shapes
- 450 mm (18 in) for other systems, or equivalent for non-circular shapes
- Use arch or oval shapes only if required by hydraulic limitations, site characteristics, structural criteria, or environmental criteria
- Where practical, the minimum height of box culverts should be 1.52 m (5 ft) for inspectability

8.4.2 Multiple Barrels

Multiple barrel culverts shall fit within the natural dominant channel with minor widening of the channel so as to avoid conveyance loss through sediment deposition in some of the barrels. They are to 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 insure adequate low flows (commonly one barrel is lowered) See Section 8.5.5.
- a high potential exists for debris problems (clogging of culvert inlet)
- a meander bend is present immediately upstream

8.4.3 Material Selection

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

- The material selected shall be based on a comparison of the total cost of alternate materials over the design life of the structure which is dependent upon the following:
 - durability (service life)
 - structural strength
 - hydraulic roughness
 - bedding conditions
 - abrasion and corrosion resistance
 - water tightness requirements
- The selection shall not be made using first cost as the only criteria.
- See Chapter 4, Culvert Repair, Materials, and Structural Design, for discussion of various pipe types and service life.

8.4.4 Culvert Skew

The culvert skew shall not exceed 45° as measured from a line perpendicular to the roadway centerline without the approval of the Hydraulics and Drainage Section. Consider structural impacts of skew. Flexible culverts may need balanced fill.

8.4.5 End Treatment (Inlet or Outlet)

The culvert inlet type shall be selected from the following list based on the considerations given and the inlet coefficient, K_E . (A table of recommended values of K_E is included in Appendix B.) Consideration shall also be given to safety since some end treatments can be hazardous to errant vehicles.

Projecting Inlets or Outlets

- extend beyond the roadway embankment and are susceptible to damage during roadway maintenance and from errant vehicles
- have low construction cost
- have poor hydraulic efficiency for thin materials
- shall include anchoring the inlet to concrete slope paving and toe wall to strengthen the weak leading edge
- are used predominantly with metal pipe

Concrete Headwalls with Bevels

- increase the efficiency of metal pipe
- provide embankment stability and embankment erosion protection
- provide protection from buoyancy
- shorten the required structure length

Improved Inlets

- shall be considered for culverts which will operate in inlet control
- can increase the hydraulic performance of the culvert, but may also add to the total culvert cost, therefore, they should only be used if practicable
- slope-taper shall not be considered where fish passage is required
- could increase downstream flows and velocities

Commercial End Sections

- are available for corrugated metal, high density polyethylene pipe and concrete pipe
- retard embankment erosion and incur less damage from maintenance
- may improve projecting metal pipe entrances by increasing hydraulic efficiency and improving their appearance
- are hydraulically equal to a headwall

Wingwalls

- 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°

Aprons

- are used to reduce scour from high headwater depths or from approach velocity in the channel
- shall extend at least two pipe diameters upstream
- shall not protrude above the normal streambed elevation

Mitered Inlets

- are hydraulically more efficient than thin edge projecting
- shall be mitered to match the fill slope
- shall include anchoring the inlet to strengthen the weak leading edge for culverts 1200 mm (48 in) in diameter and larger

Cut-off-Walls

- are generally used as primary protection against piping at culvert inlets and as secondary protection against erosion at culvert outlets (primary outlet protection is designed by the hydraulic engineer)
- shall be used at endwalls and slope paved inlets and outlets as detailed on the Standard Drawings
- shall have embedment depths which comply with the following:
 - 1) for culverts included on Standard Drawings, use dimension shown
 - 2) for culverts not included on Standard Drawings, embed cutoff walls a minimum of 1.2m (4 ft) below the streambed and 0.6m (2 ft) below the bottom of the culvert
 - 3) where riprap is required, the embedment depth should not be less than the thickness of riprap/bedding material

Weep Holes

- may be required to relieve uplift pressure and reduce hydrostatic pressure behind headwalls

8.5 Related Designs

8.5.1 Buoyancy Protection

Headwalls, endwalls, slope paving or other means of anchoring to provide buoyancy protection shall be considered for all flexible culverts. Buoyancy is more serious with steepness of the culvert slope, depth of the potential headwater (debris blockage may increase), flatness of the upstream fill slope, height of the fill, large culvert skews, or mitered ends.

8.5.2 Outlet Protection

Outlet protection consists of the construction of an erosion resistant section between a conduit outlet and a stable downstream channel. Erosion at an outlet is chiefly a function of soil type and the velocity of the conduit discharge. Therefore, in order to mitigate erosion, an adequate design must stabilize the area at the conduit outlet and reduce the exit velocity to a velocity consistent with a stable condition in the downstream channel. (See Section 11.13.)

The design procedure for outlet protection will consist of:

- the calculation of the discharge velocity for the design flow
- an assessment of the erosion potential at the outlet and other critical site factors
- the selection of an appropriate design which protects the site to a degree consistent with the standards and criteria listed herein.
- an investigation of the possibility of higher velocities than the design flood such as a lower tailwater during other storm events

8.5.3 Relief Opening

Where multiple use culverts or culverts serving as relief openings have their outlet set above the normal stream flow line, special precautions shall be required to prevent headcuts or erosion from undermining the culvert outlet.

8.5.4 Erosion And Sediment Control

Temporary measures shall be included in the construction plans. These measures include the use of silt boxes, straw silt barriers, brush silt barriers, filter cloth, temporary silt fence and check dams in accordance with the "Connecticut Guidelines for Soil Erosion and Sedimentation Control" Manual.

8.5.5 Environmental Considerations And Fishery Protection

Care must be exercised in selecting the location of the culvert site to control erosion, sedimentation and debris. Select a site that will permit the culvert to be constructed and will limit the impact on the stream or wetlands. The ConnDEP's Fisheries Division shall be contacted early in the design process if the site is considered to be a watercourse, to determine the need for fish passage and to solicit their recommendations. See Chapter 9, Section 9.2.3 for additional ConnDEP requirements.

8.5.6 Design Philosophy

The design of a culvert system for a highway crossing of a floodplain involves using information from the following chapters in this manual (hydrology, channels, bridges and storm drainage systems). Each of these should be consulted as appropriate. The discussion in this section is focused on alternative analysis and design methods.

Alternative Analysis

Culvert alternatives shall be selected to satisfy topography and design practice and criteria. Alternatives shall be analyzed for environmental impact, hydraulic equivalency, risk and cost.

Select an alternative which best integrates engineering, economic, and environmental considerations. The chosen culvert shall meet the selected structural and hydraulic criteria and shall be based on:

- utility accommodation
- construction and maintenance costs
- risk of failure or property damage
- traffic safety
- environmental or aesthetic considerations
- political or nuisance considerations
- land use requirements
- availability of detour or alternate routes
- constructability

Design Methods: The designer shall determine whether it is most appropriate:

- to assume a constant discharge or upon approval from the Hydraulics and Drainage Section, route a hydrograph
- to use nomographs or computer software

8.5.7 Performance Curves

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

8.6 Design Equations

8.6.1 General

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

- analyzing nonuniform flow with regions of both gradually varying and rapidly varying flow
- determining how the flow type changes as the flow rate and tailwater elevations change
- applying backwater and drawdown calculations, energy and momentum balance
- applying the results of hydraulic model studies
- determining if hydraulic jumps occur and if they are inside or downstream of the culvert barrel

8.6.2 Approach

The procedures in this chapter use the following:

Control Section: The control section is the location where there is a defined relationship between the flow rate and the upstream water surface elevation.

- Inlet control is governed by the inlet geometry.
- Outlet control is governed by a combination of the culvert inlet geometry, the barrel characteristics and the tailwater.

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

8.6.3 Inlet Control

For inlet control, the control section is at 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, a hydraulic jump may occur downstream of the inlet.

Headwater Factors

Headwater depth is measured from the inlet invert or the channel invert (if the culvert is buried for fish passage) of the inlet control section to the surface of the upstream pool. This measurement should be taken some distance upstream of the culvert entrance to avoid the “drawdown” curve.

Inlet area is the cross-sectional area of the face of the culvert. Generally, the inlet face area is the same as the barrel area.

Inlet edge configuration describes the entrance type. 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. Typical shapes are rectangular, circular, elliptical and arch. Check for an additional control section, if different than the barrel.

Hydraulics

Three regions of flow are shown in the Figure 8-1: unsubmerged, transition and submerged:

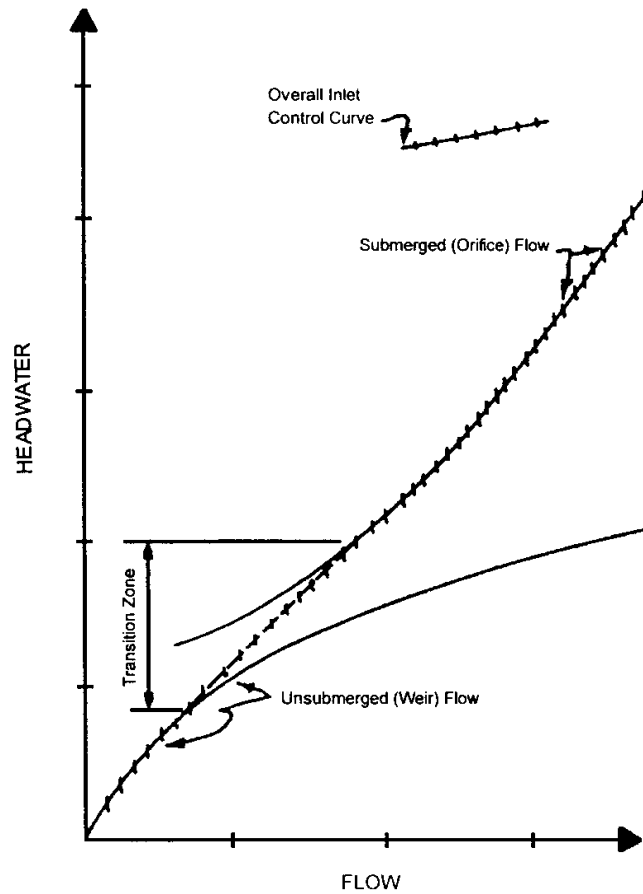


Figure 8-1 Unsubmerged, Transition And Submerged

Unsubmerged

For headwater below the inlet crown, the entrance operates as a weir.

- 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.
- These tests are then used to develop equations. Appendix A of HDS5 contains the equations which were developed from model test data, see Figure 8-2, Flow Type I:

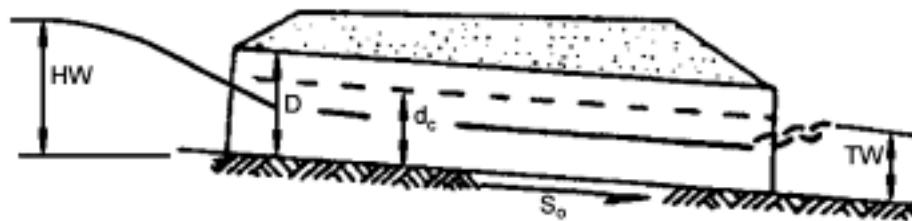


Figure 8-2 Flow Type I

Submerged

For headwaters above the inlet crown, the culvert operates as an orifice.

- An orifice is an opening, submerged on the upstream side and flowing freely on the downstream side, which functions as a control section.
- The relationship between flow and headwater can be defined based on results from model tests. Appendix A of HDS 5 contains flow equations which were developed from model test data. See Figure 8-3 flow type V.

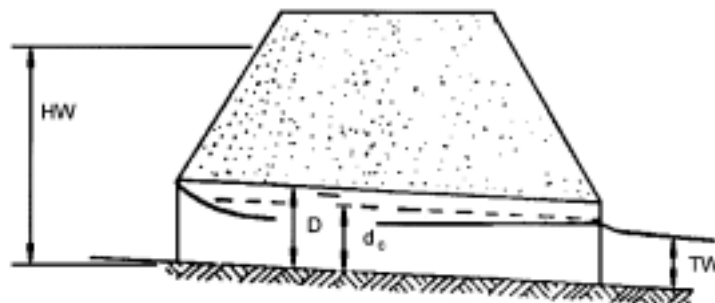


Figure 8-3 Flow Type V

Transition Zone: The transition zone is located between the unsubmerged and the submerged flow conditions where the flow 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.

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. Note that in the inlet control nomographs, HW is measured to the total upstream energy grade line including the approach velocity head.

8.6.4 Outlet Control

Outlet control has depths and velocities which are subcritical. The control of the flow is at the downstream end of the culvert (the outlet). The tailwater depth is either assumed to be critical depth near the culvert outlet or the downstream channel depth, whichever is higher. In a given culvert, the type of flow is dependent on all of the barrel factors. All of the inlet control factors also influence culverts in outlet control.

Barrel Roughness: Barrel roughness is a function of the material used to fabricate the barrel. Typical materials include concrete and corrugated metal. The roughness is represented by a hydraulic resistance coefficient such as the Manning n value. Typical Manning n values are presented in Appendix A.

Barrel Area: Barrel area is measured perpendicular to the flow.

Barrel Length: 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 slope influence the actual length, an approximation of barrel length is usually necessary to begin the design process.

Barrel Slope: 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: Tailwater is based on the downstream water surface elevation. Backwater calculations from a downstream control, a normal depth approximation, or field observations are used to define the tailwater elevation (see Section 8.3.5 and 8.3.6).

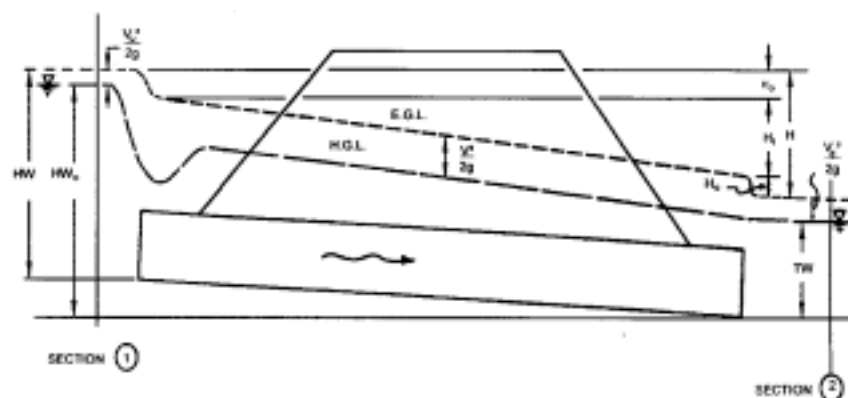


Figure 8-4 Flow Type IV

8.6.5 Hydraulics (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.

$$\text{Losses } H_L = H_E + H_f + H_v + H_b + H_j + H_g \quad (8.1)$$

Where: H_L = total energy loss, m (ft)
 H_E = entrance loss, m (ft)
 H_f = friction losses, m (ft)
 H_v = exit loss (equivalent to H_o (see equation 8.4d) m (ft))
 H_b = bend losses, m (ft) (see HDS 5)
 H_j = losses at junctions, m (ft) (see HDS 5)
 H_g = losses at grates, m (ft) (see HDS 5)

$$\text{Velocity } V = Q/A \quad (8.2)$$

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

$$\text{Velocity head } H_v = V^2/2g \quad (8.3)$$

Where: g = acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2)

$$\text{Entrance loss } H_E = K_E (V^2/2g) \quad (8.4a)$$

Where: K_E = entrance loss coefficient, See Appendix B

$$\text{Friction loss } H_f = [(19.63n^2L)/R^{1.33}] [V^2/2g] \quad (H_f = [(29n^2L)/R^{1.33}] [V^2/2g]) \quad (8.4b)$$

Where: n = Manning's roughness coefficient (See Appendix A)

L = length of the culvert barrel, m (ft)

R = hydraulic radius of the full culvert barrel = A/P , m (ft)

P = wetted perimeter of the barrel, m (ft)

$$\text{Exit loss } H_o = 1.0 [(V^2/2g) - (V_d^2/2g)] \quad (8.4c)$$

Where: V_d = channel velocity downstream of the culvert, m/s (ft/s) (usually neglected, see equation 8.4d).

$$H_o = H_v = V^2/2g \quad (8.4d)$$

Barrel losses $H = H_E + H_o + H_f$

$$H = [1 + K_e + (19.63n^2L/R^{1.33})] [V^2/2g] \quad (H = [1 + K_e + (29n^2L/R^{1.33})] [V^2/2g]) \quad (8.5)$$

Energy Grade Line: 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:

$$HW_o + (V_u^2/2g) = TW + (V_d^2/2g) + H_L \quad (8.6)$$

Where: HW_o = headwater depth above the outlet invert, m (ft)

V_u = approach velocity, m/s (ft/s)

TW = tailwater depth above the outlet invert, m (ft)

V_d = downstream velocity, m/s (ft/s)

H_L = sum of all losses (equation 8.1)

Hydraulic Grade Line: The hydraulic grade line is the depth to which water would rise in vertical tubes connected to the sides of the culvert barrel. In full flow, the energy grade line and the hydraulic grade line are parallel lines separated by the velocity head except at the inlet and the outlet.

Nomographs (full flow): The nomographs were developed assuming that the culvert barrel is flowing full and:

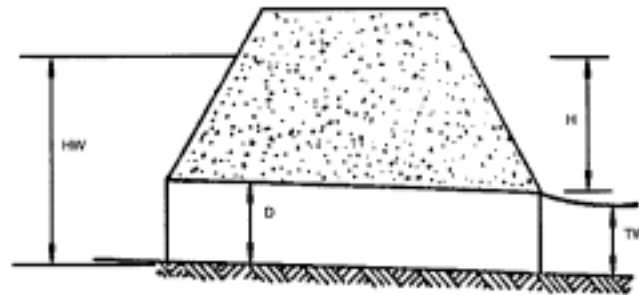


Figure 8-5 Flow Type VI

- $TW \geq D$, Flow Type IV (see Figure 8-4) or
- $d_c \geq D$, Flow Type VI (see Figure 8-5)
- 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.
- Equation (8.6) becomes:

$$HW = TW + H - S_oL \quad (8.7)$$

Where: HW = depth from the inlet invert to the energy grade line, m (ft)
 H = is the value read from the nomographs (equation 8.5), m (ft)
 S_oL = drop from inlet to outlet invert, m (ft)

Nomographs (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 8-6.

Backwater calculations may be required which begin at the downstream water surface and proceed upstream. If the depth intersects the top of the barrel, full flow extends from that point upstream to the culvert entrance.

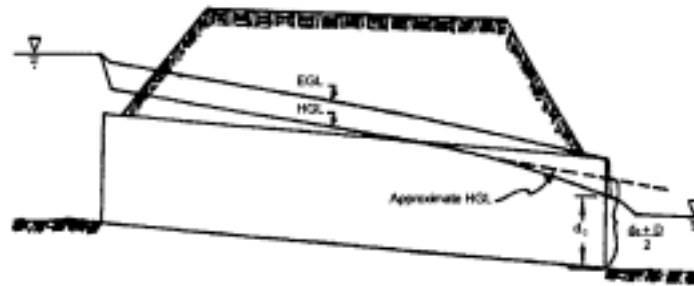


Figure 8-6 Flow Type VII

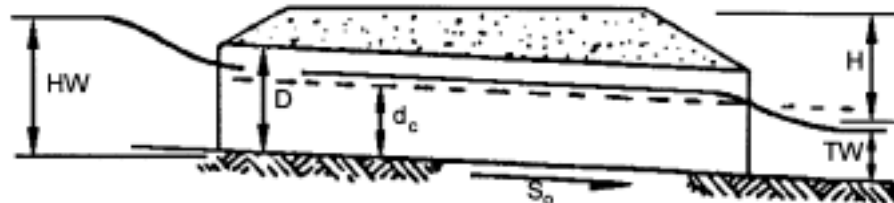
Nomographs (Partly full flow) – Approximate method: Based on numerous backwater calculations performed by the FHWA, it was found that the hydraulic grade line pierces the plane of the culvert outlet at a point approximately one-half way between critical depth and the top of the barrel or $(d_c + D)/2$ above the outlet invert. TW should be used if higher than $(d_c + D)/2$. The following equation should be used:

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

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

The approximate method renders adequate results for a $HW > 0.75D$. For lower headwaters, backwater calculations are required.

(See Figure 8-7 if $TW < d_c$ and Figure 8-8 if $TW > d_c$)

Figure 8-7 Flow Type II, $TW < d_c$

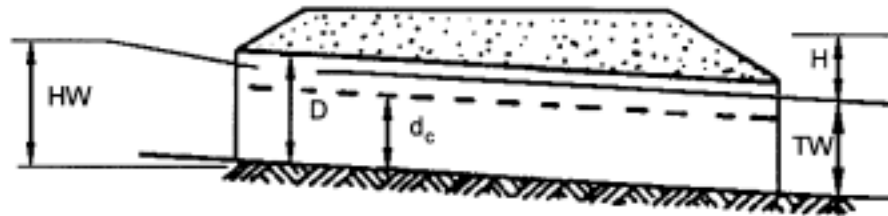


Figure 8-8 Flow Type III, $TW > d_c$

8.6.6 Outlet Velocity

Culvert outlet velocities shall be calculated to determine need for erosion protection at the culvert exit. Culverts usually result in outlet velocities which are higher than the natural stream velocities. These outlet velocities may require flow readjustment or energy dissipation to prevent downstream erosion.

Inlet Control: The velocity is calculated from equation 8.2 after determining the outlet depth. Either of the following methods may be used to determine the outlet depth.

- Calculate the water surface profile through the culvert. Begin the computation at d_c at the entrance and proceed downstream to the exit. Determine at the exit the depth and flow area.
- Assume normal depth and velocity. This approximation may be used since the water surface profile converges towards normal depth if the culvert is of adequate length. This outlet velocity may be slightly higher than the actual velocity at the outlet. Normal depths in feet may be obtained from design aids in publications such as HDS 3.

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

- 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 exceeds the top of the barrel.

8.7 Erosion, Sedimentation, and Debris Control

8.7.1 Introduction

Natural streams and manmade channels are subject to the forces of moving water. Pressure, velocity, and centrifugal forces can be significant depending on the depth of flow, and the slope and sinuosity of the watercourse. An evolutionary process is the result with the continuous occurrence and dynamic interplay of erosion, sedimentation, and debris movement. This process, referred to as fluvial geomorphology, is accelerated during storm events when stream depths and velocities are high. Inserting a culvert into this dynamic environment requires special attention to the effects of these natural phenomena on the culvert and the effects of the culvert on the stream channel. Past experience has shown significant problems, including erosion at the inlet and outlet, sediment buildup in the barrel, and clogging of the barrel with debris.

8.7.2 Scour at Inlets

A culvert barrel normally constricts the natural channel, thereby forcing the flow through a reduced opening. As the flow contracts, vortices and areas of high velocity flow impinge against the upstream slopes of the fill and may tend to scour away the embankment adjacent to the culvert. In many cases, a scour hole also forms upstream of the culvert floor as a result of the acceleration of the flow as it leaves the natural channel and enters the culvert.

Upstream slope paving, riprap protection, headwalls, wingwalls, and cutoff walls help to protect the slopes and channel bed at the upstream end of the culvert.

8.7.3 Scour at Outlets

Scour at culvert outlets is a common occurrence. The natural channel flow is usually confined to a lesser width and greater depth as it passes through a culvert barrel. An increased velocity results with potentially erosive capabilities as it exits the barrel. Turbulence and erosive eddies form as the flow expands to conform to the natural channel. However, the velocity and depth of flow at the culvert outlet and the velocity distribution upon reentering the natural channel are not the only factors which need consideration. The characteristics of the channel bed and bank material, velocity and depth of flow in the channel at the culvert outlet, and the amount of sediment and other debris in the flow are all contributing factors to scour potential. Due to the variation in expected flows and the difficulty in evaluating some of these factors, scour prediction is subjective.

Scour in the vicinity of a culvert outlet can be classified into two separate types. The first type is called local scour and is typified by a scour hole produced at the culvert outlet. This is the result of high exit velocities, and the effects extend only a limited distance downstream. Coarse material scoured from the circular or elongated hole is deposited immediately downstream, often forming a low bar. Finer material is transported further downstream. The dimensions of the scour hole change due to sedimentation during low flows and the varying erosive effects of storm events. The scour hole is generally deepest during passage of the peak flow. Methods for predicting scour hole dimensions are found in HEC No. 14, "Hydraulic Design of Energy Dissipators for Culverts and Channels." The second type of scour is classified as general stream degradation. This phenomenon is independent of culvert performance. Natural causes produce a lowering of the streambed over time. The identification of a degrading stream is an essential part of the original site investigation. Both types of scour can occur simultaneously at a culvert outlet.

Protection against scour at culvert outlets varies from limited riprap placement to complex and expensive energy dissipation devices. At some locations, use of a rougher culvert material or a flatter slope alleviates the need for a special outlet protection device. Preformed scour holes, approximating the configuration of naturally formed holes, dissipate energy while providing a protective lining to the streambed. Riprapped channel expansions and concrete aprons protect the channel and redistribute or spread the flow. Barrel outlet expansions operate in a similar manner. Headwalls and cutoff walls protect the integrity of the fill. When outlet velocities are high enough to create excessive downstream problems, consideration should be given to more complex energy dissipation devices. These include hydraulic jump-basins, impact basins, drop structures, and stilling wells. Design information for the general types of energy dissipators is provided in HEC No. 14. Riprap is the most common element for outlet protection in Connecticut. Design guidelines for smaller culverts can be found in Chapter 11, Section 11.13. Outlet protection for larger culvert is discussed in Chapter 7.

8.7.4 Sedimentation

The companion problem to erosion is sedimentation. Most streams carry a sediment load and tend to deposit this load when their velocities decrease. Therefore, barrel slope and roughness are key indicators of potential problems at culvert sites. Other important factors in sedimentation processes are the magnitude of the discharge and the characteristics of the channel material.

Culverts which are located on and aligned with the natural channel generally do not have a sedimentation problem. A stable channel is expected to balance erosion and sedimentation over time; a culvert resting on such a channel bed behaves in a similar manner. In a degrading channel, erosion, not sedimentation, is a potential problem. However, a culvert located in an aggrading channel may encounter some sediment accumulation. Stream channel aggradation and degradation, and characteristics of each type of stream are discussed in HEC 20. Fortunately, storm events tend to cleanse culverts of sediment when increased velocities are experienced. Helical corrugations tend to promote this cleansing effect if the culvert is flowing full.

Certain culvert installations may encounter sedimentation problems. The most common of these are multibarrel installations and culverts built with depressions at the entrance. Culverts with more than one barrel may be necessary for wide shallow streams and for low fills. It is well documented the one or more of the barrels will accumulate sediment, particularly the inner barrel in a curved stream alignment. It is desirable for these installations to be straight and aligned with the upstream channel. Culverts built with an upstream depression possess a barrel slope which is less than that of the natural channel. Sedimentation is the likely result, especially during times of low flow. However, self-cleansing usually occurs during periods of high discharge. Both design situations should be approached cautiously with an increased effort in the field investigation stage to obtain a thorough knowledge of stream characteristics and bed-bank materials.

8.7.5 Assessment of Erosion Potential

A field investigation of all proposed outlet locations or existing outlets to be used in a drainage design of a proposed project should be conducted to determine the erosion resistance of the soils at the outlet, the character of the downstream flow path, and any other site constraints that must be addressed by the proposed design.

Barring any unusual conditions, as determined during the field investigation, the criteria outlined in Section 11.13 should be used to determine the type of outlet protection required. When severe conditions are present, it is the responsibility of the designer to provide outlet protection as needed to safeguard against erosion damage.

8.8 Roadway Overtopping

8.8.1 General

Roadway overtopping will begin when the headwater rises to the elevation of the roadway. The overtopping will usually occur at the low point of a sag vertical curve on the roadway. The flow will be similar to flow over a broad crested weir. Flow coefficients for flow overtopping roadway embankments are found in HDS No. 1, Hydraulics of Bridge Waterways, as well as in the documentation of HY-7, the Bridge Waterways Analysis Model. Curves from the latter are included in Figure 8-9. The roadway overtopping flow rate can be determined using the following equations.

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

Where: Q_r = overtopping flow rate, m^3/s (ft^3/s)

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, m (ft)

HW_r = the upstream depth, measured above the roadway crest, m (ft)

8.8.2 Length

The determination of length versus HW is difficult to define when the length of the crest is defined by a roadway sag vertical curve.

- It is recommended to subdivide the length into a series of segments. The flow over each segment is calculated for a given headwater. The flows for each segment are added together to determine the total flow.
- As an approximation, the length can be represented by a single horizontal line (one segment). The length of the weir is the horizontal length of this segment. The depth is the average depth (area/length) of the upstream pool above the roadway.

8.8.3 Total Flow

Total weir flow is calculated for a given upstream water surface elevation using equation 8.9.

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

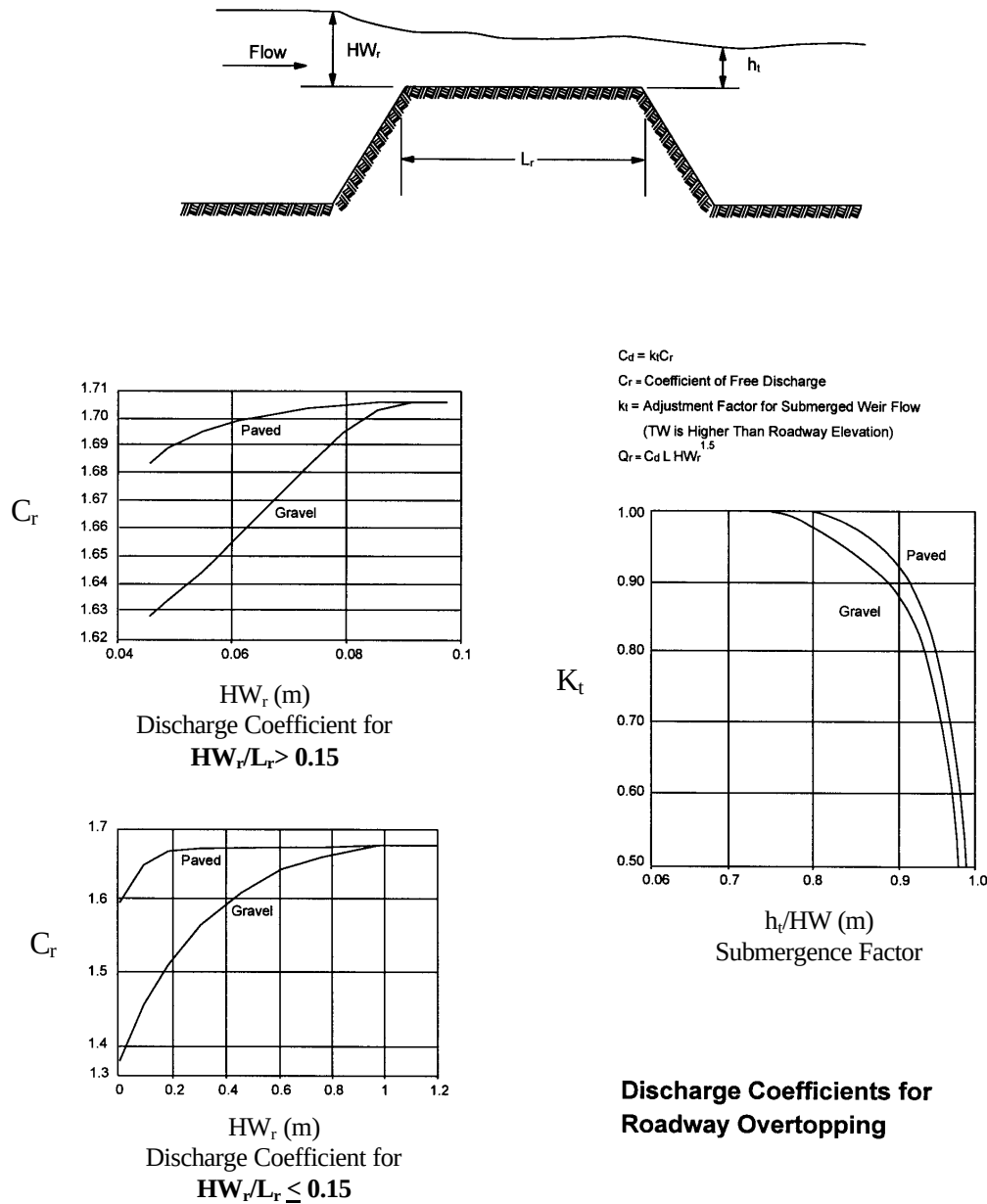
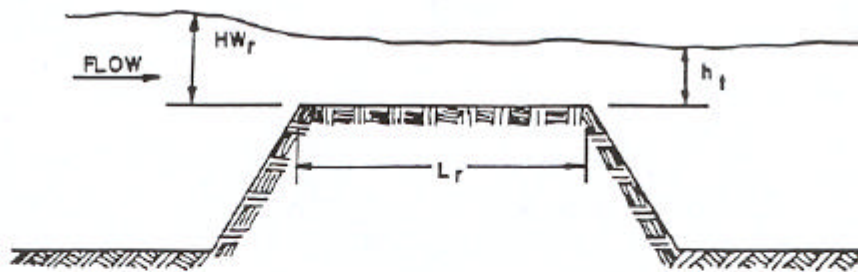


Figure 8-9 Curves for Roadway Overtopping (Metric units)

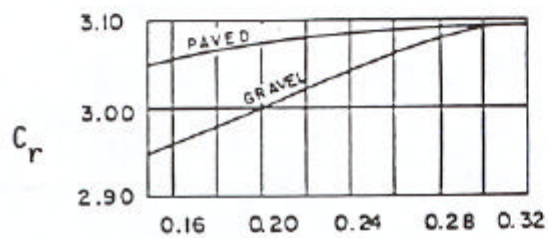


$$C_d = k_t C_r$$

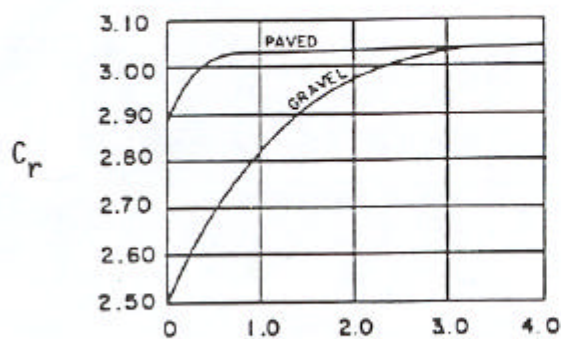
C_r = Coefficient of free discharge

K_t = Adjustment factor for submerged flow

$$Q_r = C_d L H W_r^{1.5}$$



HW_r (ft)
Discharge Coefficient for
 $HW_r/L_r > 0.15$



HW_r (ft)
Discharge Coefficient for
 $HW_r/L_r \leq 0.15$

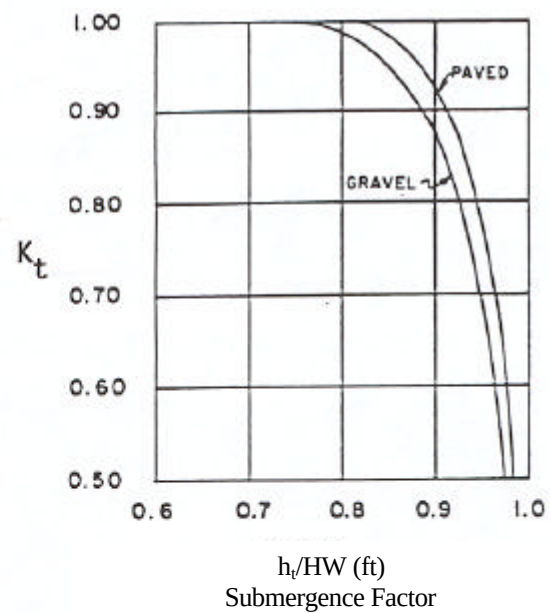


Figure 8-9.1 Curves for Roadway Overtopping (English units)

8.9 Performance Curves

Performance curves are plots of flow rate versus headwater depth or elevation, velocity, or outlet scour. The culvert performance curve is made up of the controlling portions of the individual performance curves for each of the following control sections (See Figure 8-10):

8.9.1 Inlet

The inlet performance curve is developed using the inlet control nomographs (see HDS 5).

8.9.2 Outlet

The outlet performance curve is developed using equations 8.1 through 8.7, the outlet control nomographs (see HDS 5), or backwater calculations.

8.9.3 Roadway

Roadway performance curves are developed using equation 8.9.

8.9.4 Overall

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:

1. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters shall be calculated.
2. Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.
3. When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the upstream water surface depth above the roadway for each selected flow rate. Use these water surface depths and equation (8.9) to calculate flow rates across the roadway.
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 8-10.

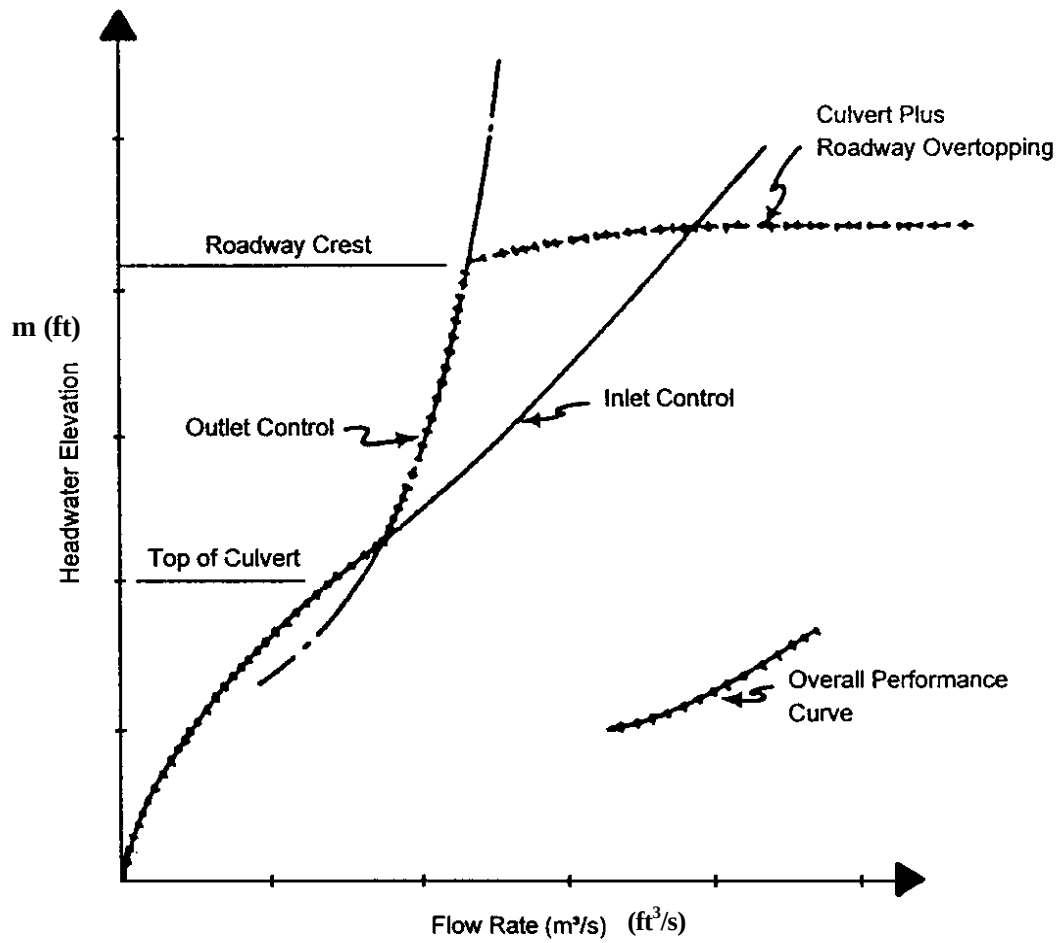


Figure 8-10 Overall Performance Curve

8.10 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 10, Storage Facilities.)

- The designer should be familiar with all the equations in Section 8.6 before using these procedures.
- Following the design method without an understanding of culvert hydraulics can result in an inadequate, unsafe, or costly structure.
- Computation forms have been provided in Appendices C through G to use as a guide. They contain blocks for the project description, designer's identification, hydrologic data, culvert dimensions and elevations, trial culvert description, inlet and outlet control HW, culvert barrel selected and comments. The hydraulic control and its elevation should be included.
- Steps 7 through 12 can be accomplished using computer programs.

Step 1 Assemble Site Data And Project File

- a. See Data Chapter — The minimum data are:
 - USGS, site and location maps
 - embankment cross section
 - roadway profile
 - photographs
 - field visit (sediment, debris)
 - design data at nearby structures
- b. Studies by other agencies including:
 - small dams — NRCS, USCE, ConnDEP
 - floodplain — NRCS, USCE, FEMA, USGS, NOAA, ConnDEP
 - storm drain — local or private
- c. Environmental constraints including:
 - commitments contained in review documents
 - fish migration
 - wetland impacts
- d. Design criteria
 - review Section 8.3 for applicable criteria
 - prepare risk assessment or analysis

Step 2 Determine Hydrology

- a. See Chapter 6, Hydrology.
- b. Minimum data are drainage area map and a discharge-frequency plot.

Step 3 Analyze Downstream Channel

- a. See Chapter 7, Channels.
- b. Minimum data are cross section of channel and the rating curve for channel.

Step 4 Summarize Data On Design Form

- a. See Appendices C through G.
- b. Data from steps 1-3.

Step 5 Select Design Discharge Q_d

- a. See Section 8.3 Design Criteria.
- b. Determine flood frequency from criteria.
- c. Determine Q from discharge-frequency plot (step 2).
- d. Divide Q by the number of barrels.

Step 6 Select Design Alternative

- a. See Section 8.4 Design Features.
- b. Choose culvert material, shape, size and entrance type.

Step 7 Determine Inlet Control Headwater Depth (HW_i)

Use the inlet control nomograph (HDS 5).

- a. Locate the size or height on the scale.
- b. Locate the discharge.
 - For a circular shape use discharge.
 - For a box shape use Q per foot of width.
- c. Locate HW/D ratio.
 - Use a straight edge.
 - Extend a straight line from the culvert size through the flow rate.
 - Mark the first HW/D scale. Extend a horizontal line to the desired scale and read HW/D .
- d. Calculate headwater depth (HW_i).
 - Multiply HW/D by D to obtain HW to energy gradeline.
 - Neglecting the approach velocity $HW_i = HW$.
 - Including the approach velocity $HW_i = HW - \text{approach velocity head}$.

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) or using a water surface profile. See Section 8.3.5 and 8.3.6.
- b. Calculate critical depth (d_c) using appropriate chart in HDS 5.
 - 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 $(d_c + D)/2$.
- d. Determine (h_o).
 $h_o = \text{the larger of } TW \text{ or } (d_c + D/2).$

- e. Determine (K_E).
 - Entrance loss coefficient from Table 2 in the Appendix B.
- f. Determine losses through the culvert barrel (H).
 - Use nomograph HDS5 or equation 8.5 or 8.6 if outside range.
 - Locate appropriate K_E scale.
 - Locate culvert length (L) or (L_1):
 - use (L) if Manning's n matches the n value of the culvert and
 - use (L_1) to adjust for a different culvert n value.

$$L_1 = L(n_1/n)^2 \quad (8.10)$$

Where: L_1 = adjusted culvert length, m (ft)
 L = actual culvert length, m (ft)
 n_1 = desired Manning n value
 n = Manning n value on chart

- Mark point on turning line:
 - use a straight edge and
 - connect size with the length.
- Read (H):
 - use a straight edge,
 - connect Q and turning point and
 - read (H) on Head Loss scale.
- g. Calculate outlet control headwater (HW_{oi}).
 - Use equation 8.18, if V_u and V_d are neglected:

$$HW_{oi} = H + h_o - S_o L \quad (8.11)$$

- 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 $(d_c + D)/2$ may not be applicable,
 - backwater calculations should be used to check the result and
 - if the headwater depth falls below 0.75D, the approximate nomograph method shall not be used.

Step 9 Determine Controlling Headwater (HW_c)

- Compare HW_i and HW_{oi} , use the higher.

Step 10 Compute Discharge Over The Roadway (Q_r) (If applicable)

- a. Calculate depth above the roadway (HW_r).

$$HW_r = HW_c - HW_{ov}$$

$$HW_{ov} = \text{height of road above inlet invert}$$
- b. If $HW_r \leq 0$, $Q_r = 0$
 If $HW_r > 0$, determine C_d from HDS5
- c. Determine length of roadway crest (L).
- d. Calculate Q_r using equation 8.9.

Step 11 Compute Total Discharge (Q_t) (If applicable)

$$Q_t = Q_d + Q_r \quad (8.12)$$

Step 12 Calculate Outlet Velocity (V_o) And 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) = 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) = 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 shall be close to the approximate length,
- the headwalls and wingwalls must fit site,
- the allowable headwater shall not be exceeded and
- the allowable overtopping flood frequency shall not be exceeded.

Step 14 Plot Performance Curve

- a. Repeat steps 6 through 12 with a range of discharges.
- b. Use the following upper limit for discharge:
 - Q_{100} if $Q_o \leq Q_{100}$
 - Q_{500} if $Q_o > Q_{100}$
 - $Q_{\max}$ if no overtopping is possible
 - $Q_{\max}$ = largest flood that can be estimated

Step 15 Related Designs

Consider the following options.

- Tapered inlets if culvert is in inlet control and has limited available headwater. (See Section 8.11.)
- Flow routing if a large upstream headwater pool exists (See Section 8.10).
- Energy dissipators or other forms of outlet protection if V_o is larger than the normal V in the downstream channel (See Section 8.7).
- Fish passage (See Section 8.5.5).
- Where practicable, some means shall be provided for personnel and equipment access to facilitate maintenance.
- Culverts shall be regularly inspected and maintained.

Step 16 Documentation

- See Section 8.3.12 and Chapter 1, Section 1.6
- Prepare report and file with background information.

8.11 Flood Routing Culvert Design

8.11.1 Introduction

Flood routing through a culvert is a practice that evaluates the effect of temporary upstream ponding caused by the culvert's backwater. It is possible that the findings from culvert analyses will be conservative if flood routing is not considered. Even if the selected allowable headwater is considered acceptable without flood routing, it is possible that costly over design of both the culvert and outlet protection may result, depending on the amount of temporary storage involved.

There are many ramifications listed below associated with altering the temporary upstream ponding caused by a culvert's backwater. Culvert routing can be helpful in dealing with these ramifications.

- requirement for ownership or easement of upstream properties
- a perceived loss of a subjective safety factor
- credibility, both in court as well as in technical negotiations
- evaluating environmental concerns
- realistic assessments of potential flood hazards
- estimation of sediment problems

Flood Hazards

Potential flood hazards increase whenever a culvert increases the natural flood stage. Some of these hazards can conservatively be assessed without flood routing. However, some damages associated with culvert backwater are time dependent and thus require an estimate of depth versus duration of inundation. Some vegetation and commercial crops can tolerate longer periods of inundation than others, and to greater depths. Such considerations become even more important when litigation is involved.

Culvert Replacement Applications

Improved hydrology methods or changed watershed conditions are factors that can cause an older, existing culvert to be determined to be inadequate. A culvert analysis that relies on findings that ignore any available temporary storage may be misleading. A flood routing analysis may show that what was thought to be an inadequate existing culvert is, in fact, adequate.

Often existing culverts require replacement due to corrosion or abrasion. This can be very costly, particularly where a high fill is involved. A less costly alternative may be to place a smaller culvert with better entrance conditions and smoother pipe inside the existing culvert. A flood routing analysis may, where there is sufficient storage, demonstrate that this is acceptable in that no increase in flood hazard results.

Limitations

There are situations where culvert sizes and velocities obtained through flood routing will not differ significantly from those obtained by designing to the selected peak discharge and ignoring any temporary upstream storage.

This occurs when:

- there is no significant temporary pond storage available (as in deep incised channels)
- the culvert must pass the design discharge with no increase in the natural channel's flood stage
- runoff hydrographs last for long periods of time such as with snowmelt runoff

8.11.2 Routing Equations

In addition to the previous Design Equations (Section 8.6), the following routing equations shall be used. The basic flood routing equation is:

$$I - O = \Delta S / \Delta t \quad \text{or} \quad (8.13)$$

$$2S_1 / \Delta t - O_1 + I_1 + I_2 = 2S_2 / \Delta t + O_2 \quad (8.14)$$

For a finite interval of time, Δt , equation 8.21 can be expressed by:

$$\Delta S = Q_i \Delta t - Q_o \Delta t \quad (8.15)$$

From these equations:

$$(I_1 + I_2) / 2 = \Delta S / \Delta t + O_1 / 2 + O_2 / 2 \quad (8.16)$$

Where:

- ΔS = $S_2 - S_1$
- S_1 = storage volume in the temporary pond at the beginning of the incremental time period Δt , m^3 (ft^3)
- S_2 = storage volume in the temporary pond at the end of the incremental time period Δt , m^3 (ft^3)
- Δt = incremental routing time interval selected to subdivide hydrograph into finite time elements, min
- I = average hydrograph inflow to the temporary pond during incremental time period Δt , m^3/s (ft^3/s)
- I_1 = instantaneous inflow to the temporary pond at the beginning of the incremental time period Δt , m^3/s (ft^3/s)
- I_2 = instantaneous inflow at the end of the time period Δt , m^3/s (ft^3/s)
- O = average outflow from the temporary pond during incremental time period Δt , m^3/s (ft^3/s)
- O_1 = instantaneous outflow at the beginning of the time period Δt , m^3/s (ft^3/s)
- O_2 = instantaneous outflow at the end of the time period Δt , m^3/s (ft^3/s)

8.11.3 Design Procedure

The design procedure for flood routing through a culvert is the same as for reservoir routing. The site data and roadway geometry are obtained (Data Collection Chapter) and the hydrology analysis completed to include estimating a hydrograph (Hydrology Chapter). Once this essential information is available, the culvert can be designed.

Flood routing through a culvert can be time consuming.

The designer should be familiar with the culvert flood routing design process. This familiarization is necessary in order to:

- recognize and test suspected software malfunctions
- circumvent any software limitations
- flood route manually where the software is limited
- understand and discuss culvert flood routing in a credible manner.

The steps are outlined in the Chapter 10, Storage Facilities.

8.12 Tapered Inlets

8.12.1 General

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section. A tapered inlet may have a depression, or FALL, incorporated into the inlet structure or located upstream of the inlet. The depression is used to exert more head on the throat section for a given headwater elevation. Therefore, tapered inlets improve culvert performance by providing a more efficient control section (the throat). Tapered inlets with FALL also improve performance by increasing the head on the throat. Tapered inlets are discussed in more detail in Hydraulic Engineering Circular No. 13, "Hydraulic Design of Improved Inlets for Culverts."

- Tapered inlets are not recommended for use on culverts flowing in outlet control because the simple beveled edge is of equal 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 are available for rectangular box culverts and circular pipe culverts.

8.12.2 Side-tapered

The side-tapered inlet has an enlarged face section with the transition to the culvert barrel accomplished by tapering the side walls (Figure 8-11). The face section is about the same height as the barrel height and the inlet floor is an extension of the barrel floor. The inlet roof may slope upward slightly, provided that the face height does not exceed the barrel height by more than 10% (1.1D).

There are two possible control sections, the face and the throat. HW_f , shown in Figure 8-11, 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. In addition, the throat is always slightly lower than the face so that more head is exerted on the throat for a given headwater elevation.

The beneficial effect of depressing the throat section below the streambed can be increased by installing a depression upstream of the side-tapered inlet. Figure 8-12 depicts a side-tapered inlet with the depression contained between wingwalls. For this type of depression, the floor of the barrel should extend upstream from the face a minimum distance of $D/2$ before sloping upward more steeply. The length of the resultant upstream crest where the slope of the depression meets the streambed should be checked to assure that the crest will not control the flow at the design flow and headwater. If the crest length is too short, the crest may act as a weir control section, the barrel is defined as the throat section.

8.12.3 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. (Figure 8-13) 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.

A slope-tapered inlet has three possible control sections, the face, the bend and the throat. Of these, only the dimensions of the face and the throat section are determined by the design procedures of this manual. The size of the bend section is established by locating it a minimum distance upstream from the throat so that it will not control the flow.

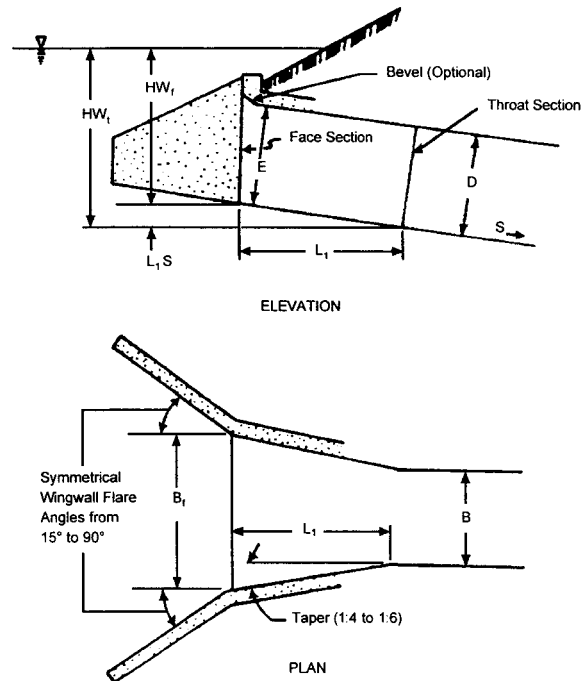


Figure 8-11 Side-Tapered Inlet

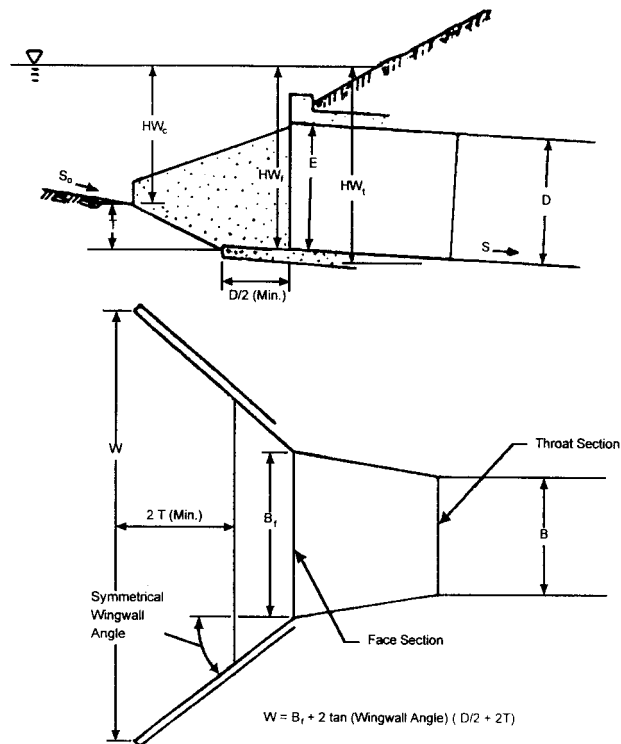


Figure 8-12 Side-tapered Inlet With Upstream Depression Contained Between Wingwalls

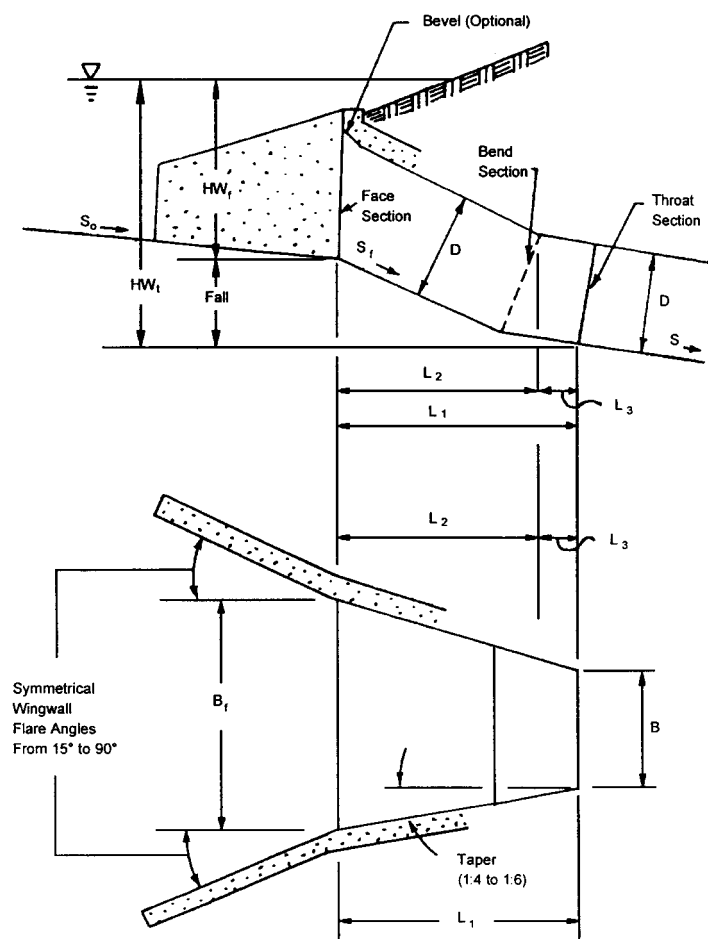


Figure 8-13 Slope-Tapered Inlet With Vertical Face

The slope-tapered inlet combines an efficient throat section with additional head 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 vertical face slope-tapered inlet design is shown in Figure 8-13.

The slope-tapered inlet is the most complex inlet improvement recommended in this manual. Construction difficulties are inherent, but the benefits in increased performance can be great. 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. For the latter application, a square to round transition is normally used to connect the rectangular slope-tapered inlet to the circular pipe.

8.12.4 Hydraulic Design

(Inlet Control)

Tapered inlets have several possible control sections including the face, the bend (for slope-tapered inlets) and the throat. In addition, a depressed side-tapered inlet has a possible control section at the crest upstream of the depression. Each of these inlet control sections has an individual performance curve. The headwater depth for each control section is referenced to the invert of the section. One method of determining the overall inlet control performance curve is to calculate performance curves for each potential control section, and then select the segment of each curve which defines the minimum overall culvert performance. (Figure 8-14)

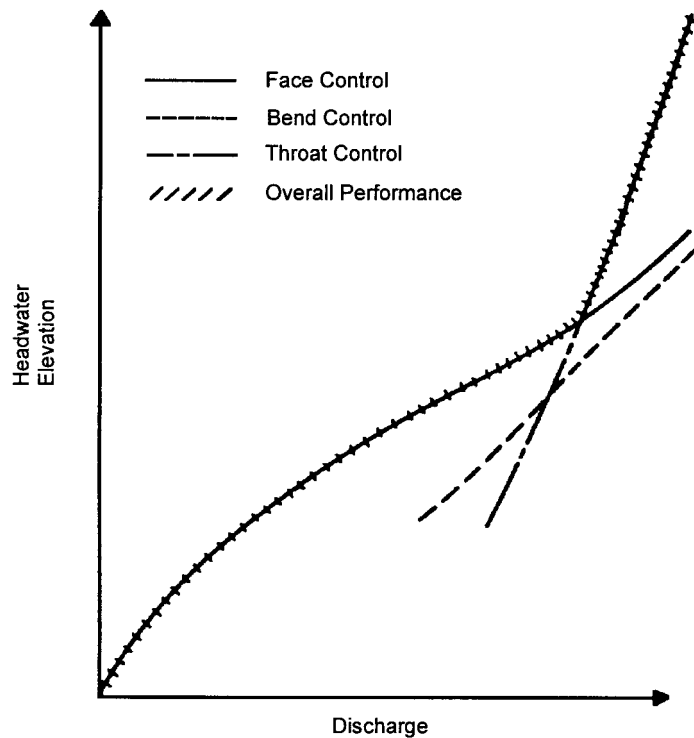


Figure 8-14 Inlet Control Performance Curves (Schematic)

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. Since the throat is only slightly lower than the face, it is likely that the face section will function as a weir or an orifice with downstream submergence within the design range. At lower flow rates and headwaters, the face will usually control the flow.

Slope-tapered Inlet

The slope-tapered inlet throat can be the primary control section with the face section submerged or unsubmerged. If the face is submerged, the face acts as an orifice with downstream submergence. If the face is unsubmerged, the face acts as a weir, with the flow plunging into the pool formed between the face and the throat. As previously noted, the bend section will not act as the control section if the dimensional criteria of this publication are followed. However, the bend will contribute to the inlet losses which are included in the inlet loss coefficient, K_E .

When a culvert with a tapered inlet performs in outlet control, the hydraulics are the same as described in Section 8.6 for all culverts. The tapered inlet entrance loss coefficient (K_E) is 0.2 for both side-tapered and slope-tapered inlets. This loss coefficient includes contraction and expansion losses at the face, increased friction losses between the face and the throat and the minor expansion and contraction losses at the throat.

8.12.5 Design Methods

Tapered inlet design begins with the selection of the culvert barrel size, shape and material. These calculations are performed using the Culvert Design Form provided in Appendix D. The design nomographs contained in HDS5 are used to design the tapered inlet. The design procedure is similar to designing a culvert with other control sections (face and throat). The result will be one or more culvert designs, with and without tapered inlets, all of which meet the site design criteria. The designer must select the best design for the site under consideration.

In the design of tapered inlets, the goal is to maintain control at the efficient throat section in the design range of headwater and discharge. This is because the throat section has the same geometry as the barrel, and the barrel is the most costly part of the culvert. The inlet face is then sized large enough to pass the design flow without acting as a control section in the design discharge range. Some slight oversizing of the face is beneficial because the cost of constructing the tapered inlet is usually minor compared with the cost of the barrel.

Performance Curves

Performance curves are of utmost importance in understanding the operation of a culvert with a tapered inlet. Each potential control section (face, throat and outlet) has a performance curve, based on the assumption that particular section controls the flow. Calculating and plotting the various performance curves results in a graph similar to Figure 8-15, containing the face control, throat control and outlet control curves. The overall culvert performance curve is represented by the hatched line. In the range of lower discharges face control governs; in the intermediate range, throat control governs; and in the higher discharge range, outlet control governs. The crest and bend performance curves are not calculated since they do not govern in the design range.

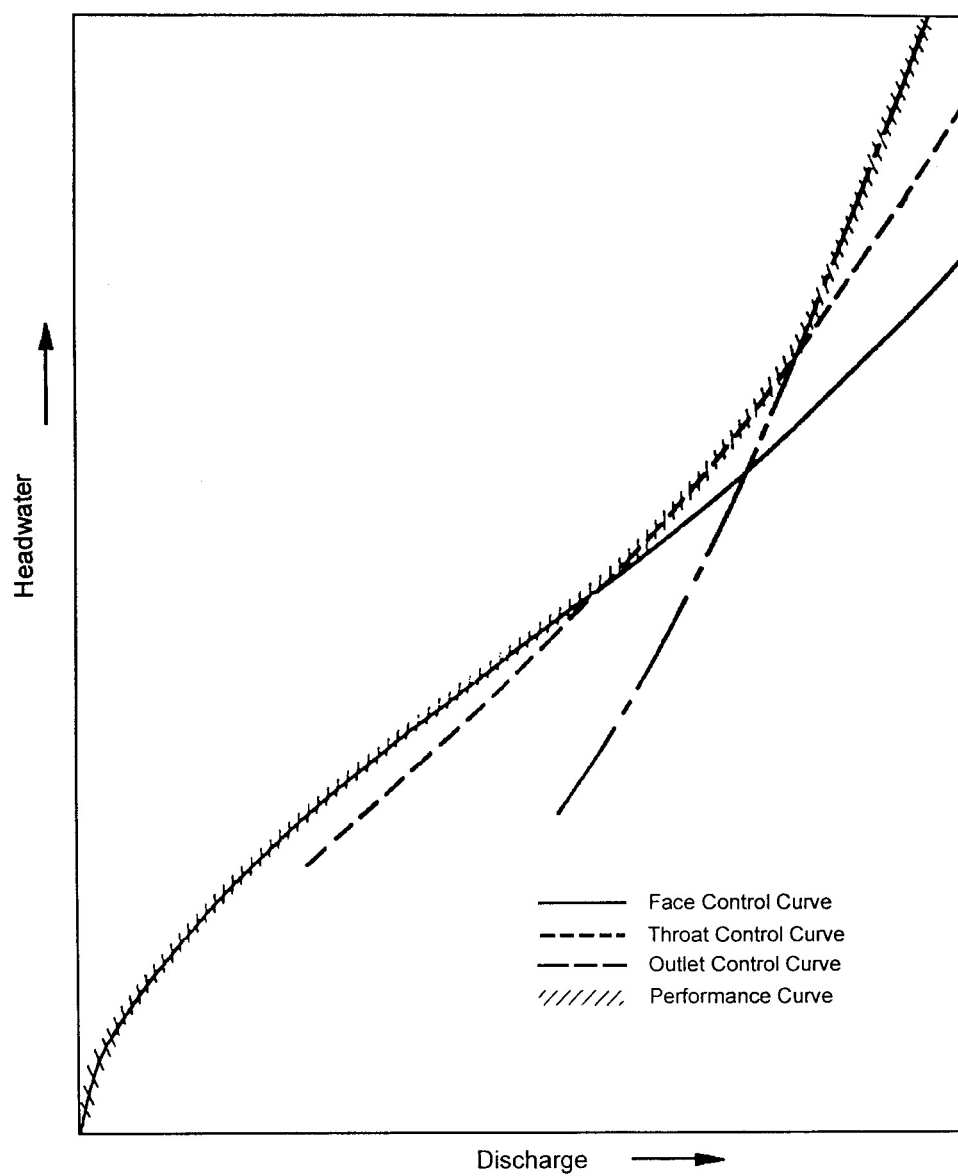


Figure 8-15 Culvert Performance Curve (Schematic)

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FHWA, Urban Drainage Design Manual, HEC-22, FHWA-SA-96-078, November 1996

Appendix A – 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	0.012-0.015
Corrugated Metal Pipes and Boxes, Annular or Helical Pipe (n varies barrel size) See HDS5	68 mm by 13 mm (2-2/3 by ½ inch) corrugations	0.022-0.027
	150 mm by 25 mm (6 by 1 inch) corrugations	0.022-0.025
	125 mm by 25 mm (5 by 1 inch) corrugations	0.025-0.026
	75 mm by 25 mm (3 by 1 inch) corrugations	0.027-0.028
	150 mm by 50 mm (6 by 2 inch) structural plate	0.033-0.035
	230 mm by 64 mm (9 by 2-1/2 inch) structural plate	0.033-0.037
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	68 mm by 13 mm (2-2/3 by ½ inch) corrugations	0.012-0.024
Spiral Rib Metal	Smooth walls	0.012-0.013
Plastic Pipe	Corrugated polyethylene, smooth	0.009-0.015
	Corrugated polyethylene, corrugated	0.018-0.025
	Polyvinyl chloride (PVC), smooth	0.009-0.011

* 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 affect 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.

Appendix B – Entrance Loss Coefficients (Outlet Control, Full or Partly Full)

$$H_e = k_e \left[\frac{v^2}{2g} \right]$$

Type of Structure and Design of Entrance	Coefficient k_e

Pipe, Concrete

Mitered to conform to fill slope	0.7
* End-Section conforming to fill slope.....	0.5
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Square-edge	0.5
Rounded (radius = 1/12D).....	0.2
Socket end of pipe (groove-end)	0.2
Projecting from fill, socket end (groove-end).....	0.2
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2

Pipe, or Pipe-Arch, Corrugated Metal

Projecting from fill (no headwall).....	0.9
Mitered to conform to fill slope, paved or unpaved slope.....	0.7
Headwall or headwall and wingwalls square-edge	0.5
* End-Section conforming to fill slope.....	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2

Box, Reinforced Concrete

Wingwalls parallel (extension of sides)	
Square-edged at crown.....	0.7
Wingwalls at 10° to 25° or 30° to 75° to barrel	
Square-edged at crown.....	0.5
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges.....	0.5
Rounded on 3 edges to radius of 1/12 barrel	
dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Crown edge rounded to radius of 1/12 barrel	
dimension, or beveled top edge.....	0.2
Side- or slope-tapered inlet	0.2

* Note: “End Section conforming to fill slope,” made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests, they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

Appendix C – Culvert Design Form

Metric Version

[illegible]

Culvert Design Form – (English Version)

[illegible]

Appendix D – Tapered Inlet Design Form

Metric Version

[illegible]

Tapered Inlet Design Form (English Version)

[illegible]

Metric Version

[illegible]

Mitered Inlet Design Form (English Version)

DESIGN DATA: N _____ ; B _____ ; D _____										MITERED INLET DESIGN FORM												
Q _____ = _____ cfs ; EL _{hi} _____ ft										DESIGNER / DATE : _____ / _____												
EL THROAT INVERT _____ ft EL STREAM BED AT CREST _____ ft										REVIEWER / DATE : _____ / _____												
FALL _____ ft ; TAPER _____ : (4:1 TO 6:1)										COMMENTS												
STREAM SLOPE, S ₀ _____ ft/ft ; BARREL SLOPE, S = _____ ft/ft																						
SLOPE OF THE EMBANKMENT S ₀ = _____ : 1 ; S _f _____ : 1 (2:1 TO 3:1)																						
BARREL SHAPE AND MATERIAL : _____																						
INLET EDGE DESCRIPTION : _____																						
Q (cfs)	EL _{hi}	EL _{throat} INVERT	y	EL _{face} INVERT	HW _f	HW _f /E	Q/B _f	MIN. B _f	SELECTED B _f	L ₃	MIN. L ₃	L ₄	L ₂	CHECK L ₂	ADJ. L ₃	ADJ. TAPER	L ₁	EL CREST INV.	HW _c	MIN. W	W	
			(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)				
(1) $\gamma = \frac{[S_0 - S_f] - 1}{(S_0 + S_f)(S_f^2 + 1)^{0.5}} \cdot D$ (2) EL. FACE INVERT = EL. STREAM BED AT CREST - γ (3) $HW_f = EL_{hi} - EL_{FACE\ INVERT}$ (4) $L_1, L_2 \geq 0$ (5) FROM DESIGN CHARTS (6) MIN. $B_f = Q/(G/B_f)$ (7) MIN. $L_3 = 0.5 NB$ (8) $L_4 = S_f \gamma + D/S_f$ (9) $L_2 = (EL_{CREST\ INVERT} - EL_{THROAT\ INVERT}) S_f - L_4$ *** IF L_2 IS NEGATIVE DO NOT USE THIS INLET (10) CHECK $L_2 = \left[\frac{B_f - NB}{2} \right] TAPER - L_3$ (11) IF (10)>(9), ADJ. $L_3 = \left[\frac{B_f - NB}{2} \right] TAPER - L_2$ (12) IF (9)>(10), ADJ. TAPER = $(L_2 + L_3) / \left[\frac{B_f - NB}{2} \right]$ (13) $L_1 = L_2 + L_3 + L_4$ (14) $HW_c = EL_{hi} - EL_{CREST\ INVERT}$ (15) MIN. $W = 0.35 Q / (HW_c)^{1.5}$ (16) $W = NB + 2 \left[\frac{L_1}{TAPER} \right]$ IF $W < MIN. W$, ADJUST TAPER																						
										SELECTED DESIGN B_f _____ L₁ _____ L₂ _____ L₃ _____ L₄ _____ BEVELS ANGLE _____° b = _____ in; d = _____ in TAPER _____ : 1 S_f _____												

Appendix F – Data Collection and Field Review Form**I. GENERAL PROJECT DATA**

Bridge No.: _____

Town: _____

Feature carried: _____

Quadrangle: _____

County: _____

Feature crossed: _____

DEP watershed basin no.: _____

Functional class:

- ☐ urban principal arterial-interstate
☐ urban principal arterial-other expwy.
☐ urban principal arterial-other
☐ urban minor arterial
☐ urban collector
☐ urban local

- ☐ rural principal arterial-interstate
☐ rural principal arterial-other expwy.
☐ rural principal arterial-other
☐ rural minor arterial
☐ rural major collector
☐ rural minor collector
☐ rural local

Year built: _____

Year of reconstruction: _____

Overall NBIS structure rating: _____

Sufficiency rating: _____

Plans available?: ☐ yes☐ no**II. HYDROLOGIC AND HYDRAULIC INFORMATION**Watershed area: _____ km² (sq. mi.)Is it tidally influenced? ☐ yes☐ no

What information is available?

- ☐ hydraulic report
☐ SCEL analysis
☐ Other: _____

- ☐ floodway analysis report
☐ FEMA F.I.S.

	Source	2 Yr. Event	10 Yr. Event	50 Yr. Event	100 Yr. Event	500 Yr. Event
Flow rates m ³ /s (cfs)						
Precipitation mm (in)						
Tidal elevations m (ft)						

Elevations m (ft.)						
At Structure		Water Surface at Approach Cross Section				
Streambed	Roadway	2 Yr. Event	10 Yr. Event	50 Yr. Event	100 Yr. Event	500 Yr. Event

Comments: _____

III. CULVERT DATA

Type

☐ concrete

☐ stone masonry

☐ corrugated metal

☐ steel

☐ aluminum

 structural plate
corrugations

☐ yes

☐ no

☐ 68mm x 13mm (2 2/3" x 1/2")

☐ 75mm x 25mm (3" x 1")

☐ 125mm x 25mm (5" x 1")

☐ 152mm x 51mm (6" x 2")

☐ 19 x 19 x 190mm (3/4" x 3/4" x 7 1/2")

☐ Other _____

Shape

☐ circular

☐ box

☐ arch

☐ elliptical

☐ other _____

Size [diameter or (width x height)]

_____ mm (inch)

_____ mm (in.) x _____ mm(in.)

_____ mm (in.) x _____ mm(in.)

_____ mm (in.) x _____ mm(in.)

End Treatment

☐ stone masonry

☐ other _____

☐ standard endwall

☐ projecting

☐ standard wing type endwall

☐ mitered ☐ slope paving

Inlet Edge

☐ square

☐ beveled

☐ socket end in headwall

Length _____ m (ft)

IV. APPROACH ROADWAY, EMBANKMENT AND CULVERT CONDITION

See ConnDOT Drainage Manual, Chapter 4, Culvert Repair, Materials, and Structural Design, Appendix A, Culvert Inspection Guideline

Approach Roadway and Embankment:Evidence of:

settlement	☐ yes	☐ no
patching or otherwise pavement built-up	☐ yes	☐ no
cracks running parallel to the culvert centerline	☐ yes	☐ no
erosion or failure of the embankment slope over the culvert	☐ yes	☐ no
sink holes over the culvert	☐ yes	☐ no

Comment on roadway alignment and sight distance at the culvert _____

Width of travelway _____ m (ft.) Width of shoulders _____ m (ft.)

Comment on objects in clear zone including culvert appurtenances _____

Safety features present:

☐ metal beam rail

☐ cable guide rail

☐ other _____

Embankment erosion protection:

☐ vegetation

☐ crushed stone

☐ modified riprap

☐ intermediate riprap

☐ standard riprap

☐ slope paving

☐ other _____

Note the overall adequacy of this protection and note any vegetation near the culvert where root systems may damage the culvert: _____

Culvert Barrel and End Treatments:

Check the culvert headwall, wingwalls, cutoff walls and footings (bottomless culverts) for any deficiencies or deterioration, undermining, scour, piping, tipping, or settlement. Note condition and/or deficiencies: _____

Check the culvert barrel for deformations, settlement, leaking or distressed joints and other deficiencies or signs of deterioration. Check for evidence that lateral earth pressure is causing bulging, flattening, peaking, sliding or rotation in the barrel. Note condition and/or deficiencies:

Note: Where practical, the floors of metal pipe culverts should be sounded with a metal rod in an attempt to locate voids due to undermining.

Dimensions should be taken at the inlet, outlet, mid-length and at 8m (26 ft.) intervals (maximum) as applicable, if access to the interior of the culvert is possible. Locations of sagging, bulging, flattening or peaking should also be measured.

inlet	_____ mm (in)	_____ mm (in)
mid-length	_____ mm (in)	_____ mm (in)
outlet	_____ mm (in)	_____ mm (in)

Any separation of the culvert barrel from the headwalls or cutoff walls. ☐ yes ☐ no

V. VISUAL SCOUR EVIDENCE

History of scour problem at outlet: ☐ yes ☐ no

Outlet protection

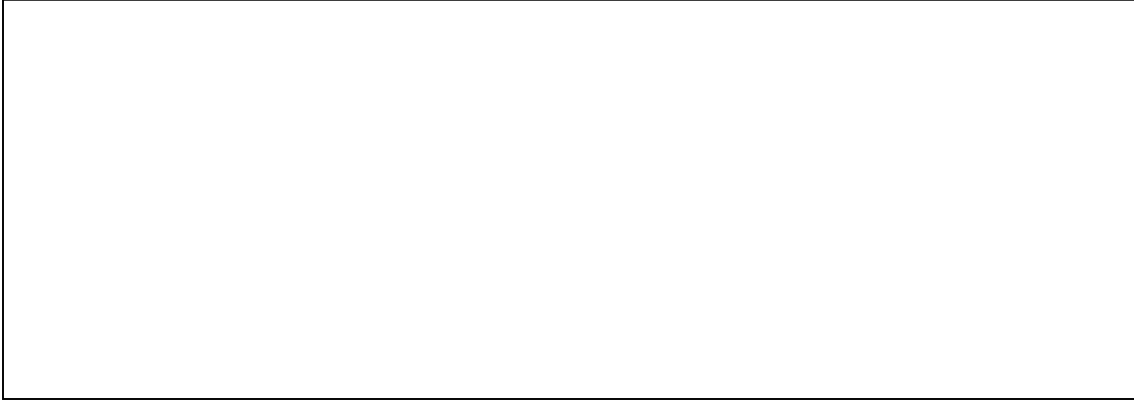
Type:	☐ modified	☐ intermediate	☐ standard	☐ slope paving
	☐ concrete	☐ other	☐ none	
Condition:	☐ good	☐ weathered	☐ slumped	☐ missing
	☐ fair	☐ poor	☐ N/A	

Comments: _____

* Note: For bottomless culverts, complete item VIII Visual Scour Evidence 9.A-9 to 9.A-11.

VI. SITE DATA

- A. Existing structure(s) – Provide sketch of culvert/structure with dimensions and brief description.



Comments: Include structure or culvert type and condition. Note particularly any scour adjacent to abutments or at culvert outlet and the presence of debris or sediment. Also note the location of any utilities in the area of the crossing.

- B. High water marks – Describe the nature and location of any apparent high water marks and relate to a date of occurrence, if possible.

- C. Maximum allowable headwater – Describe the nature of the apparent controlling feature and note its location.

- D. Fish passage requirements – Comment on the apparent need for fish passage or impediments to same; such as dams or restrictive crossings in the area.

VII. PERIPHERAL SITE DATA

- A. Hydraulic control - Note location and description.

- B. Upstream and downstream structures - Provide sketches and brief descriptions of existing bridges/culverts. Include dimensions.

Comments: _____

- C. Watershed area – Check watershed boundaries for accuracy. Note current land uses within watershed.

- D. Flow control structures within watershed – Note the location and type of all significant flow control structures (dams, etc.) within the watershed. Provide sketches with dimensions as required.



- E. Site photographs – Attach to report. Include an index and sketch of photograph locations.

VIII. STREAM CHANNEL AND RELATED ASPECTS

A. Stream characterization

Twenty Groupings of Stream Characteristics (check box)

	Identifier	Drainage Area	Streambed Slope	Streambed Soils	Land Use
☐	A	Large	Low	SD	S/F
☐	B	Large	Low	SD	Urban
☐	C	Large	Moderate	SD	Forested
☐	D	Medium	Moderate	SD	Urban
☐	E	Medium	Moderate	SD	S/F
☐	F	Medium	Moderate	CLAY	S/F
☐	G	Medium	Moderate	TILL	S/F
☐	H	Medium	Moderate	SD	Forested
☐	I	Medium	Moderate	TILL	Forested
☐	J	Small	Low	SD	Urban
☐	K	Small	Moderate	TILL	Urban
☐	L	Small	Low	SD	S/F
☐	M	Small	Moderate	SD	S/F
☐	N	Small	Moderate	SD	Forested
☐	O	Small	Low	CLAY	S/F
☐	P	Small	Steep	TILL	S/F
☐	Q	Small	Moderate	TILL	S/F
☐	R	Small	Low	TILL	S/F
☐	S	Small	Moderate	TILL	Forested
☐	T	Small	Steep	TILL	Forested

Drainage area Small $\leq 64.75\text{km}^2$ (25 mi²)
 Medium $> 64.75\text{km}^2$ (25 mi²) and $\leq 259\text{ km}^2$ (100 mi²)
 Large $> 259\text{ km}^2$ (100 mi²)

Streambed slope Low $\leq 4.76\text{ m/km}$ (25 ft/mi)
 Moderate $> 4.76\text{ m/km}$ (25 ft/mi) and $\leq 19.05\text{ m/km}$ (100 ft/mi)
 Steep $> 19.05\text{ m/km}$ (100 ft/mi)

Streambed soils SD = Stratified Drift

Land Use S/F = Suburban or Farming

B. Channel stability

Previous NBIS Item 61 rating: _____

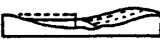
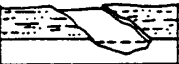
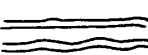
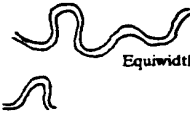
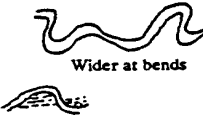
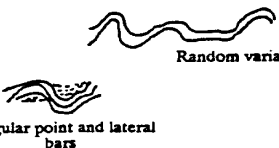
Lateral stability: ☐ stable ☐ unstable

Bank erosion:
☐ none ☐ light fluvial erosion ☐ heavy fluvial erosion ☐ mass wasting

Streambed: ☐ stable ☐ aggradating ☐ degrading

Armoring potential: ☐ none ☐ low ☐ moderate ☐ high

Geomorphic factors that affect stream stability (circle factors that apply)

STREAM SIZE	Small (< 30 m wide)		Medium (30-150 m)		Wide (> 150 m)
FLOW HABIT	Ephemeral (Intermittent)		Perennial but flashy		Perennial
BED MATERIAL	Silt-clay	Silt	Sand	Gravel	Cobble or boulder
VALLEY SETTING	 No valley; alluvial fan	 Low relief valley (< 30 m deep)		 Moderate relief (30-300 m)	 High relief (> 300 m)
FLOOD PLAINS	 Little or none (< 2X channel width)	 Narrow (2-10 channel width)		 Wide (> 10X channel width)	
NATURAL LEVEES	 Little or None	 Mainly on Concave		 Well Developed on Both Banks	
APPARENT INCISION	 Not Incised		 Probably Incised		
CHANNEL BOUNDARIES	 Alluvial		 Semi-alluvial		 Non-alluvial
TREE COVER ON BANKS	<50 percent of bankline		50-90 percent		> 90 percent
SINUOSITY	 Straight Sinuosity 1-1.05	 Sinuous (1.06-1.25)	 Meandering (1.25-2.0)	 Highly meandering (> 2)	
BRAIDED STREAMS	 Not braided (< 5 percent)	 Locally braided (5-35 percent)		 Generally braided (> 35 percent)	
ANABRANCHED STREAMS	 Not anabranching (< 5 percent)	 Locally anabranching (5-35 percent)		 Generally anabranching (> 35 percent)	
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS	 Narrow point bars	 Wide point bars		 Irregular point and lateral bars	

Source: Adapted From Brice and Blodgett, 1978

(See also FHWA HEC-20, "Stream Stability at Highway Structures" for discussion of the above factors)

Secondary bed material: ☐ sand ☐ gravel ☐ boulders ☐ manmade
☐ silt/clay ☐ cobble ☐ bedrock

Bank protection

Type ☐ none ☐ modified ☐ intermediate ☐ standard
☐ concrete ☐ slope paving ☐ absent
☐ other
Condition ☐ n/a ☐ good ☐ weathered ☐ slumped
☐ poor ☐ missing ☐ fair

Comment on the need (if any) for training walls, cutoff walls or special slope or channel protection.

C. Channel and overbank roughness coefficients

Basic channel description: ☐ channel in earth ☐ channel cut into rock
☐ channel fine gravel ☐ channel coarse gravel

Surface irregularity of channel:

- ☐ smooth – best obtainable section for materials involved
- ☐ minor – slightly eroded or scoured side slopes
- ☐ moderate – moderately sloughed or eroded side slopes.
- ☐ severe – badly sloughed banks of natural channels or badly eroded sides of man-made channels - jagged and irregular sides or bottom sections of channels in rock.

Variations in shape and size of cross sections

- ☐ changes in size or shape occurring gradually
- ☐ large and small sections alternating occasionally or shape changes causing occasional shifting of main flow from side to side.
- ☐ large and small sections alternating frequently or shape changes causing frequent shifting of main flow from side to side.

Channel obstructions – (Judge the relative effect of obstructions – consider the degree to which the obstructions reduce the average cross sectional area, character of obstructions, and location and spacing of obstructions).

NOTE: Smooth or rounded objects create less turbulence than sharp, angular objects.

The effect of obstructions is:

- ☐ negligible
- ☐ minor
- ☐ appreciable
- ☐ severe

Degree of vegetation – (Note amount and character of foliage).

The effect of vegetative growth upon flow conditions is:

☐ LOW - Dense growths of flexible turf grasses where average depth of flow is 2 to 3 times the height of vegetation. Sparse seedling tree switches where the average depth of flow is 3 to 4 times the height of the vegetation.

☐ MEDIUM - Turf grasses where the average depth of flow is 1 to 2 times the height of vegetation. Stemmy grasses, weeds or tree seedlings, (moderate cover), average depth of flow 2 to 3 times the height of vegetation. Bushy growths, (moderately dense), along channel side slopes with no significant vegetation along channel bottom.

☐ HIGH - Turf grasses where average height is about equal to the average depth of flow. Willow or Cottonwood trees 8 to 10 years old with some weeds or brush. Bushy growths about 1 year old with some weeds. No significant vegetation along channel bottom.

☐ VERY HIGH - Turf grasses where the average depth of flow is less than one half the height of vegetation. Bushy growths about 1 year old intergrown with weeds. Dense growth of cattails along channel bottom. Trees intergrown with weeds and brush (thick growth).

Additional comments:

IX. HYDRAULIC VULNERABILITY

Previous Item 71 rating: _____

Is there confluence present? ☐ yes ☐ no

Angle of attack (flood flow): ☐ yes ☐ no

Bends in channel: ☐ upstream of bridge ☐ downstream of bridge
☐ straight channel reach ☐ at bridge

Velocity order of magnitude: ____ m/s (ft/s)

Trapping potential: ☐ low ☐ medium ☐ high

Debris potential: ☐ low ☐ moderate ☐ high

Overtopping relief: ☐ none ☐ left approach ☐ right approach
☐ on bridge ☐ relief bridge ☐ cannot be determined

Primary bed material: ☐ sand ☐ gravel ☐ boulders ☐ manmade
☐ silt/clay ☐ cobble ☐ bedrock

Comments: _____

Appendix G – Culvert Design Data Form

Prepared by: _____
 Date: _____
 Checked: _____
 Date: _____

Project No. _____
 Town _____
 Route _____
 Location _____

1. DRAINAGE AREA

- a) Total area _____
- b) Special Considerations _____

- c) Existing culverts _____

2. DESIGN DISCHARGE _____ for _____ year frequency

- a) Rational Formula less than 81 ha (200 acres)
 T_c (Min) _____ Rainfall intensity mm/hr (in/hr) _____
 Coefficient of Imperviousness _____
- b) _____ HEC-1 SCS _____ TR20 _____ TR55 _____
 CN _____ T_c (Hr.) _____
 Rainfall distribution: _____ SCS Type III-24 Hr.
- c) Other _____

3. FISH PASSAGE REQUIRED? _____ Yes _____ No

- a) Special considerations _____

4. CULVERT HYDRAULIC DATA

- a) Size _____ Type _____
- b) Maximum permissible headwater elevation _____

- c) Proposed headwater elevation _____

- d) Elevation of channel bed at outlet _____ Inlet _____
- e) Length _____ Slope _____
- f) Inlet invert elevation _____ Outlet _____
- g) Improved inlet Yes _____ No _____
 _____ Beveled Edge _____ Side-Tapered _____ Slope-Tapered
 TAPER = ____:1 (4:1 TO 6:1) FALL = _____ S_f ____:1 (2:1 to 3:1)
- h) Entrance loss coefficient _____
- i) Type and location of hydraulic control _____

5. MISCELLANEOUS DATA

- a) Height of cover _____
- b) Culvert strength requirements: CMP _____ (wall or plate thickness)
RCP _____ (Class)
- c) End treatment _____

- d) Entrance channel _____
- e) Outlet channel _____
- f) Bank protection _____

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9.1 Introduction

9.1.1 Purpose of Chapter

The purpose of this chapter is to provide guidance in the hydraulic design of a stream crossing system through appropriate design practice and design criteria and technical aspects of hydraulic design. It also presents non-hydraulic factors that influence design including environmental concerns, emergency access, traffic service and consequence of catastrophic loss.

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, mitigation of impacts on stream environment and accomplishment of social, economic and environmental goals.

Bridges are defined as structures that transport traffic and/or pedestrians over waterways (streams, rivers, floodplains, etc.) or other obstructions. The bridge system includes the approach roadway and the bridge structure.

9.1.2 Structural Alternatives

A myriad of structure alternatives are available for use in a highway-stream crossing system when all of the possible combinations of bridge lengths, spans, pier types and orientation, geometries, parapet designs, and superstructure designs are considered. In addition, at many crossings, multiple bridges or a single bridge may be viable alternatives or large culverts may be used in lieu of one or more bridges.

The hydraulics of the highway-stream crossing system should be given considerable study in choosing the preferred design from the long list of available alternatives. **The selection should be based on a collective effort of bridge, soils and foundations, highway, and hydraulic engineers. This process is further outlined in Chapter 3, Design Development.**

9.1.3 Bridge or Culvert

Occasionally, the waterway opening(s) for a highway-stream crossing can be provided for by either culvert(s) or bridge(s). Estimates of costs and risks associated with each will indicate which structure alternative should be selected on the basis of economics. Other considerations which may influence structure-type selection are listed in Table 9-1 and discussed in subsequent sections and Chapter 8.

Table 9-1

BRIDGE OR CULVERT

<u>Bridges</u>	
Advantages	Disadvantages
Less susceptible to clogging with drift, ice and debris Waterway generally increases with rising water surface until water begins to submerge superstructure Flowline is flexible Minimal impact on aquatic environment and wetlands Widening does not usually affect hydraulic capacity	Require more structural maintenance than culverts Fill slopes susceptible to erosion and scour damage Piers and abutments must be designed not to fail due to scour Susceptible to ice and frost formation on deck Bridge railing and parapets hazardous as compared to recovery areas when it is possible to extend the culvert beyond the safe recovery area Deck drainage may require frequent maintenance cleanout Buoyant, drag and impact forces are hazards to bridges Susceptible to damage from stream meander migration
<u>Culverts</u>	
Advantages	Disadvantages
Provides an uninterrupted view of the road Roadside recovery area can be provided Grade raises and widening projects sometimes can be accommodated by extending culvert ends Require less structural maintenance than bridges Frost and ice usually do not form before other areas experience the same problems Capacity increases with stage Capacity can sometimes be increased by installing improved inlets Usually easier and quicker to build than bridges Scour is localized, more predictable and easier to control Can be used to arrest headcutting Storage can be utilized to reduce peak discharge	Silting in multiple barrel culverts may require periodic cleanout No increase in waterway as stage rises above soffit as the bottom is fixed May clog with drift, debris or ice Possible barrier to fish passage Susceptible to erosion of fill slopes and scour at outlets Susceptible to abrasion and corrosion damage Extension may reduce hydraulic capacity Inlets of flexible culverts susceptible to failure by buoyancy Rigid culverts susceptible to separation at joints Susceptible to failure by piping and/or infiltration

9.2 Design Practices

9.2.1 ConnDOT Design Practices

The following design practices are specific to bridges:

- The final design selection should consider the maximum backwater allowed by the National Flood Insurance Program and Stream Channel Encroachment Lines unless exceedence of the limit can be justified by special hydraulic conditions.
- The final design should not significantly alter the flow distribution in the floodplain.
- The "crest-vertical curve profile" should be considered as the preferred highway crossing profile when allowing for embankment overtopping at a lower discharge.
- A specified clearance should be established to allow for passage of ice and debris. For navigation channels, a vertical clearance conforming to Federal requirements should be established based on normally expected flows during the navigation season.
- Degradation or aggradation of the river shall be estimated and contraction and local scour shall be computed. Appropriate positioning of the foundation, below the total scour depth if practicable, shall be included as part of the final design.

9.2.2 ConnDOT Practice-Substructures for Bridges over Waterways

Substructures for bridges over waterways shall be designed to safely support the structure subjected to the design scour. This policy is based upon the design guidelines contained within the FHWA document entitled "Evaluating Scour at Bridges" (HEC-18), wherein it states that "Bridges should be designed to withstand the effects of scour from a superflood with little risk of failing."

With regard to abutment or pier foundations, two basic approaches to achieving this goal are available to the designer, listed as follows in order of preference:

- 1) Design the foundation to resist the effects of scour from a superflood.
Foundations subjected to scour shall be designed with footings supported on piles, footings founded on rock or deep footings (located below the maximum estimated scour). Structural tremies (concrete poured under water which directly supports the foundation loads) will be allowed in very limited situations, only where no other solution is feasible, and only with the approval of the Department. Preference for foundations adjacent to or within waterways will be for pile supported footings or direct foundations on rock. For pile foundations, the top of pile cap shall be set below the sum of the long term degradation and contraction scour.
- 2) Protect the substructure units with riprap or similar armoring layers.
In general, the use of riprap to provide scour protection for **new** bridges is discouraged and should be used only where it has been demonstrated that alternate, preferred means of designing bridges to be safe from scour related failures are not feasible. On bridge rehabilitation projects where the substructure is being repaired and incorporated in the reconstruction of the bridge, riprap scour countermeasures may be an effective solution for protecting the bridge from scour.

The designer should explore and incorporate into the design all reasonable methods of minimizing local scour, such as the use of embankment or "stub" abutments placed at the top of a protected slope. These types of abutments are much less susceptible to scour than full height

abutments. The use of stub abutments does not relieve the requirement for founding on piles or directly on rock. Piers which may experience local scour should be flow aligned and should have streamlined end sections.

9.2.3 ConnDEP Requirements

The following definitions were taken from Section 25-68h-1 of the Flood Management Statutes and Administrative Regulations.

Sec. 25-68h-1. Connecticut floodplain management regulations for state agencies.

(a) Definitions.

(1) As used in Sections 25-68h-1, 25-68h-2 and 25-68h-3:

"Hurricane wave wash" means the effect of wave action in a coastal flood hazard zone.

"Significant impact" means any activity that would create:

(A) A five percent increase in peak flow rates at any downstream point;

(B) A twenty percent increase in flow velocities or a change that allows a stable condition to become unstable;

(C) An activity that contributes to an unacceptable cumulative impact;

(D) Any activity that causes flooding on developed property not currently subject to flooding;

(E) An activity that could cause a downstream dam to become unsafe.

"Velocity waters" means the effect of moving water in a coastal flood hazard zone.

(2) As used in Sections 25-68h-1, 25-68h-2 and 25-68h-3, the definitions of the following terms shall be the same as the definitions in Section 25-68b of the General Statutes: activity; base flood; base flood for a critical activity; Commissioner; critical activity, floodplain; flood-proofing; freeboard.

The following requirements for culverts and bridges taken from Section 25-68h-2 and Section 25-68h-3e(1) through (8) of the Flood Management Statutes and Administrative Regulations shall be incorporated into all ConnDOT projects.

- All state activities shall conform to the Federal Emergency Management Agency National Flood Insurance Program requirements, specifically Part 60 – Criteria For Land Management and Use, Subpart A Sections 60.3, 60.4 and 60.5. (See Chapter 2, Section 2.3.)
- The following restrictions shall pertain to all new and substantially improved structures located within the floodplain.
 1. Structures shall not be designed for human habitation unless elevated with the lowest floor one foot above the level of the base flood.
 2. Structures and all stored materials which may result in damage to other structures, restriction of bridge openings or other narrow sections of the stream or river shall be anchored or restrained to prevent them from floating away.
 3. Service facilities such as electrical and heating equipment shall be constructed at or above the elevation of the base flood or flood-proofed with a passive system.
 4. Structures located within a "coastal high hazard area" as defined in 44 CFR Part 59 shall be elevated on adequately anchored pilings or columns and securely anchored to such piles or columns such that the lowest portion of the structural members of the lowest flood (excluding the pilings or columns) is elevated to one foot above the base flood and certified

- by a registered professional engineer or architect that the structure is securely anchored to piling or columns in order to withstand velocity waters and hurricane wave wash.
5. No new structures shall be permitted on undeveloped coastal barrier beaches as designated by the Federal Emergency Management Agency (FEMA).
 6. All water supply equipment shall be designed to prevent flood waters from entering and contaminating the system.
 7. All sanitary sewer collection systems located in the floodplain must have watertight manhole covers and if equipped with vents, shall extend above the elevation of the base flood.
- The following restrictions shall pertain to all filling, dumping, construction, excavating, and other activities which change the topography within the floodplain.
 1. No filling, dumping or construction or other activity shall be allowed which would increase the elevation of the base flood by more than one foot or adversely affect the hydraulic characteristics of the floodplain unless the proposed filling is fully compensated for by excavation in or contiguous to the filled area.
 2. No filling, dumping, construction or excavation will be allowed if these changes will result in a concentration of the natural flow of water such as to cause or increase drainage, erosion or sediment problems.
 3. Any fill placed in the floodplain shall not be greater than that which is necessary to achieve the intended purpose as demonstrated by a plan showing the uses to which the filled land will be put and the final dimensions of the proposed fill or other materials.
 4. Such fill or other material shall be protected against erosion as discussed in the Connecticut Guidelines for Soil Erosion and Sediment Control (1985), as may be amended.
 5. Any activity within a floodway designated by FEMA which would result in an increase of the elevation of the base flood or ten year flood profile is prohibited.
 6. The placement of fill in areas of high velocity flow or at the outside edge of a migrating river bend is discouraged.
 - The following restrictions shall pertain to the storage of materials and equipment within the floodplain.
 1. The storage of materials that are buoyant, hazardous, flammable, explosive, soluble, expansive radioactive or which could be injurious to human, animal or plant life is prohibited below the elevation of the base flood for a critical activity.
 2. Other material or equipment may be stored below the elevation of the base flood for a critical activity provided that such material or equipment is not subject to major damage by floods, and provided that such material or equipment is firmly anchored, restrained or enclosed to prevent it from floating away.
 - Culverts and bridges will be designed for flood frequencies and underclearances stipulated in this manual, except that on local (not state highways) roads and driveways with low traffic volumes and where alternate routes are available, lower design criteria is acceptable when:
 - (A) Flood discharges may be allowed to cross over roads that are at or close to the floodplain grade.
 - (B) Water surface elevations shall not be increased by more than one foot, nor allowed to cause damage to upstream properties.
 - (C) Provisions are made to barricade the road when overtopped.
 - (D) The road or driveway is posted as being subject to flooding.
 - Bridges and culverts along stocked watercourses and watercourses which may support fish shall be designed to allow passage of fish as may be recommended by the Department of Environmental Protection Fisheries and Wildlife Units.

- The location of new bridges and culverts shall minimize the relocation of watercourses.
- Where applicable, rigid structural floors at bridges and culverts should be depressed below the normal streambed, to allow an alluvial streambed to form over them, and shall anticipate if the streambed is degrading.
- The use of solid parapet walls at bridges and culverts located in the sag part of vertical curves is discouraged. *
- Debris barriers shall be used upstream of structures prone to blockage by debris.
- The use of a single large culvert or bridge opening is preferred over use of multiple small openings.
- The underclearances and maximum headwaters stipulated in this manual may be waived when decreasing the headwater depth at existing structures could increase downstream peak flows or when it can be demonstrated that satisfying these criteria would be impractical due to site conditions and other factors.

* This applies to locations where there is roadway overtopping.

- **Stream Channel Encroachment Lines (SCEL) [*Repealed October 1, 2013*]**

1. Established - On any stream where channel encroachment lines have been established by the Department of Environmental Protection, authorization shall be obtained from the ConnDEP for the proposed structure in accordance with Section 22a-342 through 22a-349 and 25-68b through 25-68h, Connecticut General Statutes. The encroachment line study discharge shall be used for analysis; however, the design discharge may vary from this quantity.
2. Not established - On streams for which channel encroachment lines have not been established Section 25-68b through 25-68h and Section 13a-94 of the Connecticut General Statutes apply. The structures shall be designed using the appropriate storm frequency as stated previously and as indicated in Table 9-2.

9.2.4 Flood Management Definitions

Sec.25-68b. Definitions. As used in sections 25-68b to 25-68h, inclusive:

- (1) "Activity" means any proposed state action in a floodplain or that impacts natural or man-made storm drainage facilities, including, but not limited to, the following: (a) Any structure, obstruction or encroachment proposed for emplacement within the floodplain area; (b) any proposal for site development which increases peak runoff rates; (c) any grant or loan which affects land use, land use planning or the disposal of state properties in floodplains, or (d) any program regulating flood flows within the floodplain;
- (2) "Base flood" means that flood which has a one per cent chance of being equaled or exceeded in any year, as defined in regulations of the National Flood Insurance Program (44 CFR 59 et seq.) or that flood designated by the commissioner pursuant to section 25-68c. Any flood so designated by the commissioner shall have at least a one per cent chance of being equaled or exceeded in any year. Such flood may be designated as the A or V zones on maps published by the National Flood Insurance Program. The "base flood for a critical activity" means the flood that has at least a .2 per cent chance of being equaled or exceeded in any year. Such flood may be designated as the B zone on maps published for the National Flood Insurance Program;

- (3) "Commissioner" means the commissioner of environmental protection;
- (4) "Critical activity" means any activity, including, but not limited to, the treatment, storage and disposal of hazardous waste and the siting of hospitals, housing for the elderly, schools, or residences, in the .2 per cent floodplain in which the commissioner determines that a slight chance of flooding is too great;
- (5) "Floodplain" means that area located within the real or theoretical limits of the base flood or base flood for a critical activity;
- (6) "Flood-proofing" means any combination of structural or nonstructural additions, changes or adjustments which reduce or eliminate flood damage to real estate or improved real property, to water and sanitary facilities, and to structures and their contents;
- (7) "Freeboard" means a safety factor, expressed in feet above a calculated flood level, that compensates for unknown factors contributing to flood heights greater than the calculated height, including, but not limited to, ice jams, debris accumulations, wave actions, obstructions or bridge openings and floodways, the effects of urbanization on the hydrology of a watershed, loss of flood storage due to the development and sedimentation of a watercourse bed.

9.3 Design Criteria

9.3.1 AASHTO General Criteria

Following are the AASHTO general criteria related to the hydraulic analyses for the location and design of bridges as stated in the 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.
- Underclearance at structure(s) designed to pass anticipated debris and ice.
- Acceptable risks of damage or viable measures to counter the vagaries of alluvial streams.
- Minimal disruption of ecosystems and values unique to the floodplain and stream.
- Provide a level of traffic service compatible with that commonly expected for the class of highway and compatible with projected traffic volumes.

9.3.2 ConnDOT General 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 the water surface profile programs such as HEC-RAS, HEC-2, or WSPRO.

- **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 the Hydrology chapter.

- **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, the environment and the infrastructure of the highway.

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 FHWA HEC-17 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. (See Section 9.6.7)

- **Design Floods** - Design floods for the purpose of the evaluation of backwater, clearance and overtopping should be established predicated on risk based assessment of local site conditions. They shall reflect consideration of traffic service, environmental impact, property damage, hazard to human life and floodplain management criteria. ConnDOT has defined various structure types based on watershed area as defined below. (See Table 9-2):

- **Backwater/Increases Over Natural Conditions** - Conform to FEMA regulations for all sites and stream channel encroachment line requirements for sites within established encroachment lines. In general, for sites not specifically identified in the FEMA or SCEL programs, backwater over natural conditions should not exceed 0.3m (1 ft.) during the passage of the 1% exceedence probability flood. (See Table 9-2)
- **Underclearance** - Where practicable a minimum clearance of 0.6 m (2 ft.) should be provided between the design approach water surface elevation and the low chord of the bridge for the final design alternative to allow for passage of ice and debris. Where this is not practicable, the clearance should be established by the designer based on the type of stream and level of protection desired as approved by the Hydraulics and Drainage section. (See Table 9-2).
- **Flow Distribution** - The conveyance of the proposed stream-crossing location shall be calculated to determine the flow distribution and to establish the location of bridge opening(s). The proposed facility shall not cause any significant change in the existing flow distribution. Relief openings in the approach roadway embankment or other appropriate measures should be investigated if there is more than a 10% redistribution of flow.
- **Scour** - Design for bridge foundation scour considering the magnitude of flood, including the 1% event, that generates the maximum scour depth. The foundation shall be evaluated by geotechnical and structural engineers for both a design event (100-yr. storm) and an extreme event (500 year) to insure that the appropriate stability criteria are met. (See Section 9.5.)

9.3.3 Minor Structures

These shall include culverts or bridges providing waterway for the drainage of adjacent land, in which there is no established watercourse, having a drainage area less than 2.59 km² (1 mile²) (See Table 9-2).

Minor structures shall be designed to pass the 25-year frequency discharge determined by the appropriate method found in the Hydrology chapter. The water surface upstream at design discharge shall not be elevated so as to damage adjacent property or endanger the highway.

9.3.4 Small Structures

These shall include culverts or bridges providing waterways for the drainage of areas of less than 2.59 km² (1 mile²) in which there is an established watercourse. (See Table 9-2).

Small structures shall be designed to pass a 50-year frequency discharge as outlined in the Hydrology chapter. The effects of a discharge equal to the 100-year flood passing through the proposed construction shall be investigated.

At locations where the stream has been studied in detail (FIS Study includes water surface profiles in the area of the design proposal), a 100-year return frequency shall be used for the design discharge.

Where a likelihood of danger to persons, extensive property damage or other than temporary interruption of traffic will exist under these conditions, increases in waterway or other improvements shall be provided to alleviate the danger.

The water surface upstream shall not be elevated so as to endanger the roadway or damage private property. At the upstream highway property boundary, the elevation shall generally be not more than 0.3m (1 ft.) over that which is achieved under natural flow conditions.

Where successive culverts are utilized on a project and the flow in upper culverts is affected by headwaters in the lower culverts, a water surface profile and appropriate computations shall be submitted for review.

9.3.5 Intermediate Structures

These shall include culverts or bridges providing waterway for the drainage of areas larger than 2.59 km^2 (1 mile²) and less than 25.9 km^2 (10 mile²) (See Table 9-2.).

Intermediate structures shall be designed to pass a discharge equal to the 100-year flood with low chord underclearance not less than 0.3m (1 ft) (not required for culverts) and a backwater usually not to exceed 0.3m (1 ft) above that which would have been obtained in the natural channel if the highway embankment were not constructed except when other limitations may be required by the General Statutes and the resulting water surface elevation which will not endanger the roadway nor cause damage to developed property upstream. The effects of a discharge equal to the 500-year flood passing through the proposed construction shall be investigated. Where a likelihood of danger to persons, extensive property damage or other than temporary interruption of traffic will exist under these conditions, increases in waterway or other improvements shall be provided to alleviate the danger, whenever possible.

Rating curves for intermediate structures are required in order to better enable reviewers to determine the probabilities of danger to persons or property for floods of various return frequencies. These rating curves shall be extended far enough to determine the effects of a recurrence equivalent to the flood of record, if such flood is greater than the design discharge. They shall in all cases show the effects of 2, 10, 50-year, 100-year and 500-year frequency storms.

9.3.6 Large Structures

These shall include culverts or bridges providing waterway for the drainage of areas larger than 25.9 km^2 (10 mile²) and less than 2590 km^2 (1000 mile²) (See Table 9-2.).

Large structures shall normally be designed to pass a discharge equal to the 100-year flood with a low chord underclearance generally not less than 0.6m (2 ft.) (not required for culverts) and an increase in water surface elevation at the upstream highway property line of not more than 0.3m (1 ft.) above that which would have been obtained in the natural channel if the highway embankment were not constructed except where other limitations may be required by the General Statutes. This elevation may be decreased as needed to avoid damage to developed property upstream. The effects of a discharge equal to the 500-year flood passing through the proposed construction shall be investigated. Where a likelihood of danger to persons, extensive property damage or other than temporary interruption of traffic will exist under these conditions, increases in waterway or other improvements shall be provided to alleviate the danger, whenever possible.

Rating curves for large structures are required in order to better enable reviewers to determine the probabilities of danger to persons or property for floods of various return frequencies. These rating curves shall be extended far enough to determine the effects of a recurrence equivalent to the flood of record, if such flood is greater than the design discharge. They shall in all cases show the effects of 2, 10, 50-year, 100-year and 500-year frequency storms.

9.3.7 Monumental Structures

Bridges providing waterway for streams draining areas greater than 2590 km² (1000 mile²). (See Table 9-2).

Monumental structures shall be designed to meet requirements established by the Inland Water Resources Division (IWRD), of the Department of Environmental Protection, the U.S. Army Corps of Engineers and the U.S. Coast Guard, communicating with these agencies for the particular location, taking into consideration channel encroachment lines, navigation requirements and past flooding history. As a minimum, the design shall conform to the requirements of large structures.

The hydraulic characteristics of monumental structures shall be established by special studies initiated during the planning or early design stages by or in cooperation with the Hydraulics and Drainage Section.

9.3.8 Tidal Structures

These shall include all structures of whatever type providing waterway on streams subject to tidal action. They shall be classified as minor, small, intermediate, etc. depending on their riverine drainage area.

Tidal structures shall normally be designed to pass the appropriate design discharge for the foregoing category into which they would normally be classified with the same limitations as apply to the appropriate class.

Unlike inland rivers where the design discharge is fixed by runoff and is virtually unaffected by the waterway provided, the size of the waterway opening of a tidal structure can modify the tidal regime and the associated tidal discharges.

Long-term aggradation and degradation at a tidal inlet or estuary are influenced primarily by the periodic tidal fluctuations associated with astronomical tides. Therefore, flow hydraulics at a bridge should be determined considering the tidal range for evaluation of long-term aggradation or degradation.

Extreme events associated with riverine floods and tidal storm surges should be used to determine the hydraulic adequacy of a bridge. These events would have a return period based on the structure classifications as previously defined. Difficulty arises in determining whether the storm surge, flood or the combination of the storm surge and flood should be considered controlling.

When inland flood discharges are small in relationship to the magnitude of the storm surge and are the result of the same event, then the flood discharge can be added to the discharge associated with the design tidal flow, or the volume of the runoff hydrograph can be added to the volume of the tidal prism. If the inland flood and the storm surge may result from different storm events, then a joint probability approach may be warranted to determine the magnitude of the design discharge.

In some cases there may be a lag time between the storm surge discharge and the stream flow discharge at the highway crossing. For this case, streamflow-routing methods such as the NRCS TR-20, USACOE HEC-1 or HEC-HMS model can be used to estimate the timing of the flood hydrograph derived from runoff of the watershed(s) draining into the sound or estuary.

The selection of the method to use to combine flood and tidal surge flows is a matter of judgement and must consider the characteristics of the site and the storm events.

For further information regarding tidal hydrology and hydraulics, the designer is directed to the manual entitled "TIDAL HYDRAULIC MODELING FOR BRIDGES", Ayres Associates, March 2002. This manual was developed under a "Pooled Fund Study" entitled "Development of Hydraulic Computer Models to Analyze Tidal and Coastal Stream Hydraulic Conditions at

Highway Structures". The Department participated in the study with the FHWA and several other coastal states with the purpose of developing improved methods to determine hydraulic conditions at bridges in tidal waterways. Chapter 3 of the referenced manual contains the storm surge hydrology results of the study. This chapter presents equations for estimating astronomical tides and storm surge hydrographs. Guidance is also presented on the combining of inland runoff (riverine discharge) with storm surges. A copy of this manual may be obtained from the Hydraulics and Drainage Section for Department funded projects. This manual should be consulted prior to developing the scope of work for the hydraulic analysis of tidal structures. See also Section 9.4.5 regarding the hydraulic analysis and modeling of coastal or tidally influenced waterways.

Table 9-2

SUMMARY OF HYDRAULIC CRITERIA⁵

CONNDOT STRUCTURE CLASS ³	DRAINAGE AREA sq. km (sq. mile)	DESIGN FREQUENCY (year)	CHECK FREQUENCY (year)	MINIMUM FREEBOARD ² FOR DESIGN FREQUENCY m (ft)	MINIMUM BRIDGE UNDER- CLEARANCE FOR DESIGN FREQUENCY m (ft)
Minor	<2.59 (1) (no established watercourse)	25	-	0.3 (1)	-
Small	<2.59 (1)	50	100	0.3 (1)	-
Intermediate ¹	≥2.59 (1) <25.9 (10)	100	500	0.3 (1)	0.3 (1)
Large ¹	≥25.9 (10) <2590 (1000)	100	500	0.3 (1)	0.6 (2)
Monumental	≥2590 (1000)	100 ⁴	500 ⁴	0.3(1) ⁴	0.6 (2) ⁴

¹ The designer shall also consider culvert alternative (see 9.1.2 and Chapter 8) for this class when area > 2.59 km (1 mi²)

² Freeboard is defined as the vertical distance between the design water surface and the upstream control such as the low point of the roadway edge, sill of a building or other controlling element.

³ See Section 9.3.3 through 9.3.8 for detailed discussions on structure classification.

⁴ Minimum requirements subject to prior approval from Hydraulics and Drainage Section.

⁵ See Section 9.3.9, Design Exceptions

9.3.9 Design Exceptions

The design criteria stipulated in this manual, including underclearance, freeboard and backwater, may be waived when it can be demonstrated that satisfying these criteria would be impractical or inappropriate due to site conditions and other factors. These factors may include but not limited to, economics, environmental impacts, constructability, rights of way and potential impacts to adjacent property.

With the exception of the specific CTDOT hydraulic design criteria being waived, projects must be demonstrated to be in compliance with the Flood Management Statutes and Regulations for State Agencies and the requirements of the National Flood Insurance Program (NFIP) for Flood Management Certification (FMC).

The reason(s) for not satisfying the design criteria shall be documented in the Hydraulic Design Report and the Floodplain/Floodway Analysis Report (if prepared separately for Flood Management Certification), along with the hydraulic analyses and other information demonstrating compliance with the Flood Management regulations and NFIP requirements..

The criteria shall be waived and the requirements considered satisfied following review and approval of the aforementioned report(s) by the Hydraulics and Drainage (H&D) section.

For projects requiring FMC approval from the DEEP, the FMC application shall advise the DEEP of the specific design criteria waived and H&D's approval of the hydraulic report(s) in conformance to this Section (9.3.9) of the Drainage Manual.

It is important that exceptions to the design criteria and the reasons be identified early in the design process so that they may be properly vetted, reviewed and obtain concurrence from H&D. Design exceptions may not be approved by H&D if it appears that reasonable design changes could have been incorporated into the project to achieve compliance to the hydraulic design criteria.

9.3.10 Documentation

The following items shall be included in the documentation file (See Chapter 1, Section 1.6). The intent is not to limit data to only those items listed, but rather establish a minimum requirement consistent with the bridge design procedures as outlined in this chapter. If circumstances are such that the hydraulic design is prepared by other than the normal procedures or is governed by factors other than hydrologic or hydraulic, a narrative summary detailing the design basis shall appear with the other data.

The following items shall be included in the documentation file:

- design discharge values and discharge rating curves
- culvert performance curves
- allowable headwater elevation and basis for its selection
- type of culvert entrance conditions
- outlet velocities and energy dissipation calculations and designs
- cross section(s) used in the design water surface determinations
- roughness coefficient assignments ("n" values)
- observed highwater marks with associated dates and discharges

- natural, existing and proposed conditions water surface profiles for intermediate structures or larger
- velocity measurements and locations or velocity estimates. Include both the through-bridge and channel
- copies of all computer analyses with 3.5" computer disk and standard computation sheets given in this chapter
- magnitude and frequency of overtopping flood if less than the design
- tailwater depth or starting water surface elevation
- roadway geometry plan and profile
- potential flood hazard to adjacent properties
- completed forms in Appendix A
- scour evaluation/analysis for bottomless structures for design 100 yr. flood and 500 yr. flood
- economic analysis of design and alternatives
- risk assessment, if required
- bridge scour results (plan and profile)
- reasons for not satisfying specific design criteria, if applicable

9.4 Design Procedure

9.4.1 Survey Accuracy (Computation Method)

The design for a stream crossing system requires a comprehensive engineering approach that includes formulation of alternatives, data collection, selection of the most cost effective alternative according to established criteria and documentation of the final design.

Water surface profiles are computed for a variety of technical uses including:

- flood insurance studies
- flood hazard mitigation investigations
- drainage crossing analysis
- longitudinal encroachments

The completed profile can affect the highway bridge design and is the mechanism for determining the effect of a bridge opening on upstream water levels. The step backwater profile method is commonly used for developing water surface profile. Errors associated with computing water surface profiles with the step backwater profile method can be classified as:

- data estimation errors resulting from incomplete or inaccurate data collection and inaccurate data estimation
- errors in accuracy of energy loss calculations depending on the validity of the energy loss equation employed and the accuracy of the energy loss coefficients
- inadequate length of stream reach investigated
- computational errors due to poor cross sectional spacings. These errors can be significant due to inaccurate integration of the energy loss-distance relationship that is the basis for profile computations. These errors may be reduced by adding interpolated or actual sections to the analysis.

9.4.2 Design Procedure Outline

Although the scope of the project and individual site characteristics make each design an unique one, this procedure shall be applied unless indicated otherwise by the Hydraulics and Drainage section.

I. Data Collection

A. Survey

- Topography/channel cross sections
- Geology
- Highwater marks
- History of debris accumulation, ice and scour
- Review of hydraulic performance of existing structures
- Maps, aerial photographs
- Rainfall and stream gage records
- Field reconnaissance

B. Studies by other agencies

- FEMA Flood Insurance Studies
- Federal Floodplain Studies by the ACOE, NRCS, etc.

- Local Floodplain Studies
 - Hydraulic performance of existing bridges
 - ConnDEP Inland Water Resources Division
 - Stream Channel Encroachment Line Reports
 - C. Influences on hydraulic performance of site
 - Other streams, reservoirs, water intakes
 - Structures upstream or downstream
 - Natural features of stream and floodplain
 - Channel modifications upstream or downstream
 - Floodplain encroachments
 - Sediment types and bed forms (Scour, Site Data, Level I Qualitative Analysis — FHWA HEC-20)
 - D. Environmental impact
 - Existing bed or bank instability
 - Floodplain land use and flow distribution
 - Environmentally sensitive areas (fisheries, wetlands, etc.)
 - Level I Qualitative Analysis (FHWA HEC-20)
 - E. Site-specific Design Criteria
 - Preliminary risk assessment
 - Application of ConnDOT criteria
- II. Hydrologic Analysis (See Chapter 3, Section 3.5, Design Development and Chapter 6, Hydrology.)
- A. Watershed morphology
 - Drainage area (attached map)
 - Watershed and stream slope
 - Channel geometry
 - B. Hydrologic computations
 - Discharge and frequency for historical flood that complements the high water marks used for calibration
 - Discharges for specified frequencies
- III. Hydraulic Analysis (See Chapter 3, Section 3.5, Design Development and Section 9.4.3.)
- A. Computer model calibration and verification
 - B. Hydraulic performance for existing conditions – including floodway analysis, if appropriate
 - C. Hydraulic performance of proposed designs – including floodway analysis, if appropriate
 - D. Hydraulic analysis of “natural” condition
 - E. Scour computations (See Section 9.5 and Appendix B – Amended Local Abutment Scour Equations for Connecticut)
- IV. Selection of Final Design
- A. Risk assessment/Least-cost alternative (LTEC) (if required see Section 9.6.7)
 - B. Measure of compliance with established hydraulic criteria
 - C. Consideration of environmental and social criteria
 - D. Design details such as riprap, scour abatement, river training

V. Documentation (See Section 9.3.10)

- A. Complete project records, permit applications, etc.
- B. Complete correspondence and reports
- C. Requirements in the ConnDEP IWRD "Model Hydraulic Analysis" document for report format
- D. Hydraulic Data Sheet presented in Appendix A.

9.4.3 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 computer program such as HEC-RAS, HEC-2 or WSPRO unless indicated otherwise by the Hydraulics and Drainage section. The hydraulic variables and flow types are defined in Figures 9-1 and 9-2.

Backwater (h_1) is measured relative to the normal water surface elevation without the effect of the bridge at the approach cross section (Section 1). Backwater is the result of contraction and re-expansion head losses and head losses due to bridge piers. Backwater can also be the result of a "choking condition" in which critical depth is forced to occur in the contracted opening with a resultant increase in depth and specific energy upstream of the contraction. This is illustrated in Figure 9-2.

The following are the three types of flow which may be encountered in bridge waterway design illustrated in Figure 9-2:

- Type I consists of subcritical flow throughout the approach, bridge and exit cross sections and is the most common condition encountered in practice.
- Type IIA and IIB both represent subcritical approach flows which have been choked by the contraction resulting in the occurrence of critical depth in the bridge opening. In Type IIA the critical water surface elevation in the bridge opening is lower than the undisturbed normal water surface elevation. In the Type IIB it is higher than the normal water surface elevation and a weak hydraulic jump immediately downstream of the bridge contraction is possible.
- Type III flow is supercritical approach flow and remains supercritical through the bridge contraction. Such a flow condition is not subject to backwater unless it chokes and forces the occurrence of a hydraulic jump upstream of the contraction.

9.4.4 Methodologies

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, essentially duplicate results can usually be achieved using both momentum and energy methods.

Momentum

- HEC-2

The Corps of Engineers HEC-2 model uses a variation of the momentum method in the special bridge routine when there are bridge piers. The momentum equation between cross sections 1 and 3 is used to detect Type II flow and solve for the upstream depth in this case with critical depth in the bridge contraction.

This model has been used for the majority of the flood insurance studies performed under the National Flood Insurance Program (NFIP). However, some feel that the bridge analysis routines in HDS-1 and WSPRO may yield a better definition of actual hydraulic performance.

- HEC-RAS

The Corps of Engineers Hydrologic Engineering Center (HEC) has developed the HEC-RAS (River Analysis System) program package. It operates under WINDOWS and has full graphic support. The finished package includes all the features inherent to HEC-2 and WSPRO plus program selected friction slope methods, mixed flow regime capability, automatic "n" value calibration, ice cover, quasi 2-D velocity distribution, superelevation around bends, bank erosion, riprap design, stable channel design, sediment transport calculations and scour at bridges. In addition to momentum balance, other methods are available in HEC-RAS for computing losses through bridges. These methods include the Energy Equation (standard step method), Yarnell equation and FHWA WSPRO method.

Energy

- HDS-1

The method developed by FHWA described in HDS-1 is an energy approach with the energy equation written between cross sections 1 and 4 as shown in Figure 9-1 for Type I flow. The backwater is defined in this case as the increase in the approach water surface elevation relative to the normal water surface elevation without the bridge.

This model utilizes a single typical cross section to represent the stream reach from points 1 to 4 on Figure 9-1. It also requires the use of a single energy gradient. This method is no longer recommended for final design analysis of bridges due to its inherent limitations but it may be useful for preliminary analysis and training. Studies performed by the Corps of Engineers for the FHWA show the need to utilize a multiple cross section method of analysis in order to achieve reasonable stage-discharge relationships at a bridge.

- WSPRO

WSPRO combines step-backwater analysis with bridge backwater calculations. This method allows for pressure flow through the bridge, embankment overtopping and flow through multiple openings and culverts. The bridge hydraulics still rely on the energy principle, but there is an improved technique for determining approach flow lengths and the introduction of an expansion loss coefficient. The flow-length improvement was found necessary when approach flows occur on very wide heavily-vegetated floodplains. The program also greatly facilitates the hydraulic analysis required to determine the least-cost alternative.

Physical Modeling

Complex hydrodynamic situations defy accurate or practicable mathematical modeling. Physical models should be considered when:

- hydraulic performance data are needed that cannot be reliably obtained from mathematical modeling

- risk of failure or excessive over-design is unacceptable
- research is needed

The constraints on physical modeling are:

- size(scale)
- cost
- time

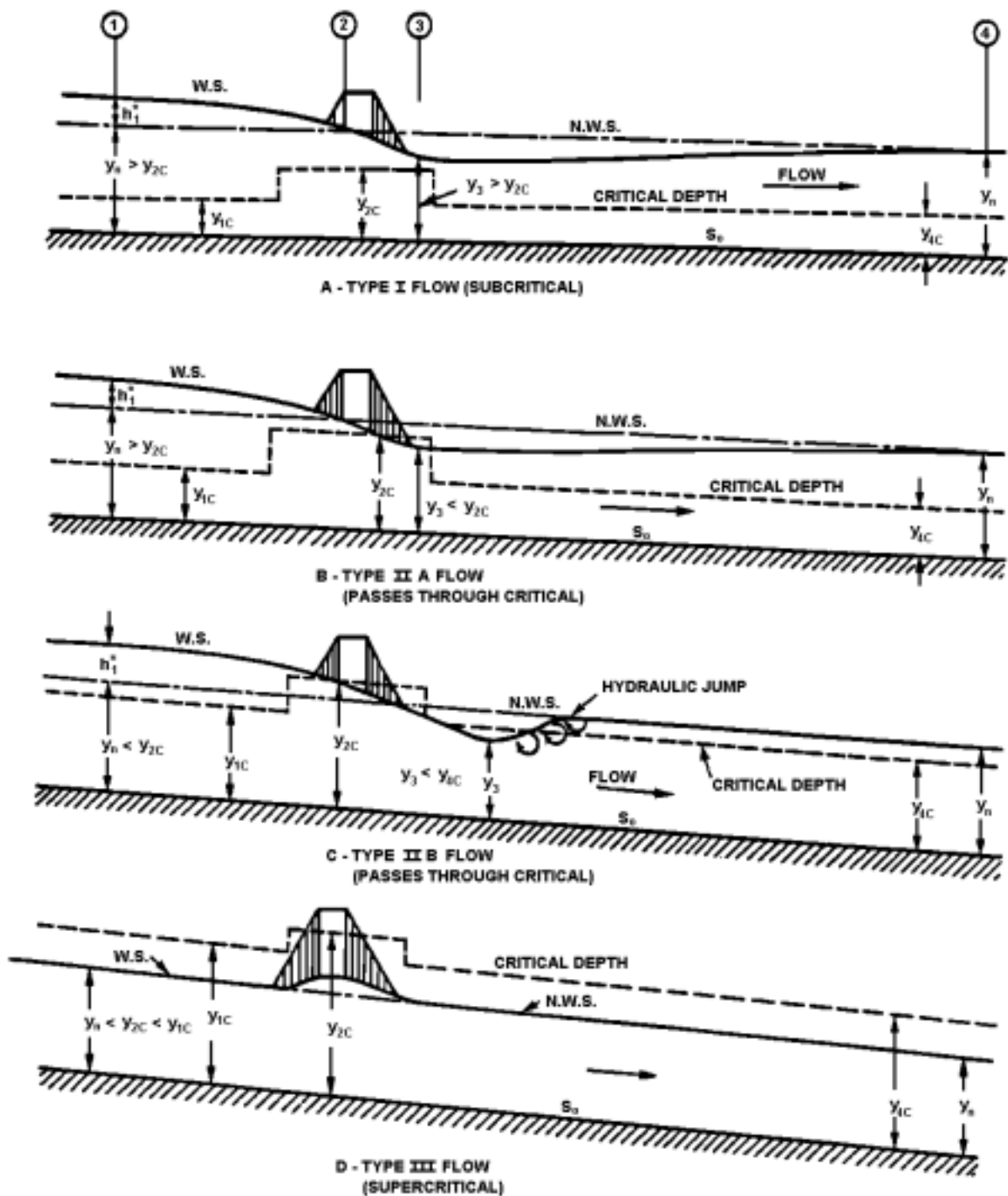


Figure 9-2 Bridge Flow Types

Source: HDS-1

9.4.5 Coastal or Tidally Influenced Waterways

Introduction

The design of coastal or tidally influenced waterway bridge openings is typically more complicated than similar designs on riverine systems. The natural system is difficult to quantify because it is typically spatially variant in two horizontal directions and it is unsteady flow. Coastal waterways are subject to storm surges and astronomical tides that play an important role in the dynamic hydraulic behavior of the system. 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 very difficult to estimate.

Tidal flow, both at flood stage and under normal conditions, may be restricted in its entrance into marshes 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 marsh or estuary and its adjacent wetlands. The highway designer should be aware of these potential impacts, particularly when planning a new facility.

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, because the total cost of a larger bridge approximates the cost of a smaller bridge considering approach embankments and abutment protection measures. This design is also desirable from an environmental perspective because 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 structure and channel exhibits hydraulic problems.

HEC-18 contains several simplified methods for tidal hydraulic analysis that are applicable to many bridge crossings in tidal waterways. When the simplified methods yield unacceptably conservative results or are not applicable due to flow complexity, tidal hydraulic analyses including one- and two-dimensional hydrodynamic computer modeling will provide a more accurate hydraulic simulation of the situations.

Computer Modeling

Existing models cover a wide range from simple analytical solutions to heavily 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 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 performed with the Corps of Engineers HEC-RAS, which has replaced previously used programs such as HEC-2 or the U.S. Geological Service – FHWA WSPRO (HY-7), or NRCS WSP-2.

In the event that tidal fluctuations are significant, simulation of the unsteady hydraulics is more appropriate. Dynamic modeling is recommended when complex geomorphic or hydraulic conditions make other methods unusable or when simplifying assumptions are violated to such a degree that the results are overly conservative.

In Phase 1 of the Pooled-Fund study entitled "Development of Hydraulic Computer Models to Analyze Tidal and Coastal Stream Hydraulic Conditions at Highway Structures" (Ayres Associates 1994), of which the Department was a participant, UNET (Barkau 1996) and FESWMS-2DH were recommended for dynamic tidal hydraulic modeling. UNET is a one-dimensional model, and FESWMS-2DH is a two-dimensional model. Although other one- and two-dimensional models are also applicable for tidal hydraulic modeling, the recommended models incorporate bridge, culvert and road overtopping hydraulics. Therefore, these models were deemed most applicable for tidal bridge hydraulic and scour evaluations. Detailed directions for model development for flow in tidal waterways are given by Ayres in manuals developed under the pooled fund study (see reference listing). This includes flow charts for development, calibration and model use and the use of the two-dimensional model within the modeling shell, SMS.

One-Dimensional Models

UNET is a powerful, unsteady-flow model that computes flow through a bifurcating or branching network of channels. UNET also includes storage areas and a wide variety of hydraulic structures. These features make this model useful in tidal and unsteady riverine applications where bridge hydraulics are an important component. Under agreement with Dr. Barkau, developer of the model, the USACE Hydrologic Engineering Center (HEC) maintains, distributes and provides training for UNET. Version 3.0 of HEC-RAS contains the one-dimensional model UNET developed by Barkau (1996). The input to the program consists of channel geometry as a series of cross sections comprising a channel reach, flow-resistance parameters, ineffective flow areas, structures located on or along the channels, storage areas located at the ends or adjacent to the channels, information on how the channel reaches and how storage areas are connected, and boundary conditions. Structures include bridges, culverts, roadway embankments, spillways, navigational dams and closed conduits.

Dynamic models perform hydraulic computations for channels, overbanks, bridges and culverts and include potentially filled or inundated bay, estuary and floodplain areas. Therefore, dynamic models yield the most accurate hydraulic analysis for flooding and scour computations and countermeasure design. The governing equations used in these models are the full dynamic equations for conservation of mass and momentum. One-dimensional modeling is applicable for estuaries with well-defined channels and for bays with single or multiple inlets. Where bays are crossed by numerous causeways, especially causeways with multiple bridge openings, two-dimensional modeling is recommended. Estuaries with multiple-branched channels and nearly well-defined channel flow can be modeled with one-dimensional network models (such as HEC-RAS), but the complexity often warrants the use of two-dimensional models. When it is necessary to employ dynamic modeling, it must be recognized that the development time will be significantly longer than for the one-dimensional model. However, it does provide a tool to develop complex and dynamic flow patterns.

Two-Dimensional Models

The computer program Finite Element Surface Water Modeling System: Two-Dimensional Flow in a Horizontal Plane (FESWMS-2DH) is maintained by FHWA.). In Version 3, FESWMS-2DH has been renamed to FESWMS Flo2DH. As indicated by its name, FESWMS Flo2DH uses a finite-element numerical method to solve the equations that describe the two-dimensional flow of shallow surface water. The governing equations include the conservation of mass, the conservation of momentum in two directions and boundary condition equations. The program solves for the flow depth and the x- and y-velocity components at discrete points, called "nodes," throughout the

network. The input to the program includes information on the network and the boundary conditions. The network is the discretized spatial description of the system being modeled and consists of elements and nodes. Elements are 3- or 4-sided cells that may be irregular in shape and size. Each element has an associated material type that contains the values of resistance parameters. Nodes are the points at the corners and midsides of the elements. Each 4-sided element also has a node at its center. Every node has a bed elevation assigned to it to describe the bathymetry of the system.

Another effective two-dimensional model that can be used is RMA2 (AYERS, 1994). Like FESWMS Flo2DH, this model is a two-dimensional depth, averaged finite-element hydrodynamic numerical model. It computes water surface elevations and horizontal velocity components for subcritical, free-surface flow in two-dimensional flow fields. RMA2 computes a finite-element solution of the Reynolds form of the Navier-Stokes equations for turbulent flows. Friction is calculated with the Manning's or Chezy Equation, and eddy viscosity coefficients are used to define turbulence characteristics. The model is used to compute dynamic response to tidal, storm surge, winds and stream inflow hydrographs. Similar to FESWMS Flo2DH, the output from the model is water surface elevation and velocity throughout the domain.

Surface-Water Modeling System

Surface-water Modeling System (SMS) is a pre- and post-processor that can be used with both FESWMS and RMA2. The SMS software is a comprehensive graphical user environment for two- and three-dimensional numerical modeling. The Environmental Modeling Research Laboratory at Brigham Young University developed it in cooperation with the USACE Waterways Experiment Station, now ERDC (Engineering Research and Development Command) and FHWA. The finite-element mesh or cross section entities, plus the associated boundary conditions necessary for analysis, are created within SMS and then saved to model specific files. These files are used as input to the hydrodynamic, wave mechanic, contaminant migration and sediment-transport analysis engines of the models supported by SMS, such as RMA2 and FESWMS Flo2DH. The numerical models create solution files that contain the water surface elevations, velocities, contaminant concentrations, sediment concentrations or other functional data at each node, cell or section. Plots and animations can then be easily created within SMS to view the simulation results. SMS can also be used as a pre- and post-processor for other finite-element or finite-difference programs if the programs can read and write files in a supported format. SMS is capable of constructing large, complex meshes (up to hundreds of thousands of elements) of arbitrary shape.

SMS can also be used as a pre- and post-processor for many other surface water modeling tools for analysis and design. Supported models include the USACE-WES supported TABS-MD (GFGEN, RMA4, RMA10, SED2D-WES) in addition to the RMA2 mentioned above. It also supports ADCIRC, CGWAVE, STWAVE and HIVEL2D. Comprehensive interfaces have also been developed for facilitating the use of the FHWA-commissioned analysis packages FESWMS Flo2DH and WSPRO. SMS also includes a generic interface, which can be used to support models that have not been officially incorporated into the interface. The SMS pre- and post-processor includes two-dimensional finite element, two-dimensional finite difference, three-dimensional finite element and one-dimensional backwater modeling tools.

Method Selection

Chapter 4 of the FHWA Pooled Fund Study manual "TIDAL HYDRAULIC MODELING FOR BRIDGES" (Ayres Associates, March 2002), also provides a review of the available methods for

tidal waterway hydraulic analyses, from the simple methods presented in HEC-18 to the complex modeling introduced in the preceding paragraphs of this manual. Based on the geomorphic characteristics of the waterway, recommendations are made on the selection of the most appropriate method. Chapter 5 contains guidance on HEC-RAS application for tidal hydraulics.

The hydraulic engineer is responsible for determining the most suitable method for analyzing tidal structures. The decision should be made early in the design phase with approval from the Hydraulics and Drainage Section.

9.5 Bridge Scour Or Aggradation

9.5.1 Introduction

Reasonable and prudent hydraulic analysis of a bridge design requires that an assessment be made of the proposed bridge's vulnerability to undermining due to potential scour. Because of the extreme hazard and economic hardships posed by a rapid bridge collapse, special considerations must be given to selecting appropriate flood magnitudes for use in the analysis. The hydraulic engineer must endeavor to always be aware of and use the most current scour forecasting technology.

The FHWA issued a Technical Advisory (TA 5140.20) on bridge scour in September 1988. The document "Interim Procedures for Evaluating Scour at Bridges" was an attachment to the Technical Advisory. The interim procedures were replaced by HEC-18 (1991, 1993, 1995). Users of this manual should consult HEC-18 for a more thorough treatise on scour and scour prediction methodology. Companion FHWA documents to HEC-18 are HEC-20, "Stream Stability at Highway Structures," and HEC-23 "Bridge Scour and Stream Instability Countermeasures."

The inherent complexities of stream stability, further complicated by highway stream crossings, requires a multilevel solution procedure. The evaluation and design of a highway stream crossing or encroachment should begin with a qualitative assessment of stream stability. This involves application of geomorphic concepts to identify potential problems and alternative solutions. This analysis should be followed with quantitative analysis using basic hydrologic, hydraulic and sediment transport engineering concepts. Such analyses could include evaluation of flood history, channel hydraulic conditions (up to and including, for example, water surface profile analysis) and basic sediment transport analyses such as evaluation of watershed sediment yield, incipient motion analysis and scour calculations. This analysis can be considered adequate for many locations if the problems are resolved and the relationships between different factors affecting stability are adequately explained. If not, a more complex quantitative analysis based on detailed mathematical modeling and/or physical hydraulic models should be considered. This multilevel approach is presented in HEC-20 (and Chapter 7, Channels).

Less hazardous perhaps are problems associated with aggradation. Where freeboard is limited, problems associated with increased flood hazards to upstream property or to the traveling public due to more frequent overtopping may occur. Where aggradation is expected, it may be necessary to evaluate these consequences. Also, aggradation in a stream reach may serve to moderate potential scour depths. Aggradation is sometimes referred to as negative scour.

All Bridge Scour Evaluation Reports, either for proposed or existing structures shall be documented in accordance with the "Bridge Scour Evaluation Report Format" found in Appendix C.

9.5.2 General Discussion ConnDOT Bridges

Prior to the early 1950's, standard bridge design practice was typically to construct short span bridges with shallow spread footing foundations. This design practice was modified in the early 1950's, particularly for high volume traffic routes, where the substructure was either shallow foundation on competent ledge or deep foundation (e.g. piles, drilled shafts, etc.). It is the population of bridges founded on spread footings or shallow pile foundations that are generally the most vulnerable to scour. The floods of 1938, 1955 and 1982 destroyed many of the bridges throughout Connecticut. Many of these failures were attributed to scour. As a consequence many of the bridges founded on piles survived the large storm events while those founded on shallow spread

footings failed. Mapping for the major flood events experienced in Connecticut since 1927 can be found in Chapter 6, Hydrology, Appendix E.

9.5.3 Data Available Through ConnDOT Scour Program

The State of Connecticut has approximately 2373 bridges over waterways that are included in the National Bridge Inspection Standards (NBIS). Incorporating the FHWA guidelines and recommendations to address the potential for scour at a bridge, the Department initiated an ambitious scour evaluation program that has been in place since the early nineties. Below is a summary of the various scour programs conducted by the ConnDOT and the associated data available for the respective projects.

- **USGS Reports** - United States Geological Survey (USGS) conducted scour screenings on all structures not programmed for rehabilitation or replacement. Only structures on the NBIS system were included, (i.e. span widths greater than 6.1m (20 feet)). This screening process did not include culverts or structures with concrete bottom as they were considered low risk. The objective of this effort was to determine the vulnerability of a structure with respect to scour. A qualitative assessment was assigned to each bridge based on foundation information, channel stability, evidence of scour and various other variables compiled in the field. The reports provide a general overview of conditions inspected at the site.
- **Scour Evaluation Reports** - Approximately 300 bridges were evaluated for scour and assigned NBIS ratings under the Bridge Scour Evaluation Program. Each of the structures in this program was hydraulically analyzed. Scour depths were calculated from the hydraulic models. A summary of the field conditions, soil data scour depths and a recommended course of action can be found in these scour evaluation reports.
- **Comparative Scour Reports** - The Comparative Scour Program evolved by combining the information accumulated in both the USGS and Scour Evaluation reports. Remaining bridges to be evaluated in the NBIS system (and not scheduled for a project) were included in this program. NBIS ratings were assigned based on potential scour depths, with a strong emphasis on field review, history of flooding, history of scour and engineering judgement. Potential scour depths were determined by comparing the predicted scour depths of similar bridges included in the Scour Evaluation project. While these reports do not offer the comprehensive scour data found in the previous project, they do provide good information with respect to scour vulnerability, foundation information and scour history.

The data collection effort provided available information discussed above for both bridge replacements and rehabilitation projects. It will be useful in designing the foundation requirements and in developing appropriate countermeasure design. The USGS reports Scour Evaluation and Comparative Scour Reports are located in the Hydraulics and Drainage Section. An additional copy can be found in the Bridge Safety and Evaluation bridge files. Based on the conclusions noted within these documents, all bridges over water have been classified into one of three general categories, Low Risk (NBIS Item 113 rating of 8 or 9), Scour Susceptible (NBIS Item 113 rating of 4, 5, or 7*) or Scour Critical (NBIS Item 113 rating of 3 or below).

* The NBIS Item 113 rating of 7 is reserved for bridge locations at which countermeasures have been installed to mitigate a previous scour problem. If the structure is a clear span bridge

(no piers) and if the countermeasures have been designed in accordance with the procedures contained within HEC-23, the bridge may be considered "low risk." When countermeasures are placed adjacent to piers to correct a previous scour condition, the bridge is classified as "scour susceptible."

9.5.4 Scour Types

Present technology dictates that bridge scour be evaluated as interrelated components:

- long-term profile changes (aggradation/degradation) (See Chapter 7, Channels.)
- plan form change (lateral channel movement) (See Chapter 7, Channels.)
- contraction scour/deposition (HEC-18)
- local scour (See Appendix B and HEC-18)

Long-Term Profile Changes

Long-term stream bed profile changes can result from aggradation and/or degradation.

- Aggradation is the deposition of bedload due to a decrease in the energy gradient.
- Degradation is the scouring of bed material due to increased stream sediment transport capacity which results from an increase in the energy gradient.

Forms of degradation and aggradation shall be considered as imposing a permanent future change for the stream bed elevation at a bridge site whenever they can be identified.

Plan Form Changes

Plan form changes are morphological changes such as meander migration or bank widening. The lateral movement of meanders can threaten bridge approaches as well as increase scour by changing flow patterns approaching a bridge opening. Bank widening can cause significant changes in the flow distribution and thus the bridge's flow contraction ratio.

Contraction

Channel contraction scour results from a constriction of the channel which may, in part, be caused by bridge piers in the waterway. Deposition results from an expansion of the channel or the bridge site being positioned immediately downstream of a steeper reach of stream. Highways, bridges and natural channel contractions are the most commonly encountered cause of contraction scour. Two practices are provided in this manual for estimating deposition or contraction scour.

- Sediment routing practice — This practice should be considered if either bed armoring or aggradation from an expanding reach is expected to cause an unacceptable hazard.
- Empirical practice — This practice is adapted from laboratory investigations of bridge contractions in non-armoring soils and, as such, must be used considering this qualification. This practice does not consider bed armoring and its application for aggradation may be technically weak.

The same empirical practice algorithms used to evaluate a naturally contracting reach may also be used to evaluate deposition in an expanding reach provided armoring is not expected to occur. With deposition the practice of applying the empirical equations "in reverse" is required; i.e., the narrower cross section is upstream which results in the need to manipulate the use of the empirical "contraction scour" equation. This need to manipulate the intended use of an equation does not occur with the sediment routing practice which is why it may be more reliable in an expanding reach.

Local Scour

Exacerbating the potential scour hazard at a bridge site are any abutments or piers located within the flood flow prism. The amount of potential scour caused by these features is termed local scour. Local scour is a function of the geometry of these features as they relate to the flow geometry. However, the importance of these geometric variables will vary. As an example, increasing the pier or cofferdam width either through design or debris accumulation will increase the amount of local scour, but only up to a point in subcritical flow streams. After reaching this point, pier scour should not be expected to measurably increase with increased stream velocity or depth. This threshold has not been defined in the more rare, supercritical flowing streams.

9.5.5 Engineering Judgement – A discussion of the predicted local scour depths at abutments in Connecticut

The magnitude of local scour predicted at abutments is largely dependent upon the discharge intercepted by the abutment and returned to the main channel at the bridge. The predicted scour depths depend on such variables as the abutment shape, sediment characteristics, cross-section of the streambed and overbank areas and the angle of attack. The equations for local abutment scour, as presented in HEC-18, were primarily developed based upon data that has been collected in laboratory flume studies. These studies show that an increase in the projected length of an abutment into the flow increased scour. This result is caused by many factors, including an assumed uniform distribution of flow upstream of the abutment. As such, the discharge intercepted by the abutment in the laboratory is a direct function of the abutment length. This may not be the case in the field where a non-uniform flow distribution is usually experienced upstream of the abutments. This non-uniform flow distribution is caused by such factors as the relative flow depths in the main channel versus the overbank areas, and the significantly higher Manning's values that are usually representative of heavily vegetated overbank areas. The research that has been accomplished to date has not accurately replicated these field conditions. Due to this fact, the abutment scour equations are generally considered to be conservative in nature. It is important to realize that the predicted abutment scour depths would only occur in the field for conditions under which the flume experiments were conducted. The amended local abutment scour equations in Appendix B shall be used when computing local scour for abutments.

9.5.6 Armoring

Armoring may occur naturally if a stream or river is unable to move the more coarse material comprising either the bed or its underlying material. Scour may occur initially but later become arrested by armoring before the full scour potential is reached for a given flood magnitude. When armoring does occur, the coarser bed material will tend to remain in place or quickly redeposit so as

to form a layer of riprap-like armor on the stream bed or in the scour holes and thus limit further scour for a particular discharge. This armoring effect can decrease scour hole depths which were predicted based on formulae developed for sand or other fine material channels for a particular flood magnitude. When a larger flood occurs than used to define the probable scour hole depths, scour will probably penetrate deeper until armoring again occurs at some lower threshold.

Armoring may also cause bank widening. Bank widening encourages rivers or streams to seek a more unstable, braided regime. Such instabilities may pose serious problems for bridges as they encourage further, difficult to assess plan form changes. Bank widening also spreads the approach flow distribution which in turn results in a more severe bridge opening contraction.

9.5.7 Scour Resistant Materials

Caution is necessary in determining the scour resistance of bed materials and the underlying strata. With sand size material, the passage of a single flood may result in the predicted scour depths. Conversely, in scour resistant material the maximum predicted depth of scour may not be realized during the passage of a particular flood; however, some scour resistant material may be lost. Commonly, this material is replaced with more easily scoured material. Thus, at some later date another flood may reach the predicted scour depth. Serious scour has been observed to occur in materials commonly perceived to be scour resistant such as consolidated soils and glacial till, as well as so-called bed rock streams and streams with gravel and boulder beds. Certain types of weathered ledge may also be susceptible to scour (**a geologist and/or geotechnical engineer should evaluate the potential for scour within a ledge formation**).

9.5.8 Scour Evaluation Studies for ConnDOT Projects

New Bridges over Waterways

Level II Scour Evaluations shall be performed for all new bridges over waterways unless one or more of the following conditions apply:

- The bridge has been designed to span the entire floodplain for the superflood (500 year recurrence interval) or the critical design event if less than the 500 year flood.
- The structure foundations will be set directly on sound bedrock.
- The abutment footings will be protected with riprap designed in accordance with the methods outlined in the latest version of Bridge Scour and Stream Instability Countermeasures: (HEC-23) or successor documents. **It should be noted that the use of riprap as the sole means of providing scour protection for new bridges is discouraged and should be used only where it has been demonstrated that alternate, preferred means of designing bridges to be safe from scour related failure are not feasible.** (Refer to the Department's Bridge Design Manual for preferred foundation types).

Reconstructed or Rehabilitated Bridges

Generally, scour evaluations shall be performed for all bridges which are to be reconstructed or rehabilitated where significant capital investment is involved and where the bridge has been classified as scour susceptible or scour critical. A significant capital investment correlates to the following improvement categories:

- Deck replacement
- Superstructure replacement or widening
- Major repairs that involve modification to substructure units

Bridges which have been classified as scour susceptible or scour critical shall have hydrologic, hydraulic and scour evaluations performed which are sufficiently detailed to satisfy all applicable design and permitting requirements. If a detailed (Level II) scour evaluation has already been performed, the designer shall modify the results of this document as necessary to incorporate the "Modified Abutment Equations" contained in Appendix B. All necessary scour countermeasures for scour susceptible or scour critical bridges shall be incorporated into the overall project plans, as appropriate.

Scour evaluations shall not be required where structures to be reconstructed or rehabilitated have previously been classified as low risk under the Department's Bridge Scour Evaluation Program or for scour susceptible bridges which are not undergoing substructure modification and have had countermeasures installed following a Level II study.

9.6 Philosophy

9.6.1 Introduction

Any stream is a dynamic natural system which, as a result of the encroachment caused by elements of a stream-crossing system, will respond in a way that may well challenge even an experienced hydraulic engineer. The complexities of the stream response to encroachment demand that: (1) hydraulic engineers must be involved from the outset in the choice of alternative stream crossing locations, and (2) at least some of the members of the engineering design team must have extensive experience in the hydraulic design of stream-crossing systems. Hydraulic engineers should also be involved in the solution of stream stability problems at existing structures.

This section discusses qualitatively some of the design issues which contribute to the overall complexity of spanning a stream with a stream-crossing system. A much more thorough discussion of design philosophy and design considerations is found in the AASHTO Highway Drainage Guidelines, "Hydraulic Analysis for the Location and Design of Bridges."

9.6.2 Location Of Stream Crossing

Although many factors, including nontechnical ones, enter into the final location of a stream-crossing system, the hydraulics of the proposed location must have a high priority. Hydraulic considerations in selecting the location include floodplain width and roughness, flow distribution and direction, stream type (braided, straight, or meandering), stream regime (aggrading, degrading, or equilibrium) and stream controls. The hydraulics of a proposed location also affect environmental considerations such as aquatic life, wetlands, sedimentation and stream stability. Finally, the hydraulics of a particular site determine whether or not certain federal and state objectives such as floodplain use, reduction of flooding losses and preservative of wetlands can be met.

9.6.3 Coordination, Permits, Approvals

The interests of other government agencies must be considered in the evaluation of a proposed stream-crossing system, and cooperation and coordination with these agencies, especially water resources planning agencies, must be undertaken. Coordination with the Federal Emergency Management Agency (FEMA) is required when a proposed crossing encroaches on a regulatory floodway and would require an amendment to the floodway map. This is a difficult process especially in more developed areas.

Whenever practicable, the stream-crossing system shall avoid encroachment on the floodway within a floodplain. When this is not feasible, modification of the floodway itself and associated floodway maps shall be considered. The process of modifying the floodway can be difficult. If the encroachment causes additional flooding of residential structures, no map revision will be forthcoming. (See Legal Aspects, Chapter 2, Section 2.3 for additional information.)

9.6.4 Environmental Considerations

Environmental criteria which must be met in the design of stream-crossing systems include the preservation of wetlands and protection of aquatic habitat. Such considerations often require the expertise of a biologist on the design team. Water quality considerations shall also be included in the design process insofar as the stream-crossing system affects the water quality relative to beneficial uses. As a practical matter with bridges, the hydraulic design criteria related to scour,

degradation, aggradation, flow velocities and lateral distribution of flow, for example, are important criteria for evaluation of environmental impacts as well as the safety of the stream-crossing structures. Environmental consequences of the bridge construction activity must also be considered (see Section 9.2.3).

9.6.5 Stream Morphology

The form and shape of the stream path created by its erosion and deposition characteristics comprise its morphology. A stream can be braided, straight, or meandering, or it can be in the process of changing from one form to another as a result of natural or manmade influences. A historical study of the stream morphology at a proposed stream-crossing site is mandatory. (See Chapter 7, Channels, Section 7.5.) This study shall also include an assessment of any long-term trends in aggradation or degradation. Braided streams and alluvial fans shall especially be avoided for stream-crossing sites whenever possible.

9.6.6 Data Collection

The purpose of data collection is to gather all necessary site information. This shall include such information as topography and other physical features, land use and culture, flood data, basin characteristics, precipitation data, historical high-water marks, existing structures, channel characteristics and environmental data. A site plan shall be developed on which much of the data can be shown. (See Chapter 1, Section 1.6.)

9.6.7 Risk Evaluation

The evaluation of the consequence of risk associated with the probability of flooding attributed to a stream-crossing system is a tool by which site specific design criteria can be developed. This evaluation considers capital cost, traffic service, environmental and property impacts and hazards to human life.

The evaluation of risk is a two stage process. The initial step, identified as risk assessment, is more qualitative than a risk analysis and serves to identify threshold values that must be met by the hydraulic design.

In many cases where the risks are low and/or threshold design values can be met, it is unnecessary to pursue a detailed economic analysis. In those cases where the risks are high and/or threshold values cannot be met, a Least Total Expected Cost (LTEC) analysis should be considered.

The results of a least-cost analysis can be presented in a graph of total cost as a function of the overtopping discharge. The total cost consists of a combination of capital costs and flood damages (or risk costs). Risk costs decrease with increases in the overtopping discharge while capital costs simultaneously increase. The overtopping discharge for each alternative is determined from a hydraulic analysis of a specific combination of embankment height and bridge-opening length. The resulting least-cost alternative provides a tradeoff comparison. If, for example, environmental criteria or constraints due to existing development result in an alternative that is different from the least-cost alternative, the economic tradeoff cost of that alternative can be given as the difference between its cost and the minimum cost provided by a LTEC analysis.

The alternatives considered in the least-cost analysis do not require the specification of a particular design flood. This information is part of the output of the least-cost analysis. In other words, the least-cost alternative has a specific risk of overtopping that is unknown before the least-cost alternative has been determined. Therefore, design flood frequencies are used only to establish

the initial alternative. Thereafter, specific flood-frequency criteria such as the 50-year flood requirement for certain interstate highways and the 100-year floodplain requirements for flood insurance should be considered only as constraints on the final design selection. Deviation from the least-cost alternative may be necessary to satisfy these constraints and the trade-off cost for doing so can be obtained from the least-cost analysis.

Risk based analysis does not recognize some of the intangible factors that influence a design. The minimum design that results from this type of analysis may be too low to satisfy the site condition.

9.6.8 Scour

The extreme hazard posed by bridges subject to bridge scour failures dictates a different philosophy in selecting suitable flood magnitudes to use in the scour analysis. With bridge flood hazards other than scour, such as those caused by roadway overtopping or property damage from inundation, a prudent and reasonable practice is to first select a design flood to determine a trial bridge opening geometry. This geometry is either subjectively or objectively selected based on the initial cost of the bridge along with the potential future costs for flood hazards. Following the selection of this trial bridge geometry, the base flood (100-year) is used to evaluate this selected opening. This two step evaluation process is used to ensure the selected bridge opening based on the design flood contains no unexpected increase in any existing flood hazards other than those from scour or aggradation. With bridge scour, not only is it required to consider bridge scour or aggradation from the base flood, but also an even larger flood termed herein as the "super flood."

Scour prediction technology is steadily developing, but lacks at this time, the reliability associated with other facets of hydraulic engineering. Several formulae for predicting scour depths are currently available and others will certainly be developed in the future. **The designer should strive to be acquainted with the "state of practice" at the time of a given analysis and use engineering judgement.**

First discussion is warranted as to what constitutes the greatest discharge passing through the bridge opening during a particular flood. Even where there are relief structures on the floodplain or overtopping occurs, some flood other than the base flood or "super flood" may cause the worst case bridge opening scour. This situation occurs where the bridge opening will pass the greatest discharge just prior to incurring a discharge relief from overtopping or a floodplain relief opening. Conversely care must be exercised in that a discharge relief at the bridge due to overtopping or relief openings may not result in reduction in the bridge opening discharge. Should a reduction occur, the incipient overtopping flood or the overtopping flood corresponding to the base flood or "super flood" would be used to evaluate the bridge scour.

With potential bridge scour hazards a different flood selection and analysis philosophy is considered reasonable and prudent. The foregoing trial bridge opening which was selected by considering initial costs and future flood hazard costs shall be evaluated for two possible scour conditions with the worst case dictating the foundation design — and possibly a change in the selected trial bridge opening.

First, evaluate the proposed bridge and road geometry for scour using the base flood, incipient overtopping flood, overtopping flood corresponding to the base flood, or the relief opening flood whichever provides the greatest flood discharge through the bridge opening. Once the expected scour geometry has been assessed, the geotechnical engineer would recommend a foundation type. This foundation type would be designed and analyzed in accordance with the governing AASHTO specifications taking into account any stream bed and bank material displaced by scour for foundation support.

Second, impose a "super flood" on the proposed bridge and road geometry. This event shall be greater than the base flood and shall be used to evaluate the proposed bridge opening to ensure that the resulting potential scour will produce no unexpected scour hazards. The "super flood" shall be defined as the 500-year flood or a designated ratio (e.g., 1.7) times the 100-year flood. Similar to the base flood to evaluate the selected bridge opening, use either the "super flood", or the relief opening flood, whichever imposes the greatest flood discharge on the selected bridge opening. The foundation design based on the base flood would then be checked against its ultimate capacity, taking into account any stream bed and bank material displaced by scour from the "super flood".

9.6.9 Preventive/Protection Measures

Based on an assessment of potential scour provided by the Hydraulic Engineer, the structural designers can incorporate design features that will prevent or mitigate scour damage at piers. In general, circular piers or elongated piers with circular noses and an alignment parallel to the flood flow direction are a possible alternative. Spread footings should be used only where the stream bed is extremely stable below the footing and where the spread footing is founded at a depth below the maximum scour computed. Deep foundations (driven piles, drill shafts, drilled piles, etc.) shall be used where it is not practical to use a spread footing foundation. Protection against general stream bed degradation can be provided by drop structures or grade-control structures in, or downstream of the bridge opening. (See Section 9.2.2.)

Rock riprap is often used, where stone of sufficient size is available, to armor abutment fill slopes and the area around the base of piers. Riprap design information is presented in HEC-18.

Consideration should be given to burying riprap under native material to accommodate Fisheries concerns. It should be noted that this is not always possible as the inspectors must be able to determine if it is in place to ensure the substructure unit is protected from the potential scour. Provide a 1.5m (5 ft) shelf for inspection purposes.

Whenever possible, clearing of vegetation upstream and downstream of the toe of the embankment slope should be avoided as it will assist in minimizing scour potential. Embankment overtopping may be incorporated into the design but should be located well away from the bridge abutments and superstructure.

9.6.10 Deck Drainage

A major responsibility of the engineer is to provide for the safety of the traveling public. There is a much greater risk of someone being in an accident on the bridge as the result of wet pavement than there is due to the catastrophic collapse of the bridge due to floods or structural failure.

Improperly drained bridge decks can cause numerous problems including corrosion, icing and hydroplaning. Whenever possible, bridge decks should be watertight and all deck drainage should be carried to the ends of the bridge. Drains at the end of the bridge should have sufficient inlet capacity to carry all bridge drainage.

The design of pavement drainage on the bridge should use the same criteria as the approach roadway. However, it should be noted that an approach roadway with a rural typical section will be more free draining than a bridge deck with parapets where the deck will confine the runoff in a manner similar to a curbed roadway section. Careful attention must be given to spread on the bridge deck.

Where it is necessary to intercept deck drainage at intermediate points along the bridge, the design of the interceptors shall conform to the HEC-12 and HEC-21 procedures.

When deck drainage interceptors are needed, a collection system will be necessary to discharge the runoff. Some considerations for this system are:

- environment concerns for discharging pavement runoff directly into a waterway
- design and maintenance of extensive drain systems attached to the superstructure
- free drops from deck interceptors
- 200-mm (8 in) minimum projection beyond lowest adjacent superstructure component
- provide erosion control under free drops unless outlet from bridge superstructure is more than 12.2 m (40 ft) above ground

9.6.11 Construction/Maintenance

Construction plans should be reviewed jointly by Construction, Environmental Planning and the Hydraulics and Drainage Section to note any changes in the stream from the conditions used in the design. Temporary structures and crossings used during construction should be designed for a specified risk of failure due to flooding during the construction period. The impacts on normal water levels, fish passage and normal flow distribution must also be considered.

The stream crossing design shall incorporate measures which reduce maintenance costs whenever possible. These measures include riprap protection of abutments and embankments, embankment overflow at lower elevations than the superstructure, and alignment of piers with the flow.

9.6.12 Temporary Hydraulic Facilities

Temporary hydraulic facilities include, among other things, temporary bridges, bypass channels, haul roads, or channel constrictions such as cofferdams. Such facilities must be capable of conveying the temporary design discharge without endangering life or property, including the structure under construction. The temporary design discharge for temporary structures and crossings used during construction is determined by the methodology indicated in Chapter 6, Section 6.15, Hydrology for Temporary Facilities.

The temporary hydraulic facilities should be designed or specified so as not to cause roadways to be overtopped or aggravate existing flooding conditions during the temporary design discharge. Hydraulic analysis of such facilities is based on the methods described in this manual. A hydraulic analysis with the temporary facility in place is required and should be compared to an analysis of existing conditions.

A flood contingency plan is required should a storm greater than the temporary design storm occur during construction. This usually starts with the engineer evaluating what path the floodwater would take should the flow exceed the capacity of the temporary system. Some considerations include:

- Is there sufficient relief so that flood levels would not significantly impact surrounding area or cause erosion and damage downstream by sudden release of the floodwater?
- If the roadway would be overtopped, what is the path, where is the low point and what is the approximate depth of flow? Is the overflow likely to cause any significant damage? If traffic is being maintained during construction are there sufficient provisions to close the road to traffic if flooded?
- How long will a temporary system be needed or any obstructions be in place? The probability of flooding increases with the length of time.

The flood contingency plan should be described in the hydraulic report and Attachment L of the DEP permit application.

The need for temporary erosion protection, scour countermeasures and, in the case of temporary culverts, outlet protection should be addressed.

Temporary hydraulic facilities should be shown on the stage construction/water handling plans. See Chapter 3, Design Development, Section 3.7.6 regarding the information to be provided.

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Appendix A – Forms

- **Data Collection and Field Review (pages 9.A-1 to 9.A-11)**
- **Hydraulic Data (pages 9.A-13 to 9.A-16)**

DATA COLLECTION AND FIELD REVIEW**I. GENERAL PROJECT DATA**

Bridge No.: _____

Town: _____

Feature carried: _____

Quadrangle: _____

County: _____

Feature crossed: _____

DEP watershed basin no.: _____

Functional class:

☐ urban principal arterial-interstate☐ urban principal arterial-other expwy.☐ urban principal arterial-other☐ urban minor arterial☐ urban collector☐ urban local☐ rural principal arterial-interstate☐ rural principal arterial-other expwy.☐ rural principal arterial-other☐ rural minor arterial☐ rural major collector☐ rural minor collector☐ rural local

Year built: _____

Overall NBIS structure rating: _____

USGS total scour index: _____

Year of reconstruction: _____

NBIS Item 113: _____

Sufficiency rating: _____

Plans available? ☐ yes☐ no**II. SUPERSTRUCTURE INFORMATION**

Bridge width: _____ m (ft)

Number of spans: _____

Bridge length: _____ m (ft)

Bridge skew: _____ (degrees)

Bearing connection type: ☐ positive connection☐ no positive connection**III. HYDROLOGIC AND HYDRAULIC INFORMATION**Watershed area: _____ km² (sq. mi.)Is it tidally influenced? ☐ yes☐ no

What information is available?

☐ floodway analysis report☐ FEMA F.I.S.☐ hydraulic report☐ SCEL analysis☐ Other: _____☐ scour report☐ comparative report

	Source	2 Yr. Event	10 Yr. Event	50 Yr. Event	100 Yr. Event	500 Yr. Event
Flow rates m ³ /s (cfs)						
Precipitation mm (in)						
Tidal elevations m (ft)						

Elevations m (ft.)							
At Structure			Water Surface at Approach Cross Section				
Streambed	Low Chord	Roadway	2 Yr. Event	10 Yr. Event	50 Yr. Event	100 Yr. Event	500 Yr. Event

Pressure flow at design storm? ☐ yes ☐ underclearance ____ m (ft.)

Comments: _____

IV. SITE DATA

- A. Existing structure(s) – Provide sketch of culvert/structure with dimensions and brief description.



Comments: Include structure or culvert type and condition. Note particularly any scour adjacent to abutments or at culvert outlet and the presence of debris or sediment. Also note the location of any utilities in the area of the crossing.

- B. High water marks – Describe the nature and location of any apparent high water marks and relate to a date of occurrence, if possible.

- C. Maximum allowable headwater – Describe the nature of the apparent controlling feature and note its location.

- D. Fish passage requirements – Comment on the apparent need for fish passage or impediments to same; such as dams or restrictive crossings in the area.

V. PERIPHERAL SITE DATA

- A. Hydraulic control – Note location and description.

- B. Upstream and downstream structures – Provide sketches and brief descriptions of existing bridges/culverts. Include dimensions.

Comments: _____

- C. Watershed area – Check watershed boundaries for accuracy. Note current land uses within watershed.

- D. Flow control structures within watershed – Note the location and type of all significant flow control structures (dams, etc.) within the watershed. Provide sketches with dimensions as required.

- E. Site photographs – Attach to report. Include an index and sketch of photograph locations.

VI. STREAM CHANNEL AND RELATED ASPECTS

A. Stream characterization

Twenty Groupings of Stream Characteristics (check box)

	Identifier	Drainage Area	Streambed Slope	Streambed Soils	Land Use
☐	A	Large	Low	SD	S/F
☐	B	Large	Low	SD	Urban
☐	C	Large	Moderate	SD	Forested
☐	D	Medium	Moderate	SD	Urban
☐	E	Medium	Moderate	SD	S/F
☐	F	Medium	Moderate	CLAY	S/F
☐	G	Medium	Moderate	TILL	S/F
☐	H	Medium	Moderate	SD	Forested
☐	I	Medium	Moderate	TILL	Forested
☐	J	Small	Low	SD	Urban
☐	K	Small	Moderate	TILL	Urban
☐	L	Small	Low	SD	S/F
☐	M	Small	Moderate	SD	S/F
☐	N	Small	Moderate	SD	Forested
☐	O	Small	Low	CLAY	S/F
☐	P	Small	Steep	TILL	S/F
☐	Q	Small	Moderate	TILL	S/F
☐	R	Small	Low	TILL	S/F
☐	S	Small	Moderate	TILL	Forested
☐	T	Small	Steep	TILL	Forested

Drainage area	Small	$\leq 64.75 \text{ km}^2$ (25 mi^2)
	Medium	$> 64.75 \text{ km}^2$ (25 mi^2) and $\leq 259 \text{ km}^2$ (100 mi^2)
	Large	$> 259 \text{ km}^2$ (100 mi^2)
Streambed slope	Low	$\leq 4.76 \text{ m/km}$ (25 ft/mi)
	Moderate	$> 4.76 \text{ m/km}$ (25 ft/mi) and $\leq 19.05 \text{ m/km}$ (100 ft/mi)
	Steep	$> 19.05 \text{ m/km}$ (100 ft/mi)
Streambed soils	SD = Stratified Drift	
Land Use	S/F = Suburban or Farming	

B. Channel stability

Previous NBIS Item 61 rating: _____

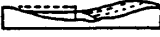
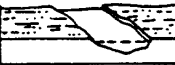
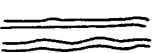
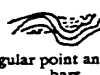
Lateral stability: ☐ stable ☐ unstable

Bank erosion: ☐ none ☐ light fluvial erosion ☐ heavy fluvial erosion ☐ mass wasting

Streambed: ☐ stable ☐ aggradating ☐ degrading

Armoring potential: ☐ none ☐ low ☐ moderate ☐ high

Geomorphic factors that affect stream stability (circle factors that apply)

STREAM SIZE	Small (< 30 m wide)		Medium (30-150 m)		Wide (> 150 m)		
FLOW HABIT	Ephemeral (Intermittent)		Perennial but flashy		Perennial		
BED MATERIAL	Silt-clay	Silt	Sand	Gravel	Cobble or boulder		
VALLEY SETTING	 No valley; alluvial fan		 Low relief valley (< 30 m deep)		 Moderate relief (30-300 m)	 High relief (> 300 m)	
FLOOD PLAINS	 Little or none (< 2X channel width)		 Narrow (2-10 channel width)		 Wide (> 10X channel width)		
NATURAL LEVEES	 Little or None		 Mainly on Concave		 Well Developed on Both Banks		
APPARENT INCISION	 Not Incised		 Probably Incised				
CHANNEL BOUNDARIES	 Alluvial		 Semi-alluvial		 Non-alluvial		
TREE COVER ON BANKS	< 50 percent of bankline		50-90 percent		> 90 percent		
SINUOSITY	 Straight Sinuosity 1-1.05		 Sinuous (1.06-1.25)		 Meandering (1.25-2.0)		 Highly meandering (> 2)
BRAIDED STREAMS	 Not braided (< 5 percent)		 Locally braided (5-35 percent)		 Generally braided (> 35 percent)		
ANABRANCHED STREAMS	 Not anabranching (< 5 percent)		 Locally anabranching (5-35 percent)		 Generally anabranching (> 35 percent)		
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS	 Narrow point bars		 Wide point bars		 Irregular point and lateral bars		 Random variation

Source: Adapted From Brice and Blodgett, 1978

(See also FHWA HEC-20, "Stream Stability at Highway Structures" for discussion of the above factors)

Secondary bed material: ☐ sand ☐ gravel ☐ boulders ☐ manmade
☐ silt/clay ☐ cobble ☐ bedrock

Bank protection

Type ☐ none ☐ modified ☐ intermediate ☐ standard
☐ concrete ☐ slope paving ☐ absent
☐ other
Condition ☐ n/a ☐ good ☐ weathered ☐ slumped
☐ poor ☐ missing ☐ fair

Comment on the need (if any) for training walls, cutoff walls or special slope or channel protection.

C. Channel and overbank roughness coefficients

Basic channel description: ☐ channel in earth ☐ channel cut into rock
☐ channel fine gravel ☐ channel coarse gravel

Surface irregularity of channel:

- ☐ smooth – best obtainable section for materials involved
- ☐ minor – slightly eroded or scoured side slopes
- ☐ moderate – moderately sloughed or eroded side slopes.
- ☐ severe – badly sloughed banks of natural channels or badly eroded sides of man-made channels - jagged and irregular sides or bottom sections of channels in rock.

Variations in shape and size of cross sections

- ☐ changes in size or shape occurring gradually
- ☐ large and small sections alternating occasionally or shape changes causing occasional shifting of main flow from side to side.
- ☐ large and small sections alternating frequently or shape changes causing frequent shifting of main flow from side to side.

Channel obstructions – (Judge the relative effect of obstructions – consider the degree to which the obstructions reduce the average cross sectional area, character of obstructions, and location and spacing of obstructions).

NOTE: Smooth or rounded objects create less turbulence than sharp, angular objects.

The effect of obstructions is:

- ☐ negligible
- ☐ minor
- ☐ appreciable
- ☐ severe

Degree of vegetation - (Note amount and character of foliage).

The effect of vegetative growth upon flow conditions is:

☐ LOW - Dense growths of flexible turf grasses where average depth of flow is 2 to 3 times the height of vegetation. Sparse seedling tree switches where the average depth of flow is 3 to 4 times the height of the vegetation.

☐ MEDIUM - Turf grasses where the average depth of flow is 1 to 2 times the height of vegetation. Stemmy grasses, weeds or tree seedlings, (moderate cover), average depth of flow 2 to 3 times the height of vegetation. Bushy growths, (moderately dense), along channel side slopes with no significant vegetation along channel bottom.

☐ HIGH - Turf grasses where average height is about equal to the average depth of flow. Willow or Cottonwood trees 8 to 10 years old with some weeds or brush. Bushy growths about 1 year old with some weeds. No significant vegetation along channel bottom.

☐ VERY HIGH - Turf grasses where the average depth of flow is less than one half the height of vegetation. Bushy growths about 1 year old intergrown with weeds. Dense growth of cattails along channel bottom. Trees intergrown with weeds and brush (thick growth).

Additional comments:

VII. HYDRAULIC VULNERABILITY

Previous Item 71 rating: _____

Is there confluence present? ☐ yes ☐ no

Angle of attack (flood flow): ☐ yes ☐ no

Bends in channel: ☐ upstream of bridge ☐ downstream of bridge
☐ straight channel reach ☐ at bridge

Velocity order of magnitude: ____ m/s (ft/s)

Trapping potential: ☐ low ☐ medium ☐ high

Debris potential: ☐ low ☐ moderate ☐ high

Overtopping relief: ☐ none ☐ left approach ☐ right approach
☐ on bridge ☐ relief bridge ☐ cannot be determined

Primary bed material: ☐ sand ☐ gravel ☐ boulders ☐ manmade
☐ silt/clay ☐ cobble ☐ bedrock

Comments: _____

VIII. VISUAL SCOUR EVIDENCE

USGS observed scour index: _____

History of scour problem: ☐ yes ☐ no

Comments: _____

Note: Comment should address any evidence of scour at ALL substructure units.

CONTRACTION SCOUR SUSCEPTIBILITY

Channel width upstream: _____ m (ft.)

Channel width under bridge: _____ m (ft.)

Channel width ratio (channel width upstream / channel width under the bridge): _____

Overbank flow: ☐ yes ☐ no

Percent of flow in main channel of the approach section:

☐ >90% ☐ 75%-90% ☐ 50%-75% ☐ 25%-50% ☐ <25%

Average bed material size (D_{50}):

@ approach section _____ mm (in) ☐ sample taken for sieve analysis

@ bridge _____ mm (in) ☐ sample taken for sieve analysis

Contraction scour susceptibility rating: ☐ low ☐ medium ☐ high

Comments: _____

ABUTMENT SUSCEPTIBILITYWhich abutment is worst?: ☐ left ☐ right

Observed scour depth: _____ m (ft.) Remaining embedment in river bed: _____ m (ft)

Abutment shape: ☐ vertical ☐ vertical with wingwalls ☐ spillthroughAbutment location: ☐ in channel ☐ at bank ☐ set backAbutment foundation: ☐ unknown ☐ spread footing ☐ pile bent
☐ friction piles ☐ EB piles ☐ set in rockPile type: ☐ metal ☐ concrete ☐ timber ☐ N/A

Pile length: _____ m (feet)

Abutment material: ☐ timber ☐ concrete ☐ metal ☐ stone

Angle of inclination: _____ (degrees)

Primary bed material: ☐ sand ☐ gravel ☐ boulders ☐ manmade
☐ silt/clay ☐ cobble ☐ bedrockAre borings available? ☐ yes ☐ noAbutment protection

Type:	☐ modified	☐ intermediate	☐ standard	☐ slope paving
	☐ concrete	☐ other	☐ absent	☐ none
Permanent or Temporary:	☐ n/a	☐ permanent	☐ temporary	
Condition:	☐ good	☐ weathered	☐ slumped	☐ missing
	☐ fair	☐ poor	☐ N/A	

Abutment exposure due to scour:

☐ none	☐ no exposure	☐ footing exposed	☐ piles exposed
☐ undermining	☐ settlement	☐ failed	

Abutment susceptibility rating ☐ low ☐ medium ☐ high
 Comments: _____

PIER SUSCEPTIBILITY

Worst pier number: _____

Observed scour depth: _____ m (ft.) Remaining embedment in river bed: _____ m (ft)

Angle of attack flood flow: _____ (degrees)

Pier foundation: ☐ unknown ☐ spread footing ☐ pile bent
☐ EB piles ☐ set in rock ☐ friction piles ☐ N/A

Pile type: ☐ metal ☐ concrete ☐ timber ☐ N/A

Pile length: _____

Pier material: ☐ concrete ☐ stone ☐ wood ☐ metal ☐ N/A

Pier shape: ☐ solid pier with square nose ☐ solid pier with round nose
☐ solid pier with sharp nose ☐ column with square nose ☐ column with round nose
☐ column with sharp nose ☐ cylinders/group of cylinders

Pier width: _____ Pier dimensions: _____

Cap/Footing dimensions: _____

Pier exposure due to scour: ☐ none ☐ no exposure ☐ footing exposed
☐ piles exposed ☐ undermining ☐ settlement
☐ failed

Pier protection

Type:	☐ modified	☐ intermediate	☐ standard	☐ slope paving
	☐ concrete	☐ other	☐ absent	☐ none
Permanent or Temporary:	☐ n/a	☐ permanent	☐ temporary	
Condition:	☐ good	☐ weathered	☐ slumped	☐ missing
	☐ fair	☐ poor	☐ N/A	

Primary bed material: ☐ sand ☐ gravel ☐ boulders ☐ manmade
☐ silt/clay ☐ cobble ☐ bedrock

Are borings available? ☐ yes ☐ no

Pier susceptibility rating ☐ low ☐ medium ☐ high

Comments: _____

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HYDRAULIC DATA1) Location

- a) Town(s): State Project No.(s):
- b) Highway: Station(s):
- c) Location Relative to Highway Landmark:
- d) Stream:
- e) Location Relative to Stream Landmark:

2) Design Flood

- a) Hydrologic Procedure Used for Design:
- b) Hydrologic Procedure Used by FEMA:
- c) Drainage Area:
- d) ConnDOT Drainage Manual Structure Classification:
- e) Design Storm Frequency:
- f) Required Underclearance at Design Discharge:
- g) Design Discharge:
 - i. D.O.T. Design:
 - ii. FEMA:
 - iii. SCEL:

3) Hydraulic Analysis Procedure

- a) Model Used and Version No.:
- b) Flow Regime:
- c) Boundary Conditions (starting water surface at the ends of the river system – i.e. known water surface, normal depth, critical depth, rating curve, etc.):

- i. Downstream:
 - ii. Upstream:
 - d) Other Method(s):
- 4) Hydraulic Control (i.e. culvert/bridge, dam (weir), channel construction, tide, known water surface elevation, etc.)
- a) Type of Control:
 - b) Location Relative to Proposed Construction:
- 5) Coefficients of Roughness
- a) Downstream: Channel _____ Overbank _____
 - b) At Crossing: Channel _____ Enclosed Conduit _____
 - c) Upstream: Channel _____ Overbank _____
- 6) Existing Structures
- Upstream:
- a) Type:
 - b) Gross Waterway Opening:
- At Site:
- a) Type:
 - b) Gross Waterway Opening:
 - c) Effective Waterway Opening:
 - d) Overall Width of Waterway Opening:
 - e) Effective Depth of Waterway Opening:
 - f) Minimum Low Chord Elevation:
 - g) Minimum Roadway Elevation:

- h) Computed Water Surface Elevation at Approach Section Upstream of Structure at Design Discharge:
- i) Underclearance at Design Discharge:
- j) Mean Velocity of Channel:

Downstream:

- a) Type:
- b) Gross Waterway Opening:

7) Proposed Structure

- a) Type:
- b) Gross Waterway Opening:
- c) Effective Waterway Opening:
- d) Overall Width of Waterway Opening:
- e) Effective Depth of Waterway Opening:
- f) Minimum Low Chord Elevation:
- g) Minimum Roadway Elevation:
- h) Computed Water Surface Elevation at Approach Section Upstream of Structure at Design Discharge:
- i) Maximum Regulatory Elevation:
- j) Other Controlling Water Surface Elevation (If Below Maximum Regulatory Elev.):
- k) Difference in Water Surface Elevation (Approach Section) Proposed vs. Existing and Proposed vs. Regulatory @ Design Discharge:
- l) Underclearance at Design Discharge with Respect to Structure Low Chord:
- m) Mean Velocity Through Structure:

8) Remarks

a) Navigational Requirements:

b) Tidal Conditions:

c) Record Floods:

d) Average Daily Flow:

$$Q_{AD}(cms) = [A (km^2)]^{0.98} * 0.0208$$

$$(Q_{AD}(cfs) = [A (sm)]^{0.98} * 1.87)$$

e) Average Spring Flow:

$$Q_{AS}(cms) = [A (km^2)]^{0.988} * 0.04$$

$$(Q_{AS}(cfs) = [A (sm)]^{0.988} * 3.62)$$

f) Flood Hazard Zone:

g) Vertical Datum:

Appendix B – Amended Local Abutment Scour Equations for Connecticut

The HEC-18 recommends the use of either Froehlich's or the HIRE abutment scour equations. The HIRE equation was developed from ACOE data relative to scour at the end of spur dikes on the Mississippi River. It is only valid when the ratio of projected abutment length (L') to flow depth (Y_a) is greater than 25. The vast majority of watercourses in Connecticut do not meet this criteria, and therefore require the use of Froehlich's equation.

Froehlich developed equations for both live bed and clear water contraction scour, however, HEC-18 recommends the use of the live bed scour equations for all evaluations. The formula is as follows:

$$Y_s / Y_a = 2.27 K_1 K_2 \left(\frac{L'}{Y_a} \right)^{0.43} Fr^{0.61} + 1$$

where:

K_1 = coefficient for abutment shape (See Table 6, Section 4.3.6 in HEC-18 Third Ed., dated Nov. 95)

K_2 = coefficient for angle of embankment to flow (Refer to Section 4.3.6, Figure 16 in HEC-18, Third Ed., dated Nov. 95)

L' = the length of abutment projected normal to flow, m (ft)

Y_a = average depth of flow in the floodplain = A_e / a' , m (ft)

A_e = the flow area of the approach cross section obstructed by the embankment, m^2 (ft^2)

Fr = the Froude Number = $V_e / (g y_a)^{0.5}$

V_e = Q_e / A_e , m/s (ft/s)

Q_e = the flow obstructed by the abutment and approach embankments, m^3/s (ft^3/s)

Y_s = scour depth, m (ft)

One of the chief contributors to the conservative nature of the abutment scour prediction is the L' / Y_a parameter. Another contributor, however, is the +1 figure on the right side of the equation. This value, intended as a factor of safety (F.S.), was not in Froehlich's original paper, and essentially increases the predicted scour depth by the depth of the overbank flow. This is based upon the fact that, when F.S. =1 is used, 98% of the laboratory scour holes would be less than those predicted. Therefore the Froehlich equation, as presented in the HEC-18 document, defines the upper limit of predicted scour depths. Given the conservative nature of the laboratory data in comparison to field conditions, the use of the +1 value on the right side of the abutment scour equation appears to create an unwarranted compounding of safety factors (particularly in pile supported structures where additional factors of safety are incorporated in the stability

analysis). Therefore, with due consideration given to the application of engineering judgement in the determination of predicted scour depths, the +1 factor of safety will not be applied in the computation of local scour depths. Rather, a factor of 0.05 will be used and the predicted scour depth will be termed the *amended scour depth*. This will place the predicted scour values at the 50th percentile based upon the results of laboratory scour studies. **The amended local abutment scour equation is, then:**

$$Y_s / Y_a = 2.27 K_1 K_2 \left(\frac{L'}{Y_a} \right)^{0.43} Fr^{0.61} + 0.05$$

The amended scour depths as computed by this equation, together with all other relevant factors, shall be considered.

GUIDELINES FOR APPLYING ENGINEERING JUDGEMENT WITH REGARD TO PREDICTED SCOUR AT ABUTMENTS FOR EXISTING BRIDGES.

Upon establishing the total scour depth, the following guidelines shall be applied to address the conservative nature of the abutment scour equations as related to the recommended NBIS Item 113 rating for the bridge in question:

Abutments on Spread Footings

- Should the total abutment scour depths as modified through the application of engineering judgement for the bridge indicate that the footings for the structure will be undermined for any of the flood events studied and the field documentation supports the presence, history or high potential for scour at the abutment(s), then a scour critical rating should be recommended.
- Should the total amended scour depths for the bridge be less than the actual embedment depths to the tops of the footings at the structure, and field documentation indicates that the bridge is not scour susceptible, then the structure should be rated low risk and assigned an NBIS Item 113 rating of 8.
- If countermeasures have apparently been installed in accordance with HEC-18 guidelines, and the countermeasures have survived at least one major flood event without showing signs of deterioration, then the bridge should be assigned an NBIS Item 113 rating of 7. (Refer to Chapter 6, Appendix E for Historical Floods of Record).
- Should the predicted scour depths indicate that the bridge will be unstable for any of the events studied and the field documentation does not support a scour critical rating, then engineering judgement should be applied to resolve the discrepancy between the predicted scour and the field evidence. This judgement shall consider, but shall not be limited to the following factors: armoring potential of the streambed, vegetated cover of the overbank areas, evidence of scour at the structure (or lack thereof), angle of inclination, hydraulic adequacy of the structure, flow distribution in the approach section and the age of the structure with respect to floods of record. Engineering judgement for this scenario are as follows:

Spread footings with less than 1.2m (4 ft.) of embedment

Should the field documentation show that the bridge does not appear to be scour critical, then the structure may be rated as scour susceptible and assigned an NBIS Item 113 rating of 5.

Spread footings with more than 1.2m (4 ft.) of embedment

Should the field documentation show that the bridge does not appear to be scour critical, then the structure may be designated as scour susceptible or low risk based on engineering judgement.

Abutments on Pile Foundations

- Should the total amended scour depths determined for the bridge indicate that the piles will be unstable for the flood events studied and the field documentation supports the presence, history or high potential for scour at the abutment(s), then the bridge should be rated scour critical and the NBIS Item 113 rating of 3.
- Should the foundation for the bridge be determined to be stable at the modified scour depths for the structure, and the field documentation indicates that the bridge is not scour susceptible, then the structure should be rated low risk.
- If countermeasures have apparently been installed in accordance with HEC-18 guidelines, and the countermeasures have survived at least one major flood event without showing signs of deterioration, then the bridge should be assigned an NBIS Item 113 rating of 7. (Refer to Chapter 6, Appendix E for Historical Floods of Record).
- Should the amended scour depths indicate that the bridge will be unstable for the flood events studied, and the field documentation does not support a scour critical rating, then engineering judgement should be applied to resolve the discrepancy between the predicted scour and the field evidence. This judgement shall consider, but shall not be limited to the following factors: armoring potential of the streambed, vegetated cover of the overbank areas, evidence of scour at the structure (or lack thereof), angle of inclination, hydraulic adequacy of the structure, flow distribution in the approach section and age of the structure relative to floods of record. Should the field documentation show that the bridge does not appear to be scour critical, then the bridge may be designated as scour susceptible or low risk based on engineering judgement.

Appendix C – Bridge Scour Evaluation Report Format

All Bridge Scour Evaluation Reports, either for proposed or existing structures, shall be presented using the following format:

A. Table of Contents

B. Executive Summary – The following items must be included:

1. A brief description of the report findings as well as the engineer's recommendations regarding scour countermeasures or countermeasure design.
2. Executive Summary Table containing the items listed below.
 - a. Recommended NBIS Item 113 Rating (Scour Critical Bridges)
 - b. Recommended NBIS Item 71 Rating (Waterway Adequacy)
 - c. Recommended NBIS Item 61 Rating (Channel and Channel Protection)
 - d. Scour Risk Designation (Low Risk, Scour Susceptible or Scour Critical)
 - e. Depth of Potential Scour (Provide the range of values computed for the various flood events analyzed)
 - f. Foundation Type (Known/Unknown)
 - g. Recommendation(s) (Monitor, Install Countermeasures or Design Foundation for Predicted Scour)
3. The signature with Connecticut seal and professional engineer number of the approved hydraulic engineer who prepared or supervised the preparation of the report.

C. Background/Site Conditions – Provide a narrative description of the existing structure (if applicable), the stream reach adjoining the bridge site and any other relevant information obtained from data gathering efforts.

D. Hydrology and Hydraulics – Provide a description of the watershed properties, hydrologic methods used in the determination of peak flows and a tabulation of the maximum flow rates for the various return frequencies. At a minimum, the 10, 50, 100 and 500 year floods shall be presented for scour evaluations of existing bridges. With respect to new bridges, it is normally acceptable to evaluate only the 100 and 500 year floods unless a flood of lesser magnitude is the maximum scour-producing event.

With respect to the hydraulic analysis, a description of the program employed to develop design water surface profiles, flow depths and velocities should be provided. Further, methodologies used in the determination of the starting water surface elevations or boundary conditions must be described.

E. Scour Results – Describe the findings of the scour evaluation in narrative and tabular formats.

F. Structural Review/Foundation Stability Analysis – Provide a narrative description, as appropriate.

G. Conclusions and Recommendations – Summarize the findings of the Bridge Scour Evaluation and provide recommendations with respect to countermeasure or foundation design.

H. Report Graphics

1. Location Plan
2. Site Plan
3. Scour Depth Cross Sections – For each flood event analyzed, provide a cross section (Elevation View) at the upstream face of the bridge on which the various components of total scour have been depicted for all substructure units. Where foundation information is available, the depth and configuration shall also be depicted. This section must be drawn to scale and must indicate the design flood elevation, the low chord elevation and the overtopping elevation.

I. Technical Appendices

1. Field Evaluation Notes or Sketches (as appropriate)
2. Photographs
3. Hydrologic Computations
4. Water Surface Profile Computations
5. Scour Calculations
6. Geotechnical Data – Riverbed and soil sample characteristics and/or subsurface investigation findings
7. Countermeasure Design Computations and Sketches (as appropriate)
8. Pile Stability Computations (as appropriate)

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10.1 Introduction

10.1.1 Overview

The traditional design of storm drainage systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As areas urbanize this type of design may result in major drainage and flooding problems downstream. The engineering community is now more conscious of the quality of the environment and the impact that uncontrolled increases in runoff can have on adjoining properties and the environment. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream flows and often the cost of the downstream conveyance system. Detention storage facilities can range from small facilities contained in parking lots or other on-site facilities to large lakes and reservoirs. This chapter provides general design criteria for detention/retention storage basins as well as procedures for performing preliminary and final sizing and reservoir routing calculations. Storage and flood routing associated with culverts is addressed in Chapter 8, Culverts (note: criteria in this chapter do not necessarily apply to routine culvert design).

10.1.2 Location Considerations

It should be noted that the location of storage facilities is very important as it relates to the effectiveness of these facilities to control downstream flooding. Small facilities will only have minimal flood control benefits and these benefits will quickly diminish as the flood wave travels downstream. Multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system which could decrease or increase flood peaks in different downstream locations. Thus it is important for the engineer to design storage facilities as a drainage structure that both controls runoff from a defined area and interacts with other drainage structures within the drainage basin. Effective stormwater management must be coordinated on a regional or basin-wide planning basis.

10.1.3 Detention And Retention

Urban stormwater storage facilities are often referred to as either detention or retention facilities. For the purposes of this chapter, detention facilities are those that are designed to reduce the peak discharge and only detain runoff for some short period of time. These facilities are designed to completely drain after the design storm has passed. Recharge basins are a special type of detention basin designed to drain into the groundwater table. See Section 10.13 for further guidance on the design of infiltration (recharge) basins and trenches. Retention facilities are designed to contain a permanent pool of water. Since most of the design procedures are the same for detention and retention facilities, the term storage facilities will be used in this chapter to include detention and retention facilities. If special procedures are needed for detention or retention facilities these will be specified.

Storage facilities may be small in terms of storage capacity and dam height when serving a single outfall from a watershed of a few hectares (acres) or they may be larger facilities serving as regional flood control. While the same principles apply to all storage facilities, Sections 10.11 and 10.12 more specifically relate to the smaller installations.

10.1.4 Computer Programs

Routing calculations needed to design storage facilities, although not extremely complex, are time consuming and repetitive. To assist with these calculations there are many available reservoir routing computer programs. Also, the storage indication method can be used which makes calculations simple. All storage facilities shall be designed and analyzed using reservoir routing calculations. (See Chapter 6, Hydrology.)

10.2 Uses

10.2.1 Introduction

The use of storage facilities for stormwater management has increased dramatically in recent years. The benefits of storage facilities can be divided into two major control categories of quality and quantity.

10.2.2 Quality

Control of stormwater quality using storage facilities offers the following potential benefits:

- decrease downstream channel erosion
- control sediment deposition
- improve water quality through stormwater filtration (wet ponds only)

10.2.3 Quantity

Controlling the quantity of stormwater using storage facilities can provide the following potential benefits:

- prevention or reduction of peak runoff rate increases caused by urban development
- mitigation of downstream drainage capacity problems
- recharge of groundwater resources
- reduction or elimination of the need for downstream outfall improvements
- maintenance of historic low flow rates by controlled discharge from storage

10.2.4 Objectives

The objectives for managing stormwater quantity by storage facilities are typically based on limiting peak runoff rates to match one or more of the following values:

- historic rates for specific design conditions (i.e., post-development peak equals pre-development peak for a particular frequency of occurrence)
- non-hazardous discharge capacity of the downstream drainage system
- a specified value for allowable discharge set by a regulatory jurisdiction

10.2.5 Downstream Effects

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.

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%. **However, it is important to be aware that the increased total volumes of water being released slowly over a longer period of time may contribute to bed and bank decay in the receiving channel.**

10.3 Symbols And Definitions

To provide consistency within this chapter as well as throughout this manual, the following symbols will be used. These symbols were selected because of their wide use in technical publications. In some cases the same symbol is used in existing publications for more than one definition. Where this occurs in this chapter, the symbol will be defined where it occurs in the text or equations.

Table 10-1 Symbols And Definitions

<u>Symbol</u>	<u>Definition</u>	<u>Units</u>
A	Cross sectional or surface area	m ² (ft ²)
C	Weir coefficient	—
d	Change in elevation	m (ft)
D	Depth of basin or diameter of pipe	m (ft)
f	Infiltration rate	mm/h (in/h)
g	Acceleration due to gravity	m/s ² (ft/s ²)
H	Head on structure	m (ft)
H _c	Height of weir crest above channel bottom	m (ft)
I	Infiltration rate	mm/h (in/h)
I	Inflow rate	m ³ /s (cfs)
L	Length	m (ft)
Q, Q _o	Flow or outflow rate	m ³ /s (cfs)
S, V _s	Storage volume	m ³ , ha-m (ft ³ , acre-ft)
t	Routing time period	s
t _b	Time base on hydrograph	h
t _i	Duration of basin inflow	h
t _p	Time to peak	h
V _s , S	Storage volume	m ³ (acre ft)
W	Width of basin	m (ft)
z	Side slope factor	—

10.4 Design Criteria

10.4.1 General Criteria

Storage may be concentrated in large basin-wide or regional facilities or distributed throughout an urban drainage system. Storage may be developed in depressed areas in parking lots, road embankments and freeway interchanges, parks and other recreation areas, and small lakes, ponds and depressions within urban developments. The utility of any storage facility depends on the amount of storage, its location within the system and its operational characteristics. An analysis of such storage facilities should consist of comparing the pre-existing discharges with the proposed design flow at a point or points downstream of the proposed storage site with and without storage. In addition to the design flow, other flows in excess of the design flow that might be expected to pass through the storage facility should be included in the analysis. The design criteria for storage facilities should include:

- release rate
- storage volume
- grading and depth requirements
- outlet works and location
- evaluation of the increase of total volume of runoff on downstream facilities

10.4.2 DEP Requirements

The following standards regarding stormwater management and stormwater detention facilities taken from Section 25-68h-3(a) and (b) of the Flood Management Statutes and Administrative Regulations shall be incorporated into all ConnDOT projects:

Sec. 25-68h-3. Stormwater management standards

(a) On-site stormwater management.

(1) The stormwater management plans for state activities shall be prepared so as to minimize any adverse increases to the peak flow rate, the timing of runoff and the volume of runoff. Hydrology studies shall be conducted at a level of detail commensurate with the probable impact of the project.

(A) A complete runoff hydrograph evaluation is required for (i) Basin Stormwater Management Plans pursuant to Section 25-68h-3 (h), (ii) Stormwater management plans for project sites resulting in significant impacts, and (iii) other state activities and critical activities as determined by the Commissioner. Hydrograph evaluations shall be conducted for existing and anticipated land use conditions for storms with average return frequencies of 2, 10 and 100 years. Where appropriate, the hydrograph analysis shall include determination of runoff for each subwatershed and routing runoff through storage impoundments and floodplain storage areas. The timing sequence of the runoff must be fully developed.

(B) Where suitable records exist, hydrographs should be developed from historic gauged flood data. For other watercourses, the hydrographs shall be developed from deterministic rainfall-runoff techniques and compared with flood flows of similar gauged watershed and an assessment made as to the need to calibrate the hydrograph based on this comparison.

- (2) Stormwater management plans for project sites shall be coordinated with Basin Stormwater Management Plans, where available.
- (b) Stormwater detention facilities
Facilities to temporarily store excess storm runoff shall be subject to the following requirements:
 - (1) Any detention facility whose failure could cause significant damage or loss of life shall be regulated as a dam pursuant to Sections 22a-401 through 22a-409 of the General Statutes.
 - (2) All detention facilities serving a watershed larger than 10 acres in size shall be analyzed with hydrograph and storage routing techniques.
 - (3) The release rates from detention facilities shall be consistent with the Basin Stormwater Management Plan for the watershed in which it is located, or comply with items 4, 5 and 6 below if there is no Basin Stormwater Management Plan.
 - (4) The release rate shall consider the existing and proposed flow rates at the site and downstream channels or structures, and the timing of runoff from other subwatersheds within the basin for the base flood.
 - (5) The waters released from a detention facility shall not increase the peak flow rate at offsite downstream points unless they have adequate flow capacity for the base flood.
 - (6) Extended duration detention facility discharges directly into alluvial or eroding channels shall not exceed the bankfull capacity or the 2 year flood frequency flow, whichever is less, unless it is determined said channel will be stable.
 - (7) Section 5-9 of the "Connecticut Guidelines for Erosion and Sediment Control" (2002) as may be amended, shall be used as a guide to construction details and materials.
 - (8) An operation and maintenance schedule shall be prepared for every detention facility identifying responsibilities and items of routine maintenance, after use and emergency operations in the event of a flood.

10.4.3 Release Rate

Control structure release rates shall approximate pre-developed peak runoff rates for the 2, 10, and 25-year storms, with emergency overflow capable of handling the 100-year discharge. If the basin is designed to enhance stormwater quality, as well as to control peak outflow rates, additional requirements as defined in Section 10.11 will apply. Design calculations are required to demonstrate that initial runoff and flow from the 2- and 10-year design storms are controlled for single stage structures. If so, runoff from intermediate storm return periods can be assumed to be adequately controlled. Multi-stage control structures may be required to control both runoff from the 2, 10, and 25-year storms.

10.4.4 Storage

Storage volume shall be adequate to attenuate the post-development peak discharge rates to pre-developed discharge rates for the 2, 10 and 25-year storms or other design storms depending on the downstream system design capacity. Routing calculations must be used to demonstrate that the storage volume is adequate. If sedimentation during construction causes loss of detention volume, design dimensions shall be restored before completion of the project. For detention basins, all detention volume shall be drained within the average period between storm events ordinarily 72 h.

10.4.5 Grading And Depth

Following is a discussion of the general grading and depth criteria for storage facilities followed by criteria related to detention and retention facilities.

- **General**

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Vegetated embankments shall be less than 6 m (20 ft) in height and shall have side slopes no steeper than 1V:4H (vertical to horizontal). Side slopes shall be benched at intervals of 1.5 m (5 ft). Riprap-protected embankments shall be no steeper than 1V:2H. Geotechnical slope stability analysis is required for embankments greater than 3 m (10 ft) in height and is mandatory for embankment slopes steeper than those given above and should be performed by a qualified Geotechnical Engineer.

A minimum freeboard of 0.3 m (1 ft) above the 100-year water surface elevation shall be provided for impoundments. Certain impoundment depths and volumes may classify the embankment structure as a dam which are governed by federal and state regulations (see Section 10.5, Dams).

Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements and required freeboard. Aesthetically pleasing features are also important. Fencing of basins is addressed in Section 10.16.

- **Detention**

Areas above the normal high water elevations of storage facilities should be sloped at a minimum of 5% toward the facilities to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2% bottom slope is recommended. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows and prevent standing water conditions wherever possible. In dual purpose basins (quality and quantity control) a sediment forebay should also be included in the design.

- **Retention**

The maximum depth of permanent storage facilities will be determined by site conditions, design constraints, and environmental needs. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage growth of weeds (without creating undue potential for anaerobic bottom conditions) should be considered. A depth of 1.5m (5 ft) to 3m (10 ft) is generally reasonable unless fishery requirements dictate otherwise. Aeration may be required in permanent pools to prevent anaerobic conditions. Where aquatic habitat is required, the appropriate wildlife experts should be contacted for site specific criteria relating to such things as depth, habitat, and bottom and shore geometry.

10.4.6 Outlet Works

Outlet works selected for storage facilities typically include a principal spillway and an emergency overflow, and must be able to accomplish the design functions of the facility. Outlet works 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. 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 outlet is the 100-year flood. The sizing of a particular outlet works shall be based on results of hydrologic routing calculations.

10.4.7 Location

To minimize design costs and to obtain regulatory approval, storage facilities should be designed to avoid inland wetlands and watercourses. A local or state inland wetlands permit will be required if a facility is proposed in a wetland area. A local inland wetland permit may also be required if the storage facility is located in the upland review area established by municipal regulation.

A state water diversion permit from the DEP's Inland Water Resources Division will be required if the contributing drainage area to the storage facility is greater than 40.5 hectares (100 acres). If the contributing drainage is less than 40.5 hectares (100 acres) and no inland wetlands or watercourses are involved then a diversion permit will not be required. However, if wetlands and/or watercourses or floodplains are involved, a diversion permit may be required and a permit need determination should be sought from the DEP.

On-site detention may be undesirable when the site is located in the lower portions of a watershed before the confluence with a perennial watercourse. If detention is located in the lower reaches of a watershed, there is a risk, depending on the size and release rate of the basin, that the peak flows from the outlet control structure could combine with the peak flows from the upper reaches, thus increasing peak flows or sustaining peak flows over longer periods. Likewise, the hydrographs of several storage facilities located within a particular watershed when combined may increase or sustain peak flows. These increases could result in prolonged flooding and channel erosion along and within the perennial stream course downstream of the site.

To avoid this problem, a hydrologic analysis is required by an engineer. Locate the area downstream that is to be targeted for protection from additional runoff. A target area might be a flood prone road crossing, eroding stream bank or reach of stream where homes are currently endangered. Delineate the watershed to the targeted area. Determine where the proposed detention basin is located within that watershed. Conduct a hydrograph analysis to determine the timing of peak discharges. Use the Natural Resources Conservation Service (NRCS) Technical Releases 20 or 55, U.S. Army Corps of Engineers HEC-1 or other appropriate methods which produce hydrographs to evaluate existing and post-development conditions.

If the hydrograph analysis shows that detention is detrimental to the target area, but nearby downstream concerns are present, other methods to decrease peak flows from the site will need to be utilized.

10.5 Dams

10.5.1 Background

National responsibility for the promotion and coordination of dam safety lies with the Federal Emergency Management Agency (FEMA) as enacted by the National Dam Safety Program Act. State responsibility for administration of the dam safety criteria lies with Connecticut Department of Environmental Protection (DEP).

10.5.2 National Dam Safety Program Act

The National Dam Safety Program Act has as its mission to "reduce the risks to life and property from dam failure in the United States through the establishment and maintenance of an effective national dam safety program to bring together the expertise and resources of the federal and non-federal communities in achieving national dam safety hazard reduction."

10.5.3 State Regulations

Rules and regulations relating to applicable dams are authorized by Connecticut General Statute 22a-401 through 411.

Under Connecticut statutes, DEP regulates all dams, dikes, or similar structures, where failure may endanger life or property.

Dams are classified as either new or existing, by hazard potential, and by height for regulatory purposes. Hazard potential categories are listed in the ConnDEP/Bureau of Water Management, Regulations Concerning Dam Safety Inspection (RCSA Section 22a-409-2).

The DEP also regulates all dam construction within the state. Contact DEP Inland Water Resources Division early in the planning process to determine the potential need for a dam construction permit pursuant to CGS Section 22a-403.

In accordance with CGS-22a-403 inland wetland, diversion and SCEL permits are **NOT** required to alter, repair or remove an existing dam if a dam permit is required. However, the construction of a new dam which requires a dam safety permit is exempt from inland wetland permitting but not exempt from diversion or SCEL permitting requirements.

Information and regulations that should be consulted for further details and engineering requirements are given in the Environmental Permit Application Package instructions for completing a permit application for programs administered by the Inland Water Resources Division, ConnDEP.

10.6 Preliminary Computations

10.6.1 Data Required

The final design of a detention facility requires three items. They are an inflow hydrograph (see Chapter 6, Hydrology), a stage vs. storage curve, and a stage vs. discharge curve (sometimes called a performance curve). However, before a stage vs. storage and a stage vs. discharge curve can be developed, a preliminary estimate of the needed storage capacity and the shape of the storage facility are required. Trial computations will be made to determine if the estimated storage volume will provide the desired outflow hydrograph.

10.6.2 Estimating Required Storage

Estimating the required volume of storage to accomplish the necessary peak reduction is an important task since an accurate first estimate will reduce the number of trials involved in the routing procedure. The following sections present four (4) methods for determining an initial estimate of the storage required to provide a specific reduction in peak discharge. **All of the methods presented provide preliminary estimates only.** It is recommended that the designer apply several of the methods and a degree of judgement to determine the initial storage estimate.

10.6.3 Hydrograph Method

To work any storage problem, the inflow hydrograph must be provided and the release rate must be assigned. With these values established, the detention basin discharge curve can be estimated and then sketched as shown in Figure 10-1. The shaded area between the curves represents the estimated storage that must be provided. To determine the necessary storage, the shaded area can be planimetered or computed mathematically.

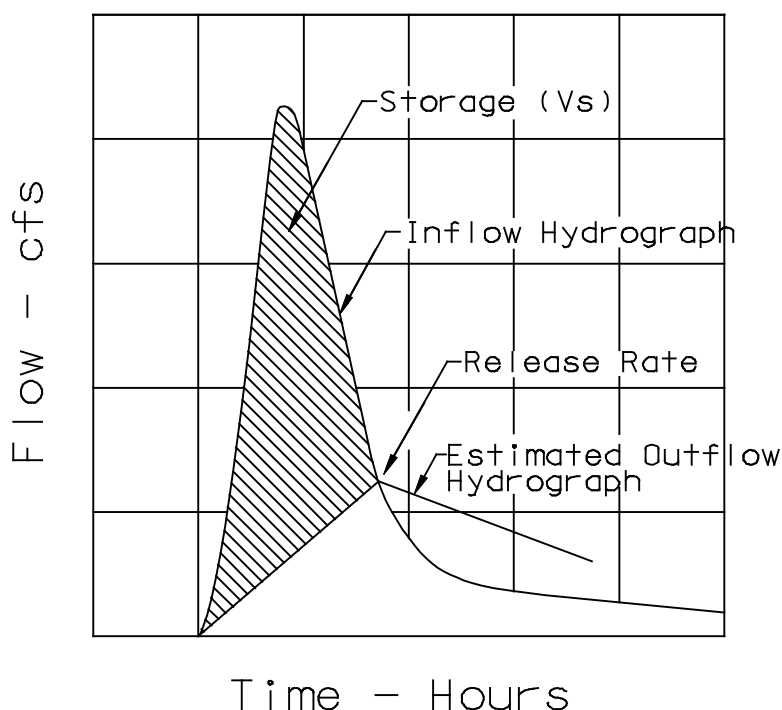


Figure 10-1 Estimating required storage hydrograph method

10.6.4 Triangular Hydrograph Method

A preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with standard triangular shapes. This method should not be applied if the hydrographs can not be approximated by a triangular shape. This would introduce additional errors of the preliminary estimate of the required storage. The procedure is illustrated by Figure 10-2. The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph as defined by equation 10.1.

$$V_s = 0.5 t_i (Q_i - Q_o) \quad (10.1)$$

where: V_s = storage volume estimate, m^3 (ft^3)
 Q_i = peak inflow rate into the basin, m^3/s (ft^3/s)
 Q_o = peak outflow rate out of the basin, m^3/s (ft^3/s)
 t_i = duration of basin inflow, s
 t_p = time to peak of the inflow hydrograph, s

The duration of basin inflow should be derived from the estimated inflow hydrograph. The triangular hydrograph procedure, originally described by Boyd, was found to compare favorably with more complete design procedures involving reservoir routing.

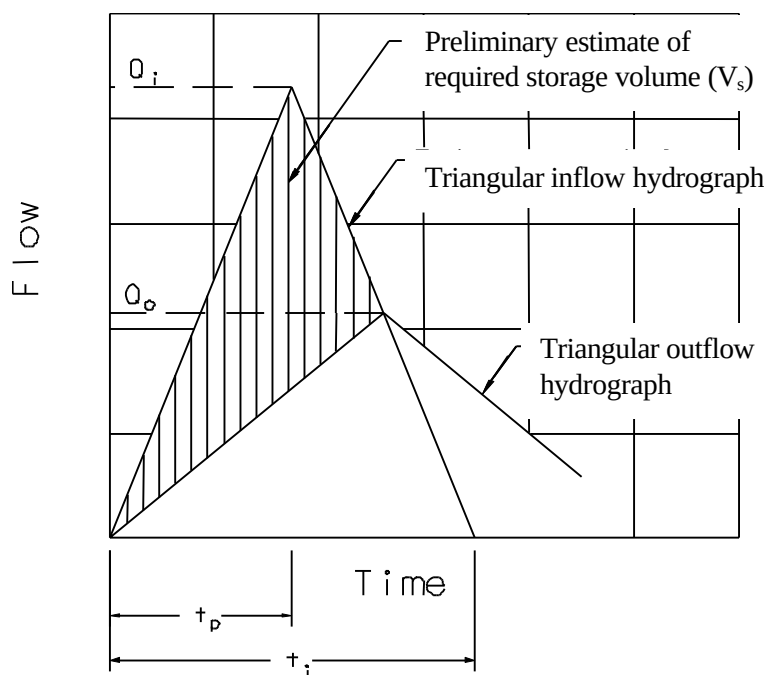


Figure 10-2 Triangular hydrograph method

10.6.5 NRCS Procedure

The NRCS, in its TR-55 Second Edition Report, describes a manual method for estimating required storage volumes based on peak inflow and outflow rates. The method is based on average storage and routing effects observed for a large number of structures. A dimensionless figure relating the ratio of basin storage volume (V_s) to the inflow runoff volume (V_r) with the ratio of peak outflow (Q_o) to peak inflow (Q_i) was developed as illustrated in Figure 10-3. **This procedure for estimating storage volume may have errors up to 25% and, therefore, should only be used for preliminary estimates.**

The procedure for using Figure 10-3 in estimating the detention storage required is described as follows:

1. Determine the inflow and outflow discharges Q_i and Q_o .
2. Compute the ratio Q_o/Q_i .
3. Compute the inflow runoff volume, V_r , for the design storm.

$$V_r = K_r Q_D A_m \quad (10.2)$$

where: V_r = inflow volume of runoff, ha-mm (ac-ft)
 K_r = 1.00 (53.33 for english units)
 Q_D = depth of direct runoff, mm (in)
 A_m = area of watershed, ha (mi^2)

4. Using Figure 10-3, determine the ratio V_s/V_r .
5. Determine the storage volume, V_s , as

$$V_s = V_r \left(\frac{V_s}{V_r} \right) \quad (10.3)$$

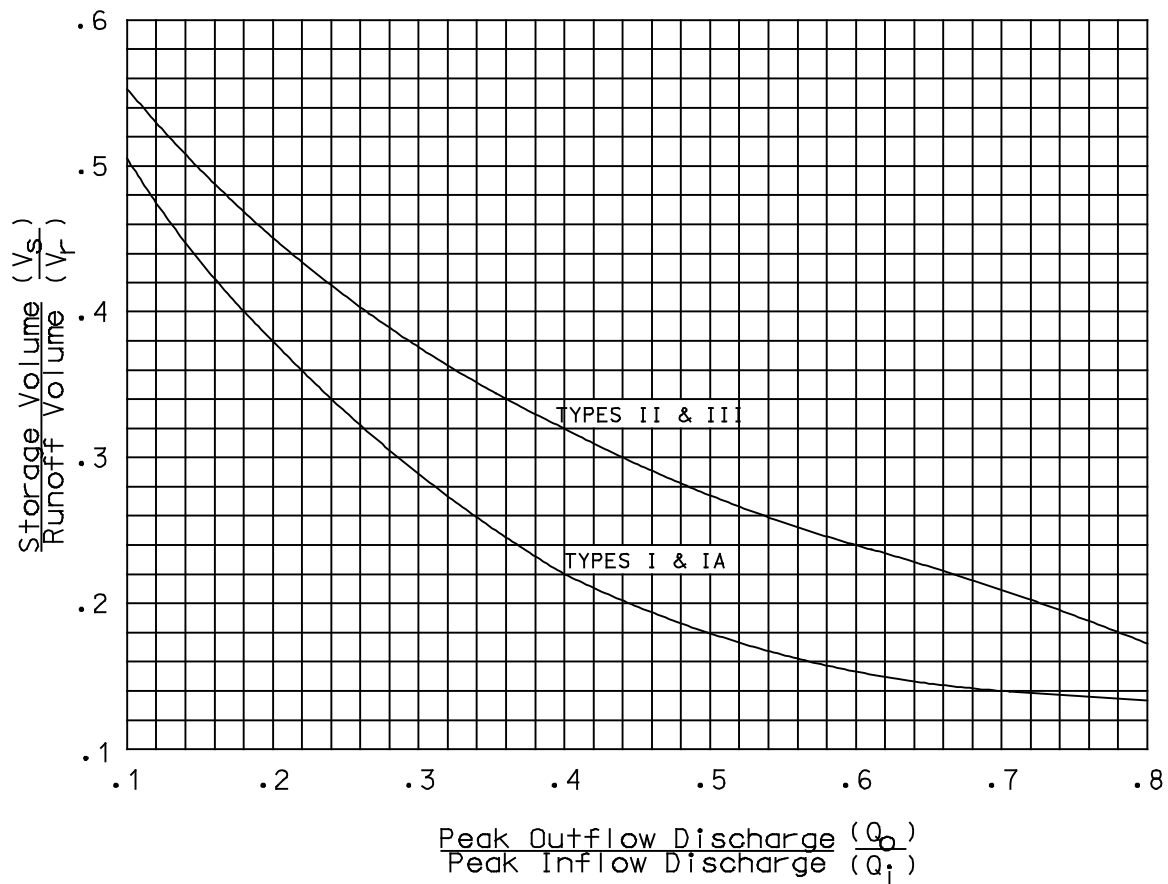


Figure 10-3 NRCS Detention Basin Routing Curves

10.6.6 Regression Equation

An estimate of the storage volume required for a specified peak flow reduction can be obtained by using the following regression equation procedure first presented by Wycoff & Singh in 1986.

1. Determine the volume of runoff in the inflow hydrograph (V_i), the allowable peak outflow rate (Q_o), the time base of the inflow hydrograph (t_i), and the time to peak of the inflow hydrograph (t_p).
2. Calculate a preliminary estimate of the ratio V_s/V_r using the input data from step 1 and the following equation:

$$\left(\frac{V_s}{V_r} \right) = \left[1.291 (1 - Q_o / Q_i)^{0.753} \right] / \left[(t_i / t_p)^{0.411} \right] \quad (\text{metric only}) \quad (10.4)$$

3. Multiply the inflow hydrograph volume (V_i) times the volume ratio computed from equation 10.4 to obtain an estimate of the required storage volume.

The following example problem demonstrates the use of some of these storage volume estimation methods.

Example 10-1

Given: The post-developed (improved conditions) hydrograph and a limiting outflow rate from the proposed detention facility of $0.55 \text{ m}^3/\text{s}$ ($19.4 \text{ ft}^3/\text{s}$). This limiting outflow is a constraint imposed by the downstream receiving water course and is the maximum outflow rate from the drainage area for unimproved conditions.

Find: The estimated required storage of a detention facility by using the:

- (1) Hydrograph Method
- (2) Triangular hydrograph method
- (3) SCS Procedure
- (4) Regression Equation

Solution: (1) - Hydrograph Method

Figure 10-4 illustrates the existing conditions and proposed conditions hydrographs. Assuming the proposed detention facility should produce an outflow hydrograph similar to existing conditions, the required detention volume is determined as the area above the existing hydrograph and below the proposed hydrograph. Planimetering this area yields an area of 9 square centimeters (1.4 square inches), which converts to the following volume:

$$V_s = (9 \text{ cm}^2) (109 \text{ m}^3/\text{cm}^2) = 980 \text{ m}^3 (34,580 \text{ ft}^3)$$

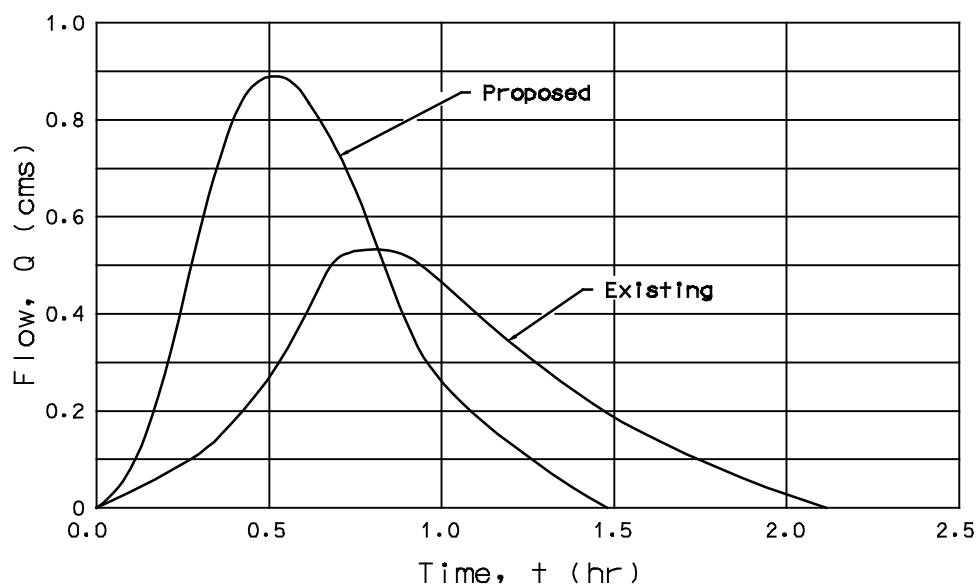


Figure 10-4 USGS Nationwide Urban Hydrograph for Existing (Unimproved) and Proposed (Improved) Conditions

Solution: (2) - Triangular hydrograph method

The duration of basin inflow (t_i) is 1.43 hours (5148 seconds) and the inflow rate into the detention basin (Q_i) is $0.88 \text{ m}^3/\text{s}$ ($31.1 \text{ ft}^3/\text{s}$). Due to a local ordinance, the peak flow rate out of the basin (Q_o) is set to be $= 0.55 \text{ m}^3/\text{s}$ ($19.4 \text{ ft}^3/\text{s}$).

Using equation 10.1, the initial storage volume is computed as:

$$V_s = 0.5 t_i (Q_i - Q_o)$$

$$V_s = (0.5)(5148)(0.88 - 0.55) = 849 \text{ m}^3 (29980 \text{ ft}^3)$$

Solution: (3) - SCS Procedure

Step 1. The inflow discharge is given to be $0.88 \text{ m}^3/\text{s}$ ($31.1 \text{ ft}^3/\text{s}$), and the outflow discharge is set to be $= 0.55 \text{ m}^3/\text{s}$ ($19.4 \text{ ft}^3/\text{s}$) by local ordinance.

Step 2. The ratio of basin inflow to basin outflow is:

$$Q_o / Q_i = 0.55 / 0.88 = 0.63$$

Step 3. The inflow runoff (V_r) is computed using equation 10.2. The depth of direct runoff (Q_D) is given to be 11 mm (0.4 in) and the area of the basin is 17.55 ha (43.37 ac).

$$V_r = K_r Q_D A_m$$

$$V_r = (1.00)(11)(17.55) = 193 \text{ ha-mm}$$

$$V_r = (193 \text{ ha-mm})(10,000 \text{ m}^2/\text{ha})(1\text{m}/1000\text{mm}) = 1930 \text{ m}^3 (68160 \text{ ft}^3)$$

Step 4. With $Q_o / Q_i = 0.63$ and a Type III Storm, use Figure 10-3 to determine V_s / V_r .

$$V_s / V_r = 0.23$$

Step 5. The preliminary estimated storage volume (V_s) is determined from equation 10.3:

$$V_s = V_r (V_s / V_r) = (1930)(0.23)$$

$$V_s = 444 \text{ m}^3 (15680 \text{ ft}^3)$$

Solution: (4) - Regression Equation

Step 1. From solution 3, the volume of direct runoff (V_r) is 1930 m³. The peak outflow rate (Q_o) is 0.55 m³/s and, the time base of the inflow hydrograph (t_i) is given to be 1.43 hours and the time to peak of the inflow hydrograph (t_p) is 0.51 hours.

Step 2. Using 10.4, the ratio V_s / V_r is:

$$V_s / V_r = [1.291 (1 - Q_o/Q_i)^{0.753}] / [(t_i / t_p)^{0.411}]$$

$$V_s / V_r = [(1.291) \{1 - (0.55/0.88)\}^{0.753}] / [(1.43/0.51)^{0.411}]$$

$$V_s / V_r = 0.40$$

Step 3. The estimated storage is:

$$V_s = V_r (V_s / V_r) = (1930)(0.40)$$

$$V_s = 772 \text{ m}^3 (27260 \text{ ft}^3)$$

The hydrograph, triangular hydrograph, and regression methods result in the most consistent estimates.

10.6.7 Estimating Peak Flow Reduction

Similarly, if a storage volume is known and you want to estimate the peak discharge, two methods can be used. First, the TR-55 method as demonstrated in Figure 10-3 can be solved backwards for the ratio of Q_o/Q_i . Secondly, a preliminary estimate of the potential peak flow reduction can be obtained by rewriting the regression equation 10.4 in terms of discharges. This use of the regression equations is demonstrated below.

1. Determine the volume of runoff in the inflow hydrograph (V_r), the peak flow rate of the inflow hydrograph (Q_i), the time base of the inflow hydrograph (t_i), the time to peak of the inflow hydrograph (t_p), and the storage volume (V_s).

2. Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation.

$$\left(\frac{Q_o}{Q_i} \right) = 1 - 0.712 (V_s / V_r)^{1.328} (t_i / t_p)^{0.546} \quad (10.5)$$

3. Multiply the peak flow rate of the inflow hydrograph (Q_i) times the potential peak flow reduction ratio calculated from step 2 to obtain the estimated peak outflow rate (Q_o) for the selected storage volume.

10.7 Stage-Storage Relationship

10.7.1 General

A stage-storage relationship defines the relationship between the depth of water and storage volume in the storage facility. The volume of storage can be calculated by using simple geometric formulas expressed as a function of storage depth. This relationship between storage volume and depth defines the stage-storage curve. A typical stage-storage curve is illustrated in Figure 10-5. After the required storage has been estimated, the configuration of the storage basin must be determined so that the stage-storage curve can be developed. The following relationships can be used for computing the volumes at specific depths of geometric shapes commonly used in detention facilities.

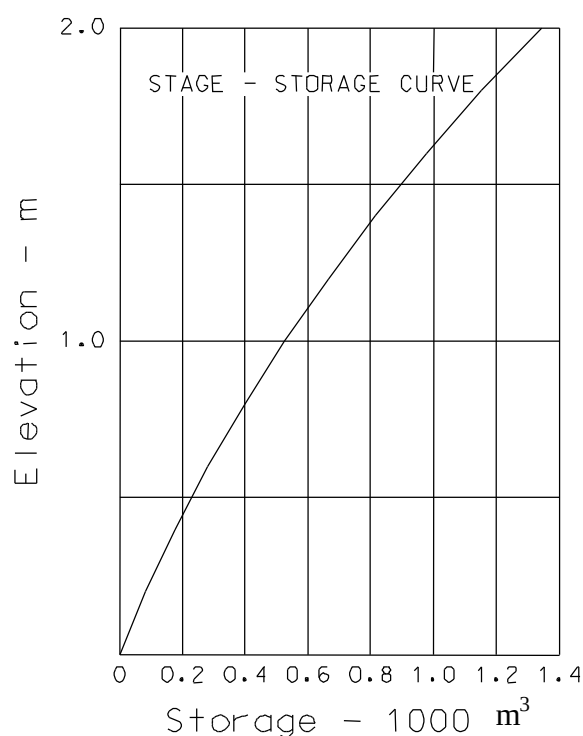


Figure 10-5 Stage-storage Curve

10.7.2 Rectangular Basins

Underground storage tanks are often rectangular. The volume of a rectangular basin can be computed by dividing the volume into triangular and rectangular shapes and using equation 10.6. The variables in equation 10.6 are illustrated in Figure 10-6.

Box:

$$V = L W D$$

Triangle:

$$V = \frac{1}{2} W \frac{D^2}{S} \quad (10.6)$$

where: V = volume at a specific depth, m³ (ft³)
 D = depth of ponding for that shape, m (ft)
 W = width of basin at base, m (ft)
 L = length of basin at base, m (ft)
 S = slope of basin, m/m (ft/ft)

If the basin is not on a slope, then the geometry will consist only of rectangular shaped boxes.

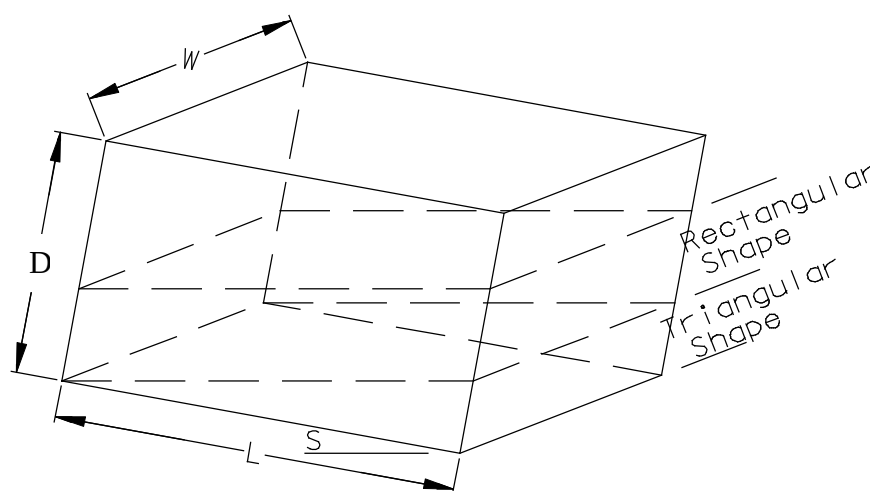


Figure 10-6 Rectangular Basin

10.7.3 Trapezoidal Basins

The volume of a trapezoidal basin can be calculated in a manner similar to that of rectangular basins by dividing the volume into triangular and rectangular shaped components and applying equation 10.7. The variables in equation 10.7 are illustrated in Figure 10-7. "Z" in this equation is the ratio of the horizontal to vertical components of the side slope. For example, if the side slope is 1 to 2 (V:H), "Z" will be equal to 2.

$$V = L W D + (L + W) Z D^2 + \frac{4}{3} Z^2 D^3 \quad (10.7)$$

where: V = volume at a specific depth, m³ (ft³)
 D = depth of ponding or basin, m (ft)
 L = length of basin at base, m (ft)
 W = width of basin at base, m (ft)
 r = ratio of width to length of basin at the base
 Z = side slope factor; ratio of horizontal to vertical components of side slope

Estimating the trial dimensions of a basin for a given basin storage volume can be accomplished by rearranging equation 10.7 as shown in equation 10.8. The use of equation 10.8 is demonstrated in example 10-2.

$$L = \frac{-ZD(r+1) + [(ZD)^2(r+1)^2 - 5.33(ZD)^2r + \frac{4rV}{D}]^{0.5}}{2r} \quad (10.8)$$

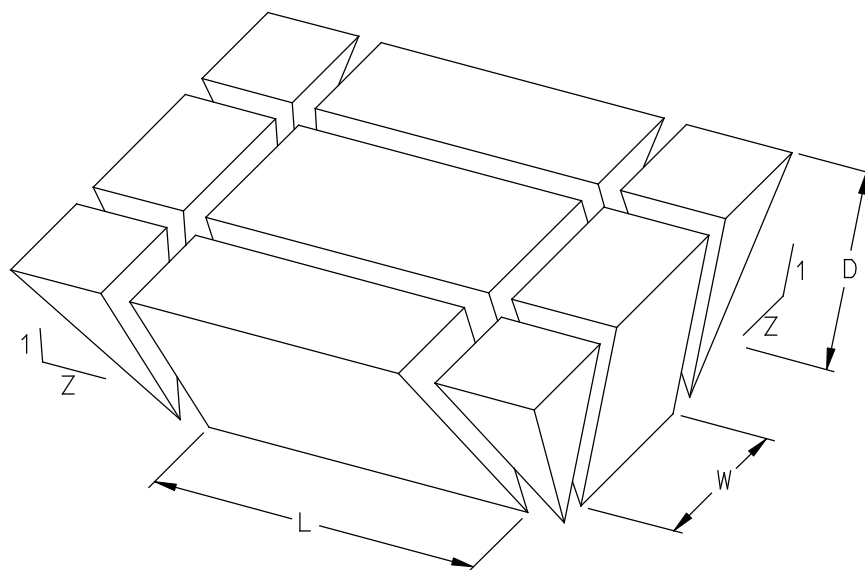


Figure 10-7 Trapezoidal Basin

10.7.4 Pipes and Conduits

If pipes or other storm drain conduits are used for storage, positive slope should be provided to transport sediment. This complicates storage calculations. The prismoidal formula presented in equation 10.9 can be used to determine the volume in sloping storm drain pipes. Figure 10-8 provides a definition sketch for the terms in equation 10.9.

$$V = \frac{L}{6} (A_1 + 4M + A_2) \quad (10.9)$$

where: V = volume of storage, m^3 (ft^3)
 L = length of section, m (ft)
 A_1 = cross-sectional area of flow at base, m^2 (ft^2)
 A_2 = cross-sectional area of flow at top, m^2 (ft^2)
 M = cross-sectional area of flow at midsection, m^2 (ft^2)

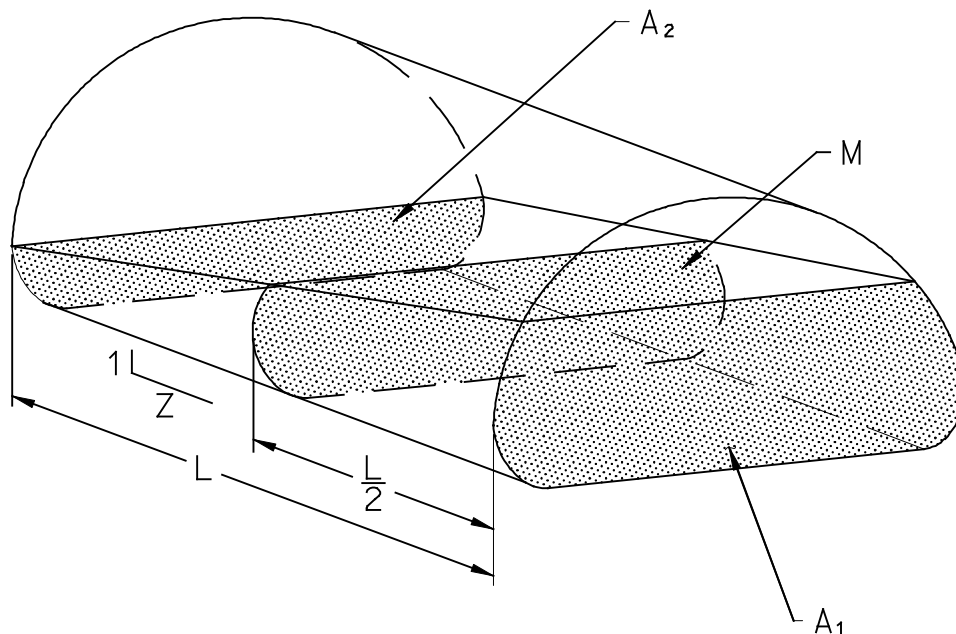


Figure 10-8 Definition Sketch for Prismoidal Formula

Calculations will be simplified if circular pipes are used, since the ungula of a cone and cylinder formulas may be used. The storage volume in a circular pipe, illustrated in Figure 10-9, can be computed using equation 10.10.

$$V = H \frac{(2/3 a^3 \pm c B)}{r \pm c} \quad (10.10)$$

where: V = volume of ungula, m^3 (ft^3)
 B = cross-sectional area of flow, m^2 (ft^2)
 H = wetted pipe length, m (ft)
 r = pipe radius, m (ft)
 a , c , and α (radians) are as defined in Figure 10-9 (a and c have units of m (ft)) and

$$a = \sqrt{(2r - d)d} \quad (10.11)$$

$$c = d - r \quad (10.12)$$

$$\alpha = 2 \sin^{-1} \left(\frac{a}{r} \right) \quad (10.13)$$

where: d = flow depth in pipe, m (ft)

To assist in the determination of the cross-sectional area of the flow (B), equation 10.14 can be used to find the area of the associated circular segment.

$$A_s = (a - \sin a) \frac{r^2}{2} \quad (10.14)$$

where: A_s = the segment area, m^2 (ft^2)

Using equation 10.10, the wetted area is computed as follows:

$$\begin{aligned} \text{For } d \leq r; \quad B &= A_s \\ \text{For } d > r; \quad B &= A - A_s \end{aligned}$$

Alternatively, various texts contain tables and charts which can be used to determine the depths and areas described in the above equations.

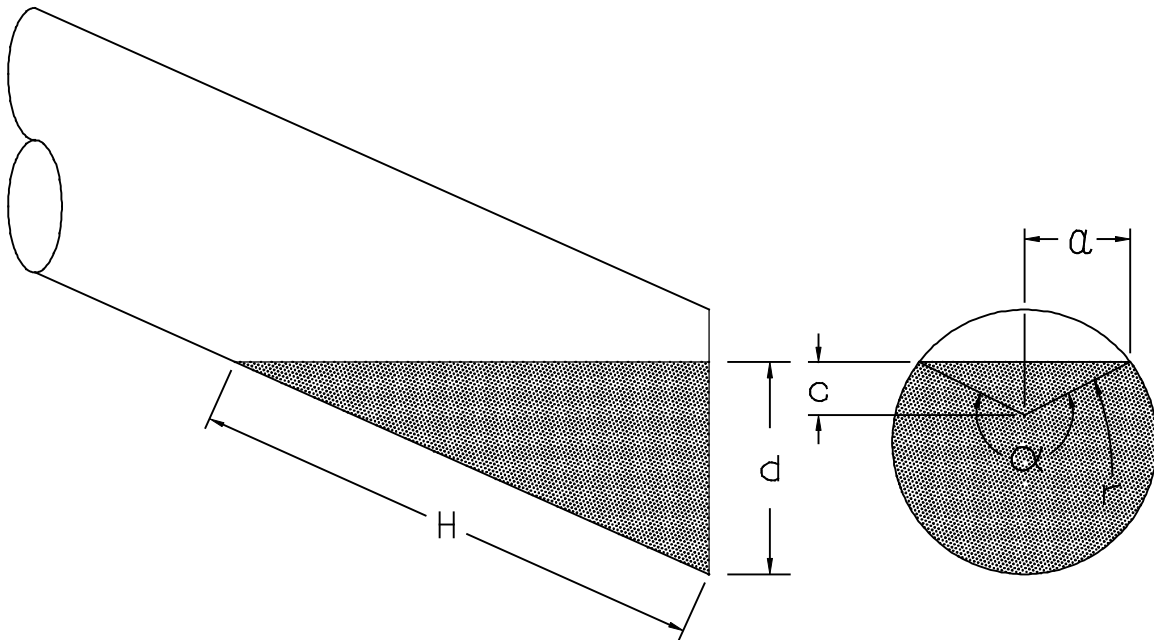


Figure 10-9 Ungula of a Cone

10.7.5 Natural Basins:

The storage volume for natural basins in irregular terrain is usually developed using a topographic map and the double-end area or frustum of a pyramid formulas. The double-end area formula is expressed as:

$$V_{1,2} = [(A_1 + A_2) / 2] d \quad (10.15)$$

where: $V_{1,2}$ = storage volume between elevations 1 and 2, m^3 (ft^3)
 A_1 = surface area at elevation 1, m^2 (ft^2)
 A_2 = surface area at elevation 2, m^2 (ft^2)
 d = change in elevation between points 1 and 2, m (ft)

The frustum of a pyramid is shown in Figure 10-10 and is expressed as:

$$V = d [A_1 + (A_1 A_2)^{0.5} + A_2] / 3 \quad (10.16)$$

where: V = volume of frustum of a pyramid, m^3 (ft^3)
 A_1 = surface area at elevation 1, m^2 (ft^2)
 A_2 = surface area at elevation 2, m^2 (ft^2)
 d = change in elevation between points 1 and 2, m (ft)

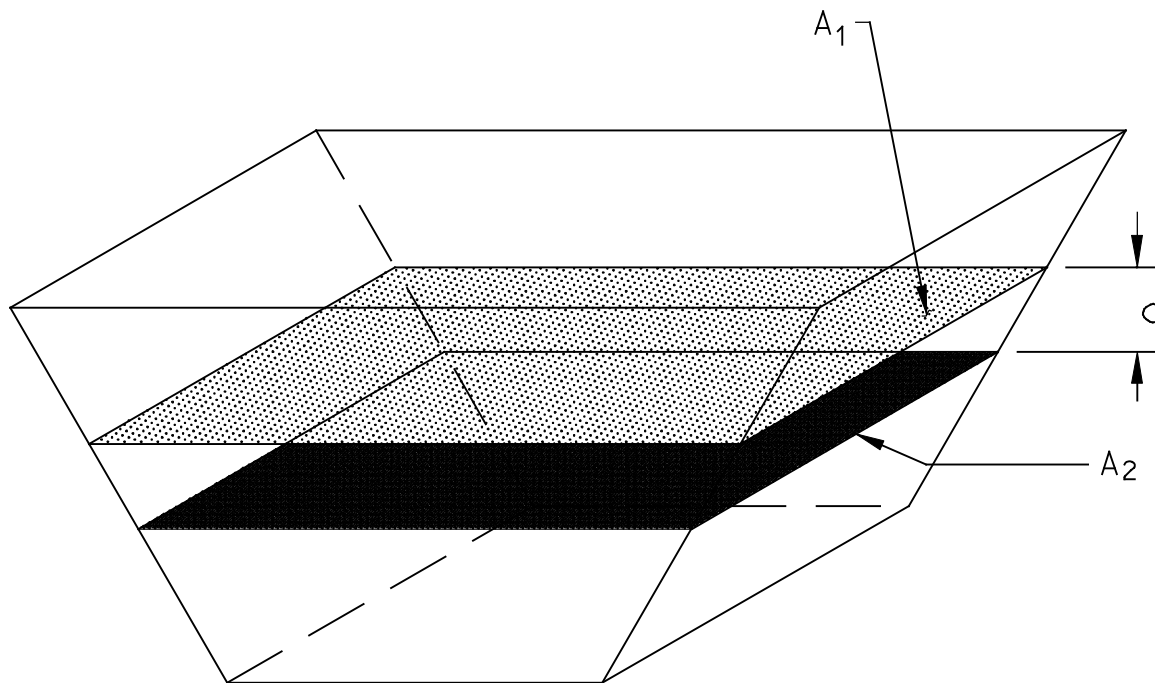


Figure 10-10 Frustum of a Pyramid

The following examples illustrate the development of a stage-storage relationship.

Example 10-2

Given: Estimated Storage Volume (V) = 849 m^3 ($29,980 \text{ ft}^3$) (from the triangular hydrograph from example 10-1 since it is the middle value). Use $V = 850 \text{ m}^3$ and $30,017 \text{ ft}^3$.
 Depth Available for Storage During 10-yr Event (D) = 1.6 m (5.3 ft)
 Available Freeboard = 0.6 m
 Basin Side Slopes (Z) = 3 ($V:H = 1:3$)
 Width to Length Ratio of Basin (r) = $1/2$

Find: (1) The length (L) and width (W) of basin at its base.

(2) Develop a stage-storage curve for the basin assuming that the base elevation of the basin is 10.0 m (32.8 ft) and the crest of the embankment is at 12.2 m (40.0 ft). (This crest elevation is determined by adding the 1.6 m of available depth plus the 0.6 m of freeboard.)

Solution: (1) Substituting the given values in equation 10.8 yields the following:

$$L = \frac{-ZD(r+1) + [(ZD)^2(r+1)^2 - 5.33(ZD)^2r + \frac{4rV}{D}]^{0.5}}{2r}$$

$$L = \frac{(-3)(1.6)(0.5+1) + [(3 \cdot 1.6)^2(0.5+1)^2 - 5.33(3 \cdot 1.6)^2(0.5) + \frac{4(0.5)(850)}{1.6}]^{0.5}}{2(0.5)}$$

$$L = 25.2 \text{ m (82.7 ft)}$$

$$\text{Use } L = 26 \text{ m (85 ft)}$$

$$W = 0.5 L = 13 \text{ m (42.7 ft)}$$

Use 26 m by 13 m basin (85 ft by 43 ft)

(2) By varying the depth (D) in equation 10.7, a stage-storage relationship can be developed for the trapezoidal basin. The following table summarizes the results:

DEPTH		STAGE		STORAGE VOLUME	
(m)	(ft)	(m)	(ft)	(m ³)	(ac-ft)
0.00	0.0	10.0	32.8	0	0.000
0.20	0.7	10.2	33.5	72	0.059
0.40	1.3	10.4	34.1	155	0.125
0.60	2.0	10.6	34.8	248	0.201
0.80	2.6	10.8	35.4	353	0.286
1.00	3.3	11.0	36.1	470	0.381
1.20	3.9	11.2	36.7	600	0.486
1.40	4.6	11.4	37.4	744	0.603
1.60	5.2	11.6	38.0	902	0.731
1.80	5.9	11.8	38.7	1075	0.871
2.00	6.6	12.0	39.4	1264	1.024

Figure 10-5 illustrates the stage-storage relationship.

Example 10-3

Given: Given a storm drain pipe having the following properties:

Diameter = 1500 mm (60 in) Pipe Slope = 0.01 m/m (ft/ft)
 Pipe Length = 250 m (820 ft) Invert Elevation = 30 m (98 ft)

Find: Develop a stage-storage tabulation between elevations 30 m (98 ft) and 31.5 m (103 ft)

Solution: Solve for the volume of storage using equations 10.10 and 10.14.

$$V = H \frac{(2/3 a^3 \pm c B)}{r \pm c}, \quad A_s = (a - \sin a) \frac{r^2}{2}$$

Note that: $B = A_s$ for $d \leq r$

$B = A - A_s$ for $d > r$

where A_s is the segment area and A is the total pipe area. The solution is provided in tabular form as follows:

Stage –Storage Tabulation

d		a		c		H		α	B		V	
(m)	(ft)	(m)	(ft)	(m)	(ft)	(m)	(ft)	(rad)	(m ²)	(ft ²)	(m ³)	(ft ³)
0.00	0.0	0.00	0.00	-0.75	-2.46	0.0	0.0	0.000	0.00	0.0	0.00	0.0
0.20	0.7	0.51	1.67	-0.55	-1.80	20.0	66.0	1.495	0.14	1.5	1.18	41.6
0.40	1.3	0.66	2.18	-0.35	-1.15	40.0	131.0	2.171	0.38	4.1	6.31	222.8
0.60	2.0	0.74	2.41	-0.15	-0.49	60.0	197.0	2.739	0.66	7.1	16.69	588.8
0.80	2.6	0.75	2.45	0.05	0.16	80.0	262.0	3.008	0.96	10.3	32.87	1159.9
1.00	3.3	0.71	2.32	0.25	0.82	100.0	328.0	2.462	1.25	13.5	54.98	1940.0
1.20	3.9	0.60	1.97	0.45	1.48	120.0	394.0	1.855	1.52	16.3	82.67	2917.3
1.40	4.6	0.37	1.23	0.65	2.13	140.0	459.0	1.045	1.72	18.5	115.09	4061.1
1.50	4.9	0.00	0.00	0.75	2.46	150.0	492.0	0.000	1.77	19.01	132.54	4676.9

10.8 Outlet Hydraulics

10.8.1 General

A stage-discharge (performance) curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility will have both a principal and an emergency outlet. The principal outlet is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway. The structure for the principal outlet will typically consist of a pipe culvert, weir, orifice, or other appropriate hydraulic control device. Multiple outlet control devices are often used to provide discharge controls for multiple frequency storms.

Development of a composite stage-discharge curve requires consideration of the discharge rating relationships for each component of the outlet structure. The following sections present design relationships for typical outlet controls.

10.8.2 Orifices

For a single orifice as illustrated in Figure 10-11 (a), orifice flow can be determined using equation 10.17.

$$Q = C_o A_o (2 g H_o)^{0.5} \quad (10.17)$$

where: Q = the orifice flow rate, m^3/s (ft^3/s)
 C_o = discharge coefficient (0.40 - 0.60)
 A_o = area of orifice, m^2 (ft^2)
 H_o = effective head on the orifice measured from the centroid of the opening, m (ft)
 g = gravitational acceleration, $9.81 m/s^2$ ($32.2 ft/s^2$)

If the orifice discharges as a free outfall, then the effective head is measured from the centerline of the orifice to the upstream water surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the upstream and downstream water surfaces. This latter condition of a submerged discharge is shown in Figure 10-11(b).

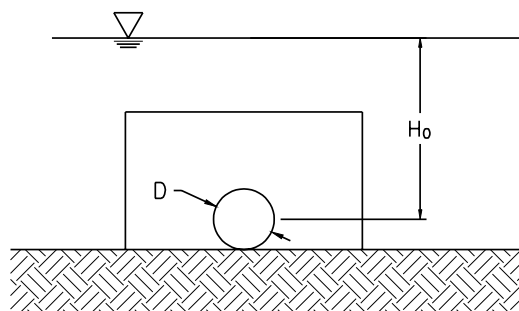
For square-edged, uniform orifice entrance conditions, a discharge coefficient of 0.6 should be used. For ragged edged orifices, such as those resulting from the use of an acetylene torch to cut orifice openings in corrugated pipe, a value of 0.4 should be used.

For circular orifices with C_o set equal to 0.6, the following equation results:

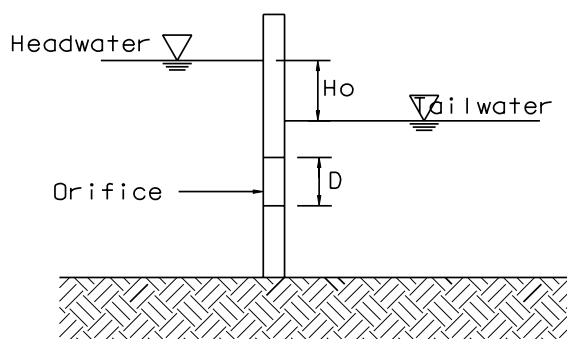
$$Q = K_{or} D^2 H_o^{0.50} \quad (10.18)$$

where: K_{or} = 2.09 in S.I. units (3.78 in English units)
 D = orifice diameter, m (ft)

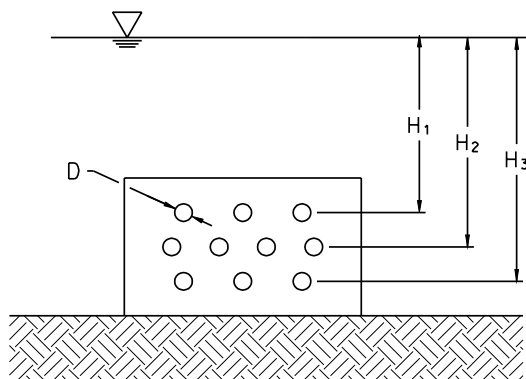
Pipes smaller than 0.3 m (1 ft) in diameter may be analyzed as a submerged orifice as long as H_o/D is greater than 1.5. Pipes greater than 0.3 m (1 ft) in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account, not just as an orifice.



a.



b.



c.

Figure 10-11 Definition Sketch for Orifice Flow

Flow through multiple orifices (see Figure 10-11(c)) can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings. The procedure is demonstrated in the following example:

Example 10-4

Given: Given the orifice plate in Figure 10-11 (c) with a free discharge and:

$$\begin{aligned}\text{orifice diameter} &= 25 \text{ mm (1.0 in)} \\ H_1 &= 1.1 \text{ m (3.6 ft)} \\ H_2 &= 1.2 \text{ m (3.9 ft)} \\ H_3 &= 1.3 \text{ m (4.3 ft)}\end{aligned}$$

Find: Total discharge through the orifice plate.

Solution: Using a modification of equation 10.18 for multiple orifices,

$$\begin{aligned}Q_i &= K D^2 (H_i)^{0.5} N_i \\ Q_i &= (2.09) (0.025)^2 (H_i)^{0.5} N_i = 0.0013 H_i^{0.5} N_i\end{aligned}$$

$$\begin{aligned}Q_1 &= 0.0013 (1.1)^{0.5} (3) = 0.0040 \\ Q_2 &= 0.0013 (1.2)^{0.5} (4) = 0.0058 \\ Q_3 &= 0.0013 (1.3)^{0.5} (3) = 0.0045\end{aligned}$$

$$Q_{\text{total}} = Q_1 + Q_2 + Q_3 = 0.0143 \text{ m}^3/\text{s} (0.50 \text{ ft}^3/\text{s})$$

Example 10-5

Given: Given the circular orifice in Figure 10-11(a) with:

$$\begin{aligned}\text{orifice diameter} &= 0.15 \text{ m (0.5 ft)} \\ \text{orifice invert} &= 10.0 \text{ m (32.8 ft)} \\ \text{discharge coeff.} &= 0.60\end{aligned}$$

Find: The stage - discharge rating between 10 m (32.8 ft) and 12.0 m (39.4 ft).

Solution: Using equation 10.17 with $D = 0.15 \text{ m}$ yields the following relationship between the effective head on the orifice (H_o) and the resulting discharge:

$$\begin{aligned}Q &= 0.047 H_o^{0.5} \\ H_o &= \text{Depth} - D/2\end{aligned}$$

The solution of this equation in table form is as follows:

Stage Discharge Tabulation

DEPTH		STAGE		DISCHARGE	
(meters)	(feet)	(meters)	(feet)	(m ³ /s)	(ft ³ /s)
0.00	0.0	10.0	32.8	0.000	0.00
0.20	0.7	10.2	33.5	0.011	0.37
0.40	1.3	10.4	34.1	0.024	0.83
0.60	2.0	10.6	34.8	0.032	1.11
0.80	2.6	10.8	35.4	0.038	1.34
1.00	3.3	11.0	36.1	0.043	1.53
1.20	3.9	11.2	36.7	0.048	1.70
1.40	4.6	11.4	37.4	0.053	1.85
1.60	5.2	11.6	38.0	0.057	2.00
1.80	5.9	11.8	38.7	0.061	2.13
2.00	6.6	12.0	39.4	0.064	2.26

10.8.3 Weirs

Relationships for sharp-crested, broad-crested, V-notch, and proportional weirs are provided in the following sections:

Sharp Crested Weirs

Typical sharp crested weirs are illustrated in Figure 10-12. Equation 10.19 provides the discharge relationship for **sharp crested weirs** with no end contractions (illustrated in Figure 12a).

$$Q = C_{scw} L H^{1.5} \quad (10.19)$$

where: Q = discharge, m^3/s (ft^3/s)
 L = horizontal weir length, m (ft)
 H = head above weir crest excluding velocity head, m (ft)
 $C_{scw} = 1.81 + 0.22 (H/H_c) [3.27 + 0.4 (H/H_c) \text{ in English units}]$

As indicated above, the value of the coefficient C_{scw} is known to vary with the ratio H/H_c (see Figure 10-12c for definition of terms). For values of the ratio H/H_c less than 0.3, a constant C_{scw} of 1.84 (3.33 in English units) is often used.

Equation 10.20 provides the discharge equation for **sharp-crested weirs with end contractions** (illustrated in Figure 10-12(b)). As indicated above, the value of the coefficient C_{scw} is known to vary with the ratio H/H_c (see Figure 10-13c for definition of terms). For values of the ratio H/H_c less than 0.3, a constant C_{scw} of 1.84 (3.33 in English units) is often used.

$$Q = C_{scw} (L - 0.2 H) H^{1.5} \quad (10.20)$$

Sharp-crested weirs will be effected by submergence when the tailwater rises above the weir crest elevation, as shown in Figure 10-12(d). The result will be that the discharge over the weir will be reduced.

The discharge equation for a **submerged sharp-crested weir** is:

$$Q_s = Q_r (1 - (H_2 / H_1)^{1.5})^{0.385} \quad (10.21)$$

where: Q_s = submerged flow, m^3/s (ft^3/s)
 Q_r = unsubmerged weir flow from equation 10.19 or 10.20, m^3/s (ft^3/s)
 H_1 = upstream head above crest, m (ft)
 H_2 = downstream head above crest, m (ft)

Flow over the top edge of a riser pipe is typically treated as flow over a sharp crested weir with no end constrictions. Equation 10.19 should be used for this case.

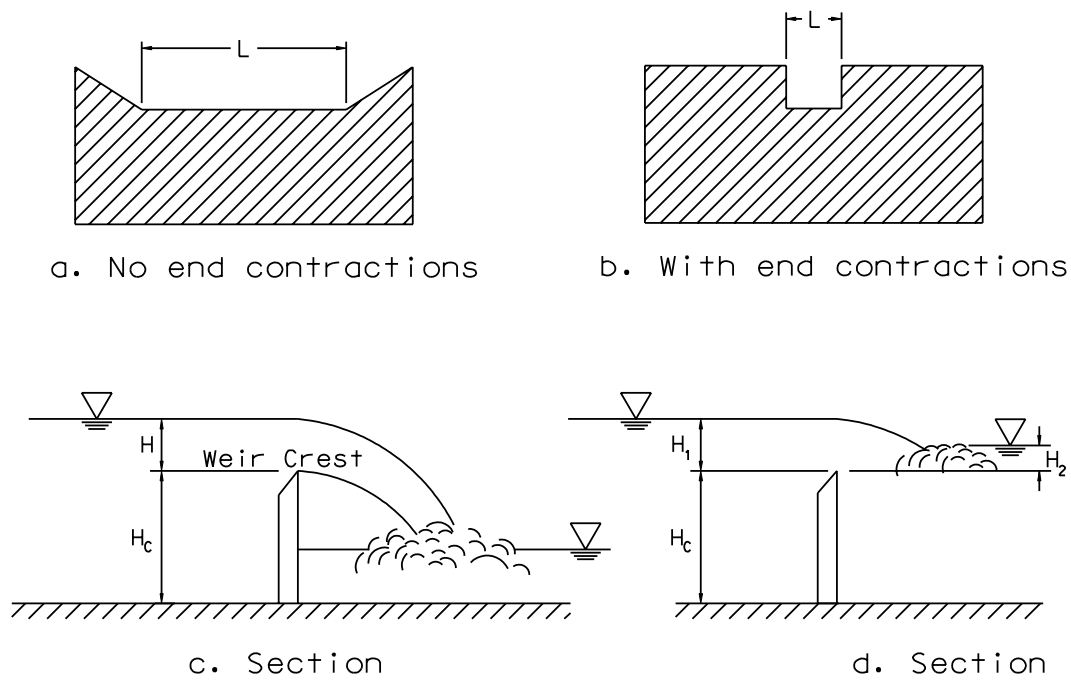
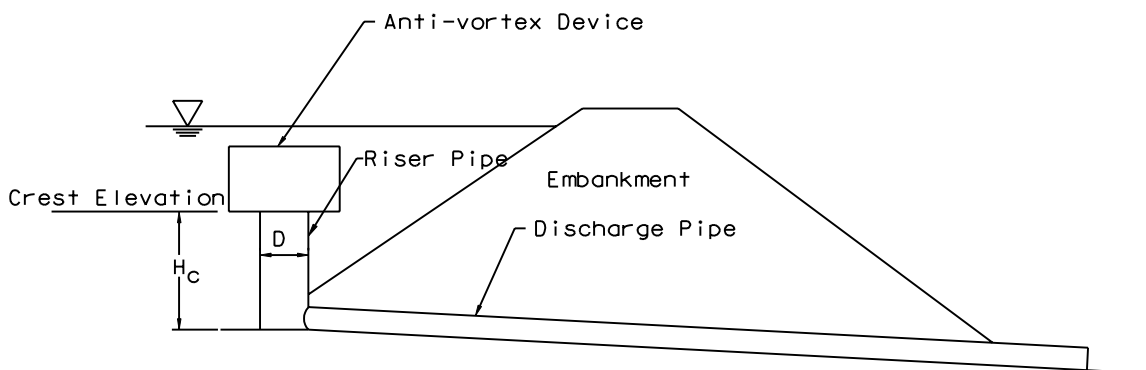


Figure 10-12 Sharp Crested Weirs

Example 10-6**Figure 10-13 Riser Pipe**

Given: A riser pipe as shown in Figure 10-13 with the following characteristics:

$$\begin{aligned} \text{diameter } (D) &= 0.53 \text{ m (1.75 ft)} \\ \text{crest elevation} &= 10.8 \text{ m (35.4 ft)} \\ \text{weir height } (H_c) &= 0.8 \text{ m (2.6 ft)} \end{aligned}$$

Find: The stage - discharge rating for the riser pipe between 10 m (32.8 ft) and 12.0 m (39.4 ft).

Solution: Since the riser pipe functions as both a weir and an orifice (depending on stage), the rating is developed by comparing the stage - discharge produced by both weir and orifice flow as follows:

Using equation 10.18 for orifices with $D = 0.53 \text{ m (1.75 ft)}$ yields the following relationship between the effective head on the orifice (H_o) and the resulting discharge:

$$\begin{aligned} Q &= K_{or} D^2 H_o^{0.50} \\ Q &= (2.09)(0.53)^2 H_o^{0.50} \\ Q &= 0.587 H_o^{0.50} \end{aligned}$$

Using equation 10.19 for sharp crested weirs with $C_{SCW} = 1.84$ (H/H_c assumed less than 0.3), and $L = \text{pipe circumference} = 1.67 \text{ m (5.5 ft)}$ yields the following relationship between the effective head on the riser (H) and the resulting discharge:

$$\begin{aligned} Q &= C_{SCW} L H^{1.5} \\ Q &= (1.84)(1.67) H^{1.5} \\ Q &= 3.073 H^{1.5} \end{aligned}$$

The resulting stage - discharge relationship is summarized in the following table:

STAGE		EFFECTIVE HEAD		ORIFICE FLOW		WEIR FLOW	
(m)	(ft)	(m)	(ft)	(m ³ /s)	(ft ³ /s)	(m ³ /s)	(ft ³ /s)
10.0	32.8	0.0	0.0	0.00	0.0	0.00	0.0
10.8	35.4	0.0	0.0	0.00	0.0	0.00	0.0
10.9	35.7	0.1	0.3	0.19	6.6	0.10	3.4
11.0	36.1	0.2	0.7	0.26	9.2	0.27	9.5
11.2	36.7	0.4	1.3	0.37	13.1	0.78	27.5
11.4	37.4	0.6	2.0	0.45	15.9	1.43	50.5
11.6	38.1	0.8	2.6	0.53	18.7	2.20	77.7
11.8	38.7	1.0	3.3	0.59	20.8	3.07	108.4
12.0	39.4	1.2	3.9	0.64	22.6	4.04	142.7

■ Designates controlling flow.

The flow condition, orifice or weir, producing the lowest discharge for a given stage defines the controlling relationship. As illustrated in the above table, at a stage of 10.9 m (35.7 ft) weir flow controls the discharge through the riser. However, at and above a stage of 11.0 m (36.1 ft), orifice flow controls the discharge through the riser.

Broad-Crested Weir

The equation typically used for a broad-crested weir is:

$$Q = C_{BCW} L H^{1.5} \quad (10.22)$$

where: Q = discharge, m³/s (ft³/s)
C_{BCW} = broad-crested weir coefficient, 1.44 - 1.70 (2.61 to 3.08)
L = broad-crested weir length, m (ft)
H = head above weir crest, m (ft)

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 1.70. For sharp corners on the broad crested weir, a minimum value of 1.44 should be used. Additional information on C values as a function of weir crest breadth and head is given in Table 10-2.

Table 10-2 Broad-crested weir coefficient C values as a function of weir crest breadth and head (coefficient has units of $m^{0.5}/sec$) (metric only)

Head ⁽¹⁾ (m)	BREADTH OF CREST OF WEIR (m)														
	0.15	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00	1.25	1.50	2.00	3.00	4.00
0.10	1.59	1.56	1.50	1.47	1.45	1.43	1.42	1.41	1.40	1.39	1.37	1.35	1.36	1.40	1.45
0.15	1.65	1.60	1.51	1.48	1.45	1.44	1.44	1.44	1.45	1.45	1.44	1.43	1.44	1.45	1.47
0.20	1.73	1.66	1.54	1.49	1.46	1.44	1.44	1.45	1.47	1.48	1.48	1.49	1.49	1.49	1.48
0.30	1.83	1.77	1.64	1.56	1.50	1.47	1.46	1.46	1.46	1.47	1.47	1.48	1.48	1.48	1.46
0.40	1.83	1.80	1.74	1.65	1.57	1.52	1.49	1.47	1.46	1.46	1.47	1.47	1.47	1.48	1.47
0.50	1.83	1.82	1.81	1.74	1.67	1.60	1.55	1.51	1.48	1.48	1.47	1.46	1.46	1.46	1.45
0.60	1.83	1.83	1.82	1.73	1.65	1.58	1.54	1.46	1.31	1.34	1.48	1.46	1.46	1.46	1.45
0.70	1.83	1.83	1.83	1.78	1.72	1.65	1.60	1.53	1.44	1.45	1.49	1.47	1.47	1.46	1.45
0.80	1.83	1.83	1.83	1.82	1.79	1.72	1.66	1.60	1.57	1.55	1.50	1.47	1.47	1.46	1.45
0.90	1.83	1.83	1.83	1.83	1.81	1.76	1.71	1.66	1.61	1.58	1.50	1.47	1.47	1.46	1.45
1.00	1.83	1.83	1.83	1.83	1.82	1.81	1.76	1.70	1.64	1.60	1.51	1.48	1.47	1.46	1.45
1.10	1.83	1.83	1.83	1.83	1.83	1.83	1.80	1.75	1.66	1.62	1.52	1.49	1.47	1.46	1.45
1.20	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.79	1.70	1.65	1.53	1.49	1.48	1.46	1.45
1.30	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.82	1.77	1.71	1.56	1.51	1.49	1.46	1.45
1.40	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.77	1.60	1.52	1.50	1.46	1.45
1.50	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.79	1.66	1.55	1.51	1.46	1.45
1.60	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.81	1.74	1.58	1.53	1.46	1.45

(1) Measured at least 2.5 H_c upstream of the weir

V-Notch Weir

The discharge through a v-notch weir is shown in Figure 10-14 and can be calculated from the following equation:

$$Q = 1.38 \tan(\theta / 2) H^{2.5} \quad (10.23)$$

where: Q = discharge, m^3/s (ft^3/s)
 θ = angle of v-notch, degrees
 H = head on apex of v-notch, m (ft)

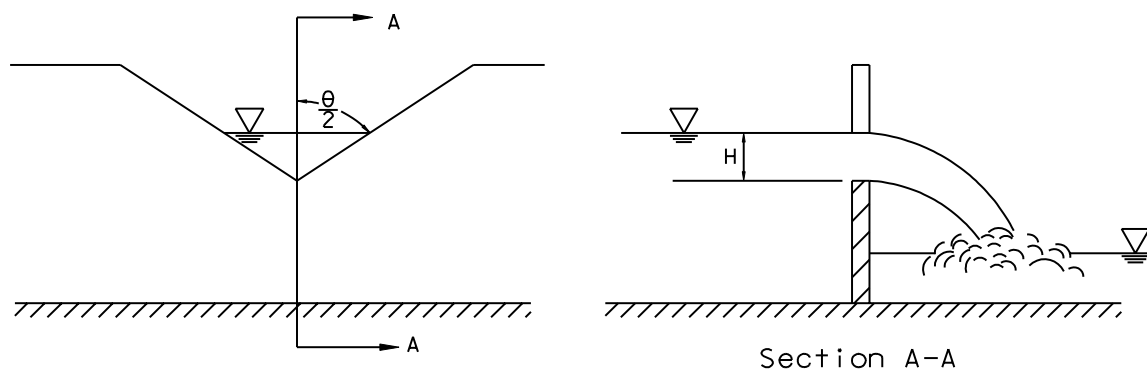


Figure 10-14 V-notch Weir

Proportional Weir

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship. This relationship is achieved by allowing the discharge area to vary nonlinearly with head.

Design equations for proportional weirs are as follows:

$$Q = 2.74 a^{0.5} b (H - a / 3) \quad (10.24)$$

$$x/b = 1 - (0.315) [\arctan(y/a)^{0.5}] \quad (10.25)$$

where: Q = discharge, m^3/s (ft^3/s)
 H = head above horizontal sill, m (ft)
 Dimensions a , b , x , and y are as shown in Figure 10-15.

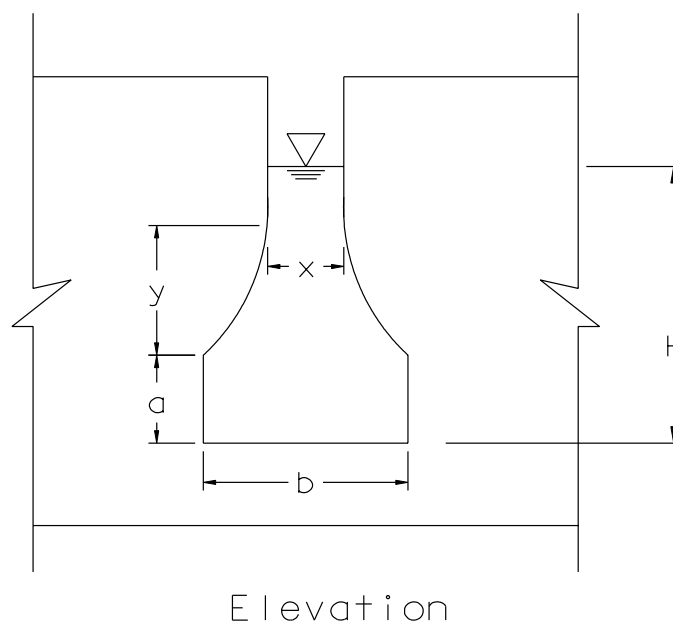


Figure 10-15 Proportional Weir Dimensions

10.8.4 Discharge Pipes

Discharge pipes are often used as outlet structures for detention facilities. The design of these pipes can be for either single or multistage discharges. A single stage discharge system would consist of a single culvert entrance system and would not be designed to carry emergency flows. A multistage inlet would involve the placement of a control structure at the inlet end of the pipe. The inlet structure would be designed in such a way that the design discharge would pass through a weir or orifice in the lower levels of the structure and the emergency flows would pass over the top of the structure. The pipe would need to be designed to carry the full range of flows from a drainage area including the emergency flows.

For single stage systems, the facility would be designed as if it were a simple culvert. Appropriate design procedures are outlined in Chapter 8, Culverts. For multistage control structures, the inlet control structure would be designed considering both the design flow and the emergency flows. A stage-discharge curve would be developed for the full range of flows that the structure would experience. The design flows will typically be orifice flow through whatever shape the designer has chosen while the higher flows will typically be weir flow over the top of the control structure. Orifices can be designed using the equations in section 10.8.2 and weirs can be designed using the equations in section 10.8.3. The pipe must be designed to carry all flows considered in the design of the control structure.

In designing a multistage structure, the designer would first develop peak discharges that must be passed through the facility. The second step would be to select a pipe that will pass the peak flow within the allowable headwater and develop a performance curve for the pipe. Thirdly, the designer would develop a stage-discharge curve for the inlet control structure, recognizing that the headwater for the discharge pipe will be the tailwater that needs to be considered in designing the inlet structure. And lastly, the designer would use the stage-discharge curve in the basin routing procedure.

Example 10-7

Given: A corrugated steel discharge pipe as shown in Figure 10-13 with the following characteristics:

maximum head on pipe = 0.75 m (2.3 ft) (conservative value of 0.05 m less than the riser height)
 inlet invert = 10.0 m (32.8 ft)
 length (L) = 50 m (164 ft)
 slope = 0.04 m/m (ft/ft)
 roughness = 0.024
 square edge entrance ($K_e = 0.5$)
 discharge pipe outfall is free (not submerged)
 Runoff characteristics as given

Find: The size pipe needed to carry the maximum allowable flow rate from the detention basin.

Solution: The maximum predeveloped discharge from the watershed is $0.55 \text{ m}^3/\text{s}$ ($19.4 \text{ ft}^3/\text{s}$). Since the discharge pipe can function under inlet or outlet control, the pipe size will be evaluated for both conditions. The larger pipe size will be selected for the final design.

Using chart 2 from HDS-5 yields the relationship between head on the pipe and the resulting discharge for inlet control. From the chart, the pipe diameter necessary to carry the flow is 750 mm (30 in).

Using chart 6 from HDS-5 yields the relationship between head on the pipe and discharge for barrel control. From the chart, the pipe diameter necessary to carry the flow is 675 mm (30 in).

10.8.5 Emergency Spillway

The purpose of an emergency spillway is to provide a controlled overflow relief for storm flows in excess of the design discharge for the storage facility. An inlet control structure is commonly used to release emergency flows. Another suitable emergency spillway for detention storage facilities for highway applications is a broad-crested overflow weir cut through the original ground next to the embankment. The transverse cross-section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated in Figure 10-16. The invert of the spillway at the outfall should be at an elevation 0.3 m (1 ft) to 0.6 m (2 ft) above the maximum design storage elevation. It is preferable to have a freeboard of 0.6 m (2 ft) minimum. However, for very small impoundments (less than 0.4 to 0.8 hectare surface area) an absolute minimum of 0.3 meter of freeboard may be acceptable.

Equation 10.26 presents a relationship for computing the flow through a broad-crested emergency spillway. The dimensional terms in the equation are illustrated in Figure 10-16.

$$Q = C_{SP} b H_p^{1.5} \quad (10.26)$$

where: Q = emergency spillway discharge, m^3/s (ft^3/s)
 C_{SP} = discharge coefficient
 b = width of the emergency spillway, m (ft)
 H_p = effective head on the emergency spillway, m (ft)

The discharge coefficient, C_{SP} , in equation 10.26 varies as a function of spillway bottom width and effective head. Figure 10-17 illustrates this relationship. Table 10-3 provides a tabulation of emergency spillway design parameters.

The critical slopes of Table 10-3 are based upon an assumed $n = 0.040$ for turf cover of the spillway. For a paved spillway, the n should be assumed as 0.015. Equations 10.27 and 10.28 can be used to compute the critical velocity and slope for spillway materials having other roughness values.

$$V_c = K_{SP} \left(\frac{Q}{b} \right)^{0.33} \quad (10.27)$$

where: V_c = critical velocity at emergency spillway control section, m/s (ft/s)
 Q = emergency spillway discharge, m^3/s (ft^3/s)
 b = width of the emergency spillway, m (ft)
 K_{SP} = 2.14 (3.18 in English units)

$$S_c = K_{SP}' n^2 \left(\frac{V_c b}{Q} \right)^{0.33} \quad (10.28)$$

where: S_c = critical slope, m/m (ft/ft)
 n = Manning's coefficient
 V_c = critical velocity at emergency spillway control section, m/s (ft/s)
 Q = emergency spillway discharge, m^3/s (ft^3/s)
 b = width of the emergency spillway, m (ft)
 K_{SP}' = 9.84 (14.6 in English units)

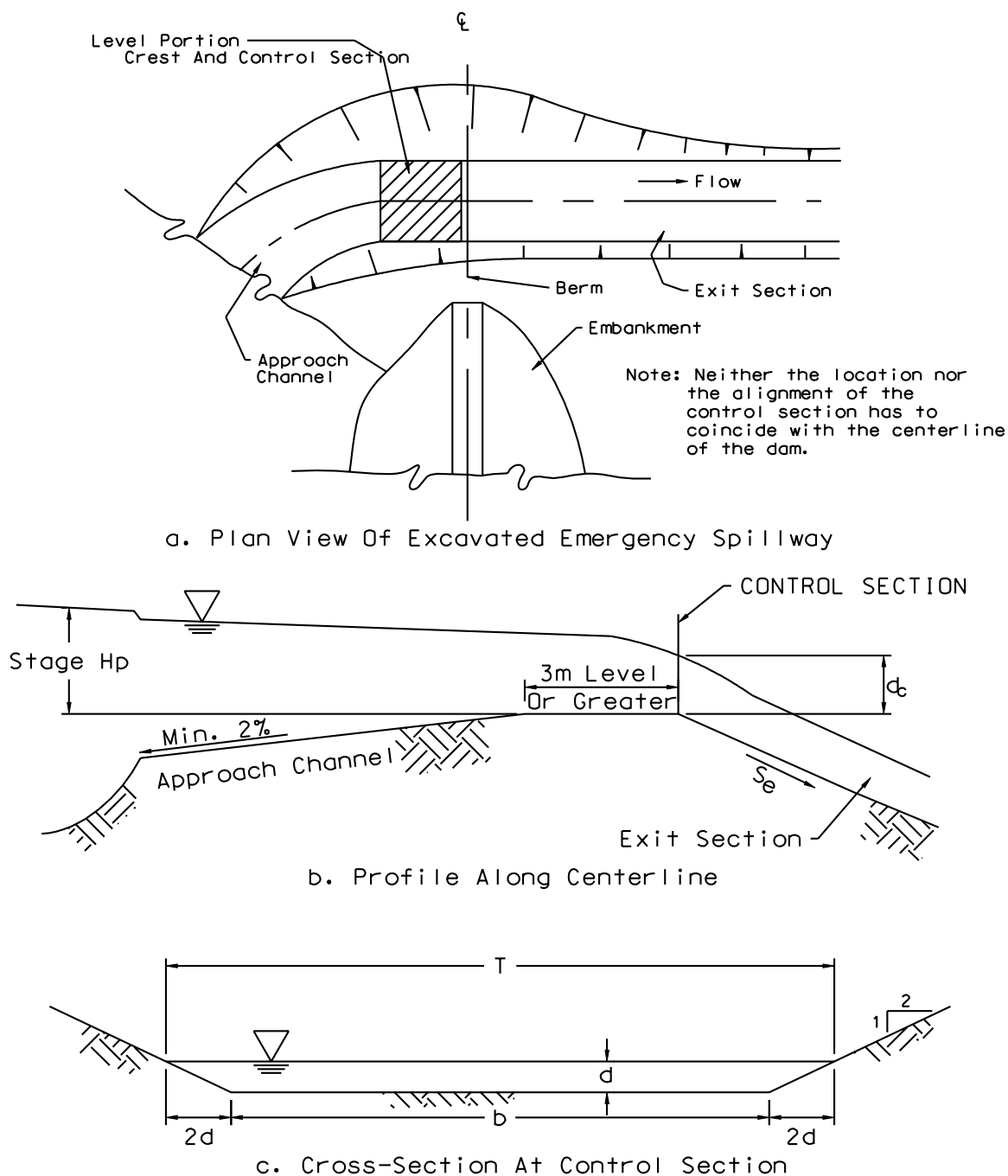


Figure 10-16 Emergency Spillway Design Schematic

Table 10-3 Emergency spillway design parameters (metric units)

H _p (m)	Spillway Bottom Width, b, meters															
	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0	
0.20	Q	0.19	0.26	0.35	0.40	0.44	0.50	0.58	0.65	0.69	0.73	-	-	-	-	-
	V _c	0.98	1.02	1.06	1.05	1.03	1.04	1.05	1.06	1.05	1.04	-	-	-	-	-
	S _c	3.4%	3.3%	3.2%	3.2%	3.3%	3.3%	3.2%	3.2%	3.2%	3.2%	-	-	-	-	-
0.25	Q	0.34	0.43	0.52	0.60	0.67	0.75	0.85	0.94	1.02	1.09	-	-	-	-	-
	V _c	1.19	1.20	1.20	1.19	1.19	1.19	1.19	1.20	1.19	1.19	-	-	-	-	-
	S _c	3.0%	3.0%	3.0%	3.0%	3.0%	3.0%	3.0%	3.0%	3.0%	3.0%	-	-	-	-	-
0.30	Q	0.53	0.63	0.72	0.83	0.95	1.06	1.18	1.29	1.41	1.52	1.64	1.75	1.87	1.96	2.07
	V _c	1.38	1.36	1.34	1.33	1.33	1.33	1.33	1.33	1.33	1.33	1.33	1.32	1.32	1.32	1.32
	S _c	2.7%	2.7%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%
0.35	Q	0.68	0.82	0.95	1.10	1.24	1.37	1.51	1.66	1.81	1.94	2.08	2.21	2.34	2.49	2.62
	V _c	1.50	1.48	1.46	1.46	1.45	1.45	1.44	1.44	1.44	1.44	1.43	1.43	1.43	1.43	1.42
	S _c	2.5%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%	2.6%
0.40	Q	0.86	1.04	1.20	1.38	1.55	1.72	1.89	2.07	2.25	2.42	2.58	2.74	2.90	3.09	3.27
	V _c	1.62	1.60	1.58	1.57	1.57	1.56	1.55	1.55	1.55	1.55	1.54	1.53	1.53	1.53	1.53
	S _c	2.4%	2.4%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%	2.5%
0.45	Q	1.05	1.27	1.48	1.69	1.90	2.11	2.32	2.53	2.74	2.95	3.15	3.35	3.56	3.78	4.00
	V _c	1.73	1.71	1.70	1.68	1.67	1.67	1.66	1.66	1.65	1.65	1.64	1.64	1.64	1.64	1.64
	S _c	2.3%	2.3%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%	2.4%
0.50	Q	1.27	1.55	1.81	2.05	2.30	2.54	2.79	3.05	3.30	3.54	3.79	4.05	4.31	4.55	4.79
	V _c	1.84	1.83	1.81	1.79	1.78	1.77	1.77	1.76	1.75	1.75	1.75	1.75	1.74	1.74	1.74
	S _c	2.2%	2.2%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%	2.3%
0.55	Q	1.54	1.85	2.13	2.43	2.73	3.03	3.33	3.60	3.91	4.19	4.47	4.75	5.02	5.30	5.58
	V _c	1.96	1.94	1.91	1.90	1.89	1.88	1.87	1.86	1.86	1.85	1.85	1.84	1.84	1.83	1.83
	S _c	2.1%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%
0.60	Q	1.84	2.18	2.48	2.81	3.17	3.52	3.86	4.19	4.53	4.85	5.18	5.52	5.85	6.17	6.50
	V _c	2.08	2.05	2.01	1.99	1.98	1.97	1.96	1.96	1.95	1.94	1.94	1.93	1.93	1.93	1.92
	S _c	2.1%	2.1%	2.1%	2.1%	2.1%	2.1%	2.1%	2.1%	2.1%	2.2%	2.2%	2.2%	2.2%	2.2%	2.2%

NOTE: 1. For a given H_p , decreasing exit slope from S_c decreases spillway discharge, but increasing exit slope from S_c does not increase discharge.

2. If a slope S_e steeper than S_c is used, velocity V_e in the exit channel will increase according to the following relationship:

$$V_e = V_c (S_e/S_c)^{0.3}$$

3. One meter is 3.28 feet; one cubic meter is 35.28 cubic feet

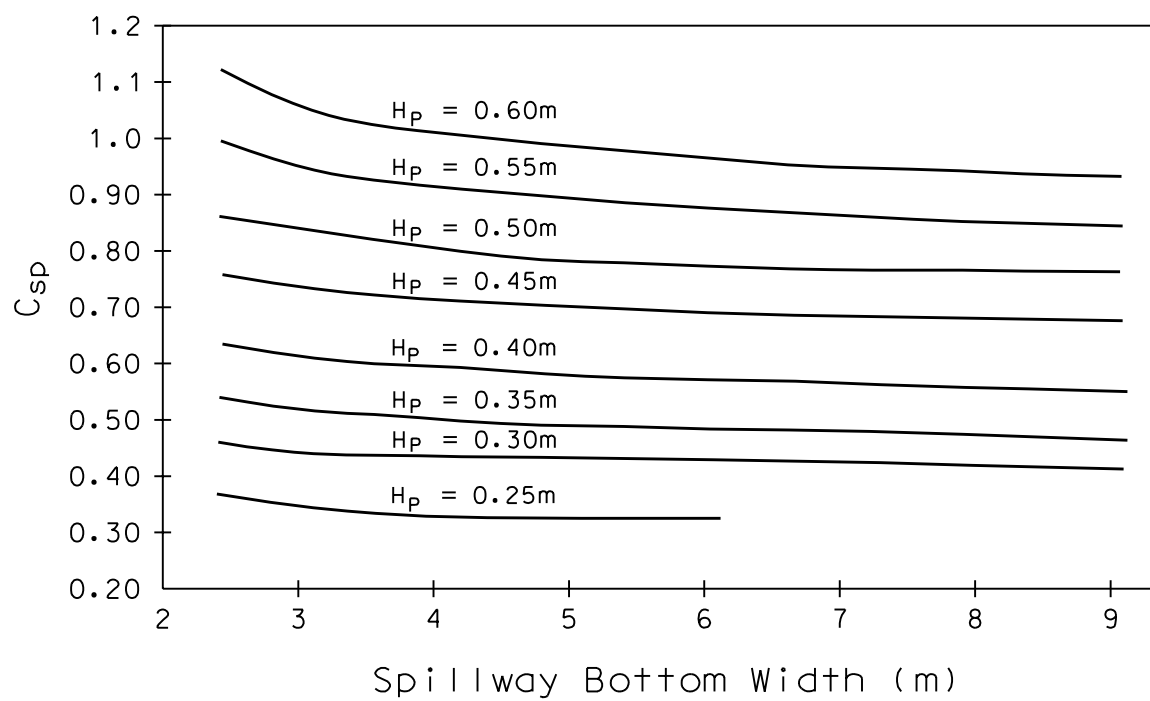


Figure 10-17 Discharge Coefficients for Emergency Spillways

Example 10-8

Given: An emergency spillway with the following characteristics:

$$\begin{aligned} \text{invert elev.} &= 11.6 \text{ m (38.0 ft)} \\ \text{width (b)} &= 5 \text{ m (16.4 ft)} \\ \text{discharge coeff. (C}_{SP}\text{)} &= 1.5 \text{ (2.7) (assumed constant)} \end{aligned}$$

Find: The stage - discharge rating for the spillway up to an elevation of 12.0 m (39.4 ft).

Solution: Using equation 10.26 with the given parameters yields:

$$\begin{aligned} Q &= C_{SP} b H_p^{1.5} \\ Q &= (1.5)(5)H_p^{1.5} = 7.5 (H_p)^{1.5} \end{aligned}$$

STAGE		EFFECTIVE HEAD ON SPILLWAY		SPILLWAY DISCHARGE	
(m)	(ft)	(m)	(ft)	(m ³ /s)	(ft ³ /s)
11.6	38.0	0.00	0.00	0.00	0.0
11.7	38.4	0.10	0.33	0.24	8.5
11.8	38.7	0.20	0.66	0.67	23.6
11.9	39.0	0.30	0.98	1.23	43.4
12.0	39.4	0.40	1.31	1.90	67.0

10.8.6 Composite Stage Discharge Curves

As indicated by the discussions in the preceding sections, development of a stage - discharge curve for a particular outlet control structure will depend on the interaction of the individual ratings for each component of the control structure. Figure 10-18 illustrates the construction of a stage - discharge curve for an outlet control device consisting of a low flow orifice and a riser pipe connected to an outflow pipe. The structure also includes an emergency spillway. These individual components are as described in examples 10-5, 10-6, and 10-8.

The impact of each element in the control structure can be seen in Figure 10-18. Initially, the low flow orifice controls the discharge. At an elevation of 10.8 m (35.4 ft) the water surface in the storage facility reaches the top of the riser pipe and begins to flow into the riser. The flow at this point is a combination of the flows through the orifice and the riser. As indicated in example 10-6, orifice flow through the riser controls the riser discharge above a stage of 11.0 m (36.1 ft). At an elevation of 11.6 meters (38.0 ft), flow begins to pass over the emergency spillway. Beyond this point, the total discharge from the facility is a summation of the flows through the low flow orifice, the riser pipe, and the emergency spillway. The data used to construct the curves in Figure 10-18 are tabulated in Table 10-4. Additionally, the designer needs to ensure that the outlet pipe from the detention basin is large enough to carry the total flows from the low orifice and the riser section. This ensures that the outlet pipe is not controlling the flow from the basin.

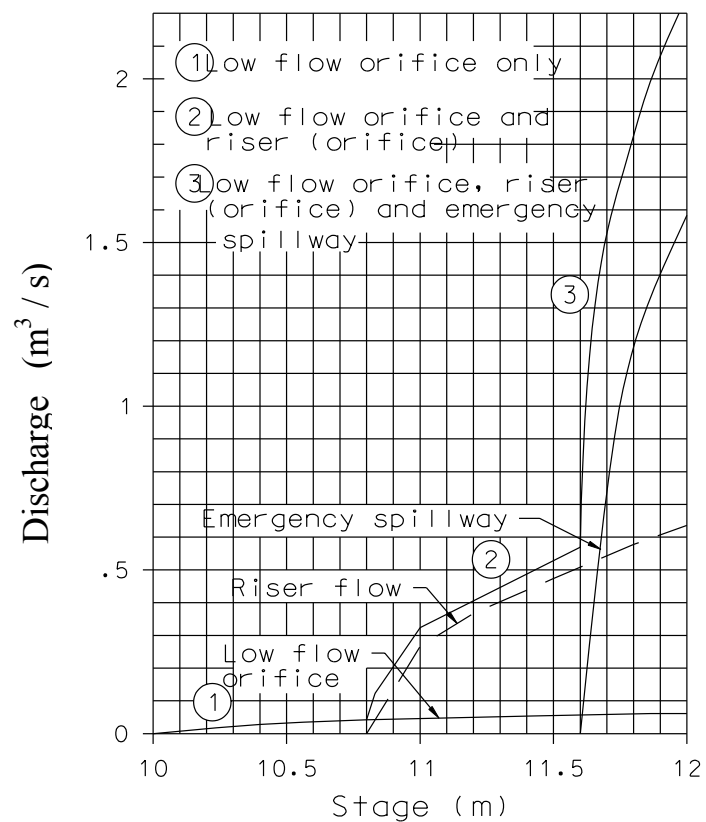


Figure 10-18 Typical Combined Stage-Discharge Relationship

Table 10-4 Stage - discharge tabulation.

STAGE		LOW FLOW ORIFICE	RISER ORIFICE FLOW	EMERGENCY SPILLWAY	TOTAL DISCHARGE	
(m)	(ft)	(m ³ /s)	(m ³ /s)	(m ³ /s)	(m ³ /s)	(ft ³ /s)
10.0	32.8	0.000	0.00	0.00	0.00	0.0
10.2	33.5	0.011	0.00	0.00	0.01	0.4
10.4	34.1	0.024	0.00	0.00	0.02	0.8
10.6	34.8	0.032	0.00	0.00	0.03	1.1
10.8	35.4	0.038	0.00	0.00	0.04	1.3
11.0	36.1	0.043	0.26	0.00	0.31	10.7
11.2	36.7	0.048	0.37	0.00	0.42	14.8
11.4	37.4	0.053	0.45	0.00	0.50	17.7
11.6	38.1	0.057	0.53	0.00	0.59	20.7
11.8	38.7	0.061	0.59	1.12	1.77	62.5
12	39.4	0.064	0.64	1.58	2.28	80.6

10.9 Routing Procedure

The most commonly used method for routing inflow hydrograph through a detention pond is the Storage Indication or modified Puls method. This method begins with the continuity equation which states that the inflow minus the outflow equals the change in storage ($I - O = \Delta S$). By taking the average of two closely spaced inflows and two closely spaced outflows, the method is expressed by equation 10.29. This relationship is illustrated graphically in Figure 10-19.

$$\frac{\Delta S}{\Delta t} = \frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} \quad (10.29)$$

where: ΔS = change in storage, m^3 (ft^3)
 Δt = time interval, min
 I = inflow, m^3 (ft^3)
 O = outflow, m^3 (ft^3)

In equation 10.29, subscript 1 refers to the beginning and subscript 2 refers to the end of the time interval.

Equation 10.29 can be rearranged so that all the known values are on the left side of the equation and all the unknown values are located on the right hand side of the equation, as shown in equation 10.30. Now, the equation with two unknowns, S_2 and O_2 , can be solved with one equation. The following procedure can be used to perform routing through a reservoir or storage facility using equation 10.30.

$$\frac{I_1 + I_2}{2} + \left(\frac{S_1}{\Delta t} + \frac{O_1}{2} \right) - O_1 = \left(\frac{S_2}{\Delta t} + \frac{O_2}{2} \right) \quad (10.30)$$

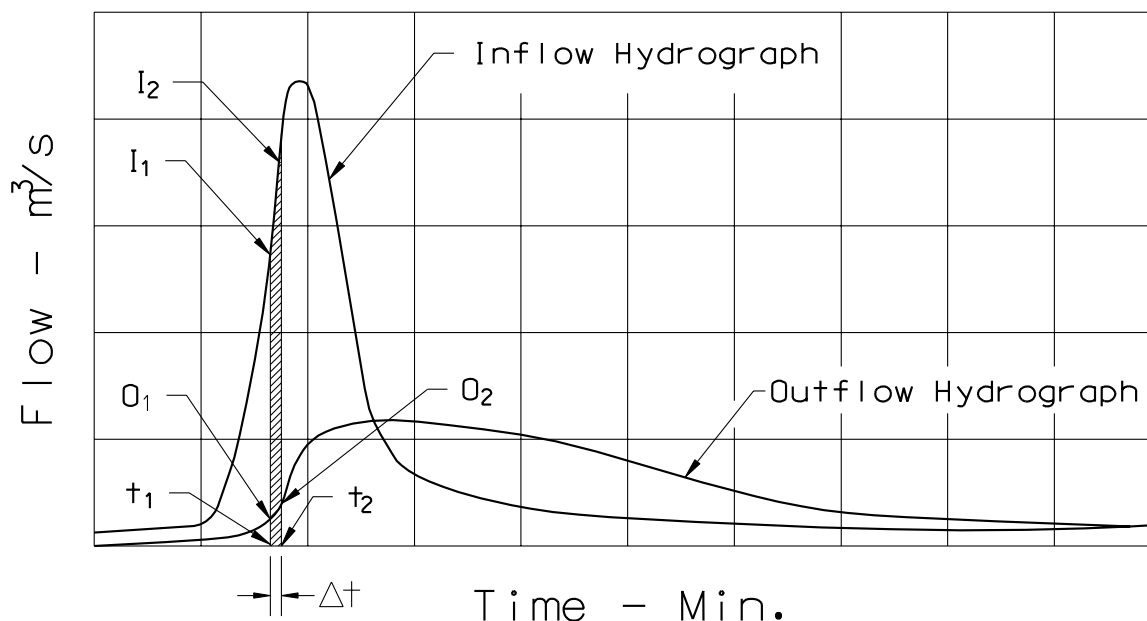


Figure 10-19 Routing hydrograph schematic

- Step 1. Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility.
- Step 2. Select a routing time period, Δt , to provide a minimum of five points on the rising limb of the inflow hydrograph.
- Step 3. Use the stage-storage and stage-discharge data from Step 1 to develop a storage indicator numbers table that provides storage indicator values, $S/(\Delta t) + O/2$, versus stage. A typical storage indicator numbers table contains the following column headings:

(1)	(2)	(3)	(4)	(5)	(6)
Stage	Discharge (O)	Storage (S)	$O_2/2$	$S_2/\Delta t$	$S_2/\Delta t + O_2/2$
(m)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	

- The discharge (O) and storage (S) are obtained from the stage-discharge and stage-storage curves, respectively.
 - The subscript 2 is arbitrarily assigned at this time.
 - The time interval (Δt) must be the same as the time interval used in the tabulated inflow hydrograph.
- Step 4. Develop a storage indicator numbers curve by plotting the outflow (column 2) vertically against the storage indicator numbers in column (6). An equal value line plotted as $O_2 = S_2/\Delta t + O_2/2$ should also be plotted. If the storage indicator curve crosses the equal value line, a smaller time increment (Δt) is needed (refer to Figure 10-20).
- Step 5. A supplementary curve of storage (column 3) vs. $S_2/\Delta t + O_2/2$ (column 4) can also be constructed. This curve does not enter into the mainstream of the routing; however, it is useful for identifying storage for any given value of $S_2/\Delta t + O_2/2$. A plot of storage vs. time can be developed from this curve.
- Step 6. The routing can now be performed by developing a routing table (see Example 10-9) for the solution of equation 10.30 as follows:
- Columns (1) and (2) are obtained from the inflow hydrograph.
 - Column (3) is the average inflow over the time interval.
 - The initial values for columns (4) and (5) are generally assumed to be zero since there is no storage or discharge at the beginning of the hydrograph when there is no inflow into the basin.
 - The left side of equation 10.30 is determined algebraically as columns (3) + (4) - (5). This value equals the right side of equation 10.30 or $S_2/\Delta t + O_2/2$ and is placed in column (6).

- e. Enter the storage indicator curve with $S_2/\Delta t + O_2/2$ (column 6) to obtain O_2 (column 7).
- f. Column (6) ($S_2/\Delta t + O_2/2$) and column (7) (O_2) are transported to the next line and become $S_1/\Delta t + O_1/2$ and O_1 in columns (4) and (5), respectively. Because ($S_2/\Delta t + O_2/2$) and O_2 are the ending values for the first time step, they can also be said to be the beginning values for the second time step.
- g. Columns (3), (4), and (5) are again combined and the process is continued until the storm is routed.
- h. Peak storage depth and discharge (O_2 in column (7)) will occur when column (6) reaches a maximum. The storage indicator numbers table developed in Step 3 is entered with the maximum value of $S_2/\Delta t + O_2/2$ to obtain the maximum amount of storage required. This table can also be used to determine the corresponding elevation of the depth of stored water.
- i. The designer needs to make sure that the peak value in column (7) does not exceed the allowable discharge as prescribed by the stormwater management criteria.

Step. 7. Plot O_2 (column (7)) versus time (column (1)) to obtain the outflow hydrograph.

The above procedure is illustrated in the following example.

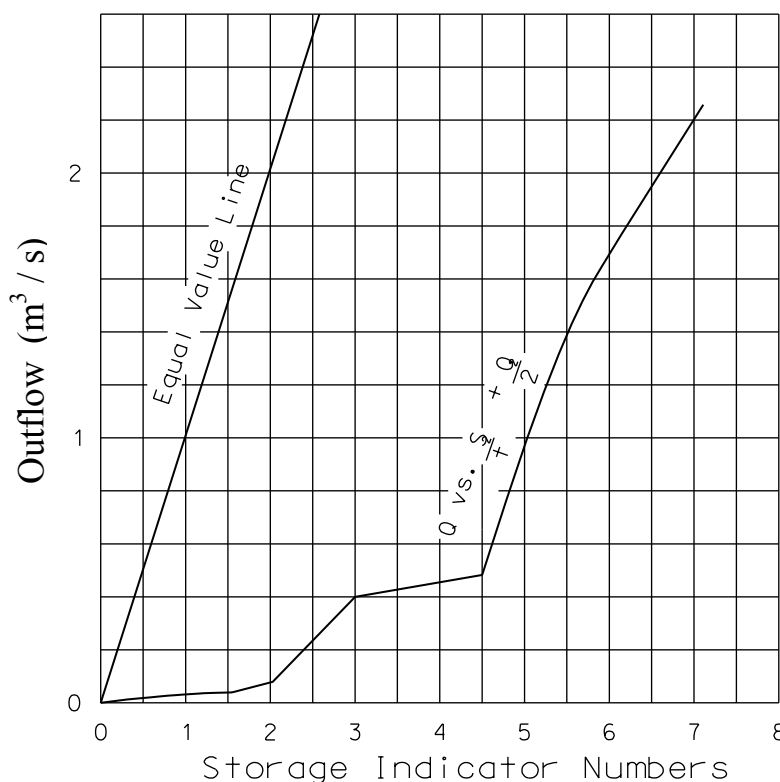


Figure 10-20 Storage Indicator Curve

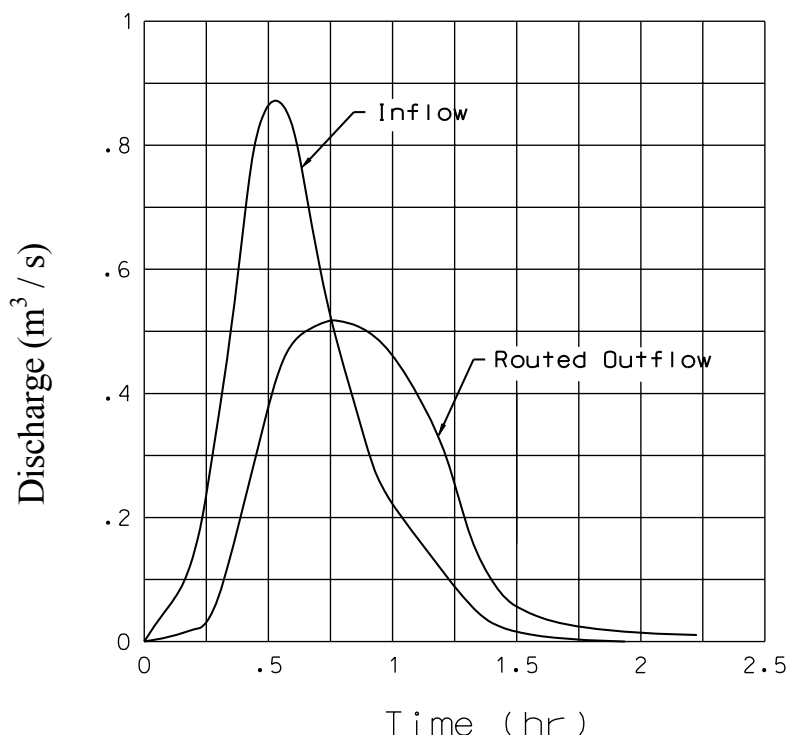


Figure 10-21 Example hydrographs

Example 10-9

Given: The inflow hydrograph from Figure 10-21, the storage basin, and a discharge control structure comprised of the components in examples 10-5 through 10-8 having a composite stage-discharge relationship defined in Table 10.4.

Find: The outflow hydrograph.

Solution: Use the generalized routing procedure outlined above.

Step 1. The inflow hydrograph is tabulated in the final routing table to follow and illustrated in Figure 10-19. The stage-storage curve for the basin in example 10-2 is illustrated in Figure 10-5. The composite stage-discharge curve tabulated in Table 10-4 is illustrated in Figure 10-18.

Step 2. A routing time interval of 0.057 hrs is selected to match the interval used in the inflow hydrograph. This interval provides 9 points on the rising limb of the hydrograph.

Step 3. Using the given stage-storage and stage-discharge curve data, and a time step of 0.057 hr, storage indicator numbers table can be developed as illustrated in the table on page 10.9-6.

Step 4. A storage indicator curve is constructed by plotting the outflow (column 2 from the table below) against the storage indicator numbers (column 6). This curve is illustrated in Figure 10-20. Note that an equal value line is also plotted in Figure 10-20, and since the storage indicator numbers curve does not cross the equal value line, the time step selected is adequate.

Step 5. (The supplementary curve of storage (column 3) vs. the storage indicator numbers (column 6) is not required for this example.)

Step 6. The final routing is shown in the final routing table below.

Step 7. The inflow and outflow hydrographs are plotted in Figure 10-21.

As indicated in the final routing table, the peak routed outflow is $0.52 \text{ m}^3/\text{s}$ ($18.4 \text{ ft}^3/\text{s}$). This peak outflow is less than the maximum allowable value of $0.55 \text{ m}^3/\text{s}$ ($19.4 \text{ ft}^3/\text{s}$). Using the table below, it can be determined that the outflow of $0.52 \text{ m}^3/\text{s}$ ($18.4 \text{ ft}^3/\text{s}$) corresponds to an approximate basin stage of 11.46 m (37.6 ft) which is less than the maximum available stage of 11.6 m (38.1 ft) established in example 10-2, and a storage volume of 794 m^3 ($28,020 \text{ ft}^3$) which is less than the estimated storage requirement of 850 m^3 ($30,000 \text{ ft}^3$) estimated in example 10-2. At the peak stage of 11.46 m (37.6 ft), the water is not flowing over the emergency spillway. Only the riser pipe and the low flow orifice are functioning.

Storage Indicator Numbers Table

(1) Stage (m)	(2) Discharge (O_2) (m^3/s)	(3) Storage (S_2) (m^3)	(4) $O_2/2$ (m^3/s)	(5) $S_2/\Delta t$ (m^3/s)	(6) $S_2/t + O_2/2$ (m^3/s)
10.0	0.000	0	0.000	0.000	0.000
10.1	0.006	35	0.003	0.171	0.174
10.2	0.011	72	0.006	0.351	0.357
10.3	0.018	109	0.009	0.531	0.540
10.4	0.024	155	0.012	0.755	0.767
10.5	0.028	199	0.014	0.970	0.984
10.6	0.032	248	0.016	1.209	1.225
10.7	0.035	299	0.018	1.457	1.475
10.8	0.038	353	0.019	1.720	1.739
10.9	0.171	414	0.086	2.018	2.104
11.0	0.303	470	0.152	2.290	2.442
11.1	0.378	540	0.189	2.632	2.821
11.2	0.418	600	0.209	2.924	3.133
11.3	0.464	672	0.235	3.275	3.510
11.4	0.503	744	0.252	3.626	3.878
11.5	0.530	824	0.265	4.016	4.281
11.6	0.587	902	0.294	4.396	4.690
11.7	1.150	986	0.575	4.805	5.380
11.8	1.771	1075	0.886	5.239	6.125
11.9	2.069	1160	1.035	5.653	6.688
12.0	2.284	1264	1.142	6.160	7.302

As a last check, it must be determined that the discharge pipe will carry the peak discharge at a head equal to or less than the final stage in the basin. At the peak discharge indicated in the Final Routing Table, the head on the discharge pipe is 1.46 m (4.8 ft). In example 10-7, it was determined that a 750 mm (30 in) discharge pipe would carry a discharge greater than $0.55 m^3/s$ ($19.4 ft^3/s$) at a stage of 0.75 m (2.51 ft). Therefore, the discharge pipe capacity is adequate, and the pipe will not control the flow of water from the basin.

Final Routing Table.

(1) Time (hr)	(2) Inflow (m ³ /s)	(3) (I ₁ +I ₂)/2 (m ³ /s)	(4) (S ₁ /t+O ₁ /2) (m ³ /s)	(5) O ₁ (m ³ /s)	(6) (S ₂ /t+O ₂ /2) (m ³ /s)	(7) O ₂ (m ³ /s)	(8) O ₂ (ft ³ /s)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
0.06	0.04	0.02	0.00	0.00	0.02	0.00	0.0
0.11	0.07	0.06	0.02	0.00	0.08	0.00	0.0
0.17	0.12	0.10	0.08	0.00	0.17	0.01	0.4
0.23	0.18	0.15	0.17	0.01	0.31	0.01	0.4
0.29	0.33	0.26	0.31	0.01	0.56	0.02	0.7
0.34	0.49	0.41	0.56	0.02	0.95	0.03	1.1
0.40	0.67	0.58	0.95	0.03	1.50	0.04	1.4
0.46	0.81	0.74	1.50	0.04	2.20	0.21	7.4
0.51	0.88	0.85	2.20	0.21	2.83	0.38	13.4
0.57	0.86	0.87	2.83	0.38	3.32	0.45	15.9
0.63	0.79	0.83	3.32	0.45	3.70	0.49	17.3
0.68	0.69	0.74	3.70	0.49	3.95	0.51	18.0
0.74	0.57	0.63	3.95	0.51	4.07	0.52	18.3
0.80	0.48	0.53	4.07	0.52	4.07	0.52	18.3
0.86	0.39	0.44	4.07	0.52	3.99	0.51	18.0
0.91	0.32	0.36	3.99	0.51	3.83	0.50	17.6
0.97	0.26	0.29	3.83	0.50	3.62	0.48	16.9
1.03	0.22	0.24	3.62	0.48	3.38	0.45	15.9
1.08	0.18	0.20	3.38	0.45	3.13	0.42	14.8
1.14	0.15	0.17	3.13	0.42	2.88	0.39	13.8
1.20	0.11	0.13	2.88	0.39	2.62	0.34	12.0
1.25	0.09	0.10	2.62	0.34	2.38	0.28	9.9
1.31	0.05	0.07	2.38	0.28	2.17	0.20	7.1
1.37	0.03	0.04	2.17	0.20	2.01	0.14	4.9
1.43	0.00	0.02	2.01	0.14	1.88	0.07	2.5
1.48	0.00	0.00	1.88	0.07	1.81	0.06	2.1
1.54	0.00	0.00	1.81	0.06	1.75	0.05	1.8
1.60	0.00	0.00	1.75	0.05	1.70	0.04	1.4
1.65	0.00	0.00	1.70	0.04	1.66	0.04	1.4
1.71	0.00	0.00	1.66	0.04	1.62	0.04	1.4
1.77	0.00	0.00	1.62	0.04	1.58	0.04	1.4
1.82	0.00	0.00	1.58	0.04	1.54	0.04	1.4
1.88	0.00	0.00	1.54	0.04	1.50	0.04	1.4
1.94	0.00	0.00	1.50	0.04	1.46	0.03	1.1
1.99	0.00	0.00	1.46	0.03	1.43	0.03	1.1
2.05	0.00	0.00	1.43	0.03	1.40	0.03	1.1
2.11	0.00	0.00	1.40	0.03	1.37	0.03	1.1
2.17	0.00	0.00	1.37	0.03	1.34	0.03	1.1

10.10 Temporary Sediment Facilities

10.10.1 Temporary Sediment Basin

Temporary sediment basins shall be designed in accordance with the "2002 Connecticut Guidelines for Soil Erosion and Sediment Control." The following general information was taken from that publication.

Definition

A temporary dam, excavated pit or dugout pond constructed across a waterway or at other suitable locations with a controlled outlet(s) such that a combination of wet and dry storage areas are created. A basin that is created by the construction of a dam is classified as an *embankment sediment basin* and a basin that is constructed by excavation is an *excavated sediment basin*. A basin that is created by a combination of dam construction and excavation is classified as an *embankment sediment basin* when the depth of water impounded against the embankment at emergency spillway elevation is three feet or more.

Purpose

- to intercept and retain sediment during construction
- to reduce or abate water pollution
- to prevent undesirable deposition of sediment in wetlands, on bottom lands and developed areas
- to preserve the capacity of reservoirs, ditches, canals, diversions, storm sewers, waterways and streams

Applicability

- below disturbed areas with a contributing drainage area less than 100 acres. For drainage areas less than five acres, a temporary sediment trap may be used.
- only for locations where failure of the temporary sediment basin will not, within reasonable expectations, result in loss of life or damage to buildings, roads, railroads or utilities
- not for use as a post-construction stormwater renovation system

Refer to the referenced publication for planning considerations and design criteria for temporary sediment basins.

10.10.2 Temporary Sediment Trap

Like temporary sediment basins, temporary sediment traps shall be designed in accordance with the "2002 Connecticut Guidelines for Soil Erosion and Sediment Control." The following general information was taken from that publication.

Definition

A temporary ponding area with a stone outlet formed by excavation and/or constructing an earthen embankment.

Purpose

- to detain sediment-laden runoff from small disturbed areas long enough to allow a majority of the sediment to settle out.

Applicability

- below disturbed areas where the contributing drainage area is 5 acres or less. For drainage areas greater than 5 acres use temporary sediment basin measure.
- where the intended use is 2 years or less. For uses greater than 2 years use temporary sediment basin measure.
- when diverting sediment-laden water with temporary diversions that meet the above limitations for use

Refer to the referenced publication for planning considerations and specifications for temporary sediment traps.

10.11 Dry Detention Basin

10.11.1 Introduction

Dry detention basins are depressed areas that store runoff during wet weather and are dry the rest of the time. They are very popular because of their comparatively low cost; few design limitations; and ability to serve large as well as small watersheds. Dry detention basins are an impoundment made by constructing a dam or an embankment (embankment detention basin), or by excavating a pit or dugout (excavated detention basin). Basins resulting from both excavation and embankment construction are classified as embankment detention basins where the depth of water impounded against the embankment at emergency spillway elevation is 0.9m (3 ft) or more. Its purpose is to regulate the rate and amount of runoff from development sites during and after construction operations and to minimize the effects of downstream erosion and flooding. They should be considered when there is a need to control or prevent downstream erosion and flooding due to site development or from other land use changes.

10.11.2 Regulatory Considerations

Design the detention basin to be compatible with State floodplain management and storm water management programs as well as local regulations for controlling sediment, erosion and runoff. The basin shall properly regulate storm discharges from a site to a safe, adequate outlet. Consider the duration of flow as well as the peak discharge. Provide adequate erosion control measures and other water-quality practices. Plan and design the basin to ensure minimal impact on visual quality and human enjoyment of the landscape. Blend structures and materials aesthetically with their surroundings.

Attempt to locate detention basins where:

1. Failure of the detention basin would not, within reasonable expectations, result in loss of life, damage all-weather roads, railroads, homes, commercial and industrial properties or interrupt the use or service of utilities. (Dams which might fail and endanger life or property are regulated by the Commissioner of the Department of Environmental Protection under the sections 22a-401 through 22a-411, CGS as discussed in Section 10.5).
2. The effective height of the dam for an embankment detention basin should be 4.6m (15 feet) or less. The effective height of the dam is defined as the difference in elevation in meters (feet) between the emergency spillway crest and the lowest point in the cross section taken along the centerline of the dam. If there is no spillway, the top of the dam becomes the upper limit; and/or the product of the storage times the effective height of the dam should be less than 1,130,000 (3,000). Storage is the volume in m³ (acre-feet) in the reservoir below the elevation of the crest of the emergency spillway.

Detention basins that exceed any one of the above conditions shall be designed to meet the criteria in Earth Dams and Reservoirs, Technical Release 60 (TR-60) (Rev. 10/85) by the Natural Resources Conservation Service (NRCS), U.S. Department of Agriculture (I).

10.11.3 Sizing Requirements

Stormwater Quality To design the basin for stormwater quality control, the water quality volume (WQV) must be routed through the basin. As described in the Connecticut Stormwater Quality Manual, “the water quality volume (WQV) is the amount of stormwater runoff from any given storm that should be captured and treated in order to remove a majority of stormwater pollutants on an average annual basis. The recommended WQV, which results in the capture and treatment of the entire runoff volume for 90 percent of the average annual storm events, is equivalent to the runoff associated with the first one-inch of rainfall.” This runoff is typically referred to as the “first-flush”.

The WQV is determined from the following equation:

$$\text{WQV} = \frac{(25.4\text{mm})(R)(A)}{1000} \text{ or } \frac{(1'')(R)(A)}{12} \quad (10.31)$$

where: WQV = Water Quality Volume, hectare meters (acre ft)
 R = Volumetric Runoff Coefficient=0.05+0.009(I)
 I = Percent of Impervious Cover (%)
 A = Site area, hectares (acres)
 (Note: 1 hectare=10,000 m²)

The WQV should be detained and released over a period of 24 h. The following equation may be used to obtain the average outflow from the basin:

$$Q_{\text{avg}} = \frac{10,000(\text{WQV})}{T} \text{ or } \frac{43560(\text{WQV})}{T} \quad (10.32)$$

Where: Q_{avg} = average outflow from the basin, m³/s (cfs)
 WQV = water quality volume, hectare meters (acre ft)
 T = detention time, s

Knowing the average outflow, one can use an appropriate orifice equation to determine the size of the outlet. To help prevent clogging, the minimum recommended size of the outlet should be 200mm (8 in). However, smaller diameters (minimum of 76 mm or 3 in) may be allowed if adequately protected from clogging by an acceptable external trash rack.

Although stormwater treatment basins presented in this chapter are sized based on WQV, some treatment practices such as grass drainage channels, hydrodynamic separators and flow diversion structures for off-line stormwater treatment practices, are more appropriately designed based on peak flow rate or Water Quality Flow (WQF). The WQF is the peak flow rate based on the WQV as defined above. The WQF is determined using the procedures outlined in Chapter 11, Appendix C of the Drainage Manual.

Stormwater Quantity For quantity purposes, the basin shall be designed to reduce the post-construction peak flow from a 2, 10, and 25-year storm (considered individually) to the preconstruction level, and it shall be able to pass a 100-year storm safely. To control these storms, the basin's storage shall be equal to the area between the pre- and post-construction hydrographs.

After a storage volume has been determined for each event, a 2, 10, and 25-year storm should be routed through the facility to ensure that the peak flows from the post-construction watershed are not greater than the corresponding preconstruction peak flows. Finally, a 100-year storm shall be routed through the facility to ensure that the embankment will not be damaged or fail during the

passage of that storm. It is common to have several outlet configurations to control the different storms: one or more for the frequently occurring storms and an emergency spillway to control anything larger, including the 100-year storm, (Figure 10-25). To improve the efficiency of the outlet, it may be necessary to include an antivortex device for the riser overflow component (Figure 10-26).

If the primary purpose of the detention basin is to minimize flooding, the peak discharge from the 2-year, 10-year, 25-year and 100-year frequency, 24-hour duration, Type III distribution storms shall be analyzed.

No increase in peak flow from the 2-year, 10-year, and 25-year storms shall be allowed unless downstream increases are compatible with an overall floodplain management system. Check local requirement for additional criteria that may include larger storms. Some of the items to consider in determining if increased peak flows are compatible with an overall floodplain management system are:

1. the timing of peak flows from sub-watershed
2. the increased duration of high flow rates
3. the stability of the downstream channels
4. the distance downstream that the peak discharges are increased

Stormwater Quantity and Quality Combined Combining the two designs (quality and quantity) will yield a dual-purpose detention basin. Several other design variations may be considered to enhance the capabilities of the facility. One consideration is shaping the basin to improve its pollutant-removal capabilities. The length-to-width ratio should be at least 3:1, and a wedge-shaped basin (wider at the outlet) can also improve pollutant removal. The inlet, outlet and side slopes should be stabilized with riprap and/or vegetation to prevent erosion. The basin floor should also be vegetated to stabilize the soil and increase biological uptake. The pond floor should be sloped no less than 2% to prevent the ponding of stormwater, and the side slopes should allow for easy maintenance access. A marsh or wetland can be established on the pond floor to increase biological uptake, and a sediment forebay (a small sediment trap at the inlet of the basin, either a depressed area or a shallow area with a very flat slope where sediment is easily deposited) can be used to catch the sediment before it fills the basin (see Figure 10-29). Design guidance for sediment forebays is outlined in Section 10.12.2.

The basin can also have two stages: one to hold the smaller storms, and a second, which is rarely inundated and can be used for other purposes, to help store the larger storms. Safety considerations include reducing the chance of drowning by fencing the basin, reducing the maximum depth and/or including ledges and mild slopes to prevent people from falling in and facilitate their escape from the basin.

10.11.4 Outlets

The outlets for the basin shall consist of a combination of an outlet control structure (sometimes referred to as a principal spillway) and an emergency spillway. These outlets shall pass the peak runoff from the contributing drainage area for the design flood. If, due to site conditions and basin geometry, a separate emergency spillway is not feasible, the outlet control structure shall pass the entire routed peak runoff expected from the design storm. However, an attempt to provide a separate emergency spillway shall always be made. An emergency spillway shall be provided on all detention basins with a contributing drainage area equal to or exceeding 8 hectares (20 acres).

Outlet Control Structure (Principal Spillway) As previously indicated, these structures are sometimes referred to as principal spillways, however, outlet control structure will be used in this manual. Outlet control structures for detention basins can be designed in a wide variety of configurations. Most outlet control structures use riser pipes of concrete or corrugated metal. These risers can be designed to control different storms through the use of several orifices on the riser, for example, a small diameter to control the WQV, an orifice to control a 2-year storm, and larger openings to control the 10 and 25 year storms. Since the WQV outlet must be small to detain the WQV long enough, it can be easily clogged; thus, the minimum recommended size of the outlet should be 200mm (8 in). However, smaller diameters (minimum of 76mm or 3 in) may be allowed if adequately protected from clogging by an acceptable external trash rack. The larger flows are usually controlled by stormwater flowing in through the top of the riser, using the entire riser circumference. Under these circumstances an antivortex and trash rack design may be necessary to improve the flow characteristics into the structure and prevent blockage from floating debris.

The base of the structure shall be firmly anchored to prevent its floating. If the riser is greater than 3.0 m (10 ft) in height, computations shall be done to determine the anchoring requirements. As a minimum, a factor of safety of 1.25 shall be used (downward forces = 1.25 X upward forces).

The barrel of the outlet structure, which extends through the embankment, shall be designed to carry the flow provided by the riser with the water level at the crest of the emergency spillway. The connection between the riser and the barrel shall be watertight. The outlet of the barrel shall be protected to prevent erosion or scour of downstream areas. (See Chapter 8, Section 8.7, Outlet Protection.)

The outlet control structure and amount of storage shall be sized to prevent the emergency spillway from overtopping for a 25-year design storm. If no emergency spillway is used, the outlet control structure shall be designed to pass the entire routed peak flow expected from the 100 year storm.

The crest of the outlet control structure shall be set at the elevation corresponding to the storage volume required. If severe sedimentation is expected, the design volume should include provisions for sediment volume.

Emergency Spillway The emergency spillway shall consist of an open channel (earthen and vegetated) constructed adjacent to the embankment over undisturbed material not within the embankment fill. Where conditions require the construction of an emergency spillway on the embankment fill, a spillway shall be constructed of a non-erodible material such as riprap.

The minimum capacity of a natural or constructed emergency spillway shall be that required to pass the peak flow expected from a design storm of 100-year frequency, 24-hour duration, Type III distribution less any reduction creditable to conduit discharge and detention storage.

Elevations If the outlet control structure is used in conjunction with an emergency spillway, the crest of the outlet control structure shall be a minimum of 0.3 m (1 ft) below the crest of the emergency spillway. If no emergency spillway is used, the crest of the outlet control structure shall be a minimum of 0.9 m (3 ft) below the top of the embankment. In either case, a minimum freeboard of 0.3 m (1 ft) should be provided between the 100 year storm and the top of the embankment elevations.

10.11.5 Embankment Cross Sections

The fill material shall be clean mineral soil, free of roots, woody vegetation, oversized stones, rocks, or other unsuitable material. Areas on which fill is to be placed shall be scarified prior to the placement of fill. Fill material will be placed in 150- to 200-mm (6-8 in) continuous layers over the entire length of the fill. Compaction shall be obtained by routing the hauling equipment over the fill

so that the entire surface of the fill is traversed by at least one wheel or tread track of the equipment, or by using a compactor.

The embankment shall have a minimum top width of 2.5 m (8 ft). The side slopes shall be 1V:2H or flatter. The embankment may have a maximum height of 3 m (10 ft) if the side slopes are 1V:2H. If the side slopes are 1V:2.5H or flatter, the embankment may have a maximum height of 4.5 m (15 ft). If there is a significant impoundment, a dam permit will be required. (See Section 10.5 for additional information.)

Table 10-5 Summary Of Considerations For A Dry Basin

Quality	Detain WQV for 24 h recommended minimum 200mm (8 in) orifice, 76mm (3 in) minimum with trash rack
Quantity	Control 2- and 10-, and 25-year peak flows and maintain non-erosive velocity
Shape	3:1 length-to-width ratio; wedge shaped (wider at outlet)
Maintenance	Inspect a minimum once a year, preferably during wet weather, mow as required; remove sediment (every 5-10 years, or as required)
Other considerations	Side slopes provide easy maintenance access (1V:4H); 2% bottom slope to prevent ponding; sediment forebay to reduce maintenance; safety requirements (depth and perimeter ledges)
Pollutant removal	Moderate

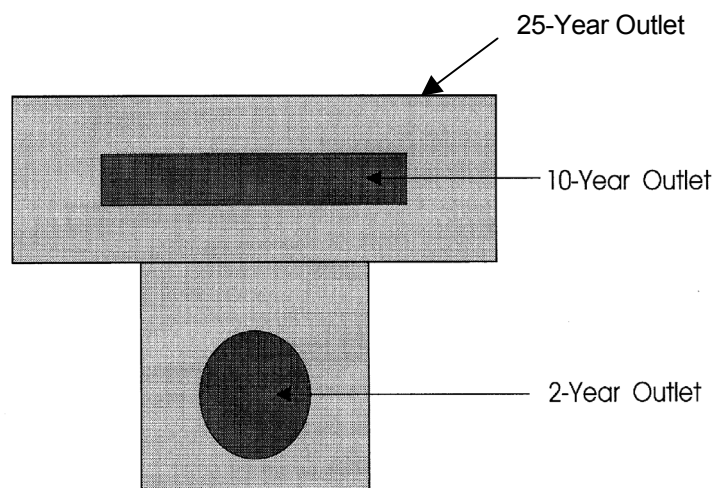


Figure 10-25 Concrete Riser

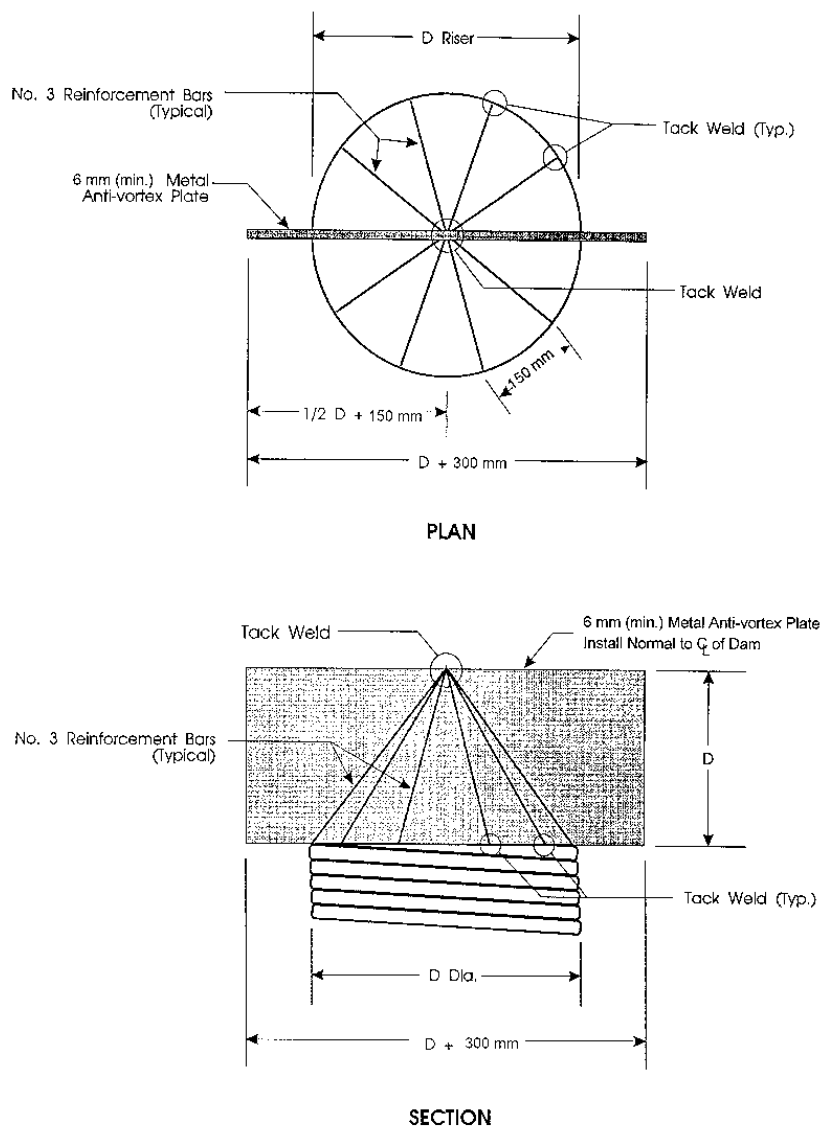


**Figure 10-26 Antivortex Plate And Trash Rack
(Design of Urban Highway Drainage, FHWA TS 79-225)**

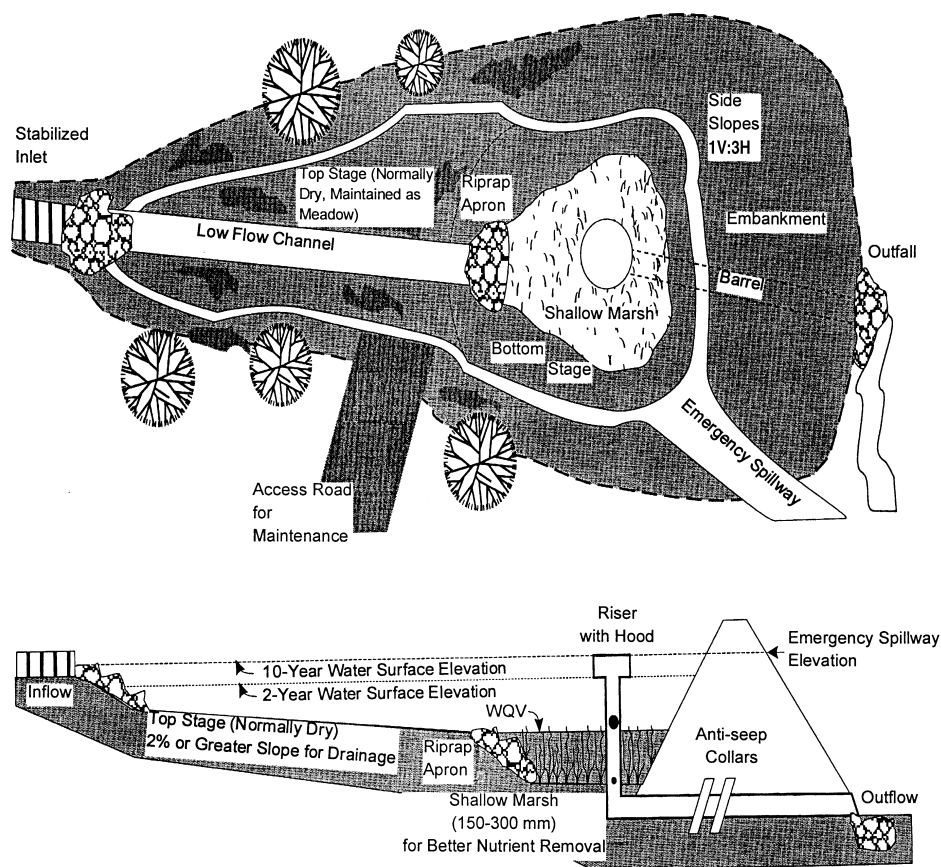


Figure 10-27 Dry Basin (after Schueler)

10.12 Wet Detention Basin

10.12.1 Introduction

A wet detention basin is very similar to a dry detention basin in that it detains stormwater, but it is different in that it retains a permanent pool during dry weather. Wet detention basins are usually more expensive to construct and maintain than dry detention basins and usually serve large watersheds.

10.12.2 Regulatory Considerations

See Section 10.11.2.

10.12.3 Sizing Requirements

Stormwater Quality For quality purposes, the permanent pool must be at least equal to the WQV for the watershed. See Section 10.11.3 for WQV calculation. The theory behind this is that incoming runoff displaces old stormwater from the basin and the new runoff is detained until it is displaced by more runoff from the next storm. A permanent pool equal to the WQV should then provide an adequate detention time for the stormwater. Watershed size, soil conditions and groundwater elevation must be evaluated to ensure the capability of the site to support a permanent wet basin. To enhance pollutant and sediment removal, several other considerations may be taken into account, including a sediment forebay.

The shape of the basin can significantly affect the pollutant-removal efficiency. The length-to-width ratio should be at least 3:1. Figure 10-28 shows basin configurations that may be used to increase the length-to-width ratio and allow for maximum flow path length. Basin depth should be between 1.5 and 3.0 m (5 and 10 ft); less could allow insect breeding and wind resuspension of settled particles, and more could lead to thermal stratification in the basin and anaerobic conditions in the deep water. A wedge-shaped basin, wider at the outlet, can also improve pollutant removal (Figure 10-29).

Stormwater Quantity For quantity purposes, the wet detention basin is designed similarly to the dry detention basin. The basin should be designed to reduce the peak flow from the 2, 10, and 25 year storms (considered individually) and be able to pass a 100-year storm safely.

10.12.4 Outlets

Outlet requirements for wet detention basins are the same as those for dry detention basins, therefore see Section 10.11.4 for outlet design.

10.12.5 Sediment Forebay

A sediment forebay is recommended for all wet detention basins. The purpose of the forebay is to provide pretreatment by settling out coarse sediment particles, which will enhance treatment performance, reduce maintenance, and increase the longevity of a stormwater basin. A forebay is a separate cell within the basin formed by a barrier such as an earthen berm, concrete weir, or gabion baskets.

- The forebay should be sized to contain at least 10% of the WQV and be of an adequate depth to prevent resuspension of collected sediments during the design storm, often being four to six feet deep. The goal of the forebay is to at least remove particles consistent with the size of medium sand. The forebay storage volume may be used to fulfill the total WQV requirement of this system. The forebay must also include additional sediment storage volume that may not be used for WQV calculations. Sediment storage volume is calculated by the Universal Soil Loss Equation in the *Connecticut Guidelines for Soil Erosion and Sediment Control*.
- The outlet from the forebay should be designed in a manner to prevent erosion of the embankment and primary pool. This outlet can be configured in a number of ways including a culvert, weir, or spillway channel. The outlet should be designed to convey the same design flow proposed to enter the basin. The outlet invert must be elevated in a manner such that 10% of the WQV can be stored below it in addition to the required sediment volume.
- The forebay should have a minimum length to width ratio of 2:1 and a preferred length to width ratio of 3:1.
- Direct access for appropriate maintenance equipment should be provided to the forebay and may include a ramp to the bottom if equipment cannot reach all points within the forebay from the top. The forebay can be lined with a concrete pad to allow easy removal of sediment and to minimize the possibility of excavating subsurface soils or undercutting embankments during routine maintenance.
- A fixed vertical sediment depth marker should be installed in the forebay to measure sediment deposition.
- A barrier, such as an earthen berm, gabions, check dam, or a concrete weir may be used to separate the forebay from the permanent pool. This barrier should be armored as necessary to prevent erosion of the embankment if it overtops. This armoring could consist of materials such as riprap, pavers, or geosynthetics designed to resist slope erosion. If a channel is used to convey flows from the forebay to the basin, the side slopes of the channel must be armored as well.
- Additional pretreatment can be provided in the forebay by raising the embankment to provide some detention of incoming flows.

10.12.6 Embankment Cross Sections

See Section 10.11.5.

Table 10-6 Summary Of Considerations For A Wet Basin

Quality	Permanent pool volume is equal to the WQV
Quantity	Control 2, 10 and 25- year peak flows
Shape	3:1 length-to-width ratio; wedge shaped (wider at outlet); permanent pool depth from 1.5 - 3.0 m (5 - 10 ft); perimeter ledges
Maintenance	Inspect once a year, preferably during wet weather; mow as required; remove sediment every 5-10 years or as required
Safety	Fence around basin; provide shallow (0.5 m (1.5 ft) deep) safety ledge around basin; post signs
Other considerations	Side slopes provide easy maintenance access (1V:4H); perimeter vegetation; sediment forebay; provide valve to drain basin for maintenance
Pollutant removal	Moderate to high

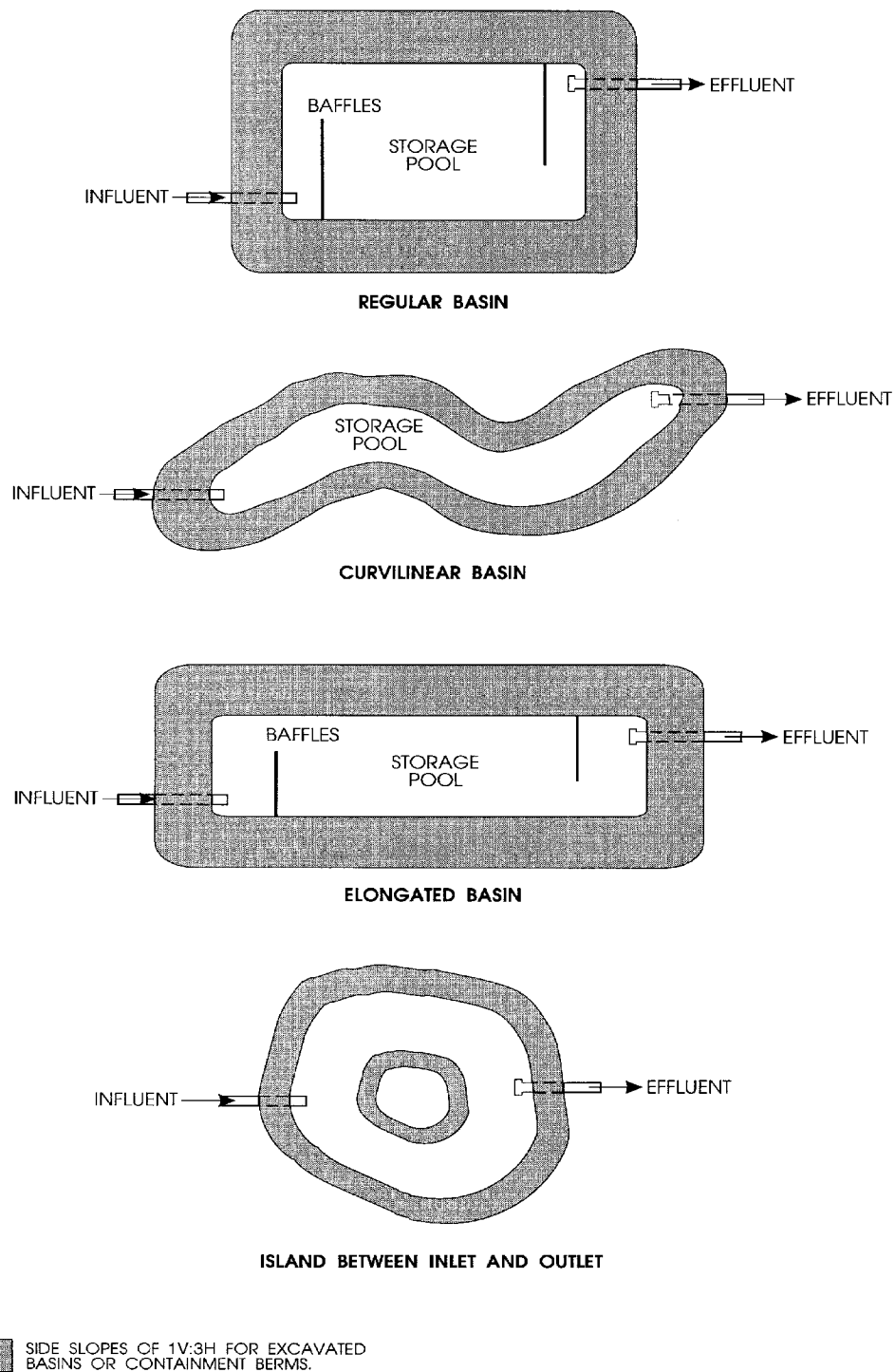


Figure 10-28 Methods Of Increasing The Length-to-Width Ratio (after Schueler, 1987)

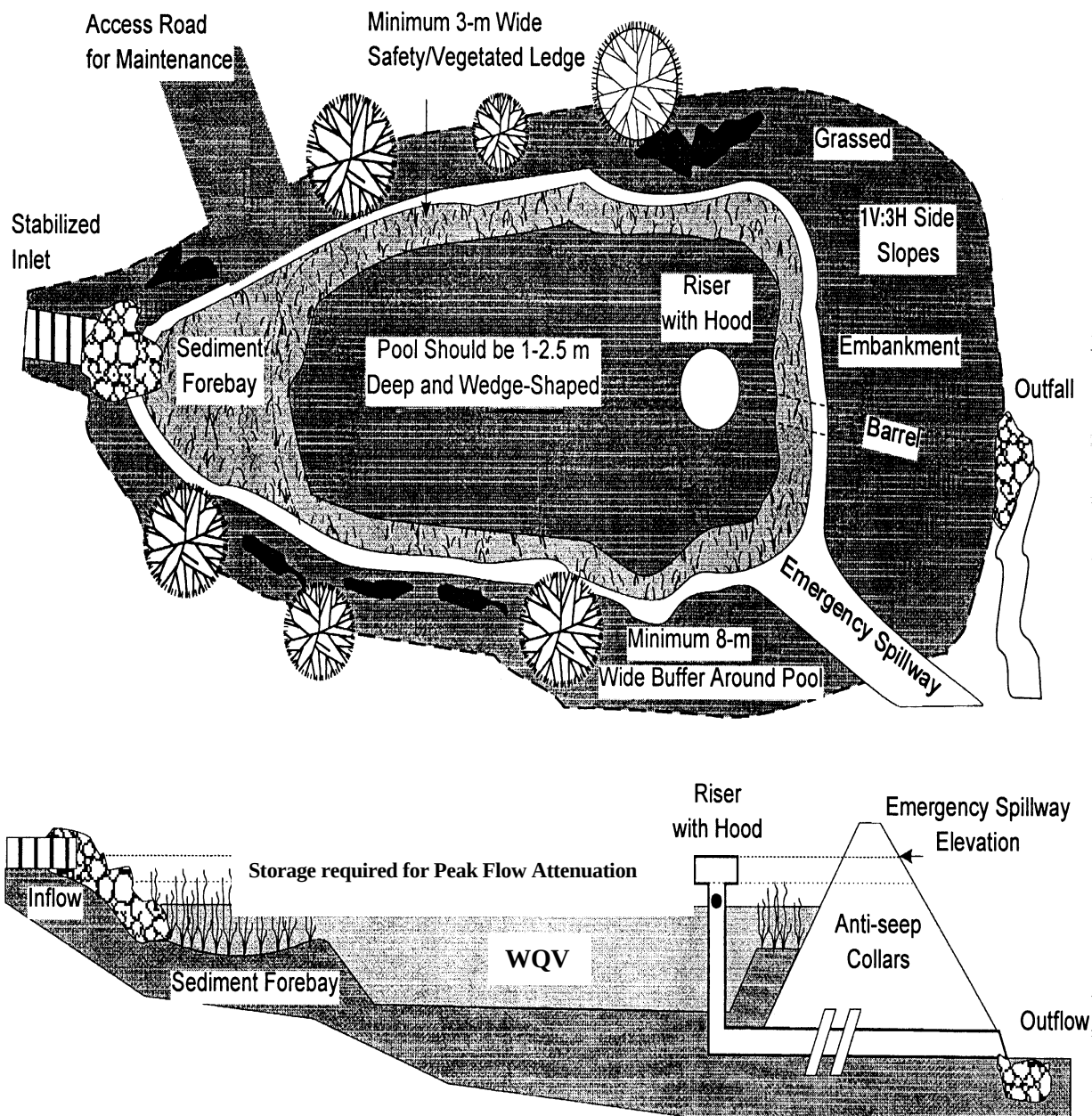


Figure 10-29 Wet Basin

10.13 Infiltration Controls

10.13.1 Introduction

Infiltration controls are best management practices (BMPs) where the primary discharge of stormwater is to the ground water table. These include infiltration trenches and basins. In some cases, the stormwater is intercepted after it has infiltrated a few meters by an under drain and is discharged to a storm sewer or surface water. One of the primary concerns with the use of infiltration BMPs is the risk of groundwater contamination especially in an Aquifer Protection Area. This is why there should be at least 1.5 m (5 ft) between the bottom of the facility to the seasonable high water table and 1.5 m (5 ft) to the underlying bedrock. Another factor is the residence time in the facility. Sources recommend that the first flush stormwater be infiltrated within 24 to 72 h. The infiltration rate is directly related to the soil type and disposition. Soil investigation should be performed at all facility locations prior to construction. Table 10-7 provides some considerations in evaluating an infiltration control.

Table 10-7 Summary Of Considerations For An Infiltration Facility

Quality	Infiltrate WQV within 24 to 72 h; minimum residence time of 24 h.
Quantity	Control 2- and 10-year peak flows (could lead to a large expensive facility; could be used with detention pond to control quantity)
Shape	Dependent on site constraints
Maintenance	Inspect once a year; preferably during wet weather; mow area as required; remove sediment every 5-10 years
Other considerations	Filter strip to remove sediments (2-5% slope with minimum 6 m (20 ft) length); infiltration rate (minimum 25 mm/h (1 in/h); depth to groundwater and bedrock (1.5 m (5 ft)); effects of facility on quality of groundwater
Pollutant removal	Moderate to high

Additional Guidance The Connecticut Stormwater Quality Manual outlines considerations and design criteria for infiltration facilities as related to stormwater quality control. The design of all such facilities shall be consistent with the Connecticut Stormwater Quality Manual and ConnDOT Drainage Manual. The more stringent criteria of the two manuals shall apply.

10.13.2 Site Selection

Refer to the Connecticut Stormwater Quality Manual for siting considerations. Additionally, the FHWA-TS-80-218 "Underground Disposal of Stormwater Runoff" describes a procedure that rates different sites by using several parameters (reader is referred to this study for details of the procedure). Upon completion of this procedure, several sites can be compared to determine which

is the best for the infiltration BMP or if infiltration is even feasible. Other selection considerations, include size of drainage area, proximity to foundations **(the facility should be no closer than 3 m (10 ft) down-gradient and 30 m (100 ft) up-gradient from a foundation and septic leaching fields)**.

Infiltration rate The infiltration rate is a very important parameter when an infiltration facility is designed. It is rarely used to determine the outflow from the facility for quantity and quality control. Infiltration rates of greater than 25 mm/h (1 in/h) are preferred for infiltration facilities. After a suitable site for the facility has been found, several soil tests must be made before the facility is designed.

First, borings should be taken at the site to determine the soil types, depth to bedrock and groundwater and infiltration rates. Infiltration rates should be determined through an appropriate field permeability test that is representative of vertical water infiltration through the soil, excluding lateral flows. A double-ring infiltrometer test is recommended (ASTM, 1994).

Observation well An observation well should be included in an infiltration facility with a covered bottom (i.e., trenches) to allow an inspector to determine how well the facility is operating (e.g., whether the stormwater is infiltrating as designed or whether maintenance is required). A schematic of a typical observation well in an infiltration trench is shown in Figure 10-30. It may also be necessary to install wells in infiltration basins to determine if they are working properly, but this can be determined visually because the stormwater is stored on the surface whereas the storage in trenches is hidden from view.

10.13.3 Infiltration Trench

An infiltration trench is a facility where a trench is excavated and then filled with a porous medium. Stormwater is stored in the voids of the fill material until it can be infiltrated. In a variation of this design, the stormwater is collected by an under drain pipe after the stormwater has been detained and filtered by the trench. Infiltration trenches can be used with a minimum offset of 3.0m (10 ft.) from the edge of pavement, or other facilities (e.g. along the centerline of a 6.09m (20 ft.) wide roadside swale).

Quality The bottom of the facility must be 1.5 m (5 ft) above the bedrock and the seasonably high groundwater table, and the bottom of the stone reservoir must be below the frost line. The WQV must be infiltrated within 48 h. The primary removal mechanisms in trenches are sedimentation and filtration, along with some biological uptake. Filtering is achieved in the top layers of the facility as stormwater enters. In the stone reservoir, the main removal mechanisms are sedimentation and adsorption.

As the stormwater leaves, it is filtered again by the underlying soil, where more pollutants will be removed. Unfortunately, all infiltration facilities are vulnerable to clogging, thereby reducing their effectiveness. Therefore, a vegetated buffer strip filtering the runoff is recommended as part of an infiltration facility. The strip would decrease the amount of suspended solids in the stormwater and thus increase the useful life of the infiltration facility. The filter strip should be at least 6 m (20 ft) wide. It should also be sloped from 2% to 5% to prevent water from ponding and to ensure a slow velocity.

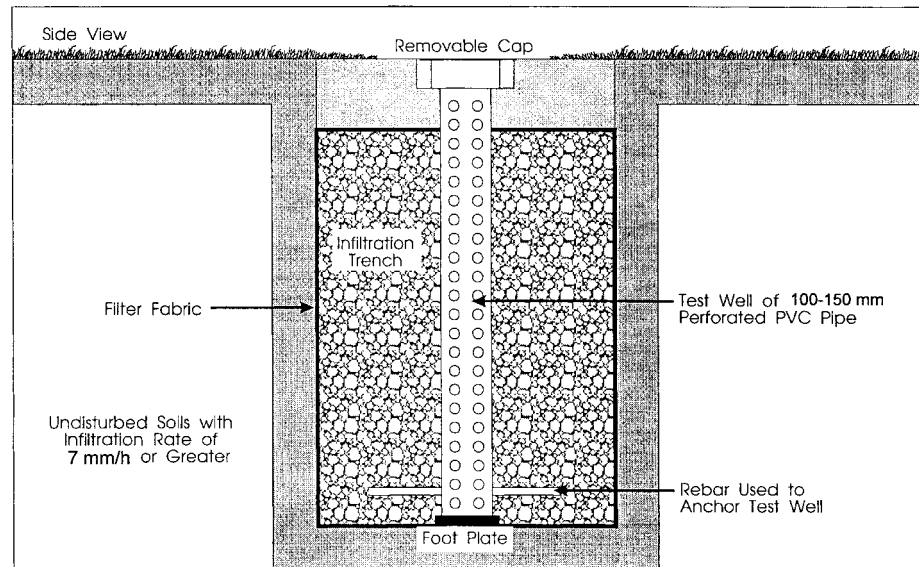


Figure 10-30 Infiltration Trench With Observation Well (after Schueler, 1987)

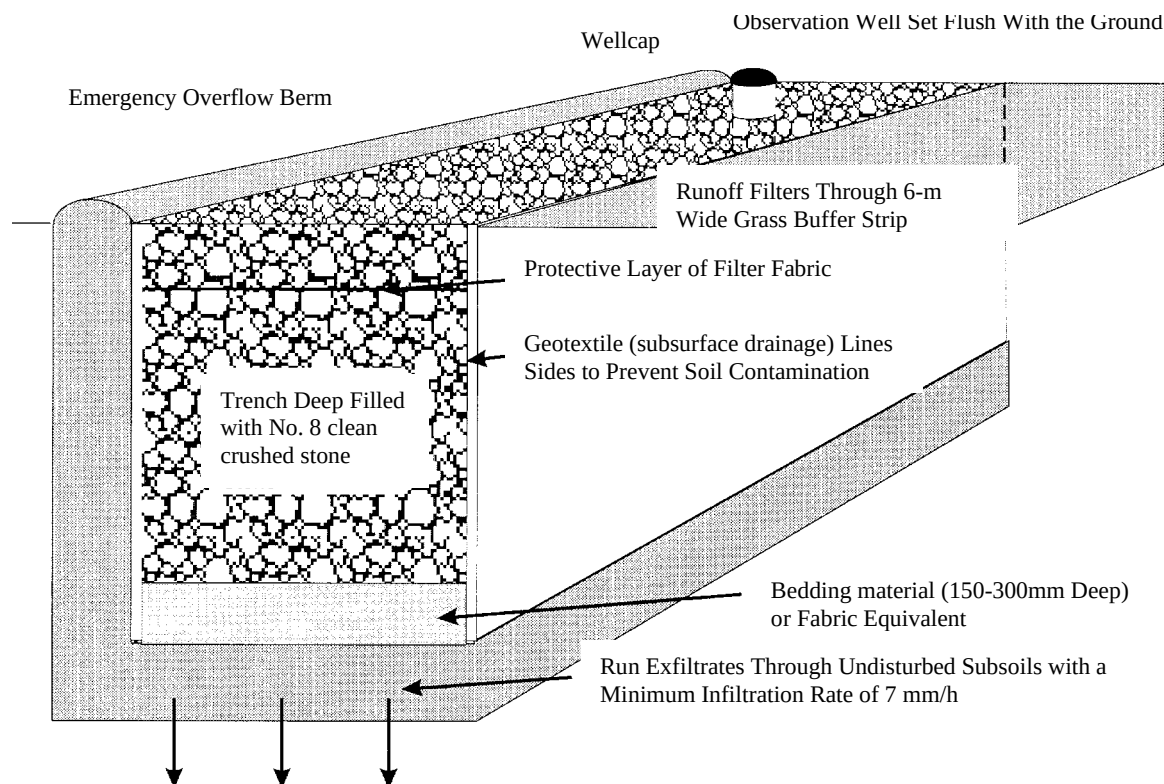


Figure 10-31 Infiltration Trench (after Schueler, 1987)

Quantity Because of the large size of the trench that would be required to control a 10-year storm, it is suggested that trenches not be used for large drainage areas or areas where the increase in peak flow, and, therefore, the amount of storage required, is very large.

A trial and error process of routing the design storms through the facility can be used to determine the amount of storage required for quantity control. Although the methods used to determine the amount of storage required for the ponds are not derived for infiltration BMPs, they can still be used to obtain an initial estimate of the required storage.

After a storage volume has been determined, the dimensions of the facility can be estimated. The depth should be designed such that the bottom is 1.5 m (5 ft) above the bedrock and high water table and below the frost line. The top of the storage area must be at or below the top of subgrade for all trenches in the vicinity of a roadway. The surface area can be manipulated to suit the site conditions so long as it yields the required storage volume. The amount of surface area required is:

$$S_a = \frac{Vol_s}{V_r d} \quad (10.33)$$

Where: S_a = surface area, m^2 (ft^2)
 Vol_s = storage volume, m^3 (ft^3)
 V_r = void ratio (= 0.4 for 37.5 to 75 mm (1.5 to 3 in.) aggregate)
 d = depth, m (ft)

The inflow hydrograph can be calculated by a number of methods, and the outflow versus storage curve can be found from a falling head test. Thus, the storms can be routed through the facility and the size of the trench can be changed to reduce the peak outflows to the pre-construction levels of the design storm. When routing the storm through the trench, one can determine whether and how much flow will bypass the trench when it is filled with stormwater. The overflow from the trench must be contained in an adequate channel.

Other considerations Detention time is an important factor in determining the effectiveness of a trench facility. A facility which drains quickly is capable of treating more stormwater volume. A maximum detention time of 72 h is recommended. The actual detention time can be estimated by:

$$T_s = \frac{d V_r}{f} \quad (10.34)$$

Where: T_s = storage time (h) or detention time
d = depth of storage in the trench, mm (in)
 V_r = void ratio of stone reservoir
f = steady infiltration rate, mm/h (in/hr)

From this equation, it can be seen that detention time is directly related to trench depth. Since the WQV will most likely be much smaller than the storage required for a 10-year storm, the depth of the WQV will be very small in the trench. Therefore, infiltration trenches are much better suited for small drainage areas where the change in peak flow between pre and post construction is small. Modifications can be made to the trench design to increase the depth of the WQV storage, but these will increase the cost and could make this BMP option infeasible.

10.13.4 Infiltration Basin

An infiltration basin looks very similar to a dry pond, see Figure 10-32. Stormwater from smaller, more frequent storms is infiltrated through the bottom of the basin. Larger storms can be controlled through infiltration and/or by a "peak shaving" outlet. The most important consideration for an infiltration basin is keeping the bottom from clogging with sediment. The clogging of basins, along with the overestimating of their infiltration rates, has led to the failure of many infiltration basins.

Quality To protect groundwater quality, the bottom of the infiltration basin must be 1.5 m (5 ft) or more above the bedrock and the seasonably high groundwater table. The WQV should be infiltrated within 48 h. The primary removal mechanisms in infiltration basins are sedimentation, filtration and biological uptake. Filtering is provided by the vegetation at the bottom of the pond and, preferably, also by a buffer strip before the stormwater runoff enters the facility. The filter strip should be at least 6 m (20 ft) wide and should also be sloped from 2% to 5% to prevent water from ponding and to ensure a slow velocity. Vegetation can also contribute to the removal of pollutants through biological uptake. As the stormwater leaves, it is filtered again by the underlying soil.

An estimation of the maximum ponding depth for a desired drain time can be found with the equation:

$$d = f T_s \quad (10.35)$$

Where: d = depth, mm (in)
 f = steady infiltration rate, mm/h (in/hr)
 T_s = time of storage, h

The recommended maximum allowable storage time is 48 h. Considering the fact that basins may fail because of clogging and an infiltration rate that is lower than expected, a shorter time of storage, say 40 h, might be used to compensate for inaccuracies in estimating infiltration rates.

Several other considerations can help enhance the pollutant removal of these facilities. First, vegetation should be established on the basin floor. A dense stand of water-tolerant grass with a deeply penetrating root system would help stabilize the bottom of the basin and help keep the soil open. Vegetation would also provide biological uptake of nutrients.

Second, the pond bottom should be sloped as close to zero as possible in order to obtain a uniform depth of stormwater over the basin. The side slopes should be sloped at 1V:4H, or flatter, to allow for easy maintenance access and prevent erosion.

A third consideration has to do with the incoming stormwater. A combination of a level spreader/sediment forebay can be constructed to spread the stormwater evenly, thereby reducing erosion, and trap sediments before they clog the basin. Riprap should also be placed at the inlet to help reduce erosion.

Quantity For an infiltration basin, quantity can be controlled very similarly to a detention basin. The basin should be designed to reduce the peak flow from a 2, 10, and a 25-year storm, considered individually and be able to pass a 100-year storm safely.

After a required storage volume is estimated, the design storms should be routed through the basin to determine if the estimated value is correct.

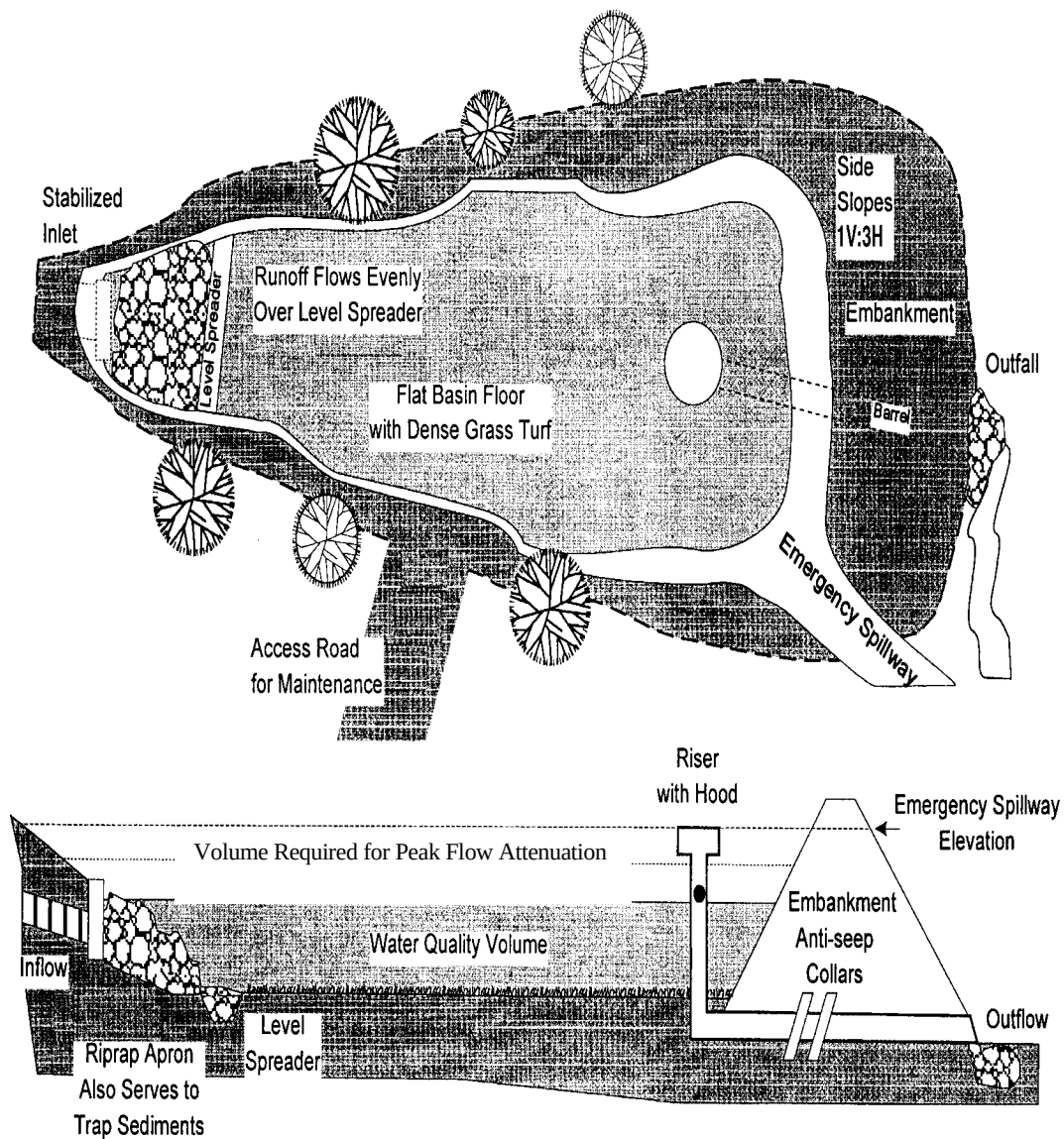


Figure 10-32 Infiltration Basin

10.14 Land-Locked Retention

10.14.1 Introduction

Watershed areas that drain to a central depression with no positive outlet can be evaluated using a mass flow routing procedure to estimate flood elevations. Although this procedure is fairly straightforward, the evaluation of basin outflow is a complex hydrogeologic phenomenon that requires good field measurements and a thorough understanding of local conditions. Since outflow rates for flooded conditions are difficult to calculate, field measurements are desirable.

10.14.2 Mass Routing

The steps presented below for the mass routing procedure are illustrated by the example given in Figure 10-33.

- Step 1 Obtain cumulative rainfall data for the 100-year frequency, 10-day duration design event from Figure 10-34.
- Step 2 Calculate the cumulative inflow to the land-locked retention basin using the rainfall data from Step 1 and runoff procedure from the Hydrology Chapter. Plot the mass inflow to the retention basin.
- Step 3 Develop the basin outflow from field measurements of hydraulic conductivity, taking into consideration worst-case water table conditions. Hydraulic conductivity should be established using insitu test methods, then results compared to observed performance characteristics of the site. Plot the mass outflow as a straight line with a slope corresponding to worst-case outflow in centimeters (inches) per hour.
- Step 4 Draw a line tangent to the mass inflow curve from Step 2, which has a slope parallel to the mass outflow line from Step 3.
- Step 5 Locate the point of tangency between the mass inflow curve of step 2 and the tangent line drawn for Step 4. The distance from this point of tangency and the mass outflow line represents the maximum storage required for the design runoff.
- Step 6 Determine the flood elevation associated with the maximum storage volume determined in Step 5. Use this flood elevation to evaluate flood protection requirements of the project. The zero volume elevation shall be established as the normal wet season water surface or water table elevation or the pit bottom, whichever is highest.
- Step 7 If runoff from the project area discharges into a drainage system tributary to the land-locked depression, detention storage facilities are required to comply with the pre-development discharge requirements for the project.

Unless the storage facility is designed as a retention facility, including water budget calculations, environmental needs and provisions for preventing anaerobic conditions, relief structures shall be provided to prevent standing water conditions.

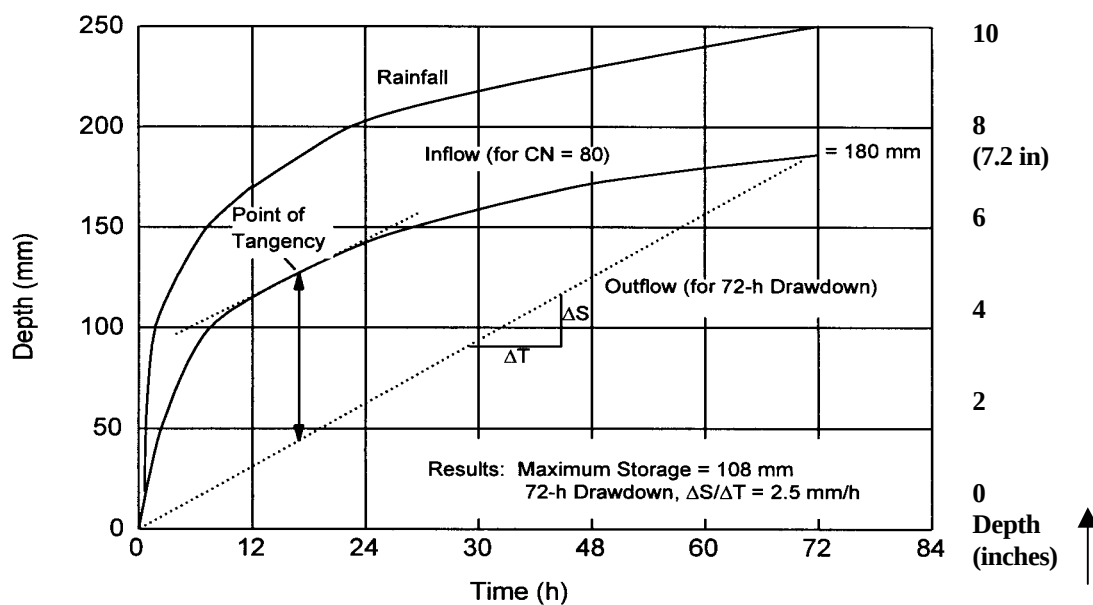


Figure 10-33 Mass Routing Curve

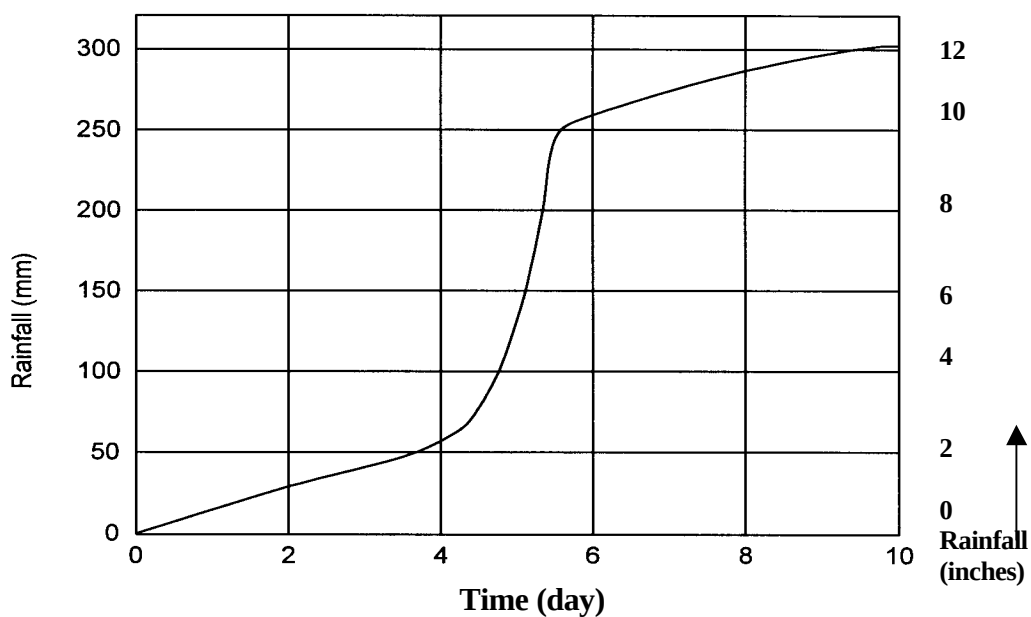


Figure 10-34 Cumulative Rainfall Data For 100-Year 10-day Design Storm

10.15 Construction And Maintenance Considerations

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 extensive maintenance are discouraged. The following maintenance problems are typical of urban detention facilities and facilities shall be designed to minimize problems:

- weed growth
- grass and vegetation maintenance
- sedimentation control
- bank deterioration
- standing water or soggy surfaces
- mosquito control
- blockage of outlet structures
- litter accumulation
- maintenance of fences and perimeter plantings

Proper design should focus on the elimination or reduction of maintenance requirements by addressing the potential for problems to develop.

- Both weed growth and grass maintenance may be addressed by constructing side slopes that can be maintained using available power-driven equipment, such as tractor mowers.
- Sedimentation may be controlled by constructing traps to contain sediment for easy removal or low-flow channels to reduce erosion and sediment transport.
- Bank deterioration can be controlled with protective lining or by limiting bank slopes.
- Standing water or soggy surfaces may be eliminated by sloping basin bottoms toward the outlet, constructing low-flow pilot channels across basin bottoms from the inlet to the outlet, or by constructing underdrain facilities to lower water tables.
- In general, when the above problems are addressed, mosquito control will not be a major problem.
- Outlet structures should be selected to minimize the possibility of blockage (i.e., very small pipes tend to block quite easily and should be avoided).
- Finally, one way to deal with the maintenance associated with litter and damage to fences and perimeter plantings is to locate the facility for easy access where this maintenance can be conducted on a regular basis.

10.16 Protective Treatment - Fencing

Protective treatment may be required to prevent entry to facilities that present a hazard to children and, to a lesser extent, all persons. Fences may be required for detention areas where one or more of the following conditions exist:

- Rapid stage increases would make escape practically impossible where small children frequent the area.
- Water depths either exceed 1 m (3 ft) for more than 24 h or are permanently wet and have side slopes steeper than 1V:4H.
- A low-flow watercourse or ditch passing through the detention area has a depth greater than 1.5 m (5 ft) or a flow velocity greater than 1.5 m/s (5 ft/s).
- Side slopes equal or exceed 1V: 2H.

Guards or grates may be appropriate for other conditions, but in all circumstances heavy debris must be transported through the detention area. In some cases, it may be advisable to fence the watercourse or ditch rather than the detention area.

Fencing should be considered for dry retention areas with design depths in excess of 1 m (3 ft) for 24 h, unless the area is within a fenced, limited access facility.

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11.1 Overview

11.1.1 Introduction

This chapter provides guidance for the design of a storm sewer system. 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, both in longitudinal and transverse directions, shallow watercourses, absence of side channels
- the more costly property damages 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 will interfere with or possibly halt the passage of highway traffic

11.1.2 Inadequate Drainage

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

- damage to surrounding or adjacent property, resulting from water overflowing the roadway section and entering such property
- risk and delay to traffic caused by excessive ponding in sags or excessive spread along the roadway
- weakening of base and subgrade due to saturation from frequent ponding of long duration and erosion of pavement edges

11.1.3 System Planning

System planning prior to commencing the design of a storm drain system is essential. The basics required are discussed in Section 11.4 and include the general design approach, type of data required, information on initiating a cooperative agreement with a municipality, the importance of a preliminary sketch and some special considerations.

11.1.4 Costs

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

11.2 Symbols and Definitions

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in storm drainage publications.

Table 11-1 Symbols And Definitions

<u>Symbol</u>	<u>Definition</u>	<u>Units</u>
A	Area of cross section	m ² (ft ²)
A	Watershed area	ha (acres)
C	Runoff coefficient, weir or orifice coefficient or correction factor	-
d	Depth of gutter flow at the curb line	m (ft)
D	Diameter of pipe	m (ft)
H	Head loss	m (ft)
I	Rainfall intensity	mm/h (in/h)
K	Coefficient	-
L	Length of curb opening inlet or slotted drain	m (ft)
L	Pipe length	m (ft)
L	Pavement width	m (ft)
L	Length of runoff travel	m (ft)
n	Roughness coefficient in Manning formula	-
P	Perimeter of grate opening, neglecting bars and side against curb	m (ft)
Q	Rate of discharge in gutter	m ³ /s (cfs)
Q _i	Intercepted flow	m ³ /s (cfs)
Q _s	Gutter capacity above the depressed section	m ³ /s (cfs)
Q _T	Total flow	m ³ /s (cfs)
R _h	Hydraulic radius	m (ft)
S or S _x	Pavement cross slope	m/m (in/in)
S or S _L	Longitudinal slope of pavement	m/m (in/in)
T	Top width of water surface (spread on pavement)	m (ft)
t _c	Time of concentration	min
V	Velocity of flow	m/s (fps)
y	Depth of flow in approach gutter	m (ft)
Z	T/d, reciprocal of the cross slope	-

11.3 Concept Definitions

Following are discussions of concepts which will be important in the conduct of a storm drainage analysis and design. These concepts will be used throughout the remainder of this chapter in dealing with different aspects related to storm drainage analysis.

Bypass	Flow which bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade. Inlets can be designed to allow a certain amount of bypass. Also, inlets may be designed to allow a certain amount of bypass for one design storm and larger or smaller amounts for other design storms.
Combination Inlet	A drainage inlet composed of a grate inlet with a curb opening, a Type "C" catch basin.
Equivalent Cross Slope	An imaginary straight cross slope having conveyance capacity equal to that of the given compound cross slope.
Flanking Inlets	Inlets placed upgradient and on either side of the low point inlet in a sag vertical curve. The purpose of these inlets is to intercept debris as the slope decreases and to act in relief of the inlet at the low point.
Frontal Flow	The portion of the flow which passes over the upgradient side of a grate.
Grate Inlet	A Type C-L catch basin composed of a grate at the roadside in a low point, swale or channel.
Grate Perimeter	The sum of the lengths of all sides of a grate, except that any side adjacent to a curb is not considered a part of the perimeter in weir flow computations.
Gutter	That portion of the roadway section adjacent to the curb which is utilized to convey stormwater runoff. It may include a portion, or all, of a traveled lane, shoulder or parking lane, and a limited width, adjacent to the curb, may be of different materials and have a different cross slope.
Hydraulic Grade Line	The hydraulic grade line is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run (pressure head plus elevation head).
Pressure Head	Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.
Scupper	A grated inlet or vertical hole through a bridge deck for the purpose of deck drainage. Sometimes, a horizontal opening in the curb or barrier is called a scupper.
Side-Flow Interception	Flow which is intercepted along the side of a grate inlet, as opposed to frontal interception. The Department disregards this minimal amount of flow entering the inlet.

Slotted Drain Inlet	A drainage inlet composed of a continuous slot built into the top of a pipe which serves to intercept, collect and transport the flow.
Storm Drain	A storm drain is that portion of the storm drainage facilities that receives runoff from the inlets and conveys the runoff to an adequate outfall. Culverts discharging to the storm drainage system are considered part of the system.
Splash-Over	Portion of frontal flow at a grate which skips or splashes over the grate and is not intercepted.
Spread	The width of flow in the gutter measured laterally from the roadway curb.
Velocity Head	Velocity head is a quantity proportional to the kinetic energy of flowing water expressed as a height or head of water.

11.4 System Planning

11.4.1 Introduction

The design of any storm drainage system involves the accumulation of basic data, familiarity with the project site, and a basic understanding of the hydrologic and hydraulic principles and drainage policy associated with that design.

11.4.2 General Design Approach

The design of a storm drain system is generally a process which evolves as a project develops. The primary ingredients to this process are listed below in a general sequence by which they may be carried out.

- data collection
- preliminary sketch
- inlet location and spacing
- plan layout of storm drain system
- locate main outfall
- determine direction of flow
- locate existing utilities
- locate connecting mains
- locate manholes
- size the pipes
- review hydraulic grade line
- prepare the plan
- provide documentation

11.4.3 Required Data

The designer should be familiar with land use patterns, 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 overland and by storm drains) to some existing outfall location. Furthermore, there should be an understanding of the nature of the outfall since it usually has a significant influence on the storm drainage system. In environmentally sensitive areas, there may be water quality requirements to consider as well.

Actual surveys of these and other features 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, particularly for busy urban roadways with all the attendant urban development. Existing topographic maps, available from the U. S. Geological Survey, the Natural Resource Conservation Service, many municipalities, and private developers are also valuable sources of the kind of data needed for a proper storm drainage design. Developers and governmental planning agencies should be consulted regarding 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. Comprehensive Stormwater Management Plans and Floodplain Ordinances should be reviewed when they are available.

11.4.4 Preliminary Sketch

Preliminary sketches or schematics, featuring the basic components of the intended design, are useful to the designer. Such sketches should indicate watershed areas and land use, existing drainage patterns, plan and profile of the roadway, street and driveway layout with respect to the project roadway, underground utility locations and elevations, locations of proposed retaining walls, bridge abutments and piers, logical inlet and manhole locations, preliminary lateral and trunk line layouts and a clear definition of the outfall location and characteristics. This sketch should be reviewed with the traffic staging plans and soils recommendations for areas which are incompatible with required construction staging. With this sketch or schematic, the designer is able to proceed with the detailed process of storm drainage design calculations, adjustments and refinements.

Unless the proposed system is very simple and small, the designer should not ignore a preliminary plan as described above. Upon completion of the design, documentation of the overall plan is facilitated by the preliminary schematic.

11.4.5 Special Considerations

Consideration and planning should be directed toward avoidance of utilities and deep cuts. In some cases, traffic must be maintained or temporary bypasses constructed and temporary drainage provided for during the construction phase (See Chapter 6, Hydrology, for Temporary Facilities). Further consideration should be given to the actual trunk line layout and its constructibility. For example, will the proposed location of the storm drain interfere with in-place utilities or disrupt traffic? Some instances may dictate a trunk line on both sides of the roadway with very few laterals while other instances may call for a single trunk line. Such features are usually a function of economy but may be controlled by other physical features.

Except in unusual circumstances, storm drains should discharge to a single outfall. A storm drain which branches, thereby distributing the discharge, should be avoided.

Storm drain pipes should not decrease in size in a downstream direction regardless of the available pipe gradient.

11.4.6 ConnDEP Requirements

The following requirements for storm drainage systems taken from Section 25-68h-3c of the Flood Management Statutes and Administrative Regulations shall be incorporated into all ConnDOT projects.

All subsurface storm drainage systems shall be designed in accordance with the methods and procedures defined in this manual as may be amended and shall meet the following requirements:

- 1) Storm drainage systems for parking lots, driveways, and roads shall be designed for a ten year frequency storm without closing the facility.
- 2) The design of storm drainage systems for depressed roads and driveways shall comply with this manual.
- 3) Use of curbing shall be minimized in order to encourage overland disbursed flow through stable vegetated areas.
- 4) The hydrology and hydraulic design of catch basins, gutters, and storm drain pipes shall comply with this manual.
- 5) Design computations shall be prepared on the appropriate forms contained in this manual.

- 6) The foundation drains and floor drains of buildings connected into storm drainage systems shall be designed to prevent backflow for the 100 year frequency flood into the building.
- 7) Surface runoff shall be directed through vegetated filter strips or grass swales wherever possible prior to storm drain inlets.
- 8) The design of the storm drainage system should be coordinated with the soil erosion and sediment control plan.
- 9) Storm drainage discharges shall be coordinated with the National Pollution Discharge Elimination System permit program administered by the Water Compliance Unit of DEP.
Note: To this end, guidance has been provided within this manual relative to the design of water quality basins and hydrodynamic separators. See Chapter 10 (Storage Facilities) and Appendix C of this chapter.
- 10) Storm drainage systems discharging into watercourses tributary to public water supply reservoirs shall be in compliance with the Public Health Code.
- 11) Storm drains shall be extended to a suitable discharge point into a watercourse or public drainage system, or to where drainage rights have been secured

11.4.7 Documentation

The following items shall be included in the documentation file (see Chapter 1, Section 1.6). The intent is not to limit data to only those items listed, but rather establish a minimum requirement consistent with the storm drainage system design procedures as outlined in this chapter. If circumstances are such that the system is prepared by other than the normal procedures or is governed by factors other than the design criteria, a narrative summary detailing the design basis shall appear with the other data.

- computations for inlets and pipes, including hydraulic grade lines
- copies of the standard computation sheets given in this chapter
- complete drainage area map
- design frequency
- information concerning outfalls, existing storm drains and other design considerations
- computations for temporary facilities where applicable (Chapter 6, Appendix F)
- a properly labeled schematic indicating storm drain system layout

11.5 Hydrology

11.5.1 Introduction

The rational method is the most common method in use for the design of storm drains when the momentary peak flow rate is desired. Its use should be limited to systems with drainage areas of 81 ha (200 acres) or less. Drainage systems involving detention storage and pumping stations require the development of a runoff hydrograph. (See Chapters 6, 10 and 12 – Hydrology, Storage Facilities, and Pump Stations).

11.5.2 Rational Method

The Rational Equation is written as follows:

$$Q = 0.00278CIA = 0.00278 (\sum CA) I \quad (Q = CIA) \quad (11.1)$$

Where: Q = discharge, m³/s (ft³/s)

C = runoff coefficient

I = rainfall intensity, mm/h (in/h)

A = drainage area, ha (ac)

11.5.3 Runoff Coefficient

The runoff coefficients for various types of surfaces are discussed in Chapter 6, with tables of appropriate values. The weighted C value is to be based on a ratio of the drainage areas associated with each C value as follows:

$$\text{weighted C} = [A_1C_1 + A_2C_2 + A_3C_3] / [A_1 + A_2 + A_3] \quad (11.2)$$

11.5.4 Rainfall Intensity

Rainfall intensity (I): Rainfall intensity is the intensity of rainfall in millimeters (inches) per hour for a duration equal to the time of concentration. Intensity is a rate of rainfall over an interval of time such that intensity multiplied by duration equals amount of rain, i.e., an intensity of 130 mm/h for a duration of 5 min indicates a total rainfall amount of $130 \times 5/60 = 10.8$ mm. See Chapter 6 Hydrology for a more complete discussion and data to be used for determining the intensity of rainfall.

11.5.5 Time of Concentration

The time of concentration is defined as the period required for water to travel from the most hydraulically distant point of the watershed to the point of the storm drain system under consideration. The designer is usually concerned about two different times of concentration: one for inlet spacing and the other for pipe sizing. There is a major difference between the two times.

- **Inlet Spacing**

The time of concentration (t_c) for inlet spacing is the time for water to flow from the hydraulically most distant point of the drainage area to the inlet, which is known as the inlet time. Usually this is the sum of the time required for water to move across the pavement or overland back of the curb to the gutter, plus the time required for flow to move through the length of gutter to the inlet. **For pavement drainage, when the total time of concentration to the upstream inlet is less than 5 min, a minimum t_c of 5 min should be used to estimate the intensity of rainfall.** The time of concentration for the second downstream inlet and each succeeding inlet should be determined independently, the same as the first inlet. In the case of a constant roadway grade and relatively uniform contributing drainage area, the time of concentration for each succeeding inlet could also be constant.

- **Pipe Sizing**

The time of concentration for pipe sizing is defined as the time required for water to travel from the most hydraulically distant point of the watershed to the point of the storm drain system under consideration. It generally consists of two components: (1) the time to flow to the inlet which can consist of overland and channel or gutter flow and (2) the time to flow through the storm drain to the point under consideration.

Travel time within the storm drain pipes can be estimated by the relation:

$$t_t = L / 60V \quad (11.3)$$

Where: t_t = travel time, min

L = length of pipe in which runoff must travel, m (ft)

V = estimated or calculated normal velocity, m/s (ft/s)

Methods for determining time of concentration are further described in Chapter 6 Hydrology.

To summarize, the time of concentration for any point on a storm drain is the inlet time for the inlet at the upper end of the line plus the time of flow through the storm drain from the upper end of the storm drain to the point in question. In general, where there is more than one source of runoff to a given point in the storm drainage system, the longest t_c is used to estimate the intensity (I). There could be exceptions to this generality, for example where there is a large inflow area at some point along the system, the t_c for that area may produce a larger discharge than the t_c for the summed area with the longer t_c . The designer should be cognizant of this possibility when joining drainage areas and determine which drainage area governs. To determine which drainage area controls, compute the peak discharge for each t_c . Note that when computing the peak discharge with the shorter t_c , not all the area from the basin with the longest t_c will contribute runoff. One way to compute the contributing area, A_c , is as follows:

$$A_c = A [t_{c1} / t_{c2}] \quad (11.4)$$

Where: $t_{c1} < t_{c2}$ and A is the area of the basin with the longest t_c .

In municipal areas, a minimum time of concentration of 5 min is recommended for calculation of runoff from paved areas and 10 min. for areas mostly grass. All other areas should be calculated on a case by case basis.

11.5.6 Detention Storage

Reduction of peak flows can be achieved by the storage of runoff in detention basins, storm drains, swales and channels, and other detention storage facilities. Stormwater is then released to the downstream conveyance facility at a reduced flow rate. The concept should be considered for use in highway drainage design where existing downstream conveyance facilities are inadequate to handle peak flow rates from highway storm drainage facilities, where the highway would contribute to increased peak flow rates and aggravate downstream flooding problems, and as a technique to reduce the right-of-way, construction, and operation costs of outfalls from highway storm drainage facilities. See Chapter 10, Storage Facilities.

11.6 Pavement Drainage

11.6.1 Introduction

Highway storm drainage facilities collect stormwater runoff and convey it through the roadway right-of-way in a manner which adequately drains the roadway and minimizes the potential for flooding and erosion to properties adjacent to the right-of-way. Storm drainage facilities consist of curbs, gutters, storm drains, channels and culverts. The placement and hydraulic capacities of storm drainage facilities should be designed to take into consideration damage to adjacent property and to secure as low a degree of risk of traffic interruption by flooding as is consistent with the importance of the road, the design traffic service requirements and available funds.

Following are the procedures which should be followed for storm drain design and analysis. For a general discussion of policies and guidelines for storm drainage, the designer is referred to the publication, "A Policy on Geometric Design of Highways and Streets," published by the American Association of State Highway and Transportation Officials. For more design and engineering guidance, refer to AASHTO, Highway Drainage Guidelines, Volume 9, or the FHWA publications HEC 12 and HEC 21.

Roadway features considered during gutter, inlet and pavement drainage calculations include:

- longitudinal and cross slopes
- curb and gutter sections
- roadside and median ditches
- bridge decks

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

11.6.2 Longitudinal Slope

A minimum longitudinal gradient is more important for a curbed pavement, since it is susceptible to stormwater spread. Flat gradients on uncurbed pavements can also lead to a spread problem if vegetation is allowed to build up along the pavement edge.

Gutter grades should not be less than 0.5 percent. Minimum grades can be maintained in very flat terrain by use of a sawtooth profile.

11.6.3 Cross Slope

The ConnDOT "Highway Design Manual" establishes the cross slopes for highways. Median areas should not be drained across traveled lanes. A careful check should be made of designs to minimize the number and length of flat pavement sections in cross slope transition areas.

Shoulders should generally be sloped to drain away from the pavement.

11.6.4 Roadside And Median Channels

Roadside channels are commonly used with uncurbed roadway sections to convey runoff from the highway pavement and from areas which drain toward the highway. Due to right-of-way limitations, roadside channels cannot be used on most urban arterials. They can be used in cut sections, depressed sections, and other locations where sufficient right-of-way is available and driveways or intersections are infrequent.

Curbed highway sections are relatively inefficient at conveying water, and the area tributary to the gutter section should be kept to a minimum to reduce the hazard from water on the pavement. Where practicable, the flow from major areas draining toward curbed highway pavements should be intercepted by channels as appropriate.

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

11.6.5 Bridge Decks

Drainage of bridge decks is similar to other curbed roadway sections. It is often less efficient, since parapets collect large amounts of debris, and drainage inlets have a higher potential for clogging by debris, therefore the designer should consider the type of inlet and piping system being proposed. The frequent gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal.

Zero gradients and sag vertical curves should be avoided on bridges. The minimum desirable longitudinal slope for bridge deck drainage should be 0.5 percent. When bridges are placed at a vertical curve and the longitudinal slope is less than 0.5 percent, the gutter spread should be checked to ensure a safe, reasonable design. Inlets are the recommended method of deck drainage. Inlets should not be outletted over embankments, slope pavement, slope protection, navigation channels, driving lanes, or railroad tracks. Runoff collected and transported to the end of the bridge should generally be collected by inlets and down drains.

Grates for bridge inlets should be the same as for catch basins. If they are different then a separate analysis may be required using HEC 12 or HEC 21 to determine their interception characteristics including splashover.

Short, continuous span bridges, particularly over-passes, may be built without inlets. The water from the bridge roadway should be intercepted by an inlet beyond the end of the bridge structure. Some type of bridge end drainage must be provided at all bridges.

11.6.6 Hydroplaning

NCHRP research project 1-29, "Improved Surface Drainage of Pavements," suggests that hydroplaning conditions can develop for relatively low vehicle speeds and at low rainfall intensities for storms that frequently occur each year. Analysis methods developed through this research effort provide guidance in identifying potential hydroplaning conditions. 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 wide pavement during high intensity rainfall and under some relationship of the primary controlling factors of:

- vehicle speed
- tire conditions (pressure and tire tread)
- pavement type
- 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
- maximize pavement roughness

Gutter Flow

- limit street spread (adequate inlet spacing)
- maximize interception of gutter flow on the high side of superelevation transitions

Sag Areas

- limit pond duration and depth

Overtopping

- limit depth and duration of overtopping flow

In the event that suitable measures cannot be implemented to address an area of high potential for hydroplaning, or an identified existing problem area, consideration should be given to installing advance warning signs.

11.6.7 Underdrains

In certain areas ground water can be a significant problem as it attacks foundations, substructures, subgrades and other aspects of highway components. In most soils where ground water is a problem, a system of underdrains, installed for the removal of excess moisture, can be a very useful feature in the overall roadway design. Underdrains may take the form of networks of perforated (or otherwise permeable) pipe, French drains, or collector fields. Where such appurtenances are needed, the additional expense in their installation is usually fully justified in terms of future savings in roadway and structure maintenance costs.

Percolation rates for ground water may be obtained from Natural Resource Conservation Service offices, measured, or simply estimated. Collector pipe sizes and networks may then be established

for the removal of that water. French drains can be very useful where the unwanted ground water percolation rates are relatively high. Collector fields may be useful where reasonable outfalls for ground water are not available. All of the above appurtenances may be enhanced by the use of some type of geotextile filter material.

11.7 Design Frequency And Spread

The major considerations for selecting a design frequency and spread include highway classification, because it defines and reflects public expectations for finding water on the pavement surface. Ponding should be prevented on the traffic lanes of high-speed, high-volume highways, where it is not expected.

Highway speed is another major consideration, because at speeds greater than 70 km/h, (45 mi/h) even a shallow depth of water on the pavement can cause hydroplaning. Design speed is recommended for use in evaluating hydroplaning potential. When the design speed is selected, consideration should be given to the likelihood that legal posted speeds may be exceeded. It is clearly unreasonable and not cost effective to provide the same level of protection for low speed facilities as for high speed facilities.

Other considerations include inconvenience, hazards and nuisances to pedestrian traffic and buildings adjacent to roadways which are located within the splash zone. These considerations should not be minimized and, in some locations (such as commercial areas), may assume major importance.

The design criteria for various types of Connecticut roadways are outlined in Table 11-2.

Table 11-2 Pavement Drainage Design Criteria

ROADWAY	ADT	SPEED km/hr (mi/hr)	DESIGN FREQUENCY yr	ALLOWABLE DESIGN SPREAD
State Arterial Highways and Expressways	≥ 3000	≥ 80 (≥ 50)	10	shoulder
	≥ 3000	≤ 70 (≤ 45)	10	½ of lane
	< 3000	---	10	½ of lane
Sag Condition	any	any	50*	all except one lane width
State Collector Highways and State-owned service Roads	≥ 3000	≥ 80 (≥ 50)	10	shoulder
	≥ 3000	≤ 70 (≤ 45)	10	½ of lane
	< 3000	---	10	½ of lane
Sag Condition	any	any	25*	all except one lane width
Town Roads	≥ 3000	any	10	½ of lane
	< 3000		5	½ of lane
Sag Condition	≥ 3000	any	25	all except one lane width
	< 3000		10	
One Lane Ramps	any	any	10	0.3m (1 ft) of lane
Ramps > one lane	any	any	10	1m (3 ft) of lane

* Sag condition is defined as sag vertical curves where the water cannot escape over berms and down an embankment. The procedure is to design the drainage inlets and storm system for a 10 year frequency and then to impose the higher frequency storm on the inlets and storm system. If the higher frequency storm closes the facility to traffic then additional inlets or the storm system will have to be changed.

11.8 Inlets

11.8.1 General

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

11.8.2 Standard ConnDOT Inlets

- Type “C” catch basin – This inlet consists of both a curb-opening inlet and a grate placed in a side-by-side configuration. In computing the inlet capacity, the curb opening is neglected and only grate opening is considered. The curb opening provides a relief if the grate should become clogged.

Type "C" catch basin double grate-Type I - This catch basin can be used on grades to reduce bypass since the frontal width increases by a factor of 2. The placement of this inlet needs to be evaluated to ensure that the grates do not protrude into the travelway which would cause traffic to veer away from it.*

Type "C" catch basin double grate Type II - these inlets are predominantly used at sags. They provide greater interception capacity than the single or Type I structures since the curb opening is longer and the perimeter of the grate to accept flow is greater.

- Slotted Drain - This inlet consists of a pipe cut along the longitudinal axis with a grate of spacer bar to form slot openings. Slotted inlets function as weirs with flow entering from the sides.
- Type “C-L” catch basin – This catch basin is a grate inlet placed in a swale or edge of roadway without curbing.

Type "C-L" catch basin double grate Type I - This is similar to the Type “C” catch basin double Grate Type I inlet except is more commonly used in swales where additional interception is required.*

Type "C-L" catch basin double grate Type II - these inlets are predominantly used at sags. They provide greater interception capacity than the single or Type I structures since the perimeter of the grate to accept flow is greater.

A typical detail showing grading adjacent to a type “C-L” catch basin located in a swale can be found in Appendix B.

- End Sections & End Walls - these inlets will be designed using the procedures found in the culvert chapter.
- Lawn Drains - These inlets vary in size dependant on discharge. They are economical to use in residential areas and traffic islands where minimal flow occurs.
- "D-G" type endwall - This inlet was developed primarily to drain earth and paved ditches at the top of cut slopes. These inlets should only be located beyond the area required for safe recovery of vehicles and where they will not be encountered by pedestrians. The capacity of these can be computed using the formula, $Q = 3.33 LH^{1.5}$ (English version). Flow should not submerge the grate and the water surface developed by the outlet pipe should be low enough not to cause the weir to be submerged.
- Catch Basins without sumps – should be used where utility conflicts exist; storm sewer systems that convey watercourses; and where pipe 36 inches and larger is required. This structure has a formed invert.
- Drop Inlets – may be used where utility conflicts exist. This system has no inlet pipes other than underdrains.

*Note: These catch basins can also be used for connecting those pipes which cannot fit into the single grate structures. This avoids the need for special junction box designs in most instances. When these inlets are used for this purpose, they should be specified without sump to decrease junction losses.

11.8.3 Standard ConnDOT Grates

There are two grates currently used by the Department. These are designated Type "A" and "B" in the Roadway Standards. Type "B" grate will be used on limited access highways. Type "A" grate will be used in all other locations which are subject to bicycle and pedestrian traffic.

11.8.4 Inlets On Grade

The capacity of an inlet depends upon its geometry and the cross slope, longitudinal slope, total flow, depth of flow and pavement or swale roughness. The depth of water next to the curb is the major factor in the interception capacity of gutter inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted.

Experience indicates that for design purposes, it is safe to consider interception of the entire frontal flow and disregard side flow. The remainder of the flow outside the width of the inlet will bypass to the next inlet. All inlets are to be designed without considering depression unless the plans specify it, with contours.

11.8.5 Inlets At Low Points

In locations such as underpasses and in sag vertical curves in depressed sections, where significant ponding can occur, flanking inlets shall be placed on each side of the inlet at the low point in the sag. The flanking inlets should be placed so that they will limit spread on low gradient approaches to the level point and act in relief of the inlet at the low point if it should become clogged. (See Section 11.8.7.) Flanking inlets are not usually considered as intercepting flow in design computations.

11.8.6 Curbing

Curbs are used where runoff from the pavement would erode fill slopes. Curbs could be eliminated where other permanent erosion control measures such as rock embankments are provided. Curbs may also be required in developed areas, with sidewalks and in cut sections with positive backslopes at the gutter. Where curbs are used with positive backslopes, a minimum shoulder width of 1.2 meters (4 ft) is required to achieve the proper cross slope to contain the flow. Curbing is required for any slope steeper than 1 (vertical) : 4 (horizontal) with the exception of the high side of superelevated sections. If curbing is not possible then erosion control matting should be used to stabilize the embankments until vegetation is established. Refer to the Highway Design Manual for additional guidance for the use of curbing.

11.8.7 Inlet Locations

This section stipulates where the location of inlets should be.

- Inlets are generally located immediately upgrade of a bridge. If on the downgrade of a bridge there is no extension or curbing, an inlet may be needed to prevent erosion.
- On vertical sags, flanker basins shall be located 0.06 m (0.2 ft) higher than the low point catch basin.
- Inlets should not be located in driveways or within 1.5 meters (5 ft) from driveway returns to preclude loss of traction.
- Bridge inlets may not be able to be located to meet the hydraulic requirements. Bridge inlets should be located with the approval of the bridge engineer. Keep in mind that an increase in width or spread may be justified for a short length of roadway to avoid the use of bridge inlets.
- At sag vertical curves of expressways where curbing is not used, a type "C-L" catch basin shall be placed at the outer edge of the shoulder and within the shoulder. This inlet will collect rain and snow melt which cannot reach the actual sag location due to the dam created by snow plowing operations. **No interception should be computed for this inlet and a 300 mm (12 in) pipe is to be used for an outlet.**
- Drainage inlets are sized and located to limit the spread on traffic lanes to tolerable widths as is indicated in Table 11-2. Grate inlets should be located outside the through-traffic lanes to minimize the shifting of vehicles attempting to avoid them.
- Where there is a danger of damage to adjacent property by flow overtopping the curb in a sag, flanking inlets should be used and the location checked to insure that the curb is not overtopped due to insufficient inlet capacity.
- Inlets should be located so that concentrated flow and sheet flow will not cross traffic lanes. Where pavement surfaces are warped, as at cross streets or ramps, surface water should be intercepted just before the change in cross slope.
- Inlets should be located just upgrade of pedestrian crossings if required near the crossing.
- Special care should be given to inlet placement to insure adequate capacity at bridge approaches and at sag vertical curves where ponding deeper than the curb height could occur.
- The maximum depth of flow in a gutter or shoulder shall be limited to 25 mm (1 in) below the top of curb, except at sag locations.
- Where driveways descend from the highway, the maximum depth of flow will be limited to 0.10 meters (0.3 ft).
- At intersections where the grades of both roads are positive, there usually is a vertical sag created at the curb line. To determine the exact location of these inlets and to determine their capacity, it is necessary to develop a profile of the gutter and a contour of the impacted quadrant of the intersection.
- Positive slopes, channelization islands, gore areas, etc. can cause snow melt from the roadside to freeze, creating ice conditions which require application of abrasives and chemicals. These designs can be avoided in most instances when thought is given to winter highway operations. A few measures which can be used are:
 1. Insure that slopes either flow to gutters or that they are negative from the roadway.
 2. Islands which do not flow to gutters should be depressed and drained with an inlet.
 3. Careful inlet designs at the optimum location to control flows in gore areas.
- At roadway transitions from cut to fill at the downhill terminus of a negative shelf should be drained with an inlet or properly designed channel to preclude erosion.

11.9 Gutter Flow Calculations

11.9.1 Introduction

Gutter flow calculations are necessary in order to relate the quantity of flow (Q) in the curbed channel to the spread of water on the shoulder, parking lane, or pavement section. The nomograph on Figure 11-1 can be utilized to solve uniform cross slope channels, composite gutter sections and V shape gutter sections. Figure 11-3 is also very useful in solving composite gutter section problems. Computer programs such as the FHWA HEC 12 program is also very useful for this computation as well as inlet capacity. Example problems for each gutter section are shown in the following sections.

11.9.2 Manning's n For Pavements

Table 11-3 Manning's n For Streets and Pavement Gutters

Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.012
Asphalt Pavement: Smooth texture	0.013
Rough texture	0.016
Concrete gutter-asphalt pavement Smooth	0.013
Rough	0.015
Concrete pavement Float finish	0.014
Broom finish	0.016
For gutters with small slope, where sediment may accumulate, increase above n values by:	0.002
Reference: USDOT, FHWA, HDS-3 (1961)	

11.9.3 Uniform Cross Slope Procedure

The nomograph in Figure 11-1 is used with the following procedures to find gutter capacity for uniform cross slopes:

CONDITION 1: Find spread (T), given gutter flow (Q).

- Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter flow (Q) and Manning's n.
- Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.

- Step 3 Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not, use the product of Q and n .
- Step 4 Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

CONDITION 2: Find gutter flow (Q), given spread (T).

- Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T) and Manning's n .
- Step 2 Draw a line between the S and S_x scales and note where it intersects the turning line.
- Step 3 Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.
- Step 4 For Manning's n values of 0.016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values (see Table 11-3), the gutter capacity times n (Qn) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

11.9.4 Composite Gutter Sections Procedure

Figure 11-3 can be used to find the flow in a gutter section with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets.

CONDITION 1: Find spread (T), given flow (Q).

- Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), depressed section slope (S_w), depressed section width (W), Manning's n , gutter flow (Q) and a trial value of the gutter capacity above the depressed section (Q_s). (Example: $S = 0.01$; $S_x = 0.02$; $S_w = 0.06$; $W = 0.6$ m; $n = 0.016$; $Q = 0.057$ m³/s; try $Q_s = 0.020$ m³/s)
- Step 2 Calculate the gutter flow in W (Q_w), using the equation:
- $$Q_w = Q - Q_s \quad (Q_w = 0.057 - 0.020 = 0.037 \text{ m}^3/\text{s}) \quad (11.5)$$
- Step 3 Calculate the ratios Q_w/Q and S_w/S_x and use Figure 11-2 to find an appropriate value of W/T . ($Q_w/Q = 0.037/0.057 = 0.65$ $S_w/S_x = 0.06/0.02 = 3$ From Figure 11-2, $W/T = 0.27$)
- Step 4 Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3. ($T = 0.6/0.27 = 2.22$ m)
- Step 5 Find the spread above the depressed section (T_s) by subtracting W from the value of T obtained in Step 4. ($T_s = 2.22 - 0.6 = 1.62$ m)

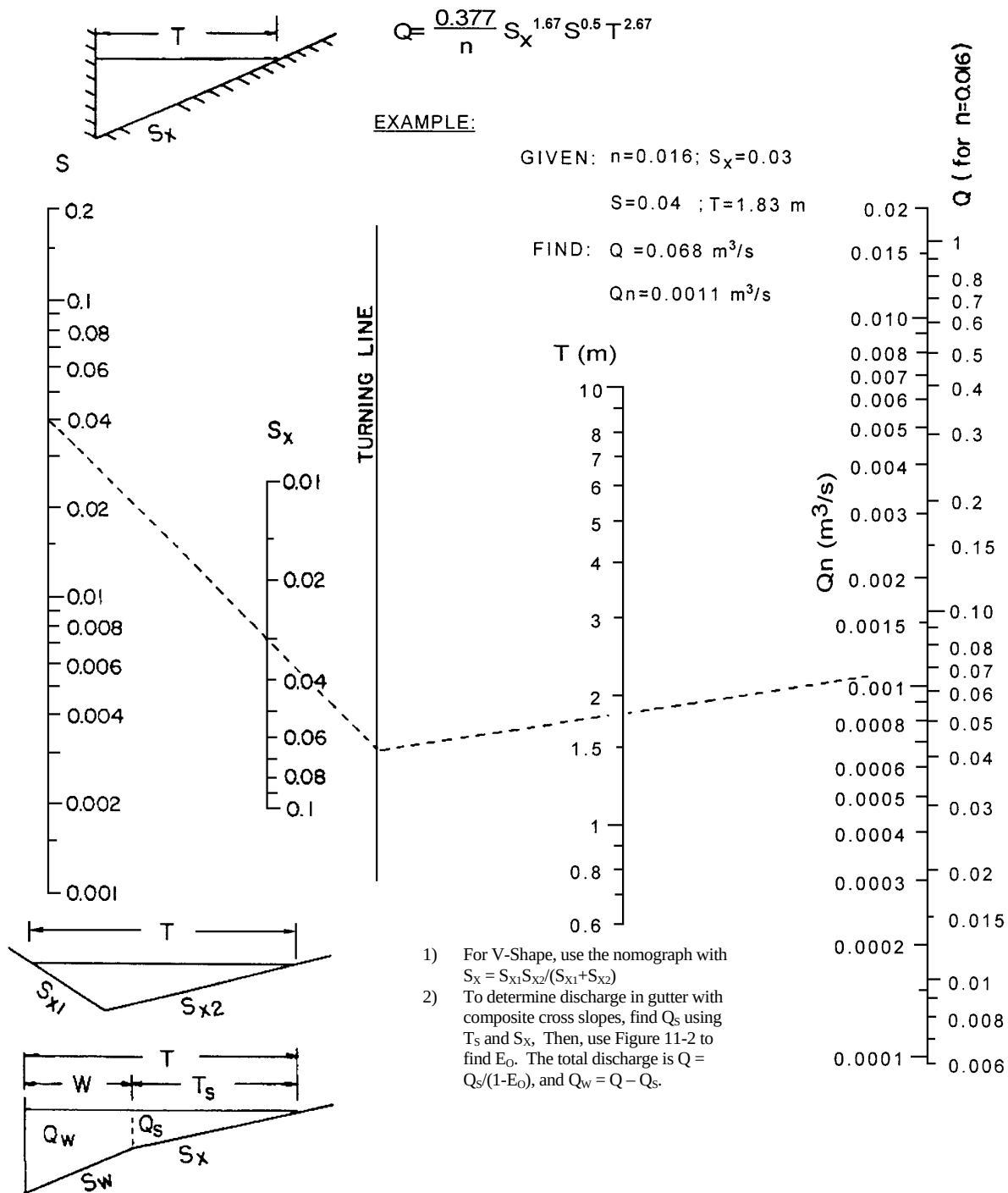


Figure 11-1 Flow In Triangular Gutter Sections – Metric units

Source: HEC 12

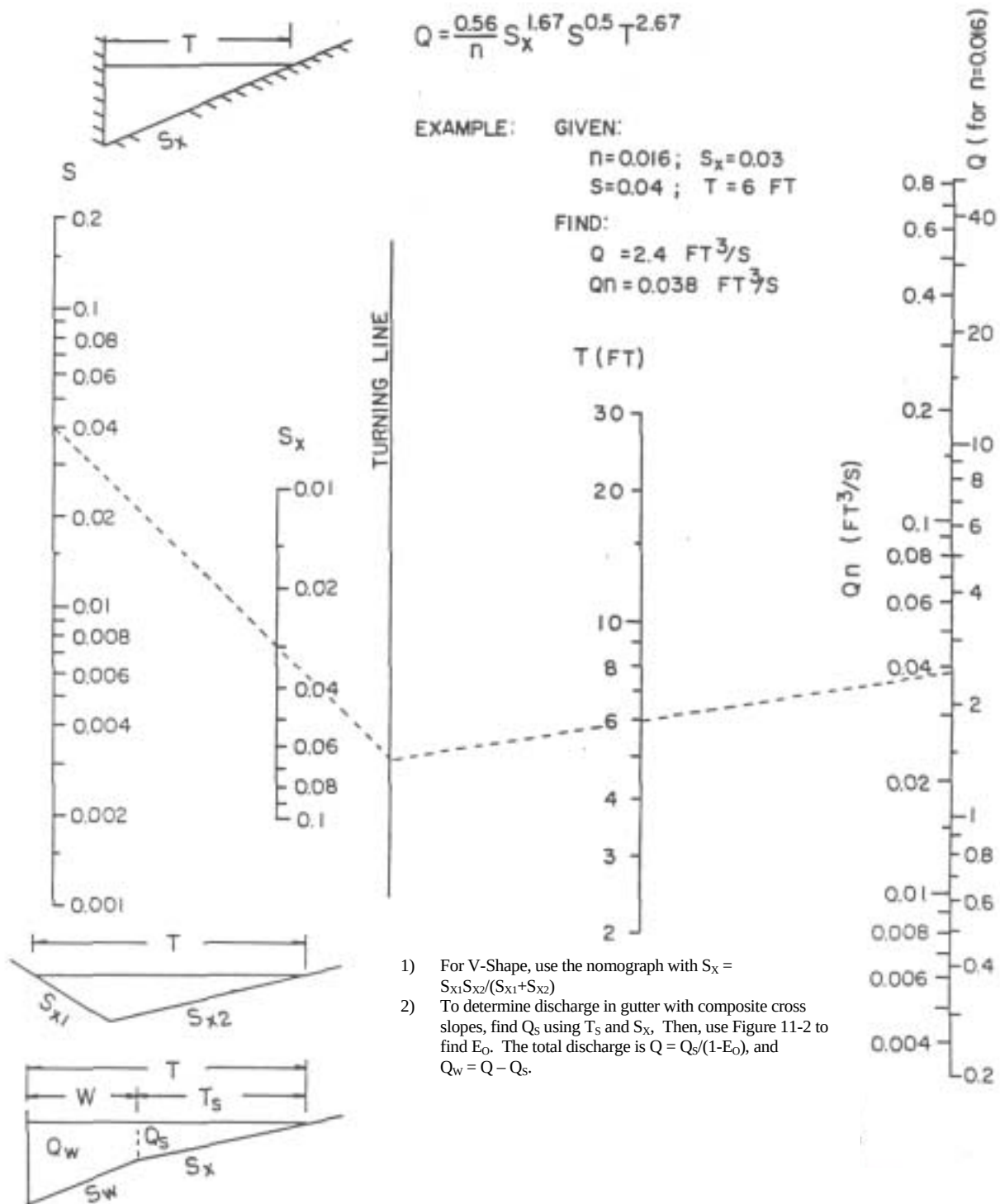


Figure 11-1.1 Flow In Triangular Gutter Sections – English units

Source: HEC-12

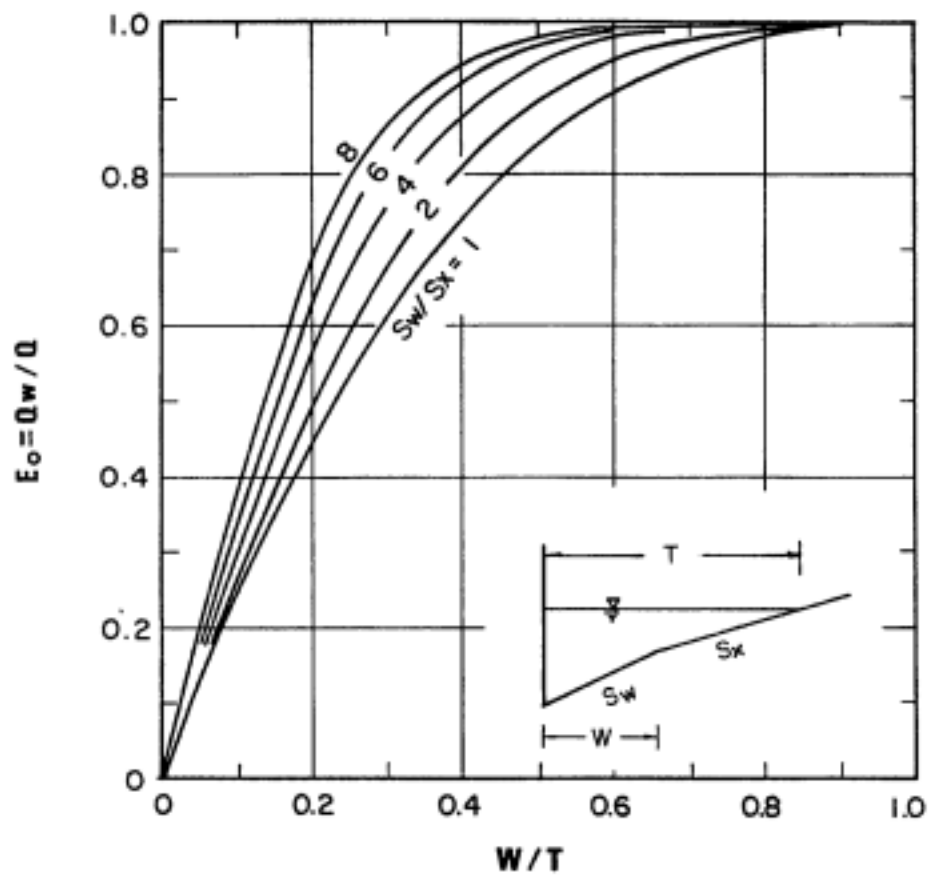


Figure 11-2 Ratio Of Frontal Flow To Total Gutter Flow

Source: HEC-12

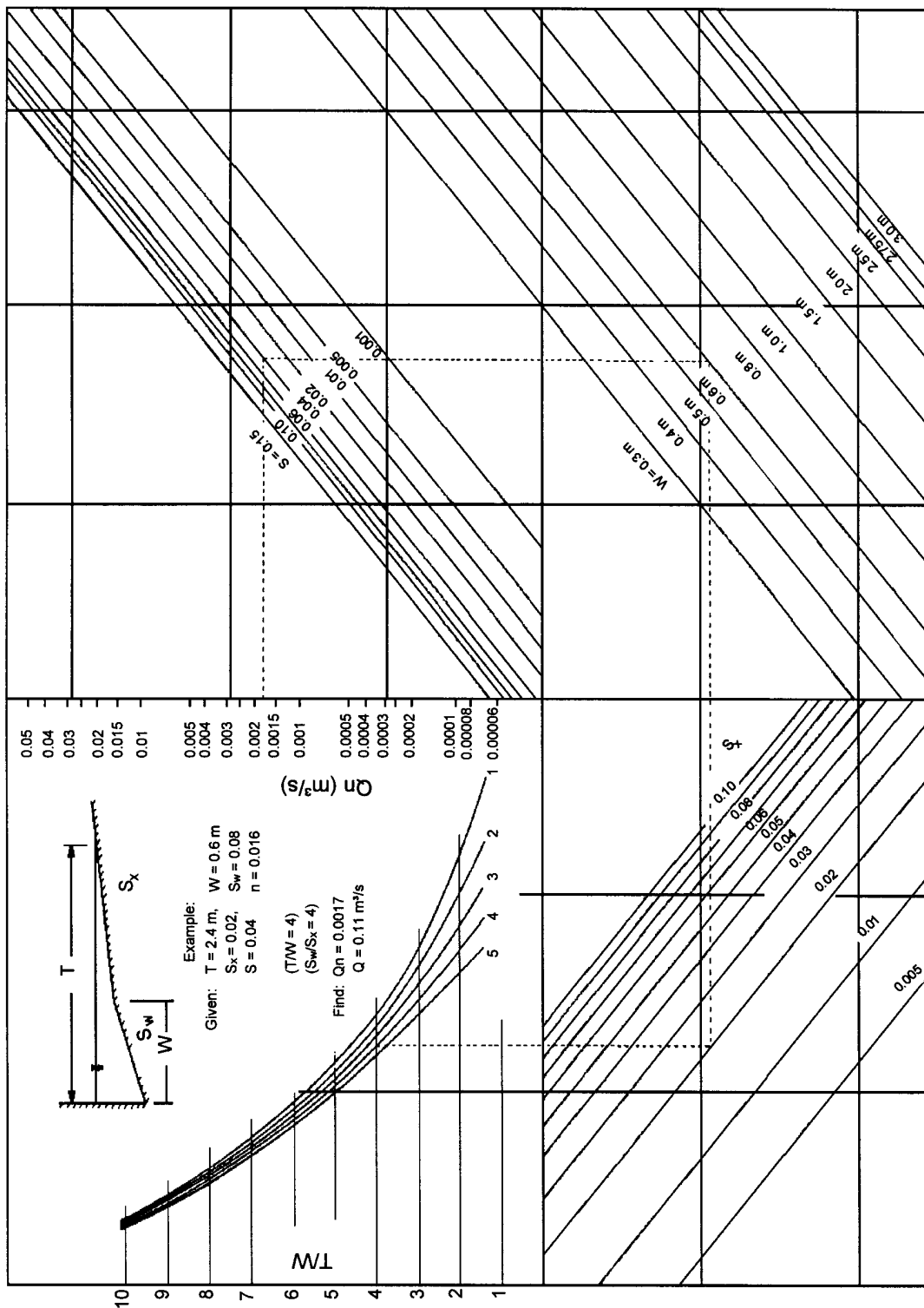


Figure 11-3 Flow in Composite Gutter Sections – Metric units

Source: HEC 12

- Step 6 Use the value of T_s from Step 5 along with Manning's n , S and S_x to find the actual value of Q_s from Figure 11-1. (From Figure 11-1 $Q_s = 0.014 \text{ m}^3/\text{s}$)
- Step 7 Compare the value of Q_s from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1.

(Compare 0.014 to 0.020 "no good," Try $Q_s = 0.023$; then $0.057 - 0.023 = 0.034$; and $0.034/0.057 = 0.6$; From Figure 11-2 $W/T = 0.23$, then $T = 0.6/0.23 = 2.61 \text{ m}$ and $T_s = 2.61 - 0.6 = 2.01 \text{ m}$. From Fig 11-1, $Q_s = 0.023 \text{ m}^3/\text{s}$ OK)

ANSWER: Spread $T = 2.61 \text{ m}$

CONDITION 2: Find gutter flow (Q), given spread (T).

- Step 1 Determine input parameters, including spread (T), spread above the depressed section (T_s), cross slope (S_x), longitudinal slope (S), depressed section slope (S_w), depressed section width (W), Manning's n and depth of gutter flow (d).

EXAMPLE: (Allowable spread $T = 3.05 \text{ m}$; $W = 0.6 \text{ m}$; $T_s = 3.05 - 0.6 = 2.44 \text{ m}$; $S_x = 0.04$; $S = 0.005 \text{ m/m}$; $S_w = 0.06$; $n = 0.016$; $d = 0.13 \text{ m}$)

- Step 2 Use Figure 11-1 to determine the capacity of the gutter section above the depressed section (Q_s). Use the procedure for uniform cross slopes- Condition 2, substituting T_s for T . (From Figure 11-1, $Q_s = 0.085 \text{ m}^3/\text{s}$)
- Step 3 Calculate the ratios W/T and S_w/S_x , and from Figure 11-2, find the appropriate value of E_o (the ratio of Q_w/Q). ($W/T = 0.6/3.05 = 0.2$; $S_w/S_x = 0.06/0.04 = 1.5$; From Figure 11-2 $E_o = 0.46$)
- Step 4 Calculate the total gutter flow using the equation:

$$Q = Q_s / (1 - E_o) \quad (11.6)$$

Where: Q = gutter flow rate, m^3/s

Q_s = flow capacity of the gutter section above the depressed section, m^3/s

E_o = ratio of frontal flow to total gutter flow (Q_w/Q)

$$(Q = 0.085 / (1 - 0.46) = 0.157 \text{ m}^3/\text{s})$$

- Step 5 Calculate the gutter flow in width (W), using equation 11.5.

$$(Q_w = Q - Q_s = 0.157 - 0.085 = 0.072 \text{ m}^3/\text{s})$$

NOTE: Figure 11-3 can also be used to calculate the flow in a composite gutter section.

11.9.5 V Type Gutter Sections Procedures

Figure 11-1 can also be used to solve V Type channel problems. The spread (T) can be calculated for a given flow (Q) or the flow can be calculated for a given spread. This method can be used to calculate approximate flow conditions in the triangular channel adjacent to concrete median barriers. It assumes the effective flow is confined to the V channel with spread T_1 .

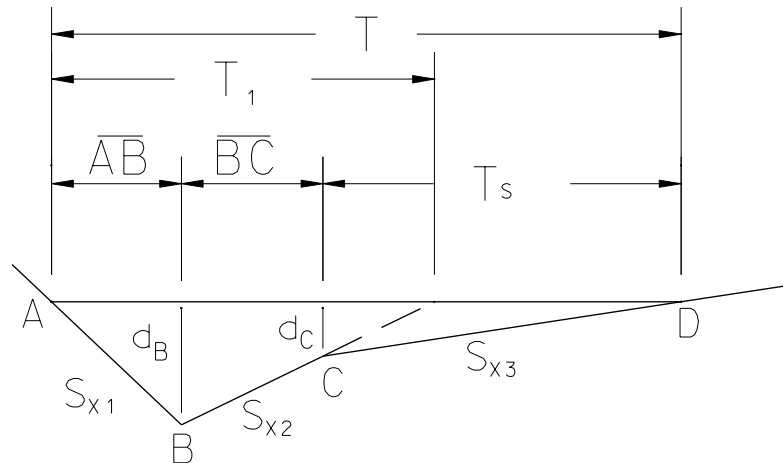


Figure 11-4 V Type Gutter

CONDITION 1: Given flow (Q), find spread (T).

- Step 1 Determine input parameters, including longitudinal slope (S), cross slope $S_x = S_{x1}S_{x2}/(S_{x1}+S_{x2})$, Manning's n, total flow (Q). (Example: $S = 0.01$, $S_{x1} = 0.25$, $S_{x2} = 0.04$, $S_{x3} = 0.015$, $n = 0.016$, $Q = 0.050 \text{ m}^3/\text{s}$, distance BC = 0.6m)
- Step 2 Calculate S_x
 $S_x = S_{x1}S_{x2}/(S_{x1} + S_{x2}) \quad S_x = (0.25)(0.04)/(0.25 + 0.04) = 0.0345$
- Step 3 Solve for T_1 using the nomograph on Figure 11-1.
- T_1 is a hypothetical width that is correct if it is contained within S_{x1} and S_{x2} . From nomograph $T_1 = 1.94\text{m}$
- Step 4 To determine if T_1 is within S_{x1} and S_{x2} , compute the flow depth, d_B , at point B and use this depth to find the horizontal distance between points A and B, AB. d_B can be computed using the following geometric relationship.

$$\begin{aligned} T_1 &= (d_B/S_{X1}) + (d_B/S_{X2}), \text{ from which} \\ d_B &= T_1(S_{X1})(S_{X2}) / (S_{X1} + S_{X2}) = (1.94(0.25)(0.04)/(0.25 + 0.04) \\ d_B &= 0.067 \text{ m (0.22 ft)} \\ AB &= d_B / S_{X1} = 0.067 / 0.25 \\ AB &= 0.27\text{m (0.9 ft)} \\ AC &= AB + 0.6\text{m} = 0.27\text{m} + 0.6\text{m} \\ AC &= 0.87 \text{ m (2.9 ft)} \\ 0.87\text{m} &< T_1 \text{ therefore, spread falls outside V-shaped gutter section.} \end{aligned}$$

- Step 5 Solve for the depth at point C, d_c , and compute the actual spread from edge of gutter section T_s

$$\begin{aligned} d_c &= d_B - BC (S_{x2}) \\ &= (0.067) - (0.60)(0.04) \\ &= 0.043 \text{ m (0.14 ft)} \end{aligned}$$

$$\text{Therefore, } T_s = d_c / S_{x3} = (0.043)/(0.015) = 2.87 \text{ m (9.4 ft)}$$

- Step 6 Find the actual total spread (T).

$$\begin{aligned} T &= T_s + AB + BC \\ T &= 2.87 \text{ m} + 0.27 \text{ m} + 0.6 \text{ m} \\ T &= 3.74 \text{ m (12.3 ft)} \end{aligned}$$

CONDITION 2: Given Spread (T), Find Flow (Q)

- Step 1 Determine input parameters such as longitudinal slope (S), Cross slope (S_x) = $S_{x1}S_{x2}/(S_{x1} + S_{x2})$, Manning's n and allowable spread. (Example: $n = 0.016$, $S = 0.015$, $S_{x1} = 0.06$, $S_{x2} = 0.04$, $T = 1.83 \text{ m}$)

- Step 2 Calculate S_x

$$S_x = S_{x1}S_{x2}/(S_{x1} + S_{x2}) = (0.06)(0.04)/(0.06 + 0.04) = 0.024$$

- Step 3 Using Figure 11-1, Solve for Q

$$\text{For } T = 1.83 \text{ m, } Q = 0.028 \text{ m}^3/\text{s}$$

The equation shown on Figure 11-1 can also be used.

11.9.6 Grate Inlets in A Sag

A type "C-L" catch basin in a sag operates as a weir up to a certain depth dependent on the bar configuration and size of the grate (Type A or B) and as an orifice at greater depths. For these types of grates, weir operation continues to a depth of about 0.12m (0.4 ft.) above the top of grate and when depth of water exceeds about 0.43m (1.4 ft.), the grate begins to operate as an orifice. Between depths of about 0.12m (0.4 ft.) and about 0.43m (1.4 ft.), a transition from weir to orifice flow occurs. For a type "C" catch basin the side against the curb is not included in computing the perimeter (P).

The capacity of grate inlets operating as a weir is:

$$Q_i = \frac{CPd^{1.5}}{C_{FS}} \quad (11.7)$$

solving for d:

$$d = \left(\frac{Q_i C_{FS}}{CP} \right)^{2/3}$$

where:

- Q_1 = rate of discharge into grate opening, m³/s (cfs)
- P = perimeter of grate excluding bar widths and the side against the curb, m (ft)
- C = 1.66 (3.0)
- d = depth of water above grate, m (ft)
- C_{FS} = factor of safety for clogging

The capacity of grate inlets operating as an orifice is:

$$Q_i = \frac{CA(2gd)^{0.5}}{C_{FS}} \quad (11.8)$$

solving for d:

$$d = \left(\frac{Q_i C_{FS}}{CA} \right)^2 / 2g$$

where:

- Q_1 = rate of discharge into grate opening, m³/s (cfs)
- C = 0.67 orifice coefficient
- A = clear opening area of the grate, m² (ft²)
- g = 9.81 m/s² (32.2 ft/s²)
- d = depth of water above grate, m (ft)
- C_{FS} = factor of safety for clogging
 - = 1.0 – Type “C” catch basin with 0% clogging
 - = 2.0 – Type “C-L” catch basin with 50% clogging – high clogging potential
 - = 1.0 < C_{FS} < 2.0 – Type “C-L” catch basin with 0%–50% clogging – low clogging potential. Typically for expressway medians, swales, and ditches where minimal tree growth is expected, a C_{FS} = 1.25 for 20% clogging is appropriate.

Between depths over the grate of about 0.12m (0.4 ft.) and about 0.43m (1.4 ft.) the operation of the grate inlet is indefinite due to vortices and other disturbances. The capacity of the grate is somewhere between that given by equations 11.7 and 11.8. The larger depth is used for design purposes.

Because of the vortices and the tendency of trash to collect on the grate, a factor of safety for clogging has been added to equations 11.7 and 11.8. For Type “C-L” catch basins with a high potential for clogging a factor of safety of 2 should be used. Where danger of clogging is slight, a factor of safety less than two might be used. When a type “C” catch basin is used, the curb opening provides the safety factor from clogging therefore the factor of safety is one (1.0).

DOT GRATE AREAS AND PERIMETERS ARE AS FOLLOWS:

Catch Basin with Type A Grate

Total Steel Frame Length	3'- 1 3/4"	or 0.9588 m	(3.1458 ft)
2 Angles 2 1/2" Wide	2 (2 1/2")	or 0.1270 m	(0.4167 ft)
8 Bars 5/8" Wide	8 (5/8")	or 0.1270 m	(0.4167 ft)
Length Clear Opening		or 0.7048 m	(2.3124 ft)

Steel Frame Width	1'- 7 5/8"	or 0.4985 m	(1.6354 ft)
9 Bars 3/8" Wide	9 (3/8")	or 0.0857 m	(0.2813 ft)
Width Clear Opening		or 0.4127 m	(1.3541 ft)

Perimeter (P)=	2(1.3541') + 2.3124' =	1.53 m	(5.02 ft)
Area (A)=	1.3541' x 2.3124' =	0.29 m ²	(3.13 ft ²)

Type "C" Catch Basin with Type A Double Grate Type II

Perimeter (P)=	2(1.3541') + (2)2.3124' =	2.24 m	(7.33 ft)
Area (A)=	1.3541' x 2.3124' x 2 =	0.58 m ²	(6.26 ft ²)

Type "C-L" Catch Basin with Type A Grate

Perimeter (P)=	2(1.3541') + (2)2.3124' =	2.24 m	(7.33 ft)
Area (A)=	1.3541' x 2.3124' =	0.29 m ²	(3.13 ft ²)

Type "C-L" Catch Basin with Type A Double Grate Type II

Perimeter (P)=	2(1.3541') + (4)2.3124' =	3.64 m	(11.96 ft)
Area (A)=	1.3541' x 2.3124' x 2 =	0.58 m ²	(6.26 ft ²)

It should be noted that these perimeters and areas are for Type A grate. These may also be used with Type B grates as the difference is insignificant.

11.9.7 Slotted Inlets On Grade

Wide experience with the debris handling capabilities of slotted inlets is not available. Deposition in the pipe is the problem most commonly encountered, and the inlet is accessible for cleaning only with a high pressure water jet. Slotted inlets are effective pavement drainage inlets which have a variety of applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations.

Slotted Inlets On Grade

Flow interception by slotted inlets is side weir and the flow is subjected to lateral acceleration due

to the cross slope of the pavement. Thus the equation with a straight cross slope is expressed by:

$$L_T = KQ^{0.42}S^{0.3}(1/nS_x)^{0.6} \quad (11.9)$$

where:

$$K = 0.817 (0.6)$$

L_T = slotted inlet length required to intercept 100% of the gutter flow, m (ft)

S = longitudinal slope m/m (ft/ft)

S_x = cross slope m/m (ft/ft)

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

$$E = 1 - (1 - L/L_T)^{1.8} \quad (11.10)$$

Where:

L = slotted inlet length, m (ft)

Figure 11-5 is a nomograph for the solution of equation 11.9, and Figure 11-6 provides a solution of equation 11.10.

The following example illustrates the use of this procedure.

Given: $Q = 0.113 \text{ m}^3/\text{s}$ $n = 0.016$
 $S = 0.010$ $S_x = 0.02$

Find: Q_{IN} = for 6.1m of slotted inlet

Solution: From Figure 11-6 $L_T = 10.36\text{m}$
 $L/L_T = 6.1/10.36 = 0.58$

From Figure 11-7 $E = 0.79$
 $Q_{IN} = (E)(Q) = 0.79 \times 0.113 = 0.089 \text{ m}^3/\text{s}$

11.9.8 Slotted Inlets In A Sag

Slotted inlets are not recommended for sag conditions since the pipe would be on a zero grade resulting in non cleaning velocities. However, if at any time it becomes necessary to design a slotted inlet in a sag, the following procedure is to be used.

Slotted inlets in sag locations perform as weirs to depths of about 0.06m (0.2 ft), dependent on slot width and length. At depths greater than about 0.12m (0.4 ft), they perform as orifices. Between these depths, flow is in a transition stage. The interception capacity of a slotted inlet operating as an orifice can be computed by the following equation:

$$Q_i = 0.8LW(2gd)^{0.5} \quad (11.11)$$

where:

- W = width of slot, m (ft)
- L = length of slot, m (ft)
- d = depth of water at slot, m (ft)
- g = 9.81 m/s² (32.2 ft/s²)

For a slot width of 44mm (1 ¾ in), the above equation becomes:

$$Q_i = 0.156Ld^{0.5} \quad (Q_i = 9.37Ld^{0.5}) \quad (11.12)$$

The interception capacity of slotted inlets at depths between 0.06m (0.2 ft) and 0.12m (0.4 ft) can be computed by use of the orifice equation. The orifice coefficient varies with depth, slot width, and the length of the slotted inlet. Figure 11-7 provides solutions for weir flow and a plot representing data at depths between weir and orifice flow.

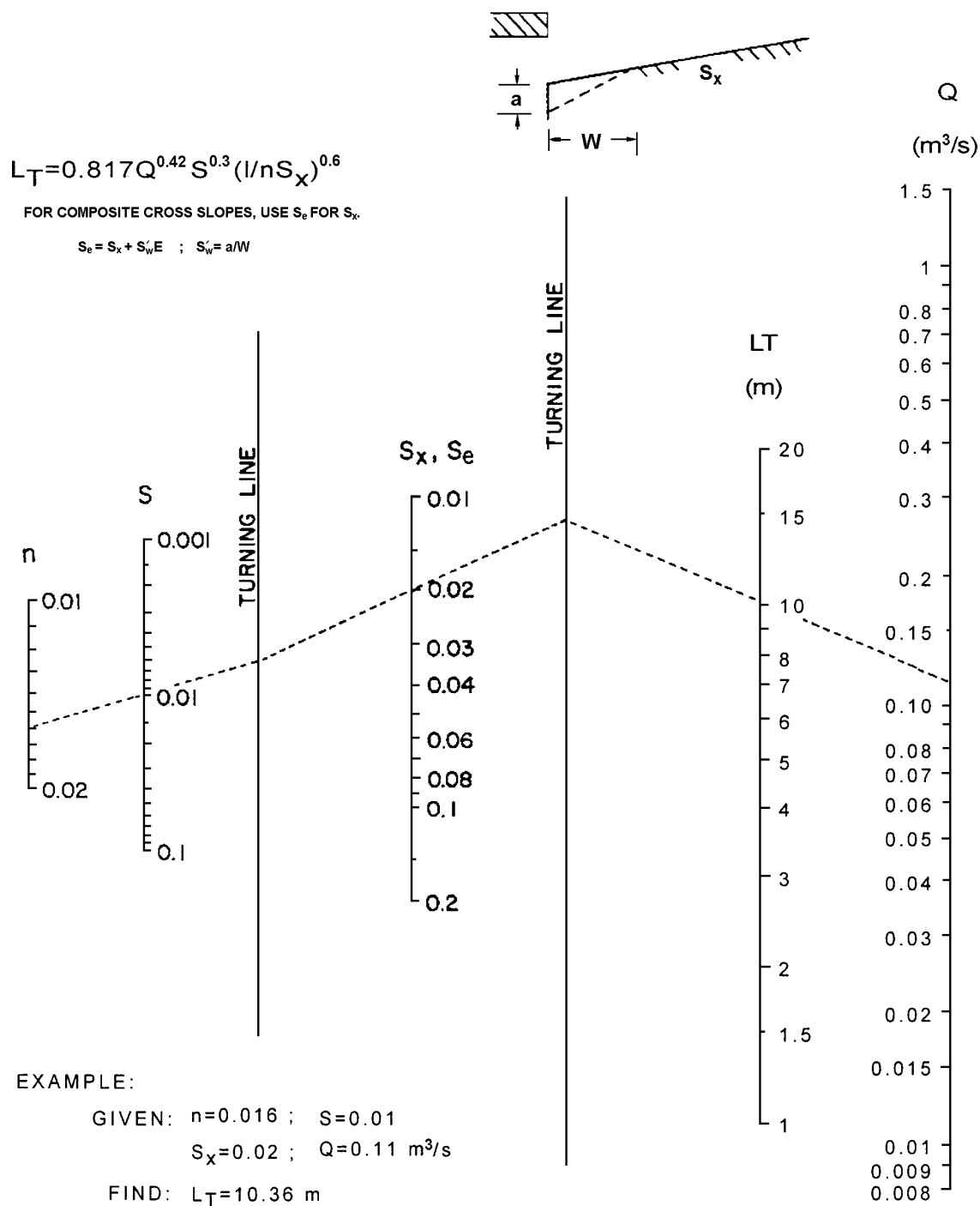


**Figure 11-5 Curb-Opening And Longitudinal Slotted Drain
Inlet Length For Total Interception – Metric units**

Source: HEC-12

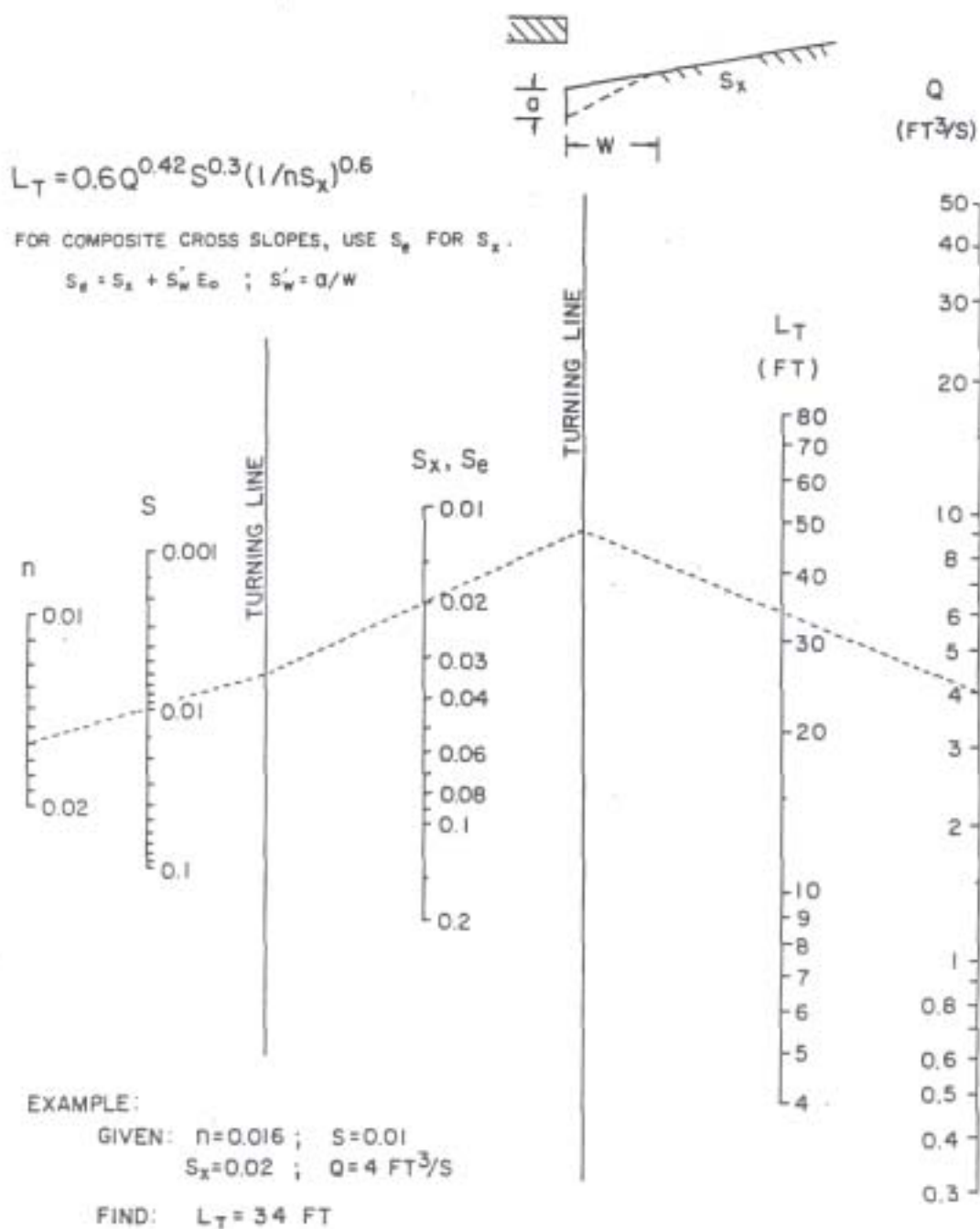


Figure 11-5.1 Curb Opening and Drain Inlet Length for Total Interception – English units

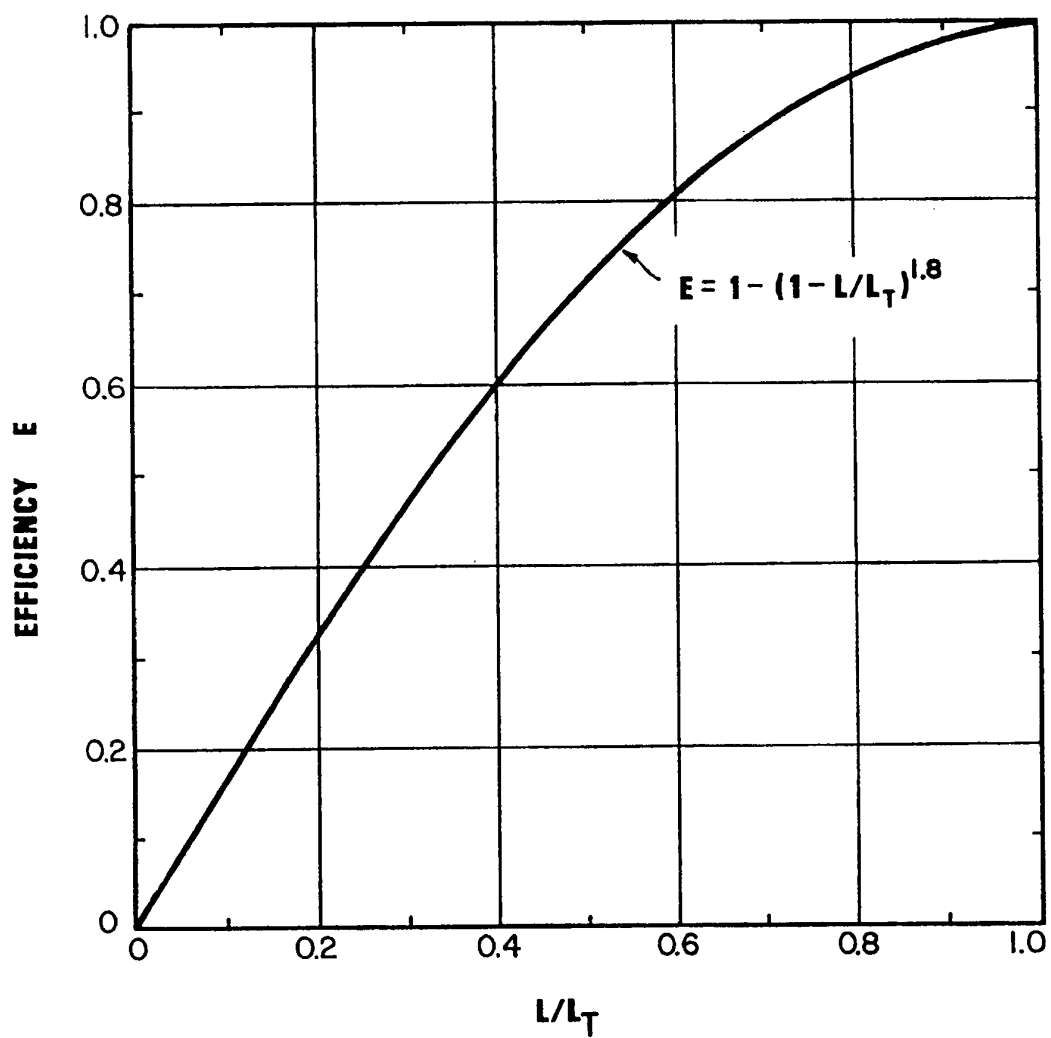


Figure 11-6 Curb-Opening And Slotted Drain Inlet Interception Efficiency
Source: HEC-12

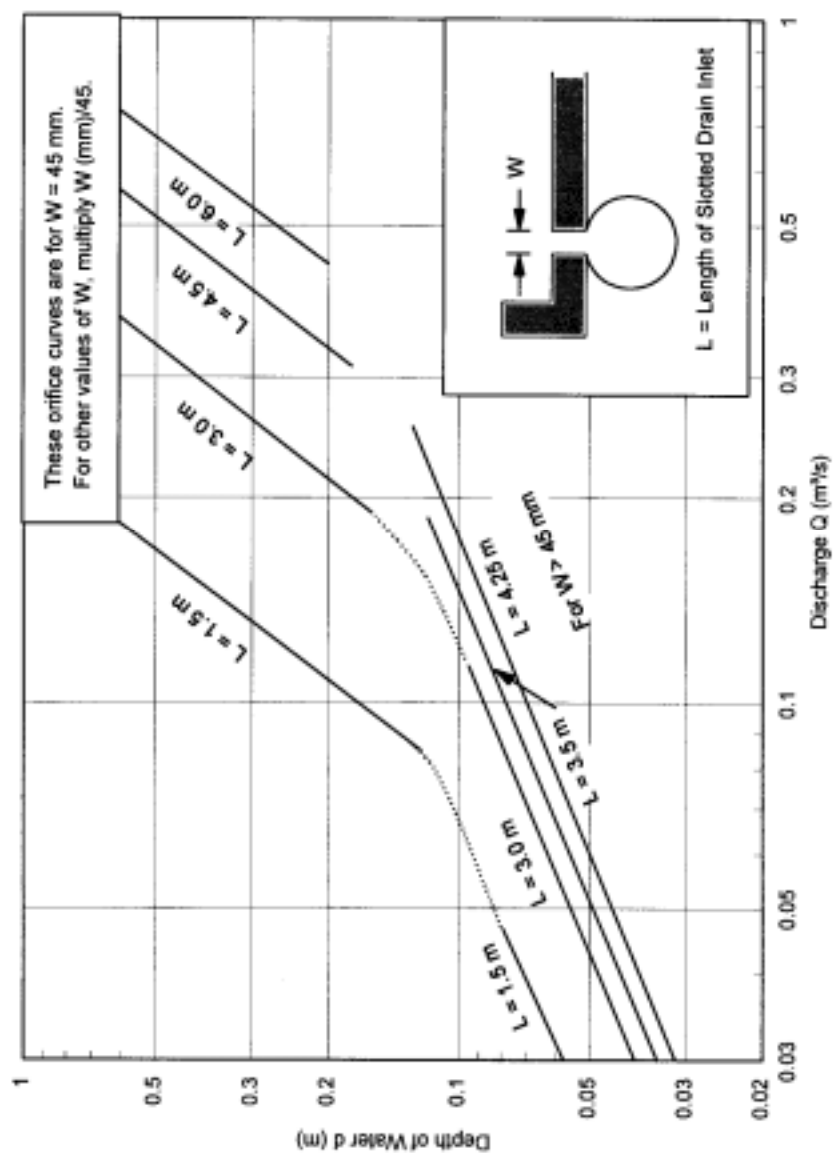


Figure 11-7 Slotted Drain Inlet Capacity In Sump Locations – Metric units

Source: HEC-12

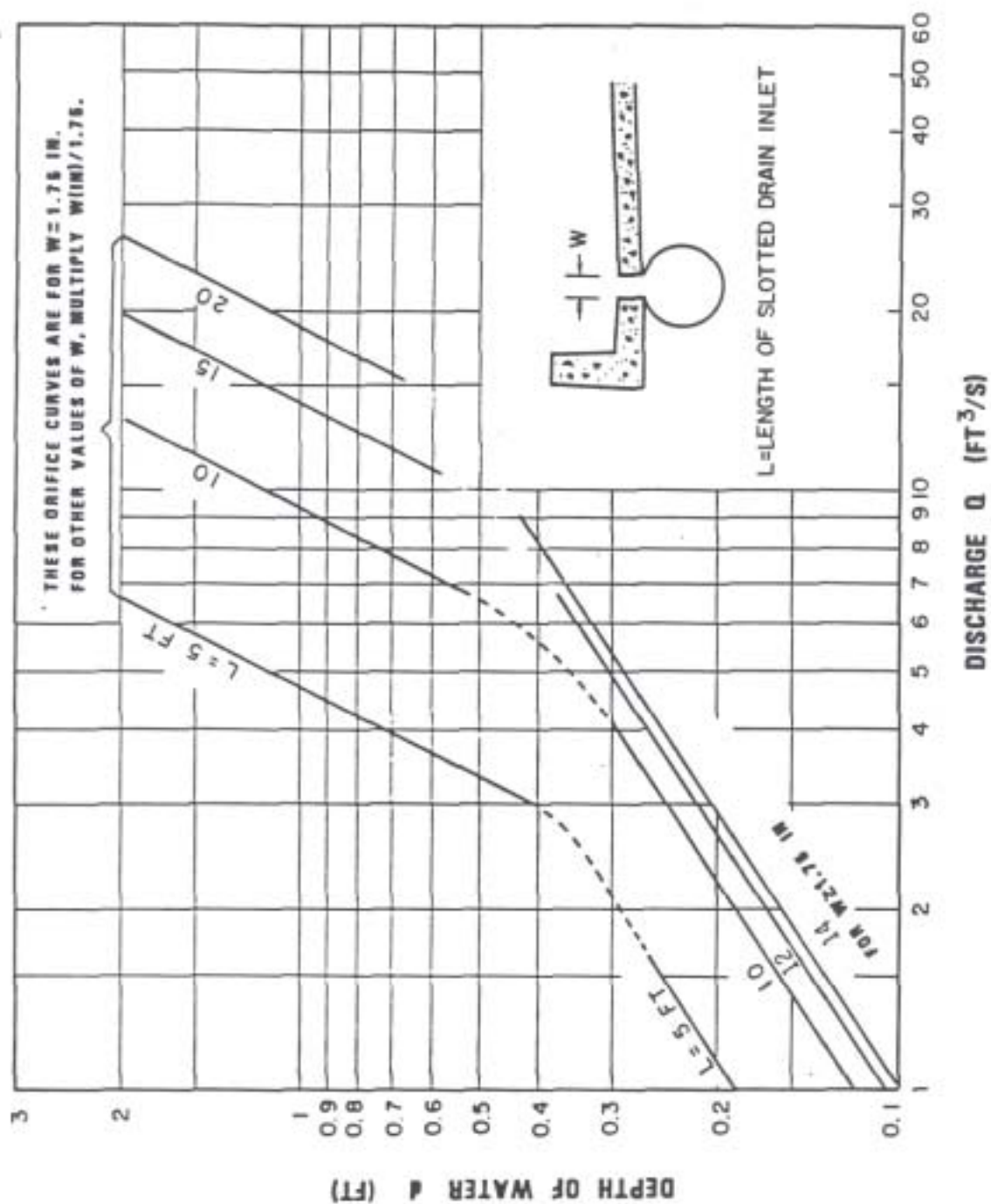


Figure 11-7.1 Slotted Drain Inlet Capacity in Sump Locations – English units
Source: HEC-12

11.9.9 Inlet Spacing Computations (Gutter Flow Analysis)

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 gutter flow analysis worksheet, Table 11-4, should be used to document the computations. A step by step procedure is as follows:

- Step 1 Complete the blanks on top of the sheet to identify the job by project, town, route, date, your initials, roadway type, ADT, design speed and allowable width of flooding.
- Step 2 Mark on the plan the location of inlets which are necessary even without considering any specific drainage area. See Section 11.8.7 Inlet Locations for additional information.
- Step 3 Start at one end of the job, at one high point and work towards the low point, then space from the other high point back to the same low point.
- Step 4 Select a trial drainage area approximately 90 to 150 m (300 to 500 ft) below the high point and outline the area including any area that may come over the curb. (Use drainage area maps.) Where practical, large areas of behind the curb drainage should be intercepted before it reaches the highway.
- Step 5 Describe the location of the proposed inlet by station and location from centerline. Enter in Col. 1. A sketch of the cross section should be provided in the open area of the computation sheet showing lane and shoulder arrangement and the available width of flow.
- Step 6 Compute the drainage area in hectares (acres) and enter in Col 2.
- Step 7 Select a C value from one of the tables in Chapter 6 and compute a weighted value based on area and cover type as described in Section 11.5.3 and enter in Col 3.
- Step 8 Compute a time of concentration for the first inlet. This will be the travel time from the hydraulically most remote point in the drainage area to the inlet. See additional discussion in Section 11.5.5 and in Chapter 6. The minimum time of concentration should be 5 min. Enter value in Col 4.
- Step 9 Select a rainfall intensity at the t_c for the design frequency (Chapter 6, Appendix B). Enter in Col. 5.
- Step 10 Compute the AC to the inlet Col 2 X Col 3. Enter in Col 6.
- Step 11 The total AC to the inlet is obtained by adding the AC bypassing the previous inlet and the AC for the inlet (Col. 6 + Col. 14). Enter in Col. 7. Note for the first inlet AC = Total AC.
- Step 12 Calculate Q by multiplying .00278 X Col. 5 X Col. 7 (1 X Col. 5 X Col. 7). Enter in Col. 8.

- Step 13 Determine the gutter slope at the inlet from the profile grade and the cross slope of shoulder and enter in Col. 9 and Col. 10, respectively (check effect of superelevation).
- Step 14 Using Figure 11-1 or an approved computer model, determine the spread T (width of flow) and enter in Col. 12 and calculate the depth d at the curb by multiplying T times the cross slope(s) and enter in Col. 11. Compare with the allowable spread as determined by the design criteria in Table 11.2. If Col. 11 is less than the curb height and Col. 12 is near the allowable spread, continue on to step 15. If not OK, expand or decrease the drainage area to meet the criteria and repeat steps 5 through 13. Continue these repetitions until column 12 is near the allowable spread then proceed to step 4.
- Step 15 Calculate Q bypassing the inlet. The portion of flow that is beyond the width of the grate will be used to determine the bypass Q. Enter in Col. 13.

$$Q_{bypass} = Q_{total} \left(\frac{(T - G)^2}{T^2} \right)$$

where

G = width of grate, m (ft.)

T = spread from Col. 12, m (ft.)

- Step 16 Determine the AC bypassing the inlet by dividing Col. 13 by Col. 5. Enter in Col. 14.
- Step 17 Calculate the AC entering the inlet. Col. 7-Col. 14. Enter in Col. 15.
- Step 18 Proceed to the next inlet downgrade. Select an area 90 to 120m (300 to 400 ft) below the first inlet as a first trail. Repeat Steps 5 through 17 considering only the area between inlets.
- Step 19 Go back to step 18 and repeat step 5 through step 17 for each subsequent inlet.
- Step 20 Inlets located in sag locations proceed to Section 11.9.10.

11.9.10 Low Point Analysis Computations

The following step by step procedure should be used to determine the depth and width of flow for inlets in sag locations. Table 11-5 should be used to document the computations.

- Step 1 Describe the location of the proposed lot point inlet by station and location from centerline. Enter in Col. 1. A sketch of the cross section should be provided in the open area of the computation sheet showing lane and shoulder arrangement and the available width of flow.
- Step 2 Compute the drainage area in hectares (acres) and enter in Col. 2.

- Step 3 Select a C value from one of the tables in Chapter 6 and compute a weighted value based on area and cover type as described in Section 11.5.3 and enter in Col 3.
- Step 4 Compute a time of concentration for the low point inlet. This will be the travel time from the hydraulically most remote point in the drainage area to the inlet. See additional discussion in Section 11.5.5 and in Chapter 6. The minimum time of concentration should be 5 min. Enter value in Col 4.
- Step 5 Select a rainfall intensity at the t_c for the design frequency (Chapter 6, Appendix B). Enter in Col. 5.
- Step 6 Compute the AC to the inlet Col 2 X Col 3. Enter in Col 6.
- Step 7 Referencing Table 11-5, select the “AC bypassing inlet” from the inlet just upstream and left of the low point inlet. Enter in Col. 7.
- Step 8 Referencing Table 11-5, select the “AC bypassing inlet” from the inlet just upstream and right of the low point inlet. Enter in col. 8.
- Step 9 Calculate the total AC by adding Col. 6 + Col. 7 + Col. 8. Enter in Col. 9.
- Step 10 Calculate the total Q to the low point inlet by multiplying .00278 X Col. 5 X Col. 9 (1 X Col. 5 X Col. 9). Enter in Col. 10.
- Step 11 Enter the cross slope of shoulder in Col. 11.
- Step 12 Calculate the depth of flow using equations 11.7 and 11.8. Enter in Col. 12.

Type “C-L” catch basins must have a clogging factor of safety included when determining depth of flow.

$C_{fs} = 2$ - Depressed expressway sag, depressed parking lots, and other depressed locations.

$C_{fs} = 1.25$ - Expressway medians, swales, and ditches where minimal tree growth is expected.

Type “C” catch basins do not require a clogging factor as the curb opening provides the necessary safety factor.

Therefore $C_{fs} = 1.0$

- Step 13 Determine the width of flow by dividing Col. 12 by Col. 11. Enter in Col. 13.

PROJECT _____ TOWN _____ ROUTE _____ DESIGNED BY: _____ DATE: _____ CHECKED BY: _____ DATE: _____	ROADWAY TYPE _____ (See Table 11-2) ADT _____ DESIGN SPEED _____ ALLOWABLE SPEED _____

GUTTER FLOW ANALYSIS															
INLET STATION AND OFFSET	AREA IN HECTARE (A)	RUNOFF COEFFICIENT (C)	TIME TO INLET MIN	RAINFALL INTENSITY MM/HR	AC	TOTAL AC	Q _T TO INLET CMS	GRADE OF GUTTER M/M	CROSS SLOPE OF SHOULDER M/M	d DEPTH OF FLOW OF GUTTER - METERS	T WIDTH OF FLOW - METERS	Q BYPASSING INLET - CMS	AC BYPASSING INLET	AC ENTERING CATCH BASIN	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	

Table 11-4 Inlet Spacing Computation Sheet – Metric units

PROJECT _____ TOWN _____ ROUTE _____ DESIGNED BY: _____ DATE: _____ CHECKED BY: _____ DATE: _____	ROADWAY TYPE _____ (See Table 11-2) ADT _____ DESIGN SPEED _____ ALLOWABLE SPEED _____

GUTTER FLOW ANALYSIS															
INLET STATION AND OFFSET	AREA IN ACRES (A)	RUNOFF COEFFICIENT (C)	TIME TO INLET MIN	RAINFALL INTENSITY IN/HR	AC	TOTAL AC	Q _T TO INLET CFS	GRADE OF GUTTER FT/FT	CROSS SLOPE OF SHOULDER FT/FT	d DEPTH OF FLOW OF GUTTER - FEET	T WIDTH OF FLOW - FEET	Q BYPASSING INLET - CFS	AC BYPASSING INLET	AC ENTERING CATCH BASIN	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	

Table 11-4.1 Inlet Spacing Computation Sheet – English units

PROJECT _____ TOWN _____ ROUTE _____ DESIGNED BY: _____ DATE: _____ CHECKED BY: _____ DATE: _____	ROADWAY TYPE _____ (See Table 11-2) ADT _____ DESIGN SPEED _____ ALLOWABLE SPEED _____

GUTTER FLOW ANALYSIS AT LOW POINT LOCATIONS													
LOW POINT INLET STATION AND OFFSET	AREA IN HECTARE (A)	RUNOFF COEFFICIENT (C)	TIME TO INLET MIN	RAINFALL INTENSITY MM/HR	AC	AC BYPASSING PREVIOUS INLET (L)	AC BYPASSING PREVIOUS INLET (R)	TOTAL AC	TOTAL Q TO LOW POINT INLET CMS	CROSS SLOPE OF SHOULDER M/M	DEPTH OF FLOW OF GUTTER M	WIDTH OF FLOW M	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12	13	

Table 11-5 Low Point Analysis Computation Sheet – Metric units

PROJECT _____ TOWN _____ ROUTE _____ DESIGNED BY: _____ DATE: _____ CHECKED BY: _____ DATE: _____	ROADWAY TYPE _____ (See Table 11-2) ADT _____ DESIGN SPEED _____ ALLOWABLE SPEED _____

GUTTER FLOW ANALYSIS AT LOW POINT LOCATIONS													
LOW POINT INLET STATION AND OFFSET	AREA IN ACRES (A)	RUNOFF COEFFICIENT (C)	TIME TO INLET MIN	RAINFALL INTENSITY IN/HR	AC	AC BYPASSING PREVIOUS INLET (L)	AC BYPASSING PREVIOUS INLET (R)	TOTAL AC	TOTAL Q TO LOW POINT INLET CFS	CROSS SLOPE OF SHOULDER FT/FT	DEPTH OF FLOW OF GUTTER FT	WIDTH OF FLOW FT	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12	13	

Table 11-5.1 Low Point Analysis Computation Sheet – English units

11.10 Manholes

11.10.1 Location

Manholes are utilized to provide entry to continuous underground storm drains for inspection and cleanout. When possible, utilize grate inlets in lieu of manholes so that the benefit of extra stormwater interception can be achieved with minimal additional cost. Typical locations where manholes should be specified are:

- where two or more storm drains converge
- at intermediate points along tangent sections
- where pipe size changes
- where an abrupt change in alignment occurs
- where an abrupt change of the grade occurs

Manholes should not be located in traffic lanes; however, when it is impossible to avoid locating a manhole in a traffic lane, care should be taken to insure it is not in the normal vehicle wheel path.

11.10.2 Spacing

The spacing of manholes should be in accordance with the following criteria:

<u>Size of Pipe (mm)</u>		<u>Maximum Distance (m)</u>
300-600	(12-24 in)	100 (350 ft)
675-900	(27-36 in)	125 (400 ft)
1050-1350	(42-54 in)	150 (500 ft)
1500-up	(60-up)	300 (1000 ft)

11.10.3 Standard ConnDOT Manholes (Standard Sheet M507-A)

The following guidelines should be followed when specifying a manhole.

- When the storm drain pipe diameter is 750 mm (30 in) or less, a 1200-mm (48 in) diameter manhole should be provided.
- When the storm drain pipe diameter is > 750mm to 1050 mm (30 in to 42 in) inclusive, a 1500-mm (60 in) diameter manhole should be provided.
- When the storm drain pipe diameter is 1050 mm to 1350 mm (42 in to 54 in), a 1800-mm (72 in) diameter manhole should be provided.
- When the storm drain pipe diameter is larger than 1350mm (54 in), a reinforced concrete pipe tee; ConnDOT standard type “C” or “C-L” catch basin double grate type I or II; or if necessary a special design junction box should be provided.

11.10.4 Sizing

When determining the minimum manhole size required for various pipe sizes and locations, two general criteria must be met.

- Manhole or inlet structure must be large enough to accept the maximum pipe as shown in Table 11-6.
- Knowing the relative locations of any two pipes, compute K as follows:

$$K = \frac{R_1 + T_1 + R_2 + T_2 + 355\text{mm}}{\Delta} \quad \left(K = \frac{R_1 + T_1 + R_2 + T_2 + 14\text{in}}{\Delta} \right) \quad (11.13)$$

Where: R_1 and T_1 are interior radius and wall thickness of Pipe #1, mm (in)

R_2 and T_2 are interior radius and wall thickness of Pipe #2, mm (in)

Δ = angle between the pipes, degrees

K = relative manhole coefficient, mm/degree (in/degree)

Table 11-6 Manhole Sizing

MH Dia. mm (in)	K * mm (in)/degree	Max Pipe Size mm (in)
1200 (48)	10.67 (.42)	750 (30)
1500 (60)	13.21 (.52)	1050 (42)
1800 (72)	16.00 (.63)	1350 (54)

- * When the computed K value exceeds 16 (.63), a junction chamber or double catch basin will be required.

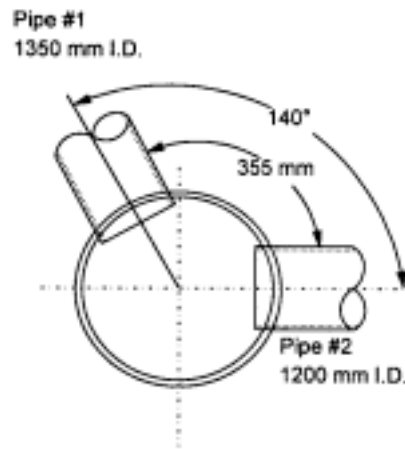


Figure 11-8

Example Problem

Given: Pipe # 1 = 1350 mm Pipe # 2 = 1200 mm
 $\Delta = 140^\circ$

Solution:

$$K = \frac{675 \text{ mm} + 140 \text{ mm} + 600 \text{ mm} + 127 \text{ mm} + 355 \text{ mm}}{140^\circ}$$

$$= 13.55 \text{ mm/degrees}$$

The table 11-6 indicates the minimum manhole barrel to be 1800 mm (72 in). For the 1500-mm (60 in) MH barrel, the table indicates a maximum pipe size of 1050 mm (42 in). As the maximum pipe size in the example is 1350 mm (54 in), an 1800-mm (72 in) MH must be used.

For this example, spacing is not critical and the pipe size governs. Had the Δ angle been 115° or less, the spacing would be critical and a larger manhole barrel would have been required. If pipes are located at substantially different elevations, pipes may not conflict and the above analysis is unnecessary.

11.11 Storm Drains

11.11.1 Introduction

After the preliminary locations of inlets, connecting pipes and outfalls with tailwaters have been determined, the next logical 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 gradient 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. For manholes where the pipe size is increased, the downstream crown should be lower than the upstream crown by the amount of the energy loss in the manhole.

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 in the design grows smaller. 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. See Section 11.5.5 for a discussion on time of concentration.

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. The Manning's formula is recommended for capacity calculations. In locations such as depressed roadway sections and underpasses where ponded water can be removed only through the storm drain system, a higher design frequency should be analyzed to ensure the roadway stays open to traffic (see Table 11-2 for design criteria). The main storm drain downstream of the depressed section should be designed by computing the hydraulic grade line and keeping the water surface elevations below the grates and/or established critical elevations for the check storm.

11.11.2 General Guidelines

The following items must be considered during the design of a storm drain system.

- Storm drains shall be designed for "just-full" condition. The head waters in structures shall be limited to 0.3 meters (1 ft) below the top of grate, taking into consideration the possible effect of headwater in the next downstream structure.
- Underdrain pipes of 100 and 150 mm (4 in and 6 in) size should be laid in straight segments or gradual curves if possible. Where bends of underdrain are necessary to enter a structure they should be no greater than 30 degrees.
- Long skew crossings of storm drain laterals under pavement should be avoided.
- All roadway drainage, including the side and slope ditches shall be carried to a suitable outlet, preferably an existing stream. Where outletting to an existing stream is impractical, or where no stream is available, appropriate drainage rights must be obtained.
- The discharge of effluent from sanitary sewers, cesspools, septic tanks, discharge of cooling water or industrial wastes into a State maintained roadway drainage system will not be permitted.
- Private connections to State drainage systems are only allowed after issuance of an encroachment permit accompanied by a special connection agreement.

- Roadway drainage shall not be outletted into existing drainage systems which are privately owned or those maintained by towns or cities except in the case where an independent outlet is not feasible due to excessive cost or other reasons. Where outletting into such a system, an agreement must be entered into with the municipality. A deeded right to drain must be secured from owners of private systems.
- All existing metal pipes to be abandoned under the travelway are to be removed. Concrete pipes to be abandoned should be plugged at the ends.
- State drainage systems shall not be outletted into municipal systems which carry both storm water and sanitary sewage, nor will any such municipal system carrying both storm water and sanitary sewage be outletted into State systems.
- Diversion of watershed area should be avoided if possible. However, in all cases where drainage is diverted from one watershed area to another, as is frequently the case in incised highways, the designer shall note the diversions in the computations and on the preliminary plans to better allow the reviewers and right of way negotiators to make proper provisions for the lawful disposal of the drainage from this area at the outlet locations.
- Utility conflicts may require design changes. New installations should be kept at least 0.3 meters (1 ft) from any utilities.
- The pertinent plans and computations for drainage systems on a project which originate or terminate on an adjacent project shall be furnished for review by the designer of the project being reviewed. The area used for runoff computation shall be shown on topographical maps also to be supplied.
- Each outlet must be carefully designed with erosion protection as needed and carried down steep slopes to lesser slopes where outlet erosion will not occur. Riprap shall be designed at all outlets not flowing over exposed rock or into deep watercourses or ponds. (See Section 11.14.)
- Storm drainage systems will be designed for the watershed which naturally drains to it. In many urban areas the existing drainage systems are inadequate and it is impossible to provide inlet capacity for the overflow, however, the trunk line system should be designed to allow the municipality to upgrade their contributing system at a future date.
- Minimum size pipe for storm drainage is 300 mm (12 in).
- Slotted drain shall be outletted into catch basins.

11.11.3 Outlets

All proposed storm drains have an outlet point where the flow is discharged. The designer should consider at least the following aspects that may affect the hydraulic design of a storm drainage system.

- The flowline elevation of the outfall should be equal to, or higher than the recipient. If this is not the case, excavation may be required to ensure positive gravity flow, or in severe cases pump stations may be required.
- Where practical, the outlet should be positioned in the outfall channel so that it is pointed in a downstream direction. This will reduce turbulence and the potential for erosion.
- When the outlet is located in a manner to allow the discharge to impinge on the opposite bank of a channel, that bank should be evaluated to determine the need for riprap.

11.11.4 Bridge Deck Outlets

The design of deck drain outlets should be such as to prevent the discharge of drainage water against any portion of the structure or on moving traffic below, and to prevent erosion at the outlet of the downspout. Deck drainage may be connected to conduits leading to stormwater outfalls at ground level. Water in a roadway gutter section should be intercepted prior to the bridge. Scuppers are not to be designed with freefall outlets except over water or where it can be demonstrated that the flow will not cause damage or become a nuisance.

Regardless of the outfall type, a design of the pipe, which is usually a vertical connection, must be performed. The depth of water required to convey the flow in the pipe should be maintained below the bottom of the grate. This will allow the flow to be intercepted, in most cases, without causing splashover. The following formula is used to determine the required pipe size. The minimum pipe size shall be 200 mm (8 in).

$$Q_I = 0.6 A (2gH)^{0.5} \quad (11.14)$$

Where:

Q_I = Flow intercepted by inlet and for the pipe design m^3/s (ft^3/s)

0.6 = Orifice coefficient

A = Area of the pipe outletting inlet m^2 (ft^2)

H = Depth of flow over pipe m (ft)

g = acceleration due to gravity, $9.81 m/s^2$ ($32.2 ft/s^2$)

Any debris entering the inlet must pass through the outlet pipe to the disposal point. Therefore, the outlet pipes must be designed to be self-cleaning and fittings that trap debris should be avoided. The following criteria should be followed in the design of pipes for bridge drainage.

- Inlets should be placed near bridge piers - Long runs of pipe to reach the pier will not function well and should be avoided.
- The minimum slope should be the maximum slope achievable, but in no case less than 8%.
- When elbows must be used, use long radius elbows. Use elbows of 45 degrees or less.
- Use smooth walled pipe which is resistant to corrosion and has watertight joint capable of withstanding the internal pressure imposed by backflushing.
- If discharge is free fall under the bridge, the pipes should be carried at least 75 mm (3 in) below the bottom of adjacent girders.
- Cleanouts should be provided at key points, considering the maintenance equipment available to access them.
- Outfall from bridge drainage should be located so that flow will not be interrupted in a manner that will preclude the conveyance of the debris.

11.11.5 Storm Drainage System Design Procedures

The design of storm drainage systems is generally divided into the following operations:

Step 1 Determine inlet location and spacing as outlined earlier in this chapter.

Step 2 Prepare plan layout of the storm drainage system establishing the following design data:

- a. Location of storm drains.
- b. Direction of flow.
- c. Location of manholes.
- d. Location of existing utilities such as water, gas, sanitary sewer, electric, communication facilities, etc.

Step 3 Perform storm drainage computations to determine flows in the drainage system and the required pipe sizes for “just-full conditions” (smallest available pipe size not flowing at full capacity for the design discharge). Use the Storm Drainage Computation Sheet, Table 11-8 or 11-8.1, described below, to facilitate the computational procedure.

- Columns 1 and 2 – Starting at the highest point in the system and proceeding downstream, enter the station and offset of the drainage structures (catch basin, manhole, junction, etc.) for each pipe segment.
- Column 3 – Determine the time of concentration for surface flow (runoff) to reach inlet. If a gutter flow analysis was performed, this information would be taken from the Gutter Flow Analysis form.
- Column 4 – Determine the time required for the design flow to pass through the pipe segment. Calculated by dividing the pipe length (13) by the velocity (19).
- Column 5 – Determine the accumulated time which is the time of concentration effective at the upstream end of the pipe segment. The longest time is to be used. This can be overland flow to an inlet, accumulation of time in pipe or a branch line entering a system.
- Column 6 – Determine the A X C entering the inlet or catch basin or enter information previously determined from the Gutter Flow Analysis.
- Column 7 – Determine the total A X C which is the sum of all the A X C entering the drainage structure and conveyed by the pipe segment.
- Column 8 – Determine the Rainfall Intensity (Chapter 6, Appendix B) based on the Accumulated Time (5).
- Column 9 – Determine the Total Flow in system which is the product of the Total A X C in system (7) and the Rainfall Intensity (8).
- Columns 10, 11, 12, 13, 14, 15 and 16 – Enter the information for the selected pipe: Size; Type; Manning’s Roughness Coefficient - “n” (See Chapter 8, Appendix A for recommended values); Length; Invert Elevation, Upstream; Invert Elevation, Downstream; Slope (expressed to the nearest thousandth).
- Column 17 – Determine the discharge which can be carried by pipe of size and type specified, flowing full.

- Column 18 – Determine the full flow velocity based on the full capacity discharge (17).
- Column 19 – Determine the velocity obtained in pipe of size, type and slope specified, at design discharge (9).

- Step 4 Complete the design by calculating the hydraulic grade line (HGL) as described in Section 11.12. If all HGL elevations meet the 0.3m (1 ft) freeboard from top of grate or rim requirement, then the hydraulic design is adequate. If the HGL exceeds this 0.3m (1 ft) freeboard, then adjustments to the design must be made to lower the water surface elevation and computations revised accordingly.
- Step 5 All computations and design sheets should be clearly identified. The engineer's initials and date of computations should be shown on every sheet. Voided or superseded sheets should be so marked. The origin of data used on one sheet but computed on another should be given.

11.11.6 Check Storm for Sag Point

As indicated above, the storm drain which drains a major sag point should be sized to accommodate the runoff from a check storm frequency rainfall. This can be done by actually computing the bypass occurring at each inlet during a check storm rainfall and accumulating it at the sag point. The inlet at the sag point as well as the storm drain pipe leading from the sag point must be sized to accommodate this additional bypass within the criteria established. In order to design the pipe leading from the sag point, it may be helpful to convert the additional bypass created by the check storm rainfall into an equivalent CA which can be added to the design CA. This equivalent CA can be approximated by dividing the check storm bypass by $0.00278 \times I_{10}$ for metric or I_{10} for English units in the pipe at the low point.

11.11.7 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:

$$V = \frac{1}{n} R^{2/3} S^{1/2} \quad \left(V = \frac{1.49}{n} R^{2/3} S^{1/2} \right) \quad (11.15)$$

Where: V = mean velocity of flow, m/s (ft/s)

n = Manning's roughness coefficient (see Appendix A of Chapter 8, Culverts)

R = hydraulic radius, m (ft) = area of flow divided by the wetted perimeter (A/WP)

S = the slope of the energy grade line, m/m (ft/ft)

In terms of discharge, the above formula becomes:

$$Q = V A = \frac{1}{n} A R^{2/3} S^{1/2} \quad \left(Q = \left(\frac{1.49}{n} \right) A R^{2/3} S^{1/2} \right) \quad (11.16)$$

Where: Q = rate of flow, m³/s (ft³/s)

A = cross sectional area of flow, m² (ft²)

For storm drains flowing full, the above equations become:

Where: D = diameter of pipe, m(ft)

$$V = \frac{0.397}{n} D^{2/3} S^{1/2} \quad Q = \frac{0.312}{n} D^{8/3} S^{1/2} \quad (11.17)$$

$$\left(V = \frac{0.592}{n} D^{2/3} S^{1/2} \right) \quad \left(Q = \frac{0.464}{n} D^{8/3} S^{1/2} \right)$$

The nomograph solution of Manning's formula for full flow in circular storm drains is shown on Figure 11-9, and Figure 11-10. Figure 11-11 has been provided to assist in the solution of the Manning's equation for part full flow in storm drains.

11.11.8 Curved Alignment

Curved storm drains are permitted where necessary. Long radius bend sections are available from many suppliers and are the preferable means of changing direction in pipes 1200 mm (48 in) and larger. Short radius bend sections are also available and can be utilized if there isn't room for the long radius bends. Deflecting the joints to obtain the necessary curvature is not desirable except in very minor curvatures. Utilizing large manholes solely for changing direction may not be cost effective on large size storm drains.

11.11.9 Minimum Grades

All storm drains should be designed such that velocities of flow will not be less than 0.9 m/s (3 ft/s) 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 of 0.9 m/s (3 ft/s) can be calculated by the Manning's formula or use values given in Table 11-7.

$$S = \frac{(nV)^2}{R^{4/3}} \quad (11.18)$$

Table 11-7

**Minimum Slopes Necessary To Ensure 0.9 m/s (3 ft/s)
In Storm Drains Flowing Full**

<u>Pipe Size, mm (in)</u>		<u>Full Pipe, m³/s (ft³/s)</u>		<u>Minimum Slopes m/m (ft/ft)</u>		
				<u>n = 0.012</u>	<u>n = 0.013</u>	<u>n = 0.024</u>
200	(8)	0.030	(1.05)	0.0064	0.0075	0.0256
250	(10)	0.046	(1.64)	0.0048	0.0056	0.0190
300	(12)	0.067	(2.36)	0.0037	0.0044	0.0149
375	(15)	0.104	(3.68)	0.0028	0.0032	0.0111
450	(18)	0.150	(5.30)	0.0022	0.0026	0.0087
525	(21)	0.204	(7.22)	0.0018	0.0021	0.0071
600	(24)	0.267	(9.43)	0.0015	0.0017	0.0059
675	(27)	0.338	(11.93)	0.0013	0.0015	0.0051
750	(30)	0.417	(14.73)	0.0011	0.0013	0.0044
825	(33)	0.505	(17.82)	0.00097	0.0011	0.0039
900	(36)	0.601	(21.21)	0.00086	0.0010	0.0034
1050	(42)	0.817	(28.86)	0.00070	0.00082	0.0028
1200	(48)	1.067	(37.70)	0.00059	0.00069	0.0023
1350	(54)	1.351	(47.71)	0.00050	0.00059	0.0020
1500	(60)	1.668	(58.90)	0.00044	0.00051	0.0017
1650	(66)	2.018	(71.27)	0.00038	0.00045	0.0015
1800	(72)	2.402	(84.82)	0.00034	0.00040	0.0014

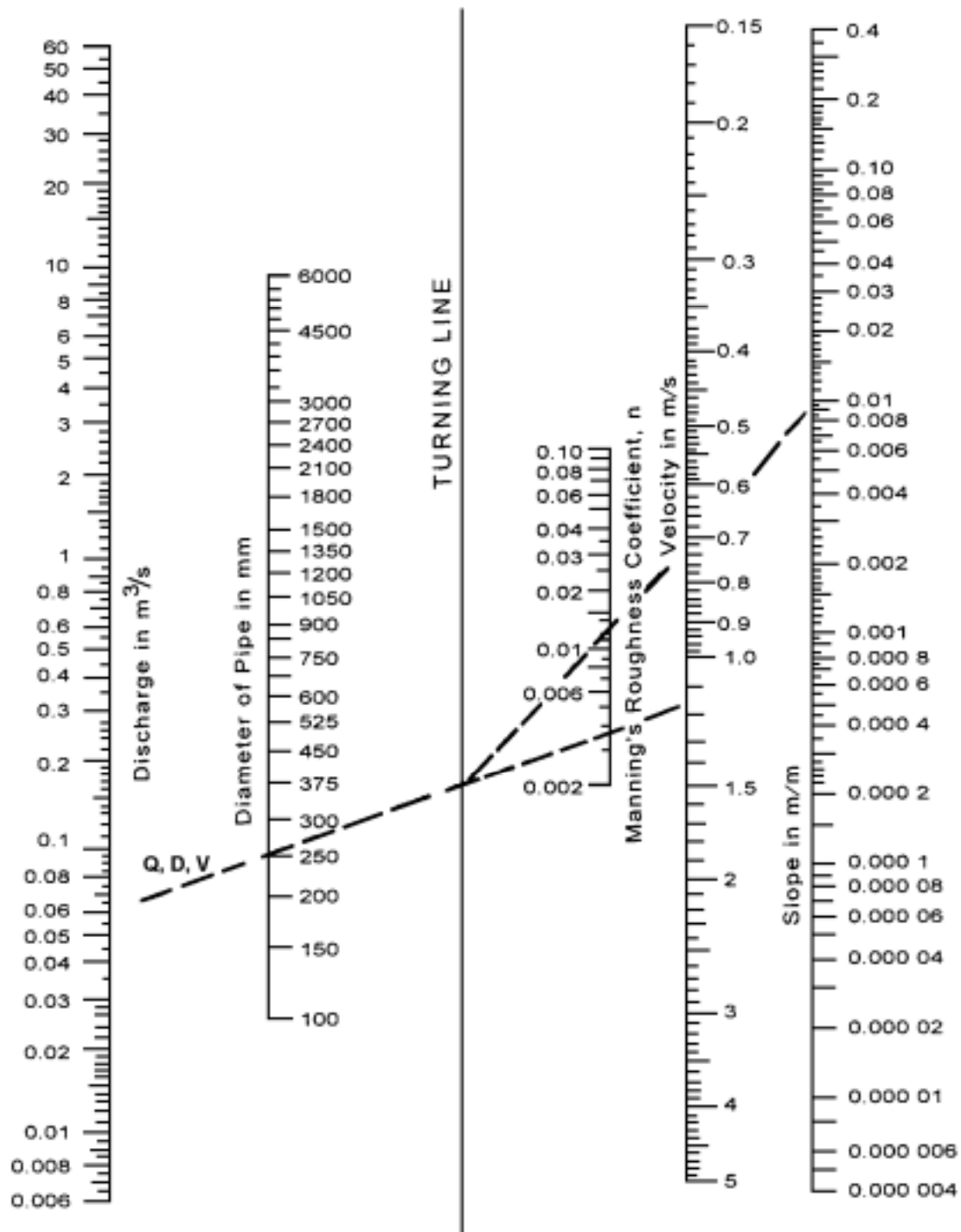


Figure 11-9 Manning's Formula For Flow In Storm Drains – Metric units

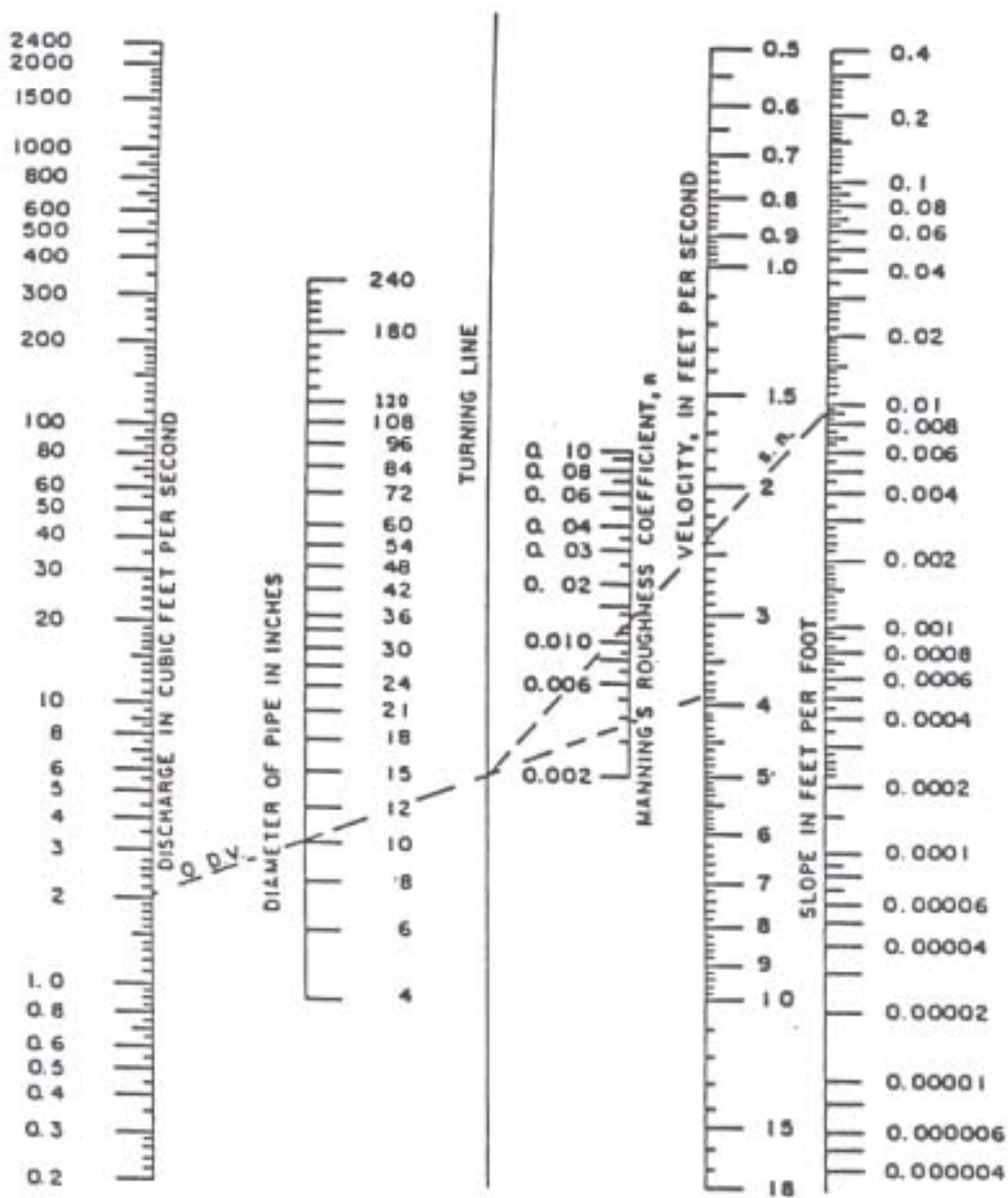


Figure 11-9.1 Manning's Formula For Flow In Storm Drains – English units

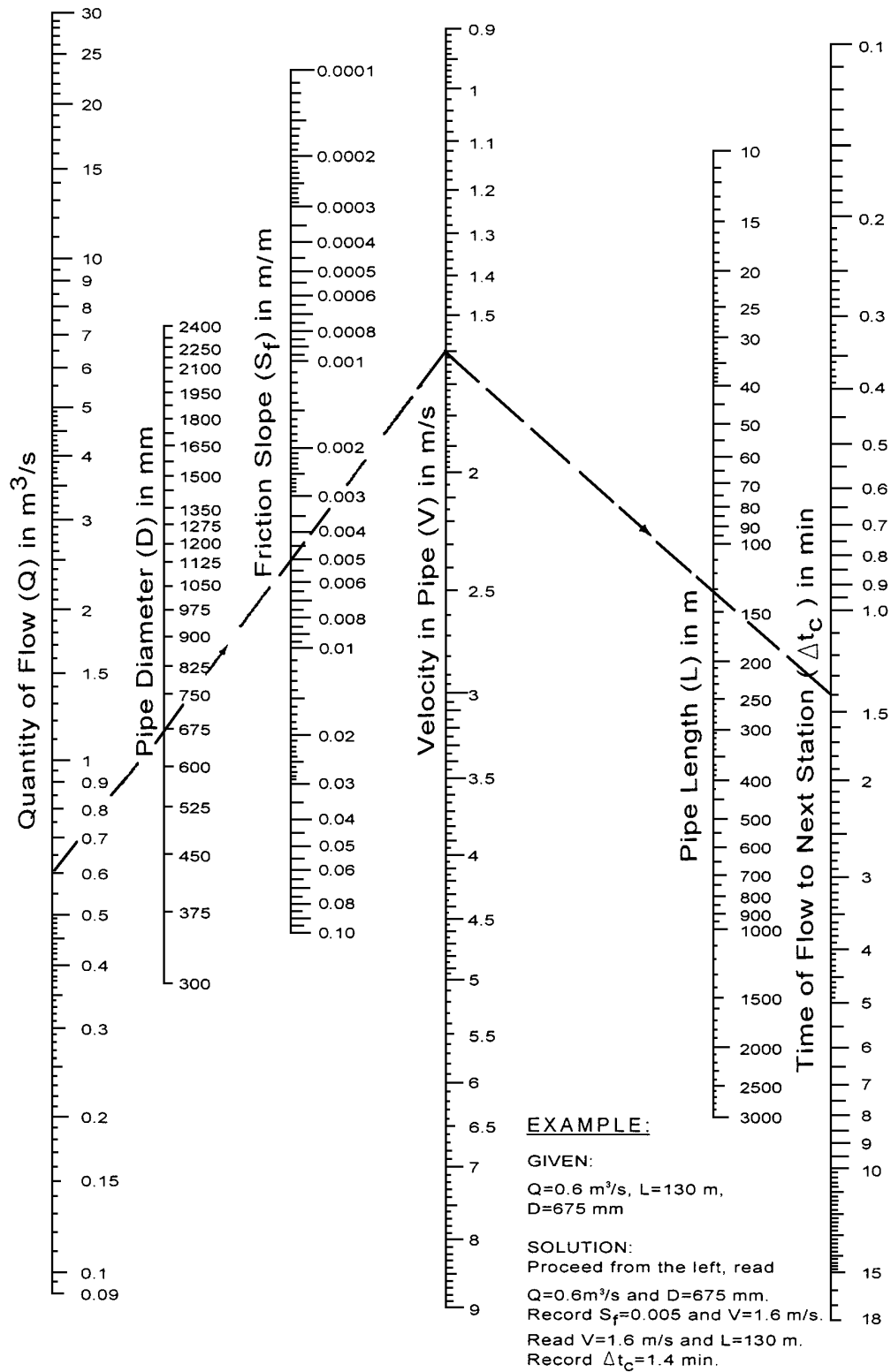


Figure 11-10 Concrete Pipe Flow Nomograph – Metric units

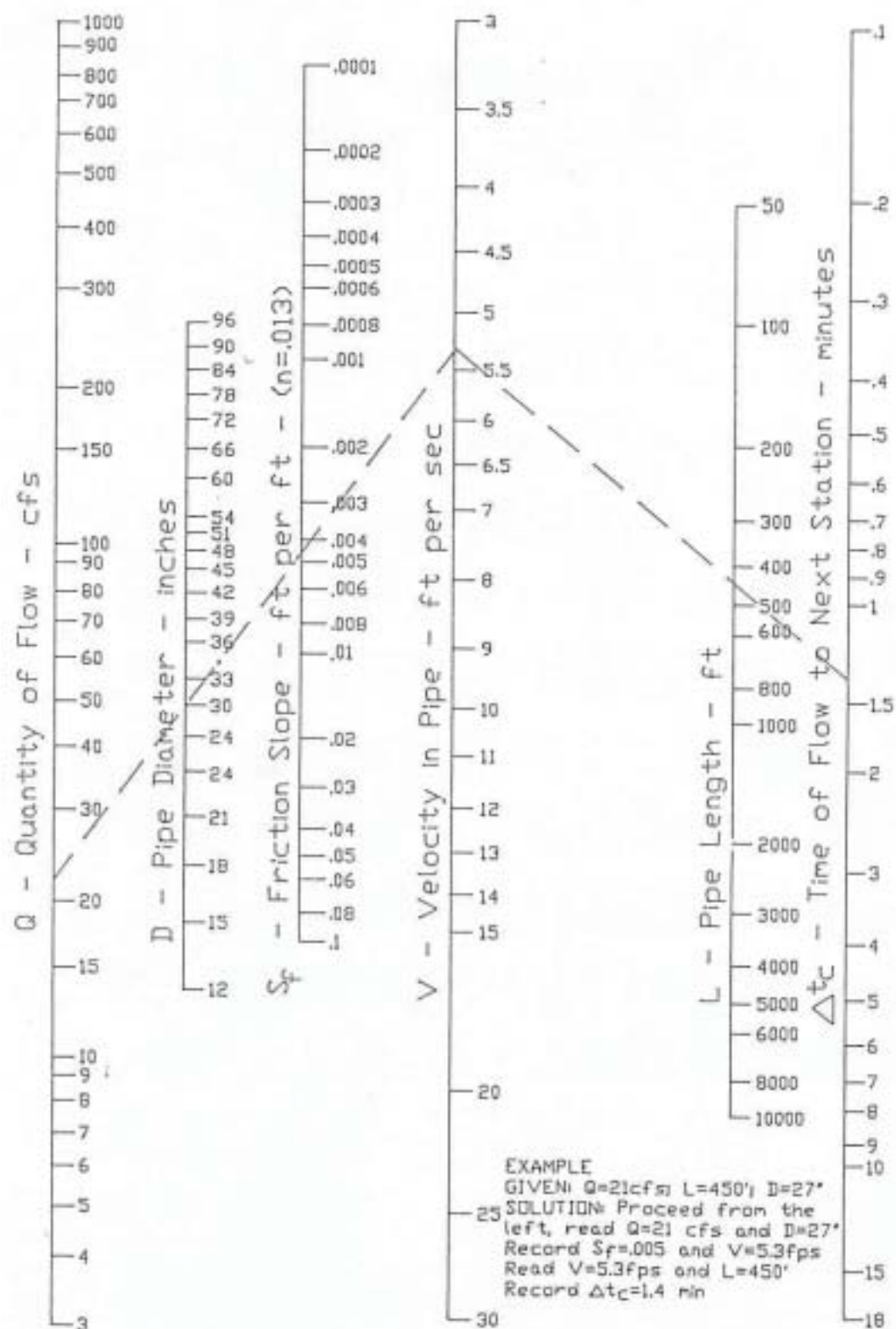


Figure 11-10.1 Concrete Pipe Flow Nomograph – English units

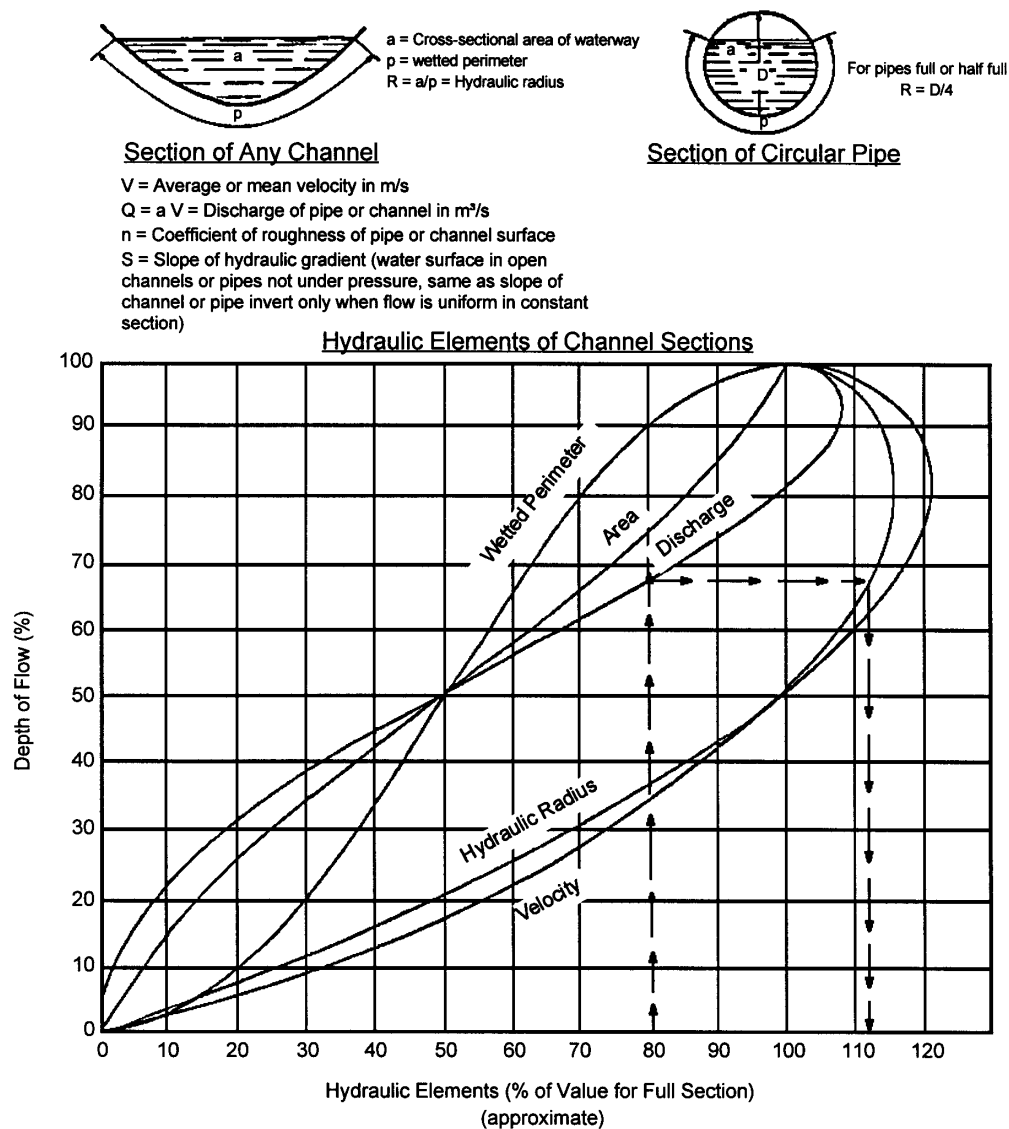


**Figure 11-11 Values Of Hydraulic Elements Of Circular Section
For Various Depths Of Flow – Metric or English units**

Computed _____ Date _____

Checked _____ Date _____

Project _____

Route _____

Station _____

Town _____

[illegible]

December 2003

Computed _____ Date _____

Checked _____ Date _____

Project _____
Route _____
Station _____
Town _____

[illegible]

Table 11-8 Storm Drain Computation Sheet – English units

11.12 Hydraulic Grade Line

11.12.1 Introduction

The hydraulic grade line (HGL) is the last important feature to be established relating to the hydraulic design of storm drains. This gradeline aids the designer in determining the acceptability of the proposed system by establishing the elevations along the system to which the water will rise when the system is operating from a flood of design frequency.

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 manhole lid displacement can occur if the hydraulic grade line rises above the ground surface. 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 this inplace system, then upstream through the proposed system to the upstream inlet. The same considerations apply to the outlet of a storm drain as to the outlet of a culvert. See Figure 8-4 for a sketch of a culvert outlet which depicts the difference between the HGL and the energy grade line (EGL). Usually it is helpful to compute the EGL first, then subtract the velocity head ($V^2/2g$) to obtain the HGL.

11.12.2 Tailwater

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 (d_c). To determine the EGL, begin with the tailwater elevation or $(d_c + D)/2$, whichever is higher, add the velocity head for full flow and proceed upstream to compute all losses such as exit losses, friction losses, junction losses, bend losses and entrance losses as appropriate.

An exception to the above might be a very large outfall with low tailwater when a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the tailwater, whichever is higher.

When estimating tailwater depth on the receiving stream, the prudent designer will 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 basin may not be critical for a larger basin. Also, it may safely be assumed that if the same storm causes peak discharges on both basins, the peaks will be out of phase. To aid in the evaluation of joint probabilities, refer to Section 8.3.6, Table 8-3, Joint Probability Analysis, Chapter 8, Culverts. For example, a main stream (receiving waters) and tributary (storm drain outfall) have a drainage area ratio of 100 to 1 and a 10-year design is required for the storm drain system. Table 8-3 indicates that:

1. When a 10-year storm is applied to the tributary, the highwater of the main stream should be determined for a 5-year storm frequency.
2. A 10-year highwater on the main stream should be applied to a 5-year storm frequency on the tributary.

The analyses should include any additional drainage area that the outfall receives between the storm drain outlet and the receiving waters.

11.12.3 Exit Loss

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

$$H_o = 1.0 \left[\frac{V^2}{2g} - \frac{V_d^2}{2g} \right] \quad (11.19)$$

Where: V = average outlet velocity, m/s (ft/s)

V_d = channel velocity downstream of outlet, m/s (ft/s)

Note that when $V_d = 0$ as in a reservoir, the exit loss is one velocity head. For part full flow where the pipe outlets in a channel with moving water, the exit loss may be reduced to virtually zero.

11.12.4 Bend Loss

The bend loss coefficient for storm drain design is minor but can be evaluated using the formula:

$$h_b = 0.0033 (\Delta) (V_o^2 / 2g) \quad (11.20)$$

Where: Δ = angle of curvature in degrees

11.12.5 Pipe Friction Losses

The friction slope is the energy gradient in m/m (ft/ft) 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:

$$S_f = [Qn/A R^{2/3}]^2 \quad (S_f = [Qn/1.486AR^{2/3}]^2) \quad (11.21)$$

The head losses due to friction may be determined by the formula:

$$H_f = S_f L \quad (11.22)$$

The Manning's equation can also be written to determine friction losses for storm drains as follows:

$$H_f = 6.35 n^2 V^2 L / D^{4/3} \quad (H_f = 2.88 n^2 V^2 L / D^{4/3}) \quad (11.23)$$

$$H_f = \frac{19.62 n^2 L}{R^{4/3}} \left(\frac{V^2}{2g} \right) \quad (H_f = \frac{29 n^2 L}{R^{4/3}} \left[\frac{V^2}{2g} \right]) \quad (11.24)$$

Where: H_F = total head loss due to friction, m (ft)

n = Manning's roughness coefficient (See Appendix A, Chapter 8, Culverts)

D = diameter of pipe, m (ft)

L = length of pipe, m (ft)

V = mean velocity, m/s (ft/s)

R = hydraulic radius, m (ft)

g = 9.81 m/s² (32.2 ft/s²)

S_f = slope of hydraulic grade line, m/m (ft./ft.)

11.12.6 Structure Losses

The head loss encountered in going from one pipe to another through a structure (catch basin, manhole, etc.) is commonly represented as being proportional to the velocity head at the outlet pipe. Using K to signify this constant of proportionality, the energy loss is approximated as $K (V_o^2/2g)$. Experimental studies have determined that the K value can be approximated as follows:

$$* K = K_o C_D C_d C_Q C_p C_B \quad (11.25)$$

Where: K = adjusted loss coefficient

K_o = initial head loss coefficient based on relative manhole size.

C_D = correction factor for pipe diameter (pressure flow only)

C_d = correction factor for flow depth (non-pressure flow only)

C_Q = correction factor for relative flow

C_B = correction factor for benching

C_p = correction factor for plunging flow

* In some cases, the intent of the methodology is to compare the size of one pipe to another pipe (or to the size of the structure). In these cases an equivalent diameter is used, which is computed from the full area of the pipe or structure.

Relative Manhole Size

K_o is estimated as a function of the relative structure size and the angle of deflection between the inflow and outflow pipes.

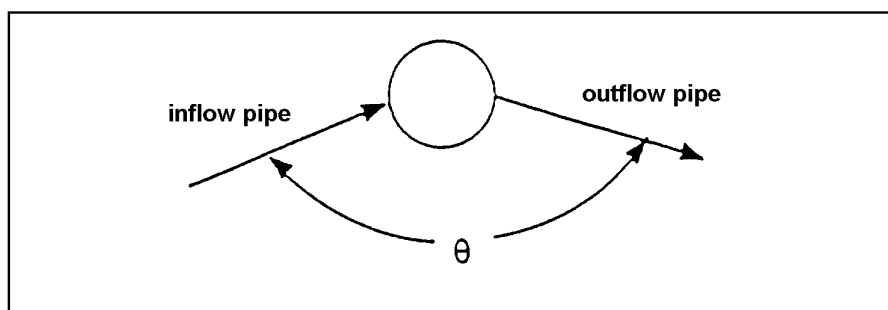
$$K_o = 0.1(b/D_o)(1 - \sin \theta) + 1.4(b/D_o)^{0.15} \sin \theta \quad (11.26)$$

Where: θ = the angle between the inflow and outflow pipes

b = structure diameter, mm (in)

D_o = outlet pipe diameter, mm (in)

Deflection Angle



Pipe Diameter

A change in head loss due to differences in pipe diameter is only significant in pressure flow situations when the depth in the structure to outlet pipe diameter ratio, d/D_o , is greater than 3.2 otherwise C_D is set equal to 1.0. Therefore, it is only applied in such cases.

$$C_D = (D_o / D_i)^3 \quad (11.27)$$

Where: D_i = incoming pipe diameter, mm (in)

D_o = outgoing pipe diameter, mm (in)

Flow Depth

The correction factor for flow depth is significant only in cases of free surface flow or low pressures, when d/D_o ratio is less than 3.2 and is only applied in such cases. In cases where this ratio is greater than 3.2, C_d is set equal to 1.0. Water depth in the manhole is approximated as the level of the hydraulic gradeline at the upstream end of the outlet pipe. The correction factor for flow depth, C_d , is calculated by the following:

$$C_d = 0.5(d/D_o)^{0.6} \quad (11.28)$$

Where: d = water depth in structure above outlet pipe invert, mm (in)

D_o = outlet pipe diameter, mm (in)

Relative Flow

The correction factor for relative flow, C_Q , is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. The correction factor is only applied to situations where there are three or more pipes entering the structure at approximately the same elevation. Otherwise, the value of C_Q is equal to 1.0. It is computed as follows:

$$C_Q = (1 - 2 \sin \theta) \left(1 - \frac{Q_i}{Q_o}\right)^{0.75} + 1 \quad (11.29)$$

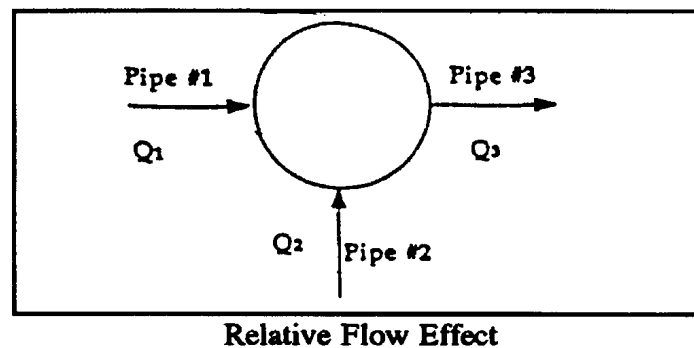
where: C_Q = correction factor for relative flow

θ = the angle between the inflow and outflow pipes (degrees)

Q_i = flow in the inflow pipe, m^3/s (ft^3/s)

Q_o = flow in the outlet pipe, m^3/s (ft^3/s)

As can be seen from the equation, C_Q is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. To illustrate this effect, consider the manhole shown in the Figure and assume the following two cases to determine the impact of pipe 2 entering the manhole.

Case 1:

$Q_1 = 0.09 \text{ m}^3/\text{s}$, $Q_2 = 0.03 \text{ m}^3/\text{s}$,
 $Q_3 = 0.12 \text{ m}^3/\text{s}$ then $C_Q = 1.35$

$$C_{Q3-1} = (1 - 2 \sin 180^\circ) \left(1 - 0.09/0.12\right)^{0.75} + 1 = 1.35$$

Case 2:

$Q_1 = 0.03 \text{ m}^3/\text{s}$, $Q_2 = 0.09 \text{ m}^3/\text{s}$,
 $Q_3 = 0.12 \text{ m}^3/\text{s}$ then $C_Q = 1.81$

$$C_{Q3-1} = (1 - 2 \sin 180^\circ) \left(1 - 0.03/0.12\right)^{0.75} + 1 = 1.81$$

Plunging Flow

The correction factor for plunging flow, C_p , is calculated by the following:

$$C_p = 1 + 0.2 \left[\frac{h}{D_o} \right] \left[\frac{(h-d)}{D_o} \right] \quad (11.30)$$

Where: C_p = correction for plunging flow

h = vertical distance of plunging flow from flow line of higher elevation incoming pipe to the center of outlet pipe, m (ft)

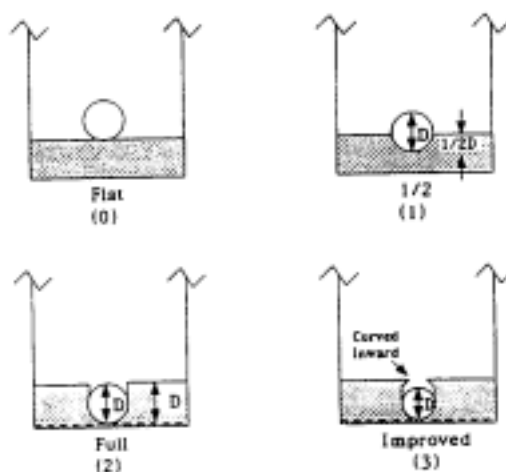
D_o = outlet pipe diameter, m (ft)

d = water depth in structure above outlet pipe invert, m (ft)

This correction factor corresponds to the effect of another inflow pipe or surface flow from an inlet, plunging into the structure, on the inflow pipe for which the head loss is being calculated. Using the notations in the above figure for the example, C_p is calculated for pipe # 1 when pipe # 2 discharges plunging flow. The correction factor is only applied when $h > d$, otherwise, the value of C_p is equal to 1.0. Additionally, the correction factor is only applied when a high elevation flow plunges into the structure that has both an inflow and outflow in the bottom of the structure.

Benching

The correction for benching in the structure, C_B , is obtained from Table 11-9. Benching tends to direct flows through the structure, resulting in reductions in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed.



Schematic Representation Of Benching Types

Table 11-9 Correction for Benching

Bench Type	Correction Factors, C_B	
	Submerged*	Unsubmerged**
Flat floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07
Improved	0.40	0.02
*pressure flow, $d/D_o > 3.2$ **free surface flow, $d/D_o < 1.0$		

Summary

In summary, to estimate the head loss through a structure from the outflow pipe to a particular inflow pipe, multiply the above correction factors together to get the head loss coefficient, K . This coefficient is then multiplied by the velocity head in the outflow pipe to estimate the minor loss for the connection.

11.12.7 Hydraulic Grade Line Design Procedure

The equations and charts necessary to manually calculate the location of the hydraulic gradeline are included in this chapter. A step by step procedure is given to manually compute the HGL. Table 11-10 can be used to document the procedure.

If the HGL is above the pipe crown at the next upstream structure, pressure flow calculations are indicated; if it is below the pipe crown, then open channel flow calculations should be used at the upstream structure. The process is repeated throughout the storm drain system. If all HGL elevations meet the 0.3m (1 ft) freeboard from top of grate requirement, then the hydraulic design is adequate. If the HGL exceeds this 0.3m (1 ft) freeboard, then adjustments to the trial design must be made to lower the water surface elevation.

See Figure 11-12 for a sketch depicting the use of energy losses in developing a storm drain system.

- Step 1 Enter in Col. 1 the station for the junction immediately upstream of the outflow pipe. HGL computations begin at the outfall and are worked upstream taking each junction into consideration.
- Step 2 Enter in Col. 2 the tailwater elevation if the outlet will be submerged during the design storm otherwise refer to the tailwater discussion in 11.12.2 for procedure.
- 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 outlet velocity of flow, V_o .
- Step 7 Enter in Col. 7 the velocity head, $V_o^2/2g$.
- Step 8 Enter in Col. 8 the exit loss, H_o .
- Step 9 Enter in Col. 9 the friction slope (SF_o) in m/m (ft/ft) of the outflow pipe. This can be determined by using the equation 11.21. Note: Assumes full flow conditions.
- Step 10 Enter in Col. 10 the friction loss (H_f) which is computed by multiplying the length (L_o) in Col. 5 by the friction slope (SF_o) in Col 9. On curved alignments, calculate curve losses by using the formula $H_c = 0.0033 (\Delta)(V_o^2/2g)$, where Δ = angle of curvature in degrees, and add to the friction loss.
- Step 11 Enter in Col. 11 the initial head loss coefficient, K_o , based on relative structure size as computed by equation 11.26.
- Step 12 Enter in Col. 12 the correction factor for pipe diameter, C_D , as computed by equation 11.27.
- Step 13 Enter in Col. 13 the correction factor for flow depth, C_d , as computed by equation 11.28. Note this factor is only significant in cases where the d/D_o ratio is less than 3.2.
- Step 14 Enter in Col. 14 the correction factor for relative flow, C_Q , as computed by equation 11.29.
- Step 15 Enter in Col. 15 the correction factor for plunging flow, C_p , as computed by equation 11.30. The correction factor is only applied when $h > d$.
- Step 16 Enter in Col. 16 the correction factor for benching, C_B , as determined in Table 11-9.
- Step 17 Enter in Col. 17 the value of K as computed by equation 11.25.
- Step 18 Enter in Col. 18 the value of the total manhole loss, $K (V_o^2/2g)$.

- Step 19 If the tailwater submerges the outlet end of the pipe, enter in Col. 19 the sum of Col. 2 (TW elevation) and Col. 7 (velocity head) to get the EGL at the outlet end of the pipe. If the pipe is flowing full, but the tailwater is low, the EGL will be determined by adding the velocity head to $(d_c + D)/2$.
- Step 20 Enter in Col. 20 the sum of the friction head (Col 10), the structure losses (Col 18), and the energy grade line (Col 19) at the outlet to obtain the EGL at the inlet end. This value becomes the EGL for the downstream end of the upstream pipe.
- Step 21 Determine the HGL (Col 21) throughout the system by subtracting the velocity head (Col 7) from the EGL (Col 20).
- Step 22 Check to make certain that the HGL is below the level of allowable high water at that point. If the HGL is above the finished grade elevation, water will exit the system at this point for the design flow.

The above procedure applies to pipes that are flowing full, as should be the condition for design of new systems. If a part full flow condition exists, the EGL is located one velocity head above the water surface.

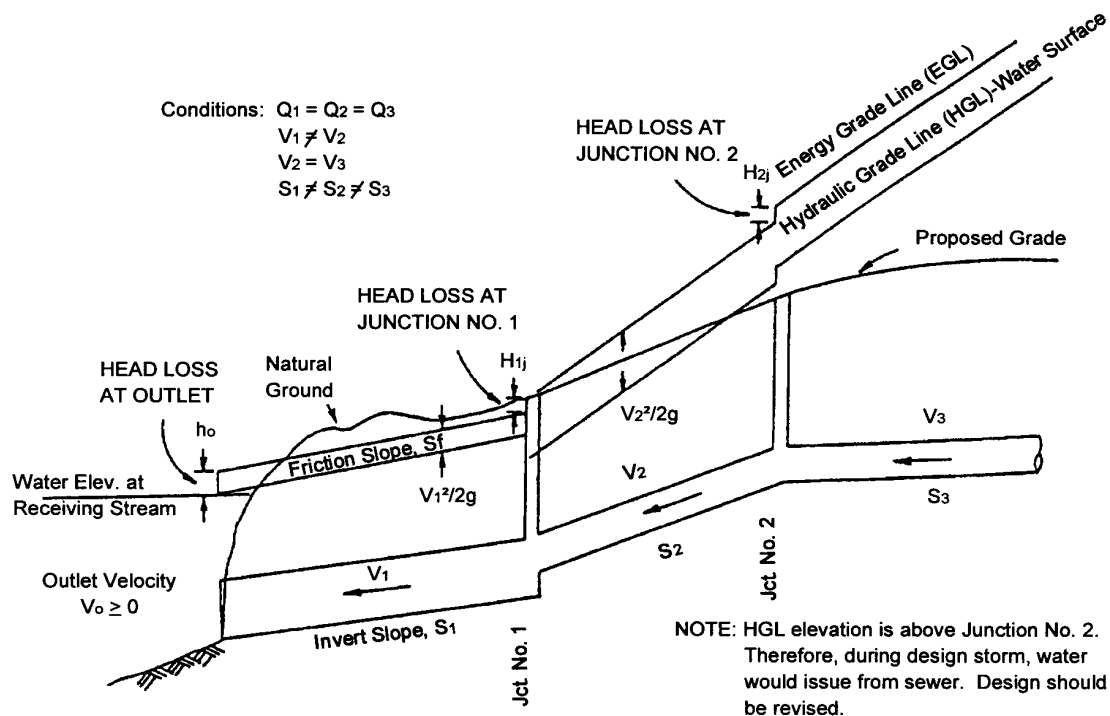
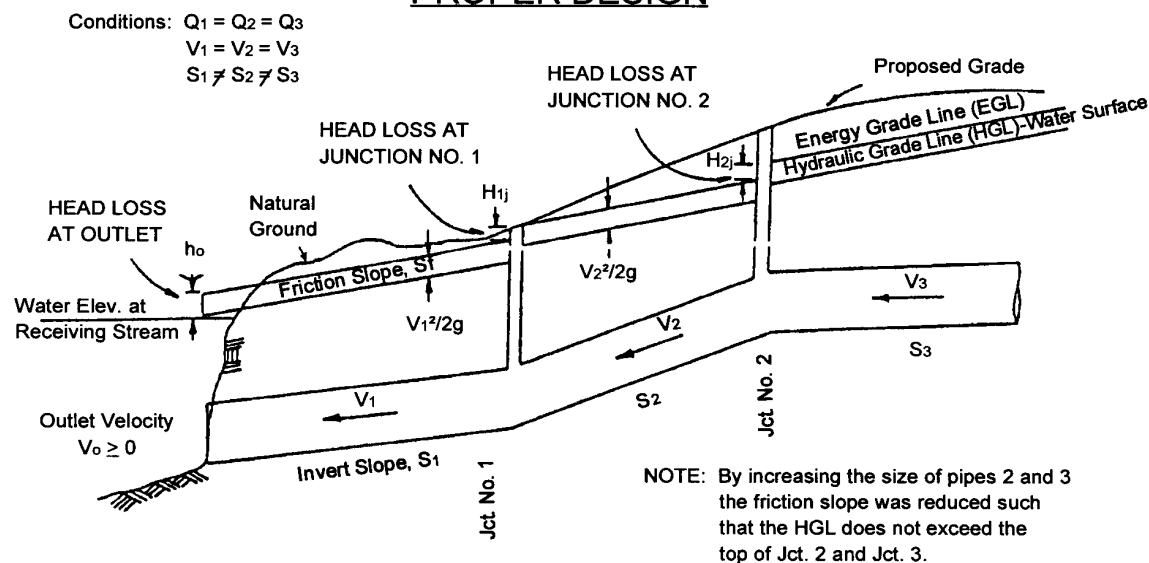
IMPROPER DESIGN**PROPER DESIGN**

Figure 11-12 Use Of Energy Losses In Developing a Storm Drain System

Computed by _____ Date _____

Checked by _____ Date _____

Page _____ of _____

Project No. _____

Route _____

Town _____

Stations _____

Station	TW	D ₀	Q ₀	L ₀	V ₀	$V_0^2/2g$	H ₀	SF ₀	H _f	K ₀	C _D	C _d	C _O	C _p	C _B	K	$K(V_0^2/2g)$	EGL ₀ 2+7	EGL _i 10+18+19	HGL EGL-7	TOC Elev
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)

Table 11-10 Hydraulic Grade Line Computation Form

11.13 Outlet Protection

11.13.1 Assessment of Erosion Potential

A field investigation of all proposed outlet locations or existing outlets to be used in a drainage design of a proposed project should be conducted to determine the erosion resistance of the soils at the outlet, the character of the downstream flow path, and any other site constraints that must be addressed by the proposed design.

Barring any unusual conditions, as determined during the field investigation, the criteria outlined in this section should be used to determine the level of outlet protection required. When severe conditions are present, it is the responsibility of the designer to provide outlet protection as needed to safeguard against erosion damage.

Pipe outlets are points of critical erosion potential. Stormwater which is transported through closed conveyance systems at design capacity generally reaches a velocity which exceeds the permissible or erosion resistant velocity of the receiving channel or overland area. To prevent scour at stormwater system outlets, a flow transition structure is needed which will absorb the initial impact of the flow and reduce the flow velocity to a level which will not erode the receiving channel or overland area.

11.13.2 Types of Outlet Protection

The most commonly used device for outlet protection is a riprap lined apron. Where practical, they are constructed at a zero grade or minimum slope to slow the outlet velocity. The type and length of the riprap lined apron is related to the outlet flow rate and the tailwater level and whether there is a defined channel downstream.

If the tailwater depth is less than half the outlet pipe rise, it shall be classified as a **Minimum Tailwater Condition**. If the tailwater depth is greater than or equal to half the outlet pipe rise, it shall be classified as a **Maximum Tailwater Condition**.

There are three types of riprap aprons to be used for outlet protection. They are designated as Type A, B and C. Type A riprap aprons would be used under minimum tailwater conditions while Type B riprap aprons would be used for maximum tailwater conditions as defined above, where the pipe outlets overland with no defined channel. Type C riprap aprons would be used when there is a well defined channel downstream of the outlet. The use of a Type C riprap apron on channels that are designated as watercourses or wetlands is discouraged due to potential wetland and fisheries impacts. See Section 11.13.3, Design Criteria, and Section 11.13.5 for the design of riprap aprons.

Where the flow rate proves to be excessive for the economical or practical use of an apron, preformed scour holes may be used. There are two types of preformed scour holes. Type 1 preformed scour holes are depressed one-half the pipe rise and Type 2 preformed scour holes are depressed the full pipe rise. See Section 11.13.3, Design Criteria and Section 11.13.6 for the design of preformed scour holes.

In most cases, a riprap apron or preformed scour hole will provide adequate outlet protection, however where design and site conditions warrant, structurally lined outlet protection or energy dissipators can be investigated. In such instances, coordination with the Hydraulics and Drainage Section early in the design phase is recommended. The design of energy dissipators is presented in HEC-14, "Hydraulic Design of Energy Dissipators For Culverts and Channels."

11.13.3 Design Criteria

The design of riprap outlet protection applies to the immediate area or reach downstream of the pipe outlet and does not apply to continuous rock linings of channels or streams. For pipe outlets at the top of exit slopes or on slopes greater than 10%, the designer should assure that suitable safeguards are provided beyond the limits of the localized outlet protection to counter the highly erosive velocities caused by the reconcentration of flow beyond the initial riprap apron. Outlet protection shall be designed according to the following criteria:

- Riprap outlet protection shall be used at all outlets not flowing over exposed rock or into deep watercourses and ponds.
- In situations not covered by the above noted criteria and where the exit velocity is ≤ 4.27 mps (14 fps), a riprap apron shall also be used. For Type A and B riprap aprons, the type of riprap specified is dependent on the outlet velocity (see Section 11.13.6) and can be determined from Table 11.5. For Type C aprons, the type of riprap specified is determined by the procedures in HEC-15 and HEC-11 depending on the design discharge. See Chapter 7, Channels.
- The type of riprap apron and dimensions are determined by the guidelines outlined in Sections 11.13.2 and 11.13.5, respectively.
- When the outlet velocity is > 4.27 mps (14 fps), the designer should first investigate methods to reduce the outlet velocity. This may be accomplished by any one or combination of the following: increasing the pipe roughness, increasing the pipe size and/or decreasing the culvert slope. When this is not possible or economical, a number of outlet protection or energy dissipation design options are available. These are presented in detail in HEC-14. In most instances, however, a preformed scour hole design should be used, as it generally can provide the necessary degree of protection at an economical cost. The design of a preformed scour hole is presented in Section 11.13.6.

The design criteria of this section should be applicable to most outlet situations. However, recognizing that design and site conditions can vary significantly depending on the project or location on a particular project, it is the responsibility of the designer to ensure that the criteria is suitable to the site or to provide an alternate design which will adequately protect the outlet area from scour and erosion. These situations should be documented in the drainage design report.

Table 11.11 Allowable Outlet Velocities for Type A and B Riprap Aprons

Outlet Velocity - mps (fps)	Riprap Specification
0-2.44 (0-8)	Modified
2.44-3.05 (8-10)	Intermediate
3.05-4.27 (10-14)	Standard

11.13.4 Tailwater Depth

The depth of tailwater immediately at the pipe outlet is required for the design of outlet protection and must be determined for the design flow rate. Manning's equation may be used to determine tailwater depth. See Sections 8.3.5 and 8.3.6 for additional information on how to determine the tailwater depth.

11.13.5 Apron Dimensions

Length

The length of an apron (L_a) is determined using the following empirical relationships (Equations 11.9 and 11.10) that were developed for the U.S. Environmental Protection Agency (1976) and modified by ConnDOT for use in Connecticut. Tables 11-12 and 11-13 show the various lengths of Type A, B and C riprap aprons based on discharge and pipe size. The tables also show the minimum and maximum lengths of aprons to be computed using Equations 11.31 and 11.32. When the table indicates that the required apron length would exceed the maximum shown, a preformed scour hole should be used in lieu of the riprap apron. As previously stated, the design of a preformed scour hole is presented in Section 11.13.6.

Type A Riprap Apron (Minimum Tailwater Condition) $TW < 0.5 R_p$

$$L_a = \frac{3.26(Q - 0.142)}{S_p^{1.5}} + 3.05 \quad \left(L_a = \frac{1.80(Q - 5)}{S_p^{1.5}} + 10 \right) \quad (11.31)$$

Type B Apron (Maximum Tailwater Condition) $TW \geq 0.5 R_p$

$$L_a = \frac{5.44(Q - 0.142)}{S_p^{1.5}} + 3.05 \quad \left(L_a = \frac{3.0(Q - 5)}{S_p^{1.5}} + 10 \right) \quad (11.32)$$

Type C Riprap Apron - The length of a Type C Riprap Apron shall be determined using the formula for a Type B Riprap Apron.

L_a = length of apron, m (ft)

S_p = inside diameter for circular sections or maximum inside pipe span for non-circular sections, m (ft)

Q = pipe (design) discharge, cms (cfs)

TW = tailwater depth, m (ft)

R_p = maximum inside pipe rise, m (ft)

Note: $S_p = R_p$ = inside diameter for circular sections

Width

For Type A or B Riprap Aprons, when there is no well defined channel downstream of the apron, the width of the apron at the pipe outlet, W_1 , should be at least three times the maximum inside pipe span and the width, W_2 of the outlet end of the apron, as shown in Figure 11-13, should be as follows:

Type A Riprap Apron (Minimum Tailwater Condition)

$$W_1 = 3S_p \text{ (min.)}$$

$$W_2 = 3S_p + 0.7L_a \quad \text{for } TW < 0.5 R_p \quad (11.33)$$

and

Type B Riprap Apron (Maximum Tailwater Condition)

$$\begin{aligned} W_1 &= 3S_p \text{ (min.)} \\ W_2 &= 3S_p + 0.4L_a \quad \text{for } TW \geq 0.5 R_p \end{aligned} \quad (11.34)$$

W_1 = width of apron at pipe outlet or upstream apron limit

W_2 = width of apron at terminus or downstream apron limit

Type C Riprap Apron

For a Type C Riprap Apron when there is a well defined channel downstream of the outlet, the bottom width of the apron should be at least equal to the bottom width of the channel and the lining should extend on the channel side slopes at least 0.3m (1 ft) above the tailwater depth (TW) or at least two-thirds of the vertical conduit dimension ($0.7 R_p$) above the invert, whichever is greater. (In all cases, the overall width of the apron shall be a minimum of $3S_p$). See Figure 11-13.

Additional guidelines:

- The type of apron to be used and length should be called out on the construction plans.
- The side slopes of the Type C riprap apron should be 2H:1V or flatter.
- The bottom grade should be level or minimum slope, where practical, for energy dissipation. Where the use of a flat apron is impractical, a preformed scour hole should be considered.
- Granular fill shall be placed between the riprap and the underlying soil to prevent soil movement into and through the riprap. Additionally, an appropriately sized geotextile (separation) can be used when field conditions dictate as determined by the engineer.
- The location of outlets and outlet protection should be carefully considered to minimize rights-of-way and wetland impacts.

11.13.6 Preformed Scour Hole

The preformed scour hole is an excavated hole or depression which is lined with rock riprap of a stable size to prevent scouring. The depression (F) provides both vertical and lateral expansion downstream of the culvert outlet to permit dissipation of excessive energy and turbulence. Equations 11.35 and 11.36 are used to determine the median stone size (d_{50}) required for the lining of the two types of preformed scour holes presented below. The first type, Type 1, represented by Equation 11.35, is depressed one-half the pipe rise and the second type, Type 2, represented by Equation 11.36, is depressed the full pipe rise. A significant reduction in stone size is achieved by the excavation. Therefore, the scour hole depressed the full pipe rise would require a smaller stone size, however the dimensions of the hole would be larger. The type that provides the most economical and practical design given the site conditions should be selected. The dimensions of a preformed scour hole are determined by the set of Equations 11.37 and Figure 11-15.

Empirical Preformed Scour Hole Equations:

Type 1: Scour Hole Depression = one-half pipe rise, m (ft)

$$d_{50} = (0.0276 R_p^2 / TW) (Q/R_p^{2.5})^{1.333} \quad (d_{50} = (0.0125 R_p^2 / TW) (Q/R_p^{2.5})^{1.333}) \quad (11.35)$$

Type 2: Scour Hole Depression = full pipe rise, m (ft)

$$d_{50} = (0.0181 R_p^2 / TW) (Q/R_p^{2.5})^{1.333} \quad (d_{50} = (0.0082 R_p^2 / TW) (Q/R_p^{2.5})^{1.333}) \quad (11.36)$$

d_{50} = median stone size required, m (ft)

For variables S_p , R_p , TW and Q , see Section 11.13.5.

Type 1 and 2 preformed scour hole dimensions (See Figure 11-15)

$$\begin{aligned} C &= 3S_p + 6F && \text{Basin Length m (ft)} \\ B &= 2S_p + 6F && \text{Basin Inlet and Outlet Width m (ft)} \\ F &= 0.5R_p \text{ (Type 1) or } R_p \text{ (Type 2)} && \text{Basin Depression m (ft)} \end{aligned} \quad (11.37)$$

Table 11-14 solves the above set of equations for Type 1 and 2 preformed scour holes for various pipe sizes.

The type of riprap required is as follows:

Modified	$d_{50} < 0.13\text{m (0.42 ft)}$
Intermediate	$0.13\text{m (0.42 ft)} < d_{50} < 0.20\text{m (0.67 ft)}$
Standard	$0.20\text{m (0.67 ft)} < d_{50} < 0.38\text{m (1.25 ft)}$
Special Design	$0.38\text{m (1.25 ft)} < d_{50}$

Reference: Report No. FHWA-RD-75-508 ("Culvert Outlet Protection Design: Computer Program Documentation")

OUTLET PROTECTION - OUTLET VELOCITY $\leq$ 4.27 meters/sec

DISCHARGE (cms)	OUTLET PIPE DIAMETER OR SPAN (mm)									
	300	375	450	600	750	900	1050	1200	1350	1500
0-0.142	3.0	3.0		USE						
0.170	3.6	3.4								
0.180		3.6	3.5							
0.190		3.7	3.6			MINIMUM				
0.210		4.0	3.8	3.5						
0.250		4.5	4.2	3.8						
0.275			4.5	4.0						
0.300			4.7	4.1				LENGTH		
0.325			5.0	4.3						
0.340				4.4	4.0					
0.350				4.5	4.1					
0.400		USE		4.8	4.3	4.0			OUTLINED	
0.450				5.2	4.6	4.2	4.0			
0.500				5.5	4.8	4.4	4.1			
0.550					5.0	4.6	4.3	4.0		
0.600					5.3	4.8	4.4	4.2		
0.650					5.5	4.9	4.6	4.3		
0.800			PREFORMED			5.5	5.0	4.6		
0.940						6.0	5.4	5.0		
1.000							5.6	5.1		
1.100							5.9	5.4	5.0	
1.250							6.3	5.7	5.3	5.0
1.300							6.5	5.9	5.4	5.1
1.500					SCOUR			6.3	5.8	5.4
1.700								6.8	6.2	5.7
1.900								7.3	6.6	6.1
2.200								8.0	7.2	6.6
2.500									7.8	7.1
2.850									8.5	7.7
3.250							HOLE			8.4
3.600										9.0

Table 11-12 - Length - L_a (meters)

Type A Riprap Apron

Notes: 1. Bold face outlined boxes indicate minimum L_a to be used for a given pipe diameter or span.
 2. Rounding and interpolating are acceptable.

OUTLET PROTECTION - OUTLET VELOCITY ≤ 14 feet/sec

DISCHARGE (cfs)	OUTLET PIPE DIAMETER OR SPAN (in)									
	12	15	18	24	30	36	42	48	54	60
0-5	10	10		USE						
6	12	11								
7		13	12							
8		14	13	12		MINIMUM				
9			14	13						
10			15	13						
11			16	14				LENGTH		
12				14						
14				16	14					
16				17	15	14			OUTLINED	
18				18	16	15				
20					17	15	14			
22		USE			18	16	15			
24						17	15	14		
26						17	16	15		
28						18	16	15		
30						19	17	16		
35						20	18	17	16	
40			PREFORMED				20	18	17	16
45							21	19	18	16
50							22	20	18	17
55								21	19	18
60								22	20	19
65								24	21	20
70					SCOUR			25	22	20
75								26	23	21
80									24	22
90									26	24
100									28	25
110										27
125							HOLE			29
130										30

Table 11-12.1 - Length - L_a (feet)

Type A Riprap Apron

Notes: 1. Bold face outlined boxes indicate minimum L_a to be used for a given pipe diameter or span.
 2. Rounding and interpolating are acceptable.

OUTLET PROTECTION - OUTLET VELOCITY ≤ 4.27 meters/sec

DISCHARGE (cms)	OUTLET PIPE DIAMETER OR SPAN (mm)									
	300	375	450	600	750	900	1050	1200	1350	1500
0-0.142	3.0	3.0			USE					
0.170	4.0	3.7	3.5							
0.180		3.9	3.7	3.5		MINIMUM				
0.190		4.2	3.9	3.6						
0.200		4.4	4.1	3.7	3.5					
0.205		4.5	4.2	3.8	3.6					
0.227			4.5	4.0	3.7			LENGTH		
0.250			5.0	4.3	3.9					
0.275				4.6	4.1					
0.300				4.9	4.3	4.0				
0.320				5.1	4.5	4.2			OUTLINED	
0.340				5.3	4.7	4.3	4.0			
0.360		USE		5.5	4.8	4.4	4.1			
0.380					5.0	4.5	4.2	4.0		
0.410					5.2	4.7	4.4	4.1		
0.440					5.5	4.9	4.5	4.3		
0.500						5.3	4.8	4.5		
0.560						5.7	5.1	4.7		
0.620			PREFORMED			6.0	5.4	5.0		
0.660							5.6	5.1		
0.730							6.0	5.4	5.0	
0.800							6.3	5.7	5.3	5.0
0.850							6.5	5.9	5.4	5.1
1.000								6.5	6.0	5.5
1.120					SCOUR			7.0	6.4	5.9
1.250								7.5	6.8	6.3
1.370								8.0	7.2	6.6
1.500									7.6	7.0
1.630									8.1	7.4
1.750									8.5	7.7
1.975							HOLE			8.4
2.200										9.0

Table 11-13 - Length - L_a (meters)

Type B or C Riprap Apron

Notes: 1. Bold face outlined boxes indicate minimum L_a to be used for a given pipe diameter or span.
 2. Rounding and interpolating are acceptable.

OUTLET PROTECTION - OUTLET VELOCITY ≤ 14 feet/sec

DISCHARGE (cfs)	OUTLET PIPE DIAMETER OR SPAN (in)									
	12	15	18	24	30	36	42	48	54	60
0-5	10	10		USE						
5.5	12	11								
6		12	12			MINIMUM				
7		14	13	12						
8			15	13						
8.5			16	14				LENGTH		
9				14						
10				15	14					
11				16	15					
12				17	15	14			OUTLINED	
13				18	16	15				
14					17	15	14			
16		USE			18	16	15	14		
18						18	16	15		
20						19	17	16		
22						20	18	16		
24							19	17	16	
26							20	18	17	16
28			PREFORMED				21	19	17	16
30							21	19	18	17
32							22	20	18	17
35								21	19	18
40								23	21	19
45								25	23	21
48					SCOUR			26	24	22
50									24	22
55									26	23
60									27	25
63									28	26
65										26
75							HOLE			29
80										30

Table 11-13.1 - Length - L_a (feet)

Type B or C Riprap Apron

Notes: 1. Bold face outlined boxes indicate minimum L_a to be used for a given pipe diameter or span.
 2. Rounding and interpolating are acceptable.

OUTLET PROTECTION

**OUTLET VELOCITY > 4.27 meters/sec or Length of Apron exceeds limits shown on
Tables 11-12 and 11-13**

Preformed Scour Hole										
(See Figure 11-15)	PIPE DIAMETER OR SPAN (mm)									
	300	375	450	600	750	900	1050	1200	1350	1500
Type 1										
B	1.5	1.9	2.3	3.0	3.8	4.6	5.3	6.1	6.9	7.6
C	1.8	2.3	2.7	3.7	4.6	5.5	6.4	7.3	8.2	9.1
d	Depends on riprap type (see Figure 11-15)									
2S_p	0.6	0.8	1.0	1.2	1.6	1.8	2.2	2.4	2.8	3.0
F = 0.5 S_p	0.2	0.2	0.2	0.3	0.4	0.5	0.5	0.6	0.7	0.8
3S_p	0.9	1.2	1.5	1.8	2.4	2.7	3.3	3.6	4.2	4.5
Type 2										
B	2.4	3.0	3.7	4.9	6.1	7.3	8.5	9.8	11.0	12.2
C	2.7	3.4	4.1	5.5	6.9	8.2	9.6	11.0	12.3	13.7
d	Depends on riprap type (see Figure 11-15)									
2S_p	0.6	0.8	1.0	1.2	1.6	1.8	2.2	2.4	2.8	3.0
F = S_p	0.3	0.4	0.5	0.6	0.8	0.9	1.1	1.2	1.4	1.5
3S_p	0.9	1.2	1.5	1.8	2.4	2.7	3.3	3.6	4.2	4.5

Table 11-14 - Dimensions of Preformed Scour Hole (Meters)

OUTLET PROTECTION
OUTLET VELOCITY > 14 feet/sec or Length of Apron exceeds limits shown on
Tables 11-12.1 and 11-13.1

Preformed Scour Hole										
(See Figure 11-15)	PIPE DIAMETER OR SPAN (in)									
	12	15	18	24	30	36	42	48	54	60
Type 1										
B	5	6	8	10	13	15	18	20	23	25
C	6	8	9	12	15	18	21	24	27	30
d	Depends on riprap type(see Figure 11-15)									
2S_p	2.0	2.6	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
3S_p	3.0	3.9	4.5	6.0	7.5	9.0	10.5	12.0	13.5	15.0
F = 0.5 S_p	0.5	0.625	0.75	1	1.25	1.5	1.75	2	2.25	2.5
Type 2										
B	8	10	12	16	20	24	28	32	36	40
C	9	11	14	18	23	27	32	36	41	45
d	Depends on riprap size (see Figure 11-15)									
2S_p	2.0	2.6	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
3S_p	3.0	3.9	4.5	6.0	7.5	9.0	10.5	12.0	13.5	15.0
F = S_p	1.0	1.3	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0

Table 11-14.1 - Dimensions of Preformed Scour Hole (Feet)

11.13.7 Design Procedure for Riprap Outlet Protection

Outlet protection consists of the construction of an erosion resistant section between a conduit outlet and a stable downstream channel. Erosion at an outlet is chiefly a function of soil type and the velocity of the conduit discharge. Therefore, in order to mitigate erosion, an adequate design must stabilize the area at the conduit outlet and reduce the outlet velocity to a velocity consistent with a stable condition in the downstream channel.

This section presents a generalized procedure for the design of riprap outlet protection. Although each project will be unique, the design outlined below will normally be applicable.

Step 1. Assess the Erosion Potential at the Outlet and other Critical Site Factors

For all proposed outlet locations including existing outlet locations to be used on the project:

- A. A field investigation should be conducted to determine the erosion resistance of the soils at the outlet, the character of the downstream flow path, and any other site constraints that must be addressed by the proposed design.
- B. Prepare a site description and a sketch (channel cross section, where appropriate) for the outlet location.
- C. Ensure that field survey limits extend far enough to adequately show the proposed outlet protection design, downstream flow path, drainage right-of-way and any other important topographic features on the design plans

Step 2. Determine Tailwater Conditions at the Outlet

- A. See Section 11.13.4 and Sections 8.3.5 and 8.3.6 for further information on how to determine the tailwater depth.
- B. If the pipe outlet discharges into a well-defined channel, estimate the existing velocity in the receiving channel using Manning's Equation (Equation 7.6, Section 7.4.11). See Section 8.3.8 regarding Maximum Velocity.

Step 3. Calculate the Outlet Velocity for the Design Discharge

Culvert outlet velocity is one of the primary indicators of erosion potential and will serve in most instances to define the outlet protection required.

The continuity equation $Q=AV$ (Equation 7.5, Section 7.4.11) can be utilized in all situations to compute the average velocity at any point within a conduit. For conduits flowing partly full, however, the location of the water surface and consequently the area of flow cannot always be easily determined.

The following procedure for the calculation of outlet velocity will produce results, which, though approximate, will be adequate for most design purposes.

- A. Determine the design discharge for the conduit based on the design return frequency.
- B. See Step 2 A. for the tailwater (TW) acting at the outlet pipe.
- C. Calculate the outlet velocity.

Step 4. Evaluate the Outlet Velocity

If the outlet velocity is considered excessive for site conditions or exceeds 4.27 mps (14 fps), the designer should investigate methods to reduce the outlet velocity. These may include any one or combination of the following:

- increasing the pipe roughness
- increasing the pipe size
- decreasing the culvert slope

It should also be noted that the above methods may be employed at velocities less than 4.27 mps (14 fps) when it desired to reduce the size of riprap required at the outlet.

For instance, a 450-mm (18-inch) pipe has a design discharge of 0.3 cms (10 cfs) and an outlet velocity of 3.66 mps (12 fps). Table 11.11 indicates that standard riprap would be required at the outlet, however, it may be more practical to employ the above methods for reducing the exit velocity, so that modified or intermediate riprap can be used in lieu of standard riprap.

Step 5. Select an Appropriate Type of Outlet Protection Design

Review Section 11.13.2 describing the Types of Outlet Protection and the Design Criteria in Section 11.13.3, which will be used in the selection of the type and size of the outlet protection. The type of outlet protection and design criteria presented in these Sections are summarized below:

TYPE	OUTLET VELOCITY mps (fps)	TAILWATER DEPTH	COMMENT
Type A Riprap Apron	≤ 4.27 (14)	$\leq \frac{1}{2}$ pipe rise (minimum condition)	Outlet has <u>no</u> well-defined channel downstream
Type B Riprap Apron	≤ 4.27 (14)	$\geq \frac{1}{2}$ pipe rise (maximum condition)	Outlet has <u>no</u> well-defined channel downstream
Type C Riprap Apron	≤ 4.27 (14)	all	Outlet has a well-defined channel downstream
Preformed Scour Hole	≥ 4.27 (14)	all	May be used for lower exit velocities as dictated by Tables 8-6 and 8-7
Structurally Lined Energy Dissipaters	≥ 4.27 (14)	all	See HEC-14 To be used only with prior approval from Hydraulics and Drainage Section.

Table 11-15 Summary of Outlet Protection Types and Selection Criteria

- A. If the outlet velocity, tailwater depth and site conditions indicate that a Type A, B or C Riprap Apron may be used, check Tables 11-12 and 11-13 to see if a Riprap Apron can be used based on the pipe size and discharge.
- B. If a Riprap Apron is adequate, Tables 11-12 and 11-13 will specify the length of apron required. Proceed to **Step 6**.
- C. If the Tables do not show an apron length, this indicates that the designer should proceed to **Step 7**, using a preformed scour hole design instead of a riprap apron.

For example, a project has two outlets.

Outlet No.1 is a 450-mm (18-inch) RCP with an outlet velocity of 2.74 mps (9 fps) and a design discharge of 0.275 cms (9.7 cfs) that outlets onto a flat area with a tailwater depth (TW) less than 200 mm (8 in).

Outlet No.2 is a 600-mm (24-inch) RCP with an outlet velocity of 3.35 mps (11 fps) and a design discharge of 0.500 cms (17.7 cfs) that outlets into a drainage channel with a tailwater depth (TW) of 500 mm (20 in).

Initially, the design parameters indicate that a Type A Riprap Apron and a Type C Riprap Apron would be appropriate for Outlet No. 1 and 2, respectively.

Next, Table 11-12 is checked for Outlet No. 1 with the design discharge and shows that a Type A Riprap Apron could be used with a required length of 4.5-m (15-ft.). Table 11-13 is checked for Outlet No. 2 and shows that the design discharge falls outside the limit for the use of a Type C Riprap Apron and that a preformed scour hole design should be used.

Step 6. Riprap Apron Dimensions

The designer has determined in **Step 5** that a riprap apron is appropriate at the outlet location. Riprap apron dimensions are discussed in Section 11.13.5 and are determined as follows:

- A. The length of apron (L_a) is determined from Tables 11-12 and 11-13 or Equations 11.31 and 11.32. **It should be noted, however, that the Tables are required to determine the minimum and maximum length of apron that can be used for a given pipe size and discharge.** The length of apron is shown on Figures 11-13 and 11-14.
- B. The width of the upstream (W_1) and downstream (W_2) apron limit for the Type A and B Riprap Apron are computed using Equations 11.33 and 11.34, respectively, or as shown on Figure 11-13. The width of a Type C Riprap Apron (W_3) is determined as described in Section 11.13.5 or as shown on Figure 11-14.

Step 7. Preformed Scour Hole Design

The designer has determined in **Step 5** that the outlet velocity, Tables 11-12 and 11-13 or site conditions dictate that a preformed scour hole is required for outlet protection. The design is discussed in Section 11.13.6 and summarized as follows:

- A. Compute the median stone size (d_{50}) required for both the Type 1 and 2 Preformed Scour Holes using Equations 11.35 and 11.36, respectively.
- B. Compute the scour hole dimensions for both types using the set of equations labeled 11.37 or Figure 11-15.
- C. Compare the values computed in Steps 7A and 7B for the two preformed scour hole types and select the one that provides the most economical and practical design given the site conditions.

Step 8. Special Design

In unusual cases where neither a riprap apron nor preformed scour hole can be used, and a special design is required, HEC-14 can be used to design an alternative energy dissipater. These designs, however, require prior approval from the Hydraulics and Drainage Section.

Step 9. Prepare Outlet Protection Computation Form

See Appendix A for form.

Step 10. Project Plans

The following information is required on the project plans for outlet protection:

TYPE	PLANS	DETAILS
Type A, B & C Riprap Apron	Call out apron type (A,B,C), riprap type & length of apron (L_a). Show apron limits.	Include detail(s) similar to Figures 11-13 & 11-14
Preformed Scour Hole Type 1 & Type 2	Call out type & riprap size. Show limits.	Include a detail similar to Figure 11-15.

Table 11-16 Outlet Protection Plan Requirements

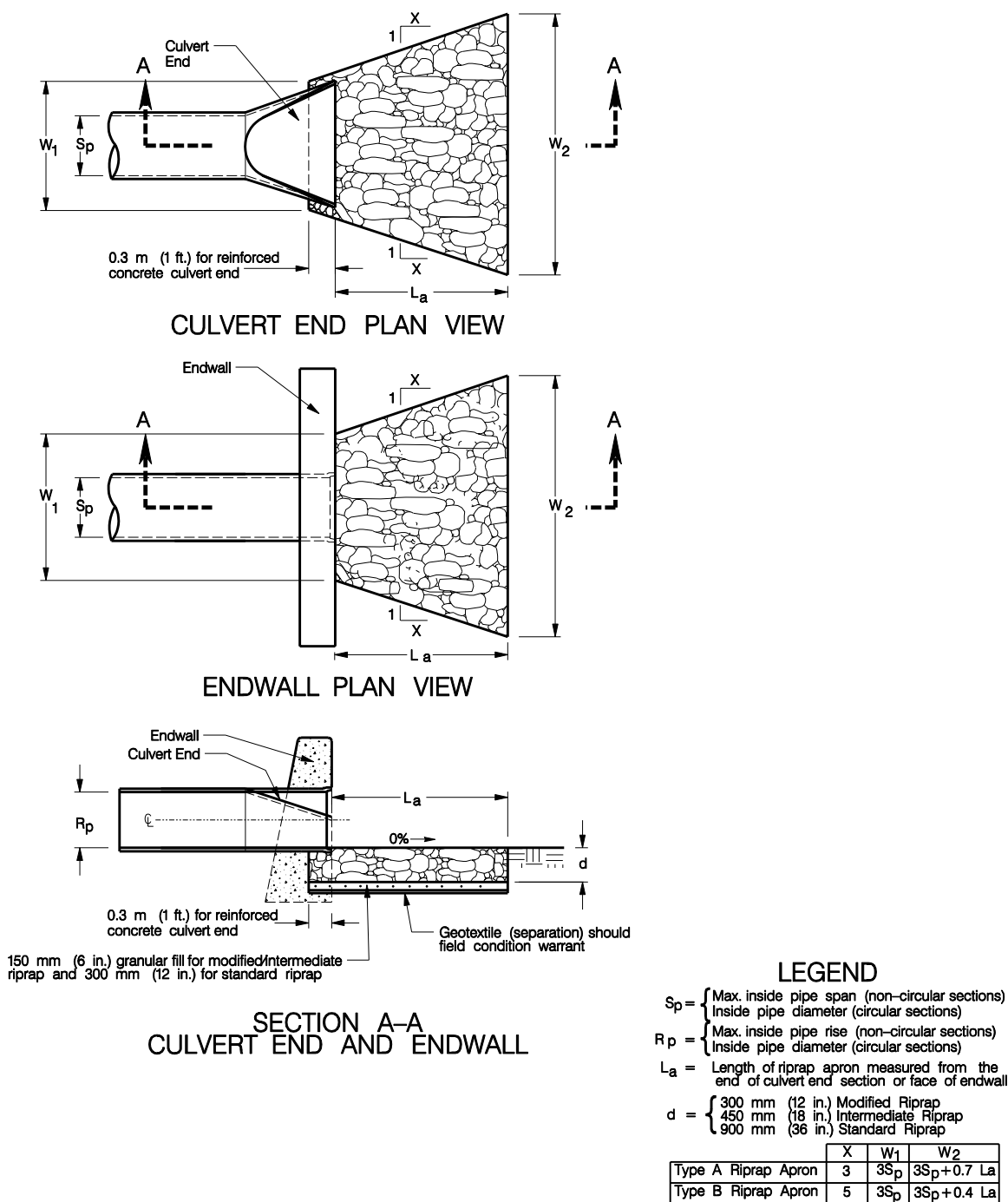


Figure 11-13 Type A and B Riprap Apron
(to be used where there is no defined channel downstream of the outlet)

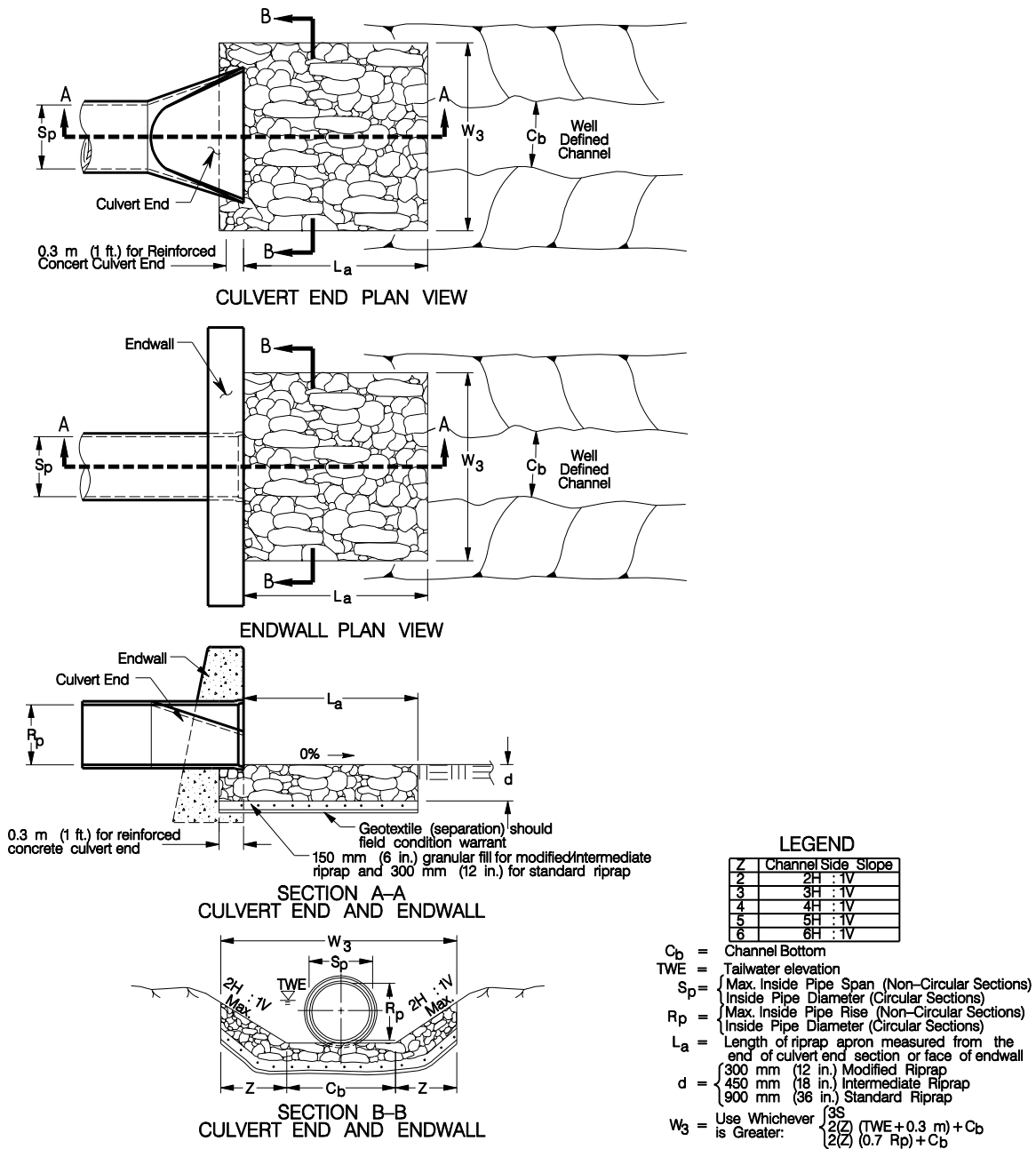


Figure 11-14 Type C Riprap Apron
 (to be used where there is a well defined channel downstream of the outlet)

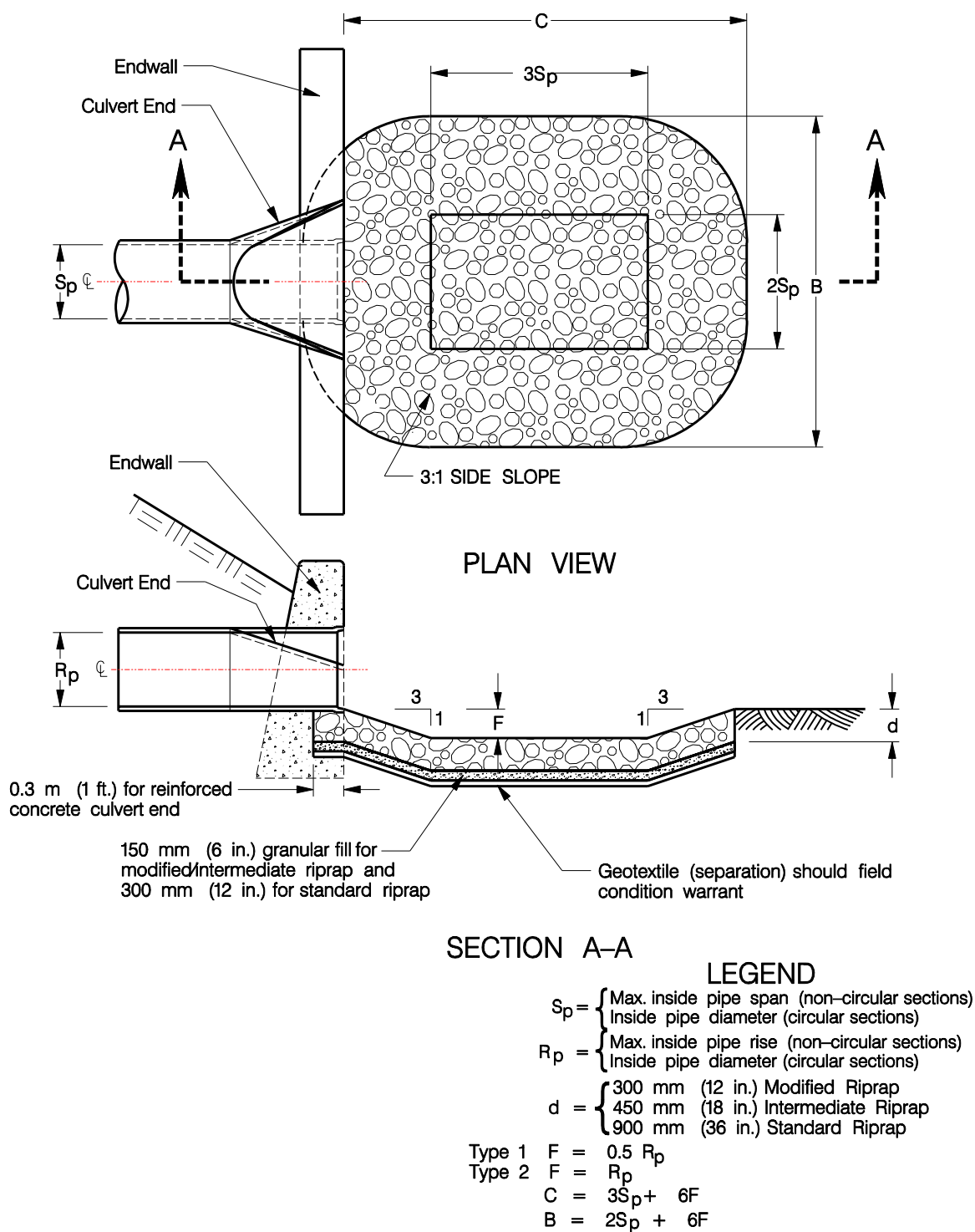


Figure 11-15 Preformed Scour Hole Type 1 and Type 2

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Pavement and Geometric Design Criteria For Minimizing Hydroplaning. FHWA Report No. RD-79-31. December 1979.

Dah-Chen Woo. Public Roads, Vol. 52, No. 2 Bridge Drainage System Needs Criteria. U. S. Department of Transportation. September 1988.

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Practical Guidance For Estimating and Controlling Erosion at Culvert Outlets, Miscellaneous Paper H-72-5, May 1972, by B. P. Fletcher, J.L. Grace, Jr., U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi

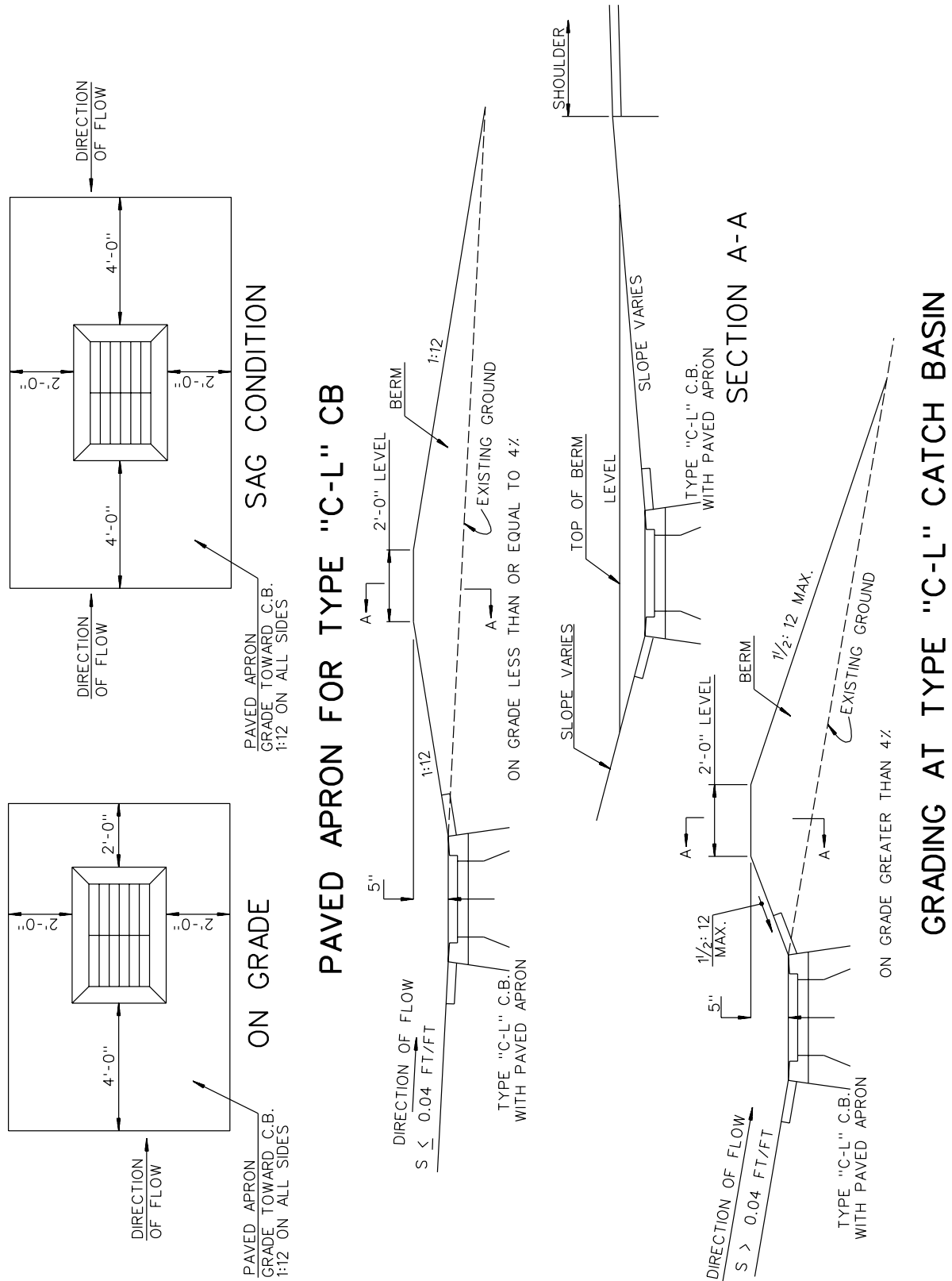
U.S. Department of Agriculture, Natural Resources Conservation Service (formerly Soil Conservation Service), Urban Hydrology for Small Watersheds, Technical Release No. 55, Washington, D.C., June 1986

Connecticut Department of Environmental Protection, Connecticut Stormwater Quality Manual, 2003

Appendix A – Outlet Protection Form

OUTLET PROTECTION	
Project No.: _____ Town: _____ Route: _____	Designed By: _____ Date: _____ Checked By: _____ Date: _____ Station: _____
1. Assess the erosion potential at the outlet and other critical site factors Describe the conditions at the outlet location: _____ _____ _____ _____ _____ _____ ☐ No well-defined channel ☐ Well-defined channel Sketch	
2. Determine the tailwater (TW) conditions at the outlet TW depth: _____ TW elevation: _____ TW computational method: _____ Channel bed elevation: _____ Estimated velocity in channel: _____	
3. Calculate and evaluate the outlet velocity for the design discharge Design Discharge: _____ Design Frequency: _____ Outlet Pipe Size: _____ Type: _____ Length: _____ Slope: _____ Outlet Invert Elevation: _____ Outlet Velocity at design discharge: _____ Velocity computational method: _____	
4. Select the type of outlet protection	
☐ Riprap Apron (See Figures 11-13 & 11-14) Type _____ (A,B,C) Riprap type: _____ Length (L_a): _____ Width (W_1): _____ Width (W_2): _____ Width-Type C (W_3): _____	☐ Preformed Scour Hole (See Figure 11-15) <u>Type 1</u> d_{50} _____ F _____ C _____ B _____ S_p _____ <u>Type 2</u> _____ Proposed Type: _____ Riprap Type: _____

Appendix B – Typical Grading for Type "C-L" Catch Basins Located in a Swale



Appendix C – Water Quality Flow (WQF) and Flow Diversion Guidance

Water Quality Flow Calculation

The water quality flow (WQF) is the peak flow rate associated with the water quality design storm. This section describes the recommended procedures for calculating the water quality flow for the design of:

- Pre-manufactured stormwater treatment devices (e.g. hydrodynamic separators, catch basin inserts, and media filters)
- Flow diversion structures for off-line stormwater treatment practices

The WQF should be calculated using the water quality volume (WQV) described in Chapter 10. This WQV, converted to watershed inches, should be substituted for the runoff depth (Q) in the Natural Resources Conservation Service (NRCS) (formerly Soil Conservation Service), TR-55 Graphical Peak Discharge Method. The procedures are based on the approach described in Claytor and Schueler, 1996.

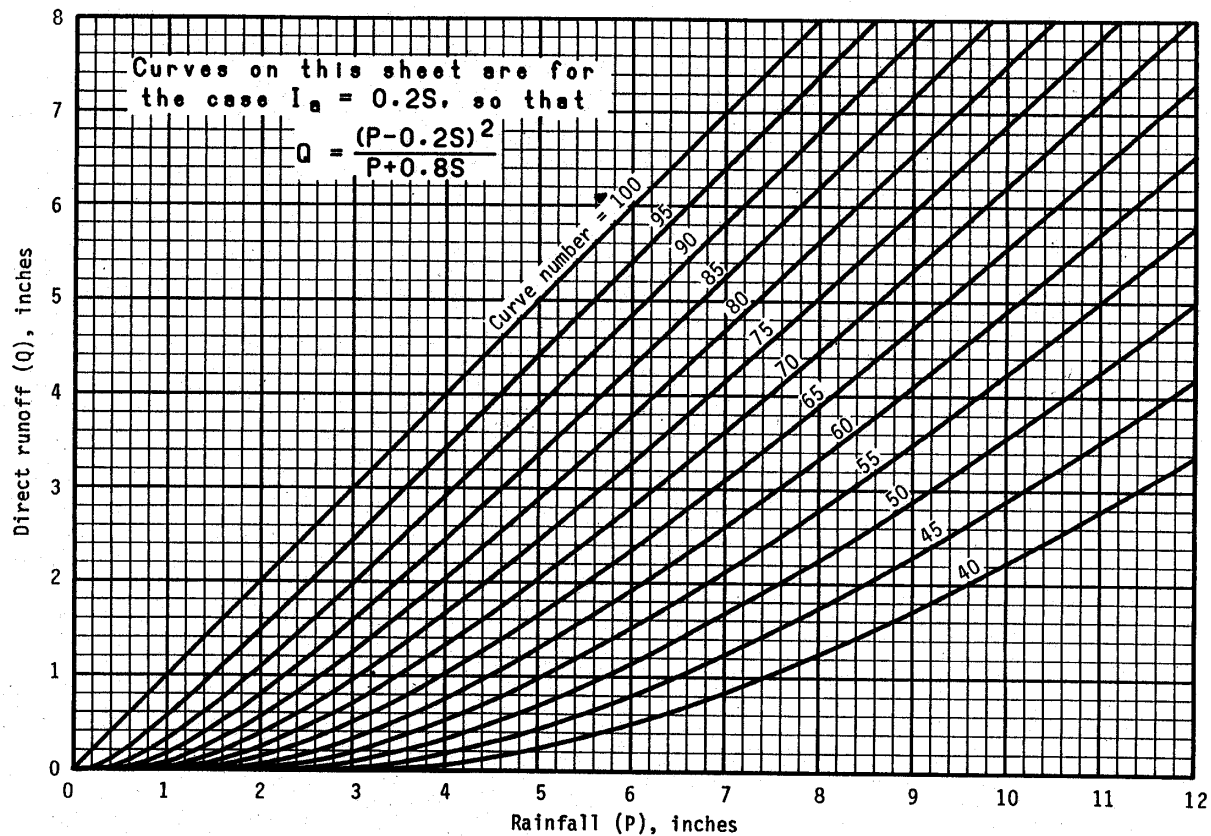
1. Compute the NRCS Runoff Curve Number (CN) using the following equation, or graphically using Figure 2-1 from TR-55 (USDA, 1986) (reproduced below):

$$CN = \left[\frac{1000}{10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}} \right] \quad (\text{English Units Only})$$

where: CN = Runoff Curve Number
P = design precipitation, inches (1" for water quality storm)
Q = runoff depth (in watershed inches)

$$Q = \frac{[WQV(acrefeet)] \times [12(inches / foot)]}{DrainageArea(acres)}$$

Solution of runoff equation (Figure 2-1 TR55)



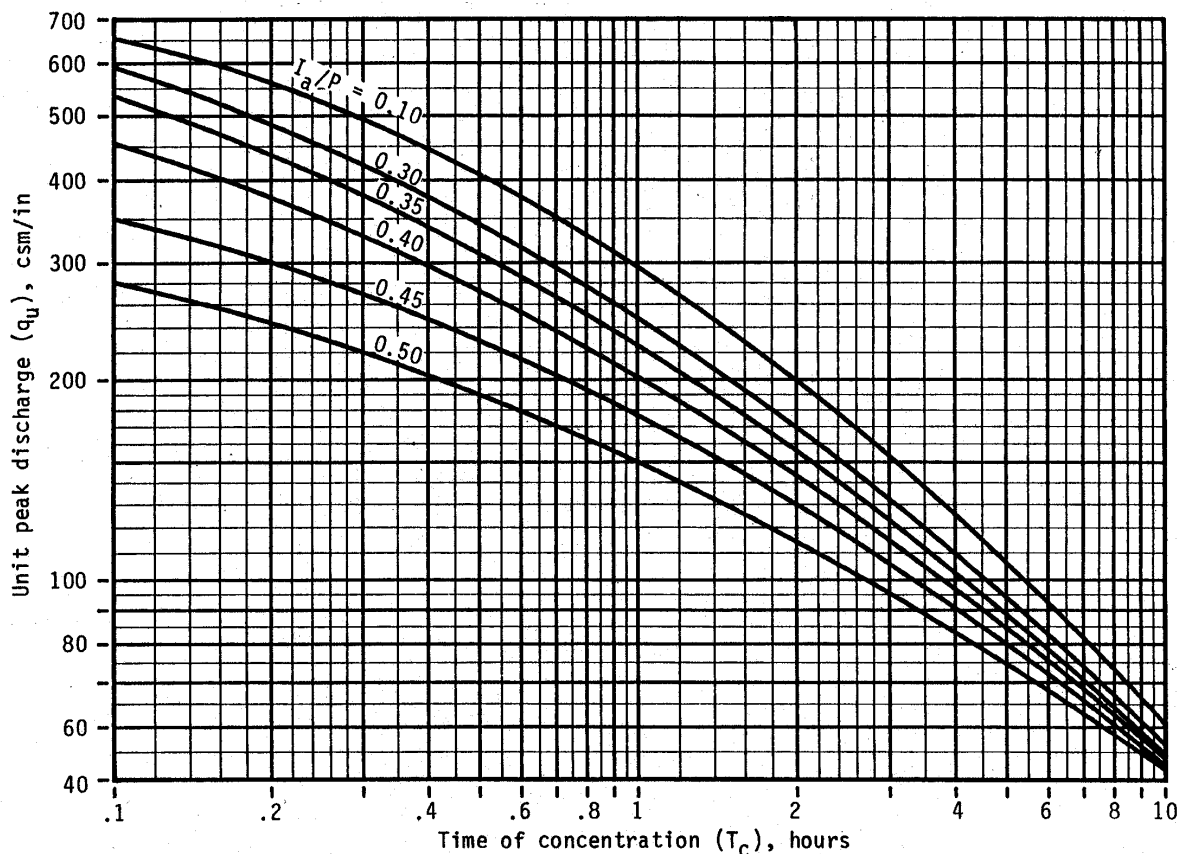
2. Compute the time of concentration (t_c) based on the methods described in Chapter 3 of TR-55. A minimum value of 0.167 hours (10 minutes) should be used. For sheet flow, the flow path should not be longer than 300 feet.
 3. Using the computed CN, t_c , and drainage area (A) in acres, compute the peak discharge for the water quality storm (i.e., the water quality flow), based on the procedures described in Chapter 4 of TR-55.
- Read initial abstraction (I_a) from Table 4-1 in Chapter 4 of TR-55 (reproduced below); compute I_a/P

I_a values for runoff curve numbers (Table 4-1 TR55)

Curve number	I _a (in)	Curve number	I _a (in)
40.....	3.000	70.....	0.857
41.....	2.878	71.....	0.817
42.....	2.762	72.....	0.778
43.....	2.651	73.....	0.740
44.....	2.545	74.....	0.703
45.....	2.444	75.....	0.667
46.....	2.348	76.....	0.632
47.....	2.255	77.....	0.597
48.....	2.167	78.....	0.564
49.....	2.082	79.....	0.532
50.....	2.000	80.....	0.500
51.....	1.922	81.....	0.469
52.....	1.846	82.....	0.439
53.....	1.774	83.....	0.410
54.....	1.704	84.....	0.381
55.....	1.636	85.....	0.353
56.....	1.571	86.....	0.326
57.....	1.509	87.....	0.299
58.....	1.448	88.....	0.273
59.....	1.390	89.....	0.247
60.....	1.333	90.....	0.222
61.....	1.279	91.....	0.198
62.....	1.226	92.....	0.174
63.....	1.175	93.....	0.151
64.....	1.125	94.....	0.128
65.....	1.077	95.....	0.105
66.....	1.030	96.....	0.083
67.....	0.985	97.....	0.062
68.....	0.941	98.....	0.041
69.....	0.899		

- Read the unit peak discharge (q_u) from Exhibit 4-III in Chapter 4 of TR-55 (reproduced below) for appropriate t_c

Unit peak discharge (q_u) for SCS type III rainfall distribution (Exhibit 4-III TR55)



- Substituting the water quality volume (WQV), converted to watershed inches, for runoff depth (Q), compute the water quality flow (WQF) from the following equation:

$$WQF = (q_u)(A)(Q)$$

where: WQF = water quality flow (cfs) (English units only)
 q_u = unit peak discharge (cfs/mi²/inch)
 A = drainage area (mi²)
 Q = runoff depth (in watershed inches)

$$Q = \frac{[WQV(acrefeet)] \times [12(inches / foot)]}{DrainageArea(acres)}$$

Flow Diversion Structures

Flow diversion structures, also called flow splitters, are designed to deliver flows up to the design water quality flow or water quality volume to off-line stormwater treatment practices. Flow in excess of the WQF or WQV are diverted around the treatment facility with minimal increase in head at the flow diversion structure to avoid surcharging the treatment facility under higher flow conditions. Flow diversion structures are typically manholes or vaults equipped with weirs, orifices, or pipes to bypass excess runoff.

The following general procedures are recommended for design of flow diversion structures:

- Locate the top of the weir or overflow structure at the maximum water surface elevation associated with the WQF, or the water surface elevation in the treatment practice when the entire WQV is being held, whichever is higher.
- Determine the diversion structure dimensions required to divert flows in excess of the WQF using standard equations for a rectangular sharp-crested weir, uniform flow in pipes or channels, or orifice depending on the type of diversion structure.
- Provide sufficient freeboard in the stormwater treatment practice and flow splitter to accommodate flow over the diversion structure.
- Limit the maximum head over the flow diversion structure to avoid surcharging the stormwater treatment practice under high flow conditions.
- Design diversion structures to withstand the effects of freezing, frost in foundations, erosion, and flotation due to high water conditions. These structures should be designed to minimize clogging potential and to allow for ease of inspection and maintenance.

Pump Stations

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12.1 Overview

12.1.1 Introduction

Stormwater pump stations are necessary to remove stormwater from highway sections that cannot be drained by gravity. Because of high costs and the potential problems associated with pump stations, their use is recommended only where other systems are not feasible. When operation and maintenance costs are capitalized, a considerable expenditure can be justified for a gravity system.

12.1.2 Design Considerations

Pump station design presents the designer with a challenge to provide a cost-effective drainage system that meets the needs of the project. There is a myriad of considerations involved in their design. Below is a listing of some of them:

- wet-pit vs. dry-pit
- type of pumps
- number and capacity of pumps
- motor vs. engine drive
- peak flow vs. storage
- force main vs. gravity
- above vs. below grade
- monitoring systems
- backup systems
- maintenance requirements

Many of the decisions regarding the above are currently based on engineering judgment and experience. To assure cost-effectiveness, the designer should assess each choice and develop economic comparisons of alternatives on the basis of annual cost. However, some general recommendations can be made which will help minimize the design effort and the cost of these expensive drainage facilities. These are discussed in the following pages of this chapter.

For further information on the design and use of pump stations see, *Highway Storm Water Pumping Stations*, Volume 1 & 2, FHWA-IP-82-17, NTIS numbers PB 84-152727 and 152735 and the other references given at the end of this chapter. The Hydraulic Institute, 9 Sylvan Way, Parsippany, New Jersey, 07054-3802 has developed standards for pumps. Pump station design should be consistent with these standards.

12.2 Symbols And Definitions

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in the technical publications.

Table 12-1 Symbols And Definitions

<u>Symbol</u>	<u>Definition</u>	<u>Units</u>
A	Minimum distance from back wall to trash rack	m (ft)
B	Maximum distance between a pump and the backwall	m (ft)
C	Average distance from floor to pump intake	m (ft)
D	Pump diameter	mm (in)
DHW	Design high water elevation	m (ft)
H _f	Friction head	m (ft)
H _l	Losses through fittings, valves, etc.	m (ft)
H _s	Maximum static head	m (ft)
H _t	Storage depth	m (ft)
H _v	Velocity head	m (ft)
H _x	Depth for storage volume	m (ft)
INV	Inlet invert elevation	m (ft)
L	Length of wet well	m (ft)
N	Number of equal size pumps	-
Q _i	Inflow	m ³ /s (cfs)
Q _p	Total capacity of all pumps Peak Discharge Rate	m ³ /s (cfs)
S	Minimum submergence at the intake	m (ft)
t _c	Minimum allowable cycle time	s
TDH	Total dynamic head	m (ft)
V _t	Total cycling storage volume	m ³ (ft ³)
V _x	Individual pump cycling volumes	m ³ (ft ³)
W	Minimum required distance between pumps	m (ft)
Y	Minimum level floor distance upstream of pump	m (ft)

12.3 Design Considerations

12.3.1 Location

Economic and design considerations dictate that the pump station be located relatively near the low point of the highway. Hopefully a frontage road or overpass is available for easy access to the station. The station and access road should be located on high ground so that access can be obtained if the highway becomes flooded. Soil borings should be made during the selection of the site to determine the allowable bearing capacity of the soil and to identify any potential problems.

Architectural and landscaping decisions should be made in the location phase for above-ground stations so the station will blend into the surrounding community. Following are some considerations that should be used in the location and design of pump stations.

- Modern pump stations can be architecturally pleasing with a minimum increase in cost.
- Clean functional lines will improve the station's appearance.
- Masonry or a textured concrete exterior can be very pleasing.
- Screening walls may be provided to hide exterior equipment and break up the lines of the building.
- A small amount of landscaping can substantially improve the overall appearance of the site.
- It may be necessary or desirable to place the station entirely underground.
- Ample parking and working areas should be provided adjacent to the station for maintenance and repair vehicles.

12.3.2 Hydrology

Because of traffic safety and flood hazards, pump stations serving major expressways and arterials are usually designed to accommodate a 50-year storm. It is desirable to check the drainage system for the 100-year storm to determine the extent of flooding and the associated risk. Every attempt should be made to keep the drainage area tributary to the station as small as possible. Bypass or pass-through all possible drainage to reduce pumping requirements. Avoid future increases in pumping by isolating the drainage area, i.e., prevent off-site drainage from possibly being diverted to the pump station. Hydrologic design should be based on the ultimate development of the area which must drain to the station.

Designers should consider storage, in addition to that which exists in the wet well, at all pump station sites. For most highway pump stations, the high flows of the inflow hydrograph will occur over a relatively short time. Additional storage, skillfully designed, may greatly reduce the peak pumping rate required. An economic analysis can be used to determine the optimum combination of storage and pumping capacity. Because of the nature of the sites where highway related pump stations are located, it is most often necessary that the storage be located well below normal ground level.

If flow attenuation is required for purposes other than reducing the size of the pump facility and cannot be obtained upstream of the station, consideration may be given to providing the storage downstream of the pump station. This will require large flows to be pumped and thus pump installation and operation costs will be higher.

If storage is used to reduce peak flow rates, a routing procedure must be used to design the system. To determine the discharge rate, the routing procedure integrates three independent elements: the inflow hydrograph, the stage-storage relationship, and the stage-discharge relationship.

12.3.3 Collection Systems

Storm drains leading to the pumping station are usually designed on a flat grade to minimize depth and cost. A minimum grade that produces a velocity of 1 m/s (3 ft/s) in the pipe while flowing one-quarter full is suggested to avoid siltation problems in the collection system. Minimum cover or local head requirements should govern the depth of the uppermost inlets. The inlet pipe should enter the station perpendicular to the line of pumps. The inflow should distribute itself equally to all pumps. Baffles may be required to ensure that this is achieved.

Storm drainage systems tributary to the pump station can be quite extensive and costly. Linear or intermediate storage along the storm drain may be used to reduce peak flows and pipe sizes. For some pump stations, the storage available in the collection system may be significant. However, it is often necessary to provide additional storage near the pump station. This may be done by oversizing the collection system or designing an underground vault.

In a wet-pit station where submersible or screw-type pumps are proposed, consideration of storage in the collection lines may be less critical, due to the less stringent submergence requirements of these alternate types of pumps. Also, submersible pumps can withstand more frequent cycling.

Using grate inlets as screens to prevent large objects from entering the system and possibly damaging the pumps is recommended. This approach has additional advantages of possibly eliminating costly trash racks and simplifying debris removal since debris can be more easily removed from the roadway than the wet well.

12.3.4 Station Types

Basically, there are two types of stations: wet-pit and dry-pit.

Wet-Pit Stations - In the wet-pit station, the pumps are submerged in a wet well or sump with the motors and the controls located overhead. With this design, the stormwater is pumped vertically through a riser pipe. The motor is commonly connected to the pump by a long drive shaft located in the center of the riser pipe. See Figure 12-1 for typical layout. Another type of wet-pit design involves the use of submersible pumps. The submersible pump commonly requires less maintenance and less kilowatts because a long drive shaft is not required. They also allow for convenient maintenance in wet-pit stations because of easy pump removal.

Submersible pumps are now available in large sizes and should be considered for use in all station designs. Rail systems are available which allow removal of pumps without entering the wet well.

Dry-Pit Stations - Dry-pit stations consist of two separate elements: the storage box or wet well and the dry well. Stormwater is stored in the wet well which is connected to the dry well by horizontal suction piping. Centrifugal pumps are usually used. Power is provided by either close-coupled motors in the dry well or long drive shafts with the motors located overhead. The main advantage of the dry-pit station for stormwater is the availability of a dry area for personnel to perform routine and emergency pump and pipe maintenance. See Figure 12-2 for typical layout.

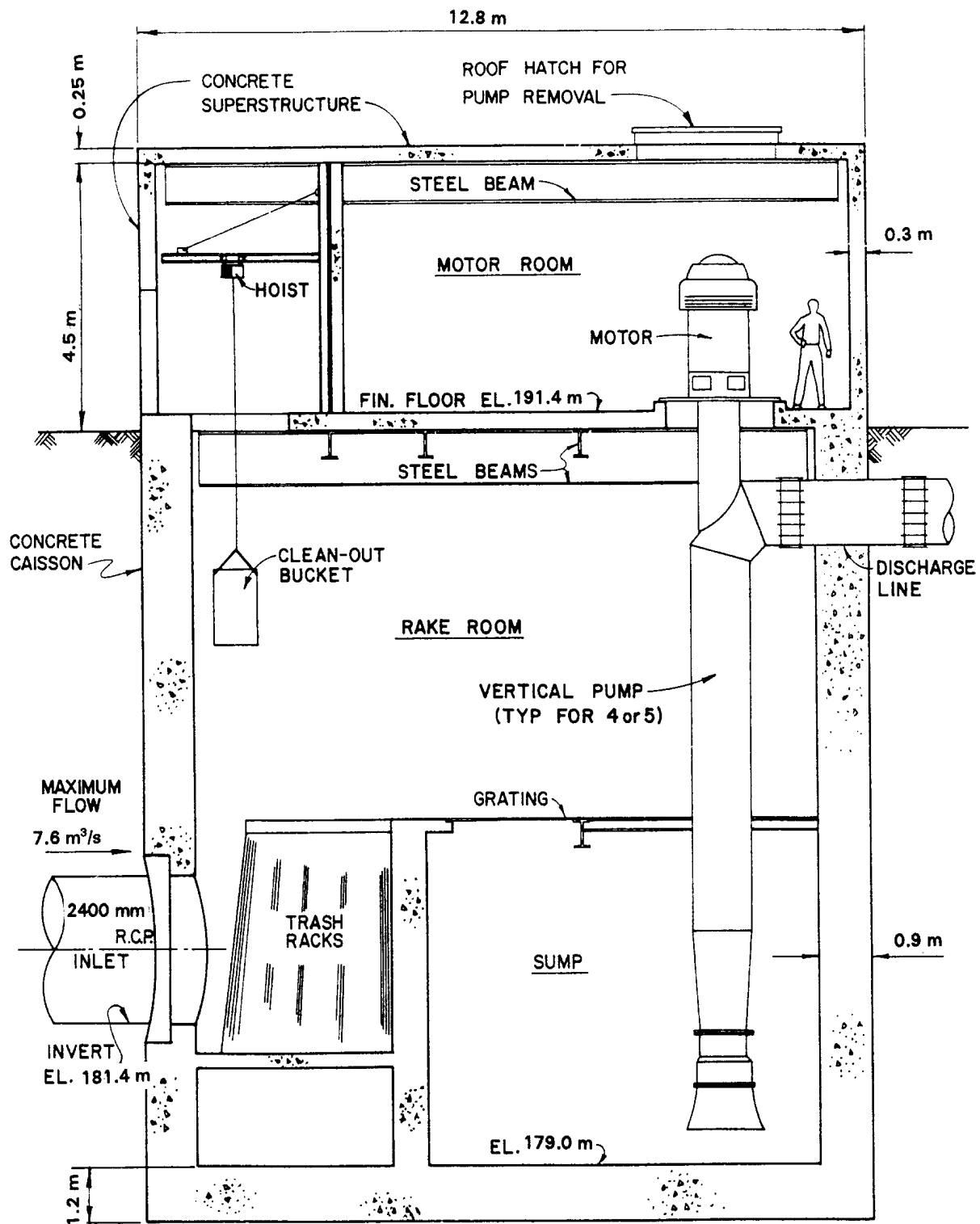


Figure 12-1 Typical Wet-Pit Station

Source: FHWA IP-82-17

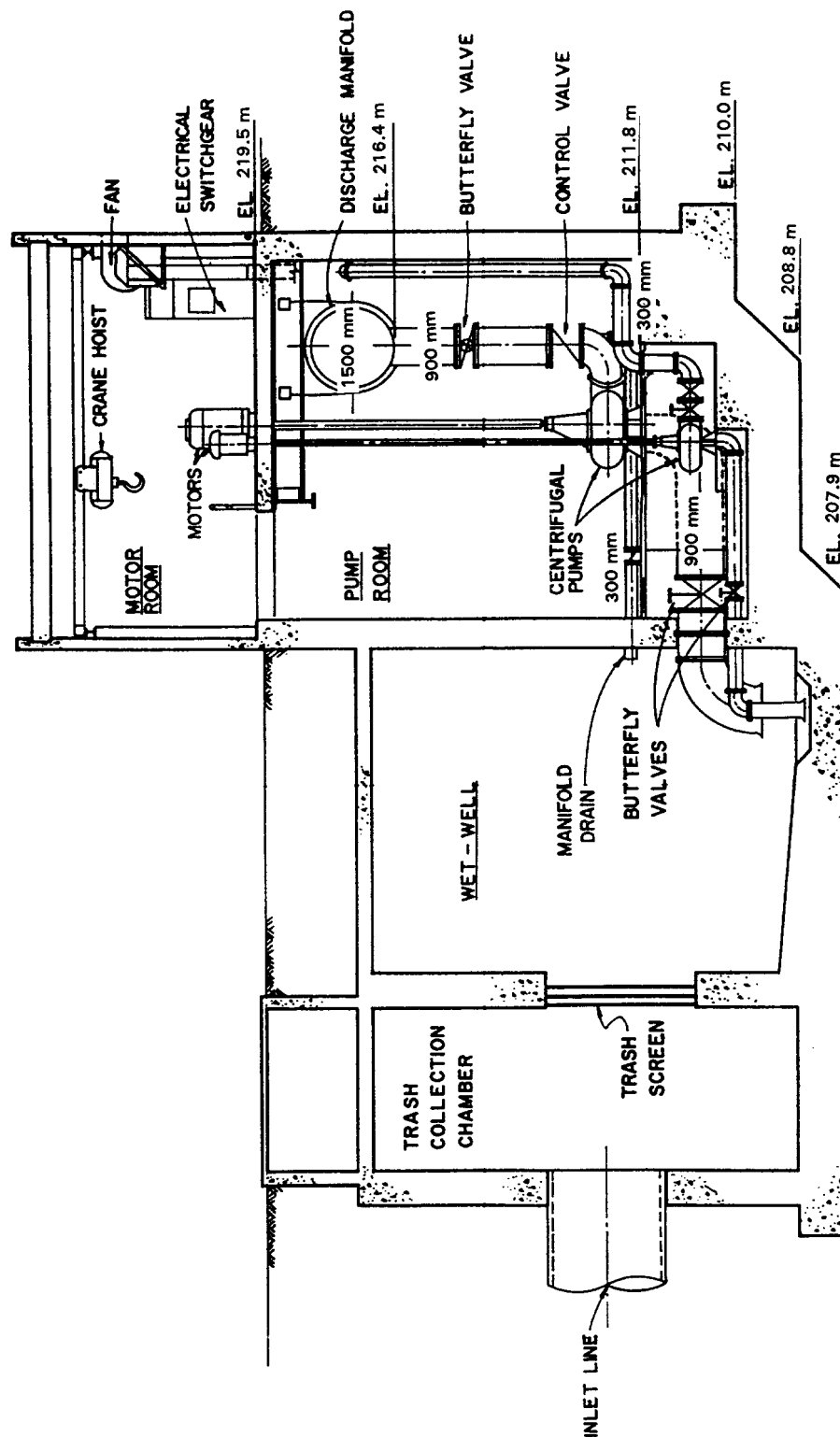


Figure 12-2 Typical Dry-Pit Station

Source: FHWA IP-82-17

12.3.5 Pump Types

The most common types of stormwater pumps are axial flow (propeller), radial flow (impeller) and mixed flow (combination of the previous two). Each type of pump has its particular merits.

Axial Flow Pumps - Axial flow pumps lift the water up a vertical riser pipe; flow is parallel to the pump axis and drive shaft. They are commonly used for low head, high discharge applications. Axial flow pumps do not handle debris particularly well because the propellers will bend or possibly break if they strike a relatively large, hard object. Also, fibrous material will wrap itself around the propellers.

Radial Flow Pumps - Radial flow pumps utilize centrifugal force to move water up the riser pipe. They will handle any range of head and discharge, but are the best choice for high head applications. Radial flow pumps generally handle debris quite well. A single vane, non-clog impeller handles debris the best because it provides the largest impeller opening. The debris handling capability decreases with an increase in the number of vanes since the size of the openings decrease.

Mixed Flow Pumps - Mixed flow pumps are very similar to axial flow except they create head by a combination of lift and centrifugal action. An obvious physical difference is the presence of the impeller "bowl" just above the pump inlet. They are used for intermediate head and discharge applications and handle debris slightly better than propellers.

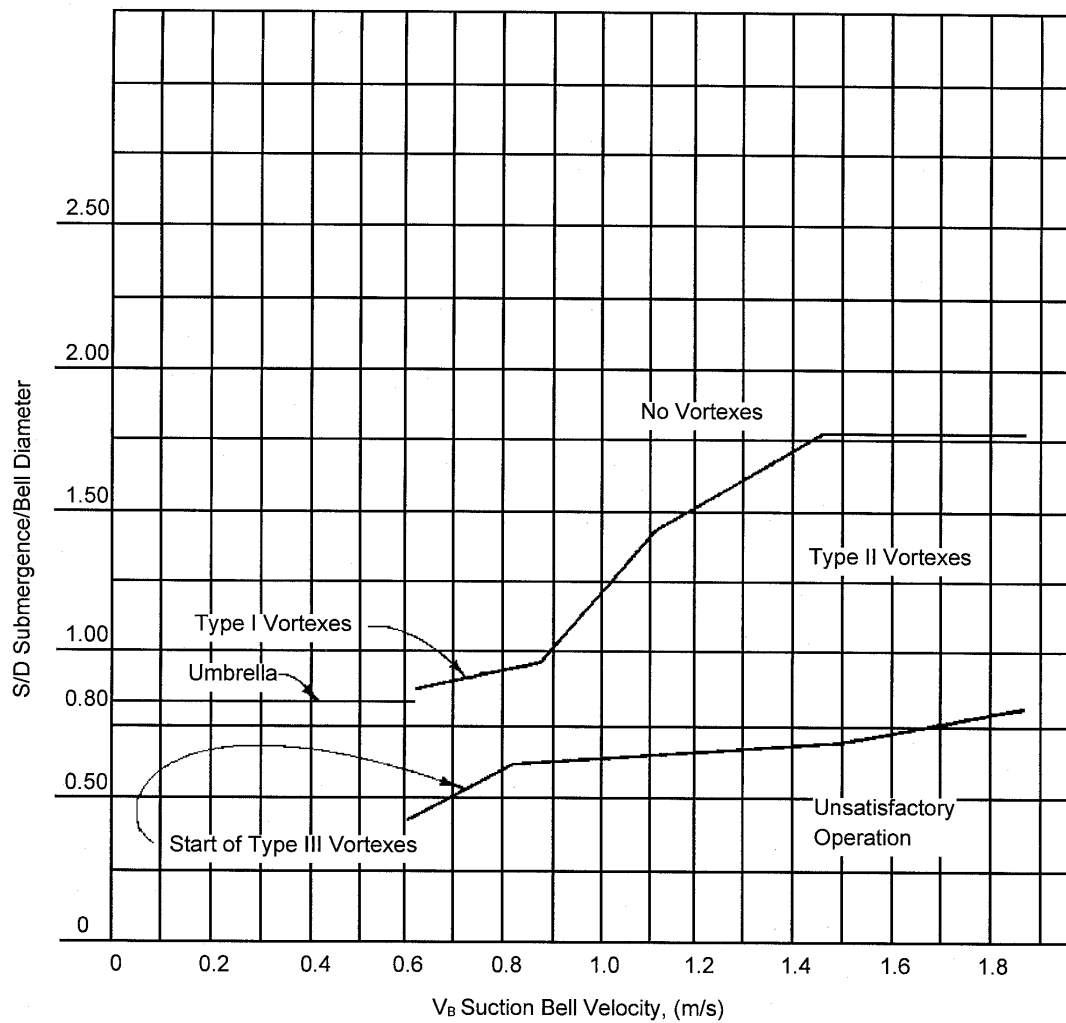
These pumps can be driven by motors or engines housed overhead or in a dry well or by submersible motors located in a wet well. Submersible pumps frequently provide special advantages in simplifying the design, construction, maintenance and, therefore, cost of the pumping station. Use of anything other than a constant speed, single stage, single suction pump would be rare.

The selection procedure is to first establish the criteria and then to select from the options available a combination that clearly meets the criteria. Cost, reliability, operating and maintenance requirements are all important considerations when making the selection. It is difficult and beyond the scope of this Manual to develop a totally objective selection procedure. First costs are usually of more concern than operating costs in stormwater pump stations since the operating periods during the year are relatively short. Ordinarily, first costs are minimized by providing as much storage as possible, with two or three small pumps, electrically-driven.

12.3.6 Submergence

Submergence is the depth of water above the pump inlet necessary to prevent cavitation and vortexing. It varies significantly with pump type and speed and atmospheric pressure. This dimension is provided by the pump manufacturer and is determined by laboratory testing. A very important part of submergence is the required net positive suction head (NPSH) because it governs cavitation. The available NPSH should be calculated and compared to the manufacturer's requirement. Additional submergence may be required at higher elevations. As a general rule, radial flow pumps require the least submergence while axial flow pumps require the most.

One popular method of reducing the submergence requirement (and therefore the station depth) for axial and mixed flow pumps, when cavitation is not a concern, is to attach a suction umbrella. A suction umbrella is a dish-shaped steel plate attached to the pump inlet which improves the entrance conditions by reducing the intake velocities. For umbrella velocities of 0.6 m/s (2 ft/s) or less, a submergence to pump bell diameter ratio of 0.80 can be used. Vortex criteria developed by Dicmas can be used to check the submergence requirements for these pumps (see Figure 12-3).



Phenomenon is not related solely to backwall as illustrated.

Vortexing occurs at random locations according to pump pit conditions.

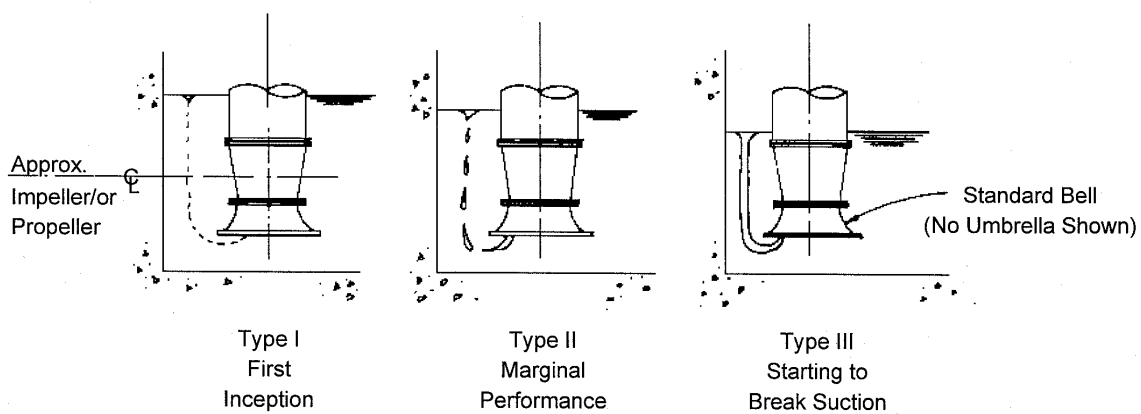


Figure 12-3 Vortex Description And Submergence Requirements

12.3.7 Water-Level Sensors

The water-level sensors activate the pumps and, therefore, are a vital component of the control system. There are a number of different types of sensors that can be used. Types include the float switch, electronic probes, ultrasonic devices, mercury switch and air pressure switch. The location or setting of these sensors controls the starting and stopping of the pump motors. Their function is critical because the pump motors or engines must not start more frequently than an allowable number of times per hour (i.e., the minimum cycle time) to avoid damage. To prolong the life of the motors, sufficient volume must be provided between the pump start and stop elevations to meet the minimum cycle time requirement. The on-off setting for the first pump is particularly important because it defines the most frequently used cycle.

12.3.8 Power

Several types of power may be available for a pump station. Examples are electric motors and gasoline, diesel or natural gas engines. The designer should select the type of power that best meets the needs of the project based on an estimate of future energy considerations and overall station reliability. A comparative cost analysis of alternatives is helpful in making this decision. However, when readily available, electric power is usually the most economical and reliable choice today.

The need for backup power is dependent upon the consequences of failure. The decision to provide it should be based on economics and safety. For electric motors, two independent electrical feeds from the electric utility with an automatic transfer switch may be the cost-effective choice when backup power is required. A standby generator is generally less cost-effective because of its initial costs. Also, standby generators require considerable maintenance and testing to ensure operation in times of need.

For extensive depressed freeway systems involving a number of electric motor-driven stations, a mobile generator may be the cost-effective choice for backup power. A trailer mounted generator can be stored at any one of the pump stations. If a power outage occurs, maintenance forces can move the generator to the affected station to provide temporary power. If a mobile generator is used as the source of backup power, it may be necessary to add additional storage to compensate for the time lag that results in moving the generator from site to site. This lag will typically be 1.0 to 1.5 h from the time the maintenance forces are notified.

12.3.9 Discharge System

The discharge piping should be kept as simple as possible. Pumping systems that lift the stormwater vertically and discharge it through individual lines to a gravity storm drain as quickly as possible are preferred. Individual pump discharge lines are the most cost-effective system for short outfall lengths. Damaging pump reversal could occur with very long forced mains. Check valves should be installed. The effect of stormwater returning to the sump after pumping stops should be considered. Individual lines may exit the pumping station either above or below grade. Frost depth shall be considered while deciding the depth of discharge piping. Frozen discharge pipe could exert additional back pressure on pumps.

It may be necessary to pump to a higher elevation using long discharge lines. This may dictate that the individual lines be combined into a force main via a manifold. For such cases, check valves must be provided on the individual lines to keep stormwater from running back into the wet well and restarting the pumps or prolonging their operation time. Check valves should preferably be located on horizontal layouts rather than vertical, to prevent sedimentation on the downstream side

after the valve closing. Gate valves should be provided in each pump discharge line to provide for continued operation during periods of repair, etc. A cost analysis should be performed to determine what length and type of discharge piping justifies a manifold. Number of valves required shall be kept to a minimum to reduce cost, maintenance and headloss through the system.

12.3.10 Flap Gates And Valving

Flap Gates - The purpose of a flap gate is to restrict water from flowing back into the discharge pipe and to discourage entry into the outfall line. Flap gates are usually not water tight so the elevation of the discharge pipe should be set above the normal water levels in the receiving channel. If flap gates are used, it may not be necessary to provide for check valves.

Check Valves - Check valves are water tight and are required to prevent backflow on forcemains which contain sufficient water to restart the pumps. They also effectively stop backflow from reversing the direction of pump and motor rotation. They must be used on manifolds to prevent return flow from perpetuating pump operation. Check valves should be "non-slam" to prevent water hammer. Types include: swing, ball, dash pot and electric.

Gate Valves - Gate valves are simply a shut-off device used on forcemains to allow for pump or valve removal. These valves should not be used to throttle flow. They should be either totally open or totally closed.

Air/Vacuum Valves - Air/Vacuum valves are used to allow air to escape the discharge piping when pumping begins and to prevent vacuum damage to the discharge piping when pumping stops. They are especially important with large diameter pipe. If the pump discharge is open to the atmosphere, an air-vacuum release valve is not necessary. Combination air release valves are used at high points in force mains to evacuate trapped air.

12.3.11 Trash Racks And Grit Chambers

Trash racks should be provided at the entrance to the wet well if large debris is anticipated. For stormwater pumping stations, simple steel bar screens are adequate. Usually, the bar screens are inclined with bar spacings approximately 40 mm (1.5 in). Constructing the screens in modules facilitate removal for maintenance. If the screen is relatively small, an emergency overflow should be provided to protect against clogging and subsequent surcharging of the collection system. As previously noted, screening large debris at surface inlets may be very effective in minimizing the need for trash racks.

If substantial amounts of sediment are anticipated, a chamber may be provided to catch solids that are expected to settle out. This will minimize wear on the pumps and limit deposits in the wet well. The grit chamber should be designed so that a convenient means of sediment removal is available.

12.3.12 Ventilation

Ventilation of dry and wet wells is necessary to ensure a safe working environment for maintenance personnel. The ventilation system can be activated by a light switch at the entrance to the station. Maintenance procedures should require personnel to wait several minutes after ventilation has started before entering the well. Some owners require the testing of the air in the wet well prior to allowing entry. Safety procedures for working in wet wells should be well established and carefully followed.

If mechanical ventilation is required to prevent buildup of potentially explosive gasses, the pump motors or any spark producing equipment should be rated explosion proof or the fans run continuously.

Heating and dehumidifying requirements are variable. Their use is primarily dependent upon equipment and station type, environmental conditions and station use.

12.3.13 Roof Hatches And Monorails

It will be necessary to remove motors and pumps from the station for periodic maintenance and repair. Removable roof hatches located over the equipment are a cost-effective way of providing this capability. Mobile cranes can simply lift the equipment directly from the station onto maintenance trucks. Monorails are usually more cost-effective for larger stations.

12.3.14 Equipment Certification And Testing

Equipment certification and testing is a crucial element of pump station design. The purchaser has a right to witness equipment testing at the manufacturer's lab. However, this is not always practical. As an alternative, the manufacturer should provide certified test results to the owner. It is good practice to include in the contract specifications the requirement for acceptance testing by the owner, when possible, to ensure proper operation of the completed pump station. The testing should be done in the presence of the owner's representative. If the representative waves the right to observe the test, a written report should be provided to give assurance that the pump equipment meets all performance requirements. Any component that fails should be repaired and retested.

12.3.15 Monitoring

Pump stations are vulnerable to a wide range of operational problems from malfunction of the equipment to loss of power. Monitoring systems such as on-site warning lights and remote alarms can help minimize such failures and their consequences.

Telemetry is an option that should be considered for monitoring critical pump stations. Operating functions may be telemetered from the station to a central control unit. This allows the central control unit to initiate corrective actions immediately if a malfunction occurs. Such functions as power, pump operations, unauthorized entry, explosive fumes and high water levels can be monitored effectively in this manner. Perhaps the best overall procedure to assure the proper functioning of a pump station is the implementation of a regular schedule of maintenance conducted by trained, experienced personnel.

12.3.16 Hazardous Spills

The possibility of hazardous spills is always present under highway conditions. In particular, this has reference to gasoline, and the vulnerability of pump stations and pumping equipment to fire damage. There is a history of such incidents having occurred and also of spills of oils, corrosive chemicals, pesticides and the like having been flushed into stations, with undesirable results. The usual design practice has been to provide a closed conduit system leading directly from the highway to the pump station without any open forebay to intercept hazardous fluids, or vent off volatile gases. With a closed system, there must be a gas-tight seal between the pump pit and the motor room in the pump station. Preferably, the pump station should be isolated from the main collection system and the effect of hazardous spills by a properly designed storage facility upstream of the station. This may be an open forebay or a closed box below the highway pavement or adjacent to it. The closed box must be ventilated by sufficient grating area at each end.

12.4 Additional Considerations

12.4.1 Construction

The method of construction has a major impact on the cost of the pump station. For near continuous operation, such as pumping sewage, it has been estimated that construction represents more than 20% of the pump station costs over a 10-year period. With a less frequently operating stormwater pump station, operating costs may be insignificant compared to construction costs. Therefore, the type of construction should be chosen carefully, between caisson construction, in which the station is usually circular, or open-pit construction. Soil conditions are the primary factor in selecting the most cost-effective alternative.

Feedback should be provided by the construction personnel on any problems encountered in the construction of the station so the designers can improve future designs. Any changes should be documented by "as-built" drawings. Personnel who are knowledgeable and experienced with such equipment should conduct construction inspections of pump stations.

12.4.2 Maintenance

Since major storm events are infrequent, a comprehensive, preventive maintenance program should be developed for maintaining and testing the equipment so that it will function properly when needed. Instruments such as hour meters and number-of-starts meters should be used on each pump to help schedule maintenance. Input from maintenance forces should be a continuous process so that each new generation of stations will be an improvement.

12.4.3 Retrofitting Stations

Retrofitting existing stormwater pump stations may be required when changes to the highway increase runoff to them. The recommended approach to this problem is to first try to increase the capacity of the station without making major structural changes. This can be achieved by using a cycling sequence that requires less cycling volume or power units that allow a greater number of starts per hour (i.e., shorter cycling time). Submersible pumps have been used effectively in retrofitting stations because of the flexibility in design and construction afforded by their frequent cycling capability.

12.4.4 Safety

All elements of the pump station should be carefully reviewed for safety of operation and maintenance. Ladders, stairwells and other access points should facilitate use by maintenance personnel. Adequate space should be provided for the operation and maintenance of all equipment. Particular attention should be given to guarding moving components such as drive shafts and providing proper and reliable lighting. It may also be prudent to provide air testing equipment in the station so maintenance personnel can be assured of clean air before entering.

Pump stations may be classified as a confined space in which case access requirements along with any safety equipment are all defined by code. Pump stations should be designed to be secure from entry by unauthorized personnel and as few windows as possible should be provided.

12.4.5 Environmental

Pump stations should be located outside the flood zone. If this is not possible, then they must be flood proofed in accordance with the Flood Management statutes and Administrative Regulations.

12.5 Recommended Design Criteria

12.5.1 Objective

The following recommendations are being made with the objective of minimizing the construction, operation and maintenance costs of highway stormwater pump stations while remaining consistent with the practical limitations of all aspects.

12.5.2 Station Type And Depth

Since dry-pit stations are as much as 60% more expensive than wet-pit stations, wet-pit stations are recommended. Dry-pit stations are more appropriate for handling sewage because of the potential health hazards to maintenance personnel. The hazards associated with pumping stormwater usually do not warrant the added expense. Some advantages associated with dry-pit stations include ease of access for repair and maintenance and the protection of equipment from fire and explosion.

The station depth should be minimum. No more depth than that required for pump submergence and clearance below the inlet invert is necessary, unless foundation conditions dictate otherwise.

12.5.3 Power

Electric is usually most desirable if available. Recommend constant speed, 3-phase induction motors (National Electrical Manufacturers Association Design B). Motor voltages between 440 and 575 are very economical for pumping applications. Consequently, the recommended maximum size is 225 kW. This size is also a good upper limit for ease of maintenance.

Consideration should be given to whether the pump station is to have standby power (SBP). If the stations have a SBP receptacle, manual transfer switch and a portable engine/generator set, then the practical power limit of the pumps becomes 55 kW since this is the limit of the power generating capabilities of most portable generator units. Two pumps would be operated by one engine/generator set.

12.5.4 Discharge Head And System Curve

Since stormwater pumps are extremely sensitive to changes in head, the head demand on the pumps should be calculated as accurately as possible. All valve and bend losses should be considered in the computations. In selecting the size of discharge piping, consideration should be given to the manufactured pump outlet size vs. the head loss produced by smaller piping. This approach should identify a reasonable compromise in balancing cost.

Once the head losses have been calculated for the range of discharges expected, the system curve (Q vs. TDH) can be plotted. This curve defines the energy required to pump any flow through the discharge system. It is especially critical for the analysis of a discharge system with a forcemain. When overlaid with pump performance curves (provided by manufacturer), it will yield the pump operating points (see Figure 12-4).

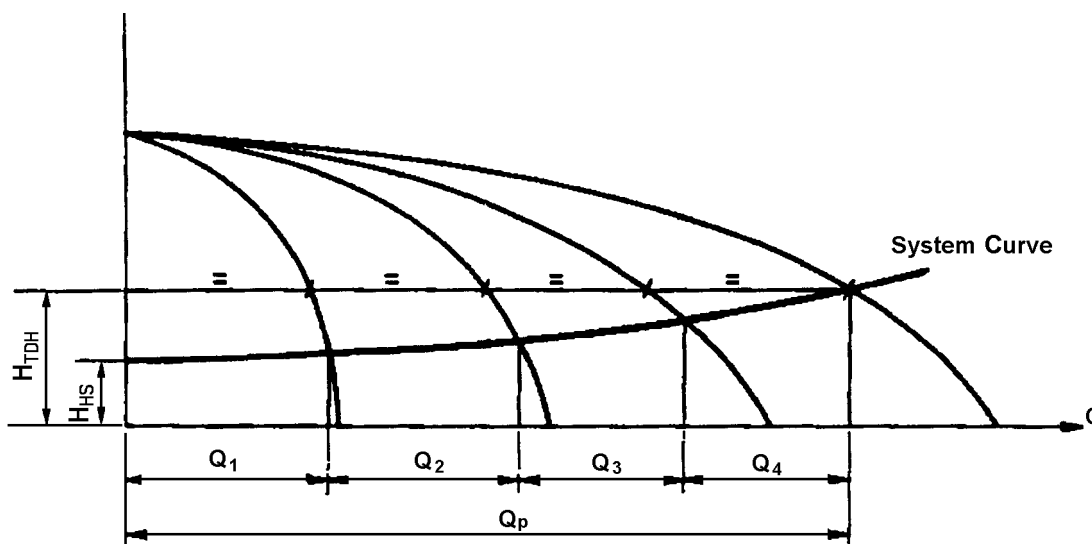


Figure 12-4 Systems Curve

When the pump is raising the water from the lowest level, the static head will be greatest and the discharge will be the least. When operating at the highest level, the static head will be the least and the discharge will be the greatest. The capabilities of pump must always be expressed in both quantity of discharge and the total dynamic head at a given level. The designer will typically specify these conditions for three points on the performance curve. One point will be near maximum TDH, the next will be the design point, and the third will be at about minimum TDH.

The combination of static head, velocity head and various head losses in the discharge system due to friction is called total dynamic head. It is usual to minimize these various head losses by the selection of correctly sized discharge lines and other components.

A pump is selected to operate with the best possible efficiency as its Design Point, corresponding to the Design Water Level of the station, and its performance is expressed as the required discharge at the resulting total dynamic head. The efficiency of a stormwater pump at its design point may be 75 or 80% or more, but this will depend on the type of pump.

When the static lift is greatest (low water in sump), the energy required (kilowatt) maybe the greatest even though the quantity of water raised is less. This is because the pump efficiency may also be much less. The pump selection should be made so that maximum efficiency is at the design point.

Pumps for a given station are selected to all operate together to deliver the Design Q at a Total Dynamic Head computed to correspond with the Design Water Level. Because pumps must operate over a range of water levels, the quantity delivered will vary significantly between the low level of the range and the high level. Typically, the designer will be required to specify at least three points on the performance curve. These will typically be the conditions for the TDH at the highest head, the design head and the lowest head expected over the full operating range of the pump. The manufacturer always plots a curve of total dynamic head versus pump capacity for each pump. When running, the pump will respond to the total dynamic head prevailing and the quantity of discharge will be in accordance with the curve. The designer must study the pump performance curves for various pumps in order to develop an understanding of the pumping conditions (head, discharge, efficiency, kilowatt, etc.) throughout the full range of head that the pump will operate under. The system specified must operate properly under the full range of specified head.

The total dynamic head (TDH) should be determined for a sufficient number of points to draw the system head curve that will be discussed later in this chapter. The TDH is computed as follows:

$$\text{TDH} = H_s + H_f + H_v + H_l \quad (12.1)$$

where: TDH = total dynamic head, m (ft)
H_s = max. static head (at lowest pump-off elevation), m (ft)
H_f = friction head, m (ft) (i.e., friction loss)
H_v = velocity head, m (ft) ($V^2/2g$)
H_l = losses through fittings, valves, etc., m (ft)

Adjustments may have to be made to these curves to account for losses within the pumping unit provided by the manufacturer.

12.5.5 Main Pumps

Number And Capacity

The designer will determine the number of pumps needed by following a systematic process defined in section 12.6. However, two to three pumps have been judged to be the recommended minimum. If the total discharge to be pumped is small and the area draining to the station has little chance of increasing substantially, the use of a two pump station is preferred. Consideration may be given to over sizing the pumps to compensate, in part, for a pump failure. The two pump system could have pumps designed to pump 66 - 100% of the required discharge and the three pump system could be designed so that each pump will pump 50% of the design flow. The resulting damage caused by the loss of one pump could be used as a basis for deciding the size and numbers of the pumps.

It is recommended that economic limitations on power unit size as well as practical limitations governing operation and maintenance be used to determine the upper limit of pump size. The minimum number of pumps used may increase due to these limitations.

It is also recommended that equal-size pumps be used. Identical size and type enables all pumps to be freely alternated into service. This equalizes wear and reduces needed cycling storage. It also simplifies scheduling maintenance and allows pump parts to be interchangeable. Hour meters and start meters should be provided to aid in scheduling needed maintenance.

Final Selection

For the typical highway application, any of the three pump types described earlier will usually suffice. If not, manufacturers' information will likely dictate the type required. However, knowing the operating RPMs, a computation can be made to check the appropriateness of the pump type (see Figure 12-5 to determine the ranges where specific impeller types should be used). Suction specific speed may be defined as that speed in revolutions per minute at which a given impeller would operate if reduced proportionally in size so as to deliver a capacity of 0.21 L/s (.055 gal/s) against TDH of 1 m (3 ft.). This is an index number descriptive of the suction characteristics of a given pump. Higher numerical values are associated with better NPSH capabilities. This number should be checked for dry pit applications and systems with suction lifts. Once the pump type and capacity have been determined, the final selection of the pump can be made.

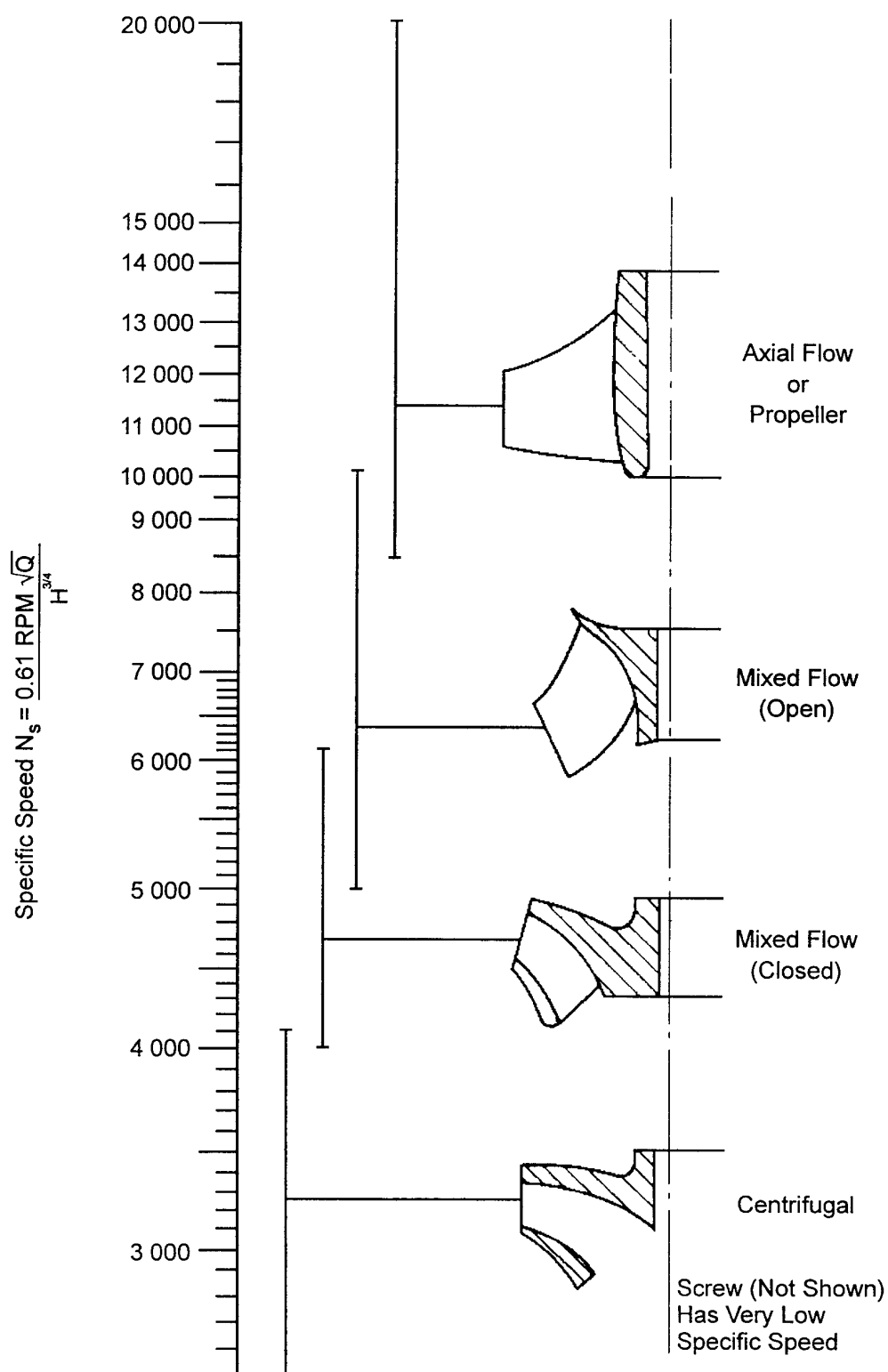


Figure 12-5 Specific Pump Speed vs. Impeller Types

12.5.6 Standby/Spare Pumps

Considering the short duration of high inflows, the low frequency of the design storm, the odds of a malfunction and the typical consequences of a malfunction, spare or standby pumps are not warranted in stormwater applications. If the consequences of a malfunction are particularly critical, it is more appropriate to add another main pump and reduce their size accordingly.

12.5.7 Storage

The total storage capacity that can or should be provided is an important initial consideration in pump station design. Using the hydrograph and pump-system curves, various levels of pump capacity can be tried and the corresponding required total storage can be determined. The basic principle is that the volume of water as represented by the shaded area of the hydrograph in Figure 12-6 is beyond the capacity of the pumps and must be stored. If a larger part or most of the design storm is allowed to collect in a storage facility, a much smaller pump station can be utilized, with anticipated cost benefits. If the discharge rate is to be limited, ample storage is essential.

Since most highway related pump stations are associated with either short underpasses or long depressed sections, it is not reasonable to consider above ground storage. Water that originates outside of the depressed areas should not be allowed to enter the depressed areas because of the need to pump all of this water. The simplest form of storage for these depressed situations is either the enlargement of the collection system or the construction of an underground storage facility. These can typically be constructed under the roadway area and will not require additional right-of-way. The pump stations can remove the stored water by either the dry pit or the wet pit approach as chosen by the designer.

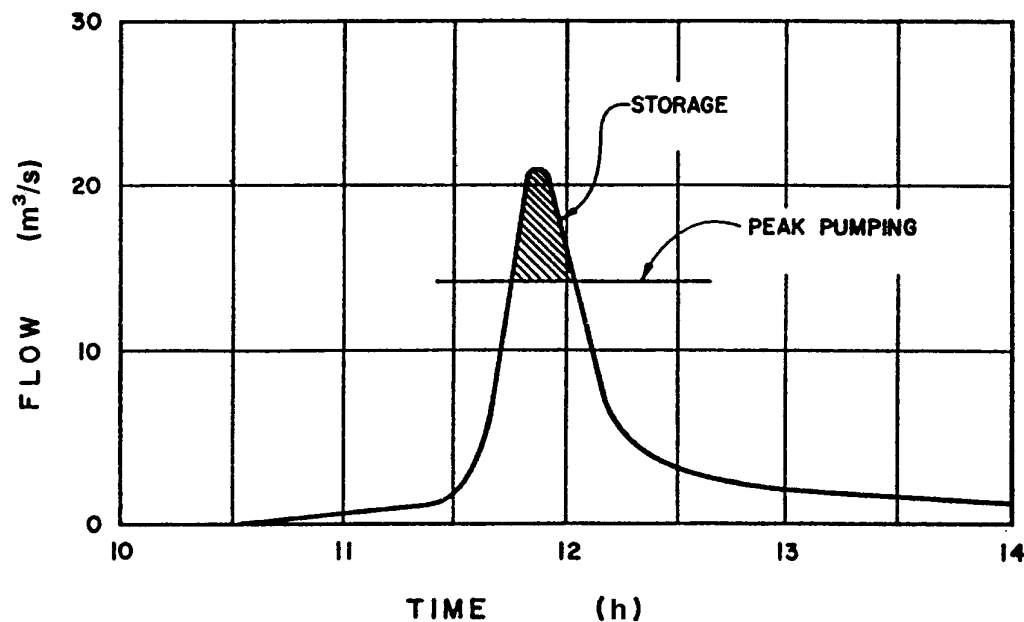


Figure 12-6 Estimating Required Storage

12.5.8 Sump Pumps

These are usually small submersible pumps necessary only in the dry well of dry-pit stations to protect equipment from seepage water damage. Because of their size, they are prone to sediment locking in wet-pit stations and, therefore, are not recommended for those installations. If it is necessary to evacuate the wet well, a portable pump can be used.

12.5.9 Wet Well Design

Cycling Sequence And Volumes

Cycling is the starting and stopping of pumps, the frequency of which must be limited to prevent damage and possible malfunction. The wet well must be designed to provide sufficient volume for safe cycling or sufficient volume must be provided outside the wet well. However, to keep sediment in suspension, it should not be oversized. The volume required to satisfy the minimum cycle time is dependent upon the characteristics of the power unit, the number and capacity of pumps, the sequential order in which the pumps operate and whether or not the pumps are alternated during operation.

There are two basic cycling sequences. One will be referred to here as the "common off elevation." In this sequence, the pumps start at successively higher elevations as required; however, they all stop at the same off elevation. This is advantageous when large amounts of sediment are anticipated. The other sequence uses a "successive start/stop" arrangement in which the start elevation for one pump is also the stop elevation for the subsequent pump, i.e., the start elevation for pump 1 is the stop elevation for pump 2, the start elevation for pump 2 is the stop elevation for pump 3, etc. (see Figure 12-7). There are countless variations between these two sequences.

There are also different alternation techniques which reduce the cycling volume requirement and equalize wear on the pumps. They range from simply alternating the first pump to start, to continuously alternating all pumps during operation, a technique referred to as "cyclical running alternation". Using this technique, each pump is stopped in the same order in which it starts, i.e., the first pump to start will be the first pump to stop, etc. (See Figure 12-8).

Alternating the first pump to start is sufficient for stormwater pump stations where more than one pump on will be rare and of short duration.

This alternation technique coupled with the successive start/stop cycling sequence requires the smallest total cycling volume possible (see Figure 12-9). This total volume is computed as follows:

$$V_t = Q_p t_c / 4N \quad (12.2)$$

where: V_t = total cycling storage volume, m^3 (ft^3)
 Q_p = total capacity of all pumps, m^3/s (ft^3/s)
 t_c = minimum allowable cycle time, s (= 3600/max. starts per hour)
 N = total number of equal-size pumps

The volume required for each pump will vary, depending upon the characteristics of the discharge system. It should be noted here that with these volumes, the minimum allowable cycle time will only be experienced when the proportionate inflow to each pump is exactly one-half the capacity of that pump. All other inflows will produce a cycle time longer than the minimum.

Lowest Pump "Off" Elevation

It is recommended that the lowest pump "off" elevation be located at or within 0.3 m (1 ft) below the inlet invert elevation unless plan dimension constraints dictate that the station floor be lowered to obtain the necessary cycling volume. This recommendation is based on the fact that it is usually less expensive to expand a station's plan dimensions than to increase its depth. This elevation represents the static pumping head to be used for pumping selection.

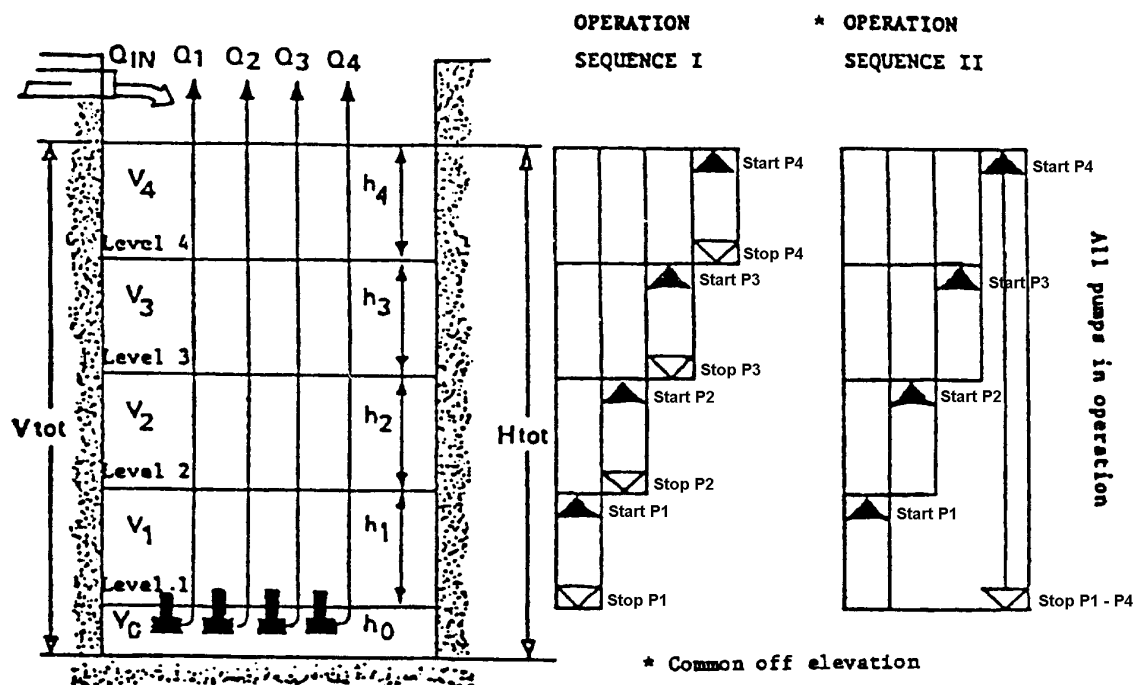
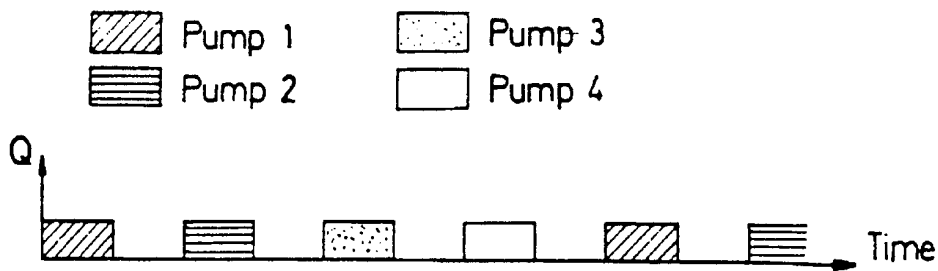


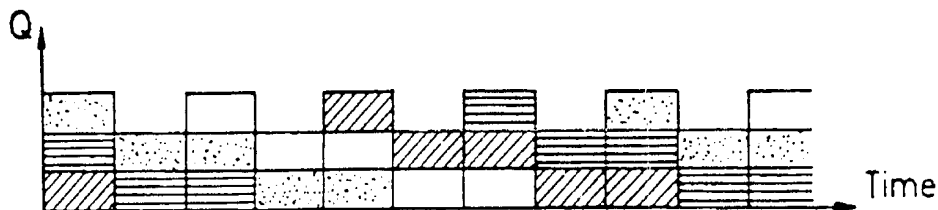
Figure 12-7 Pump Sequences

Notes to Figure 12-7.

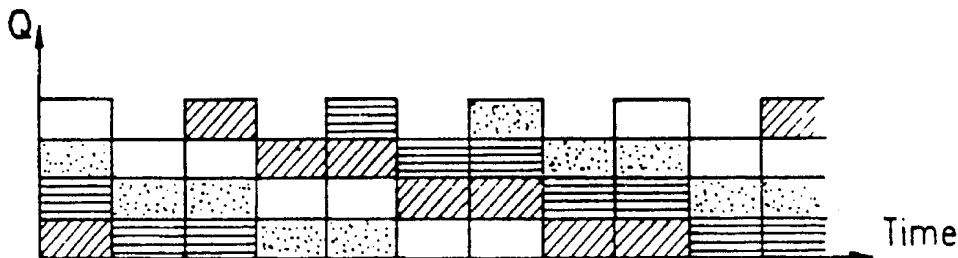
- Decreasing sump volume by pump alternation. Pumps starting in sequence and stopping in reverse order.
- By designing the control system for pump alternation, the sump volumes can be reduced as well as distribute the pump operating time more evenly between the four pumps.
- This system works for any number of pumps in a station.
- For example when four pumps are installed in the same station. If the inflow is less than the capacity of one pump, pump number one would, without alternation, do all the work.
- With alternation pump number one starts and draws down. Next start would call pump number two, etc. This means that with four pumps of the same size and operating in an alternating sequence each pump is called on to pump down sump volume V1 every fourth time. The cycle time of each pump will be four times longer than the cycle time of filling and emptying of V1.



The pumps will, however, run only 1/8 of their cycle time (provided $Q_{\text{pump}} = 2 \times Q_{\text{in}}$).

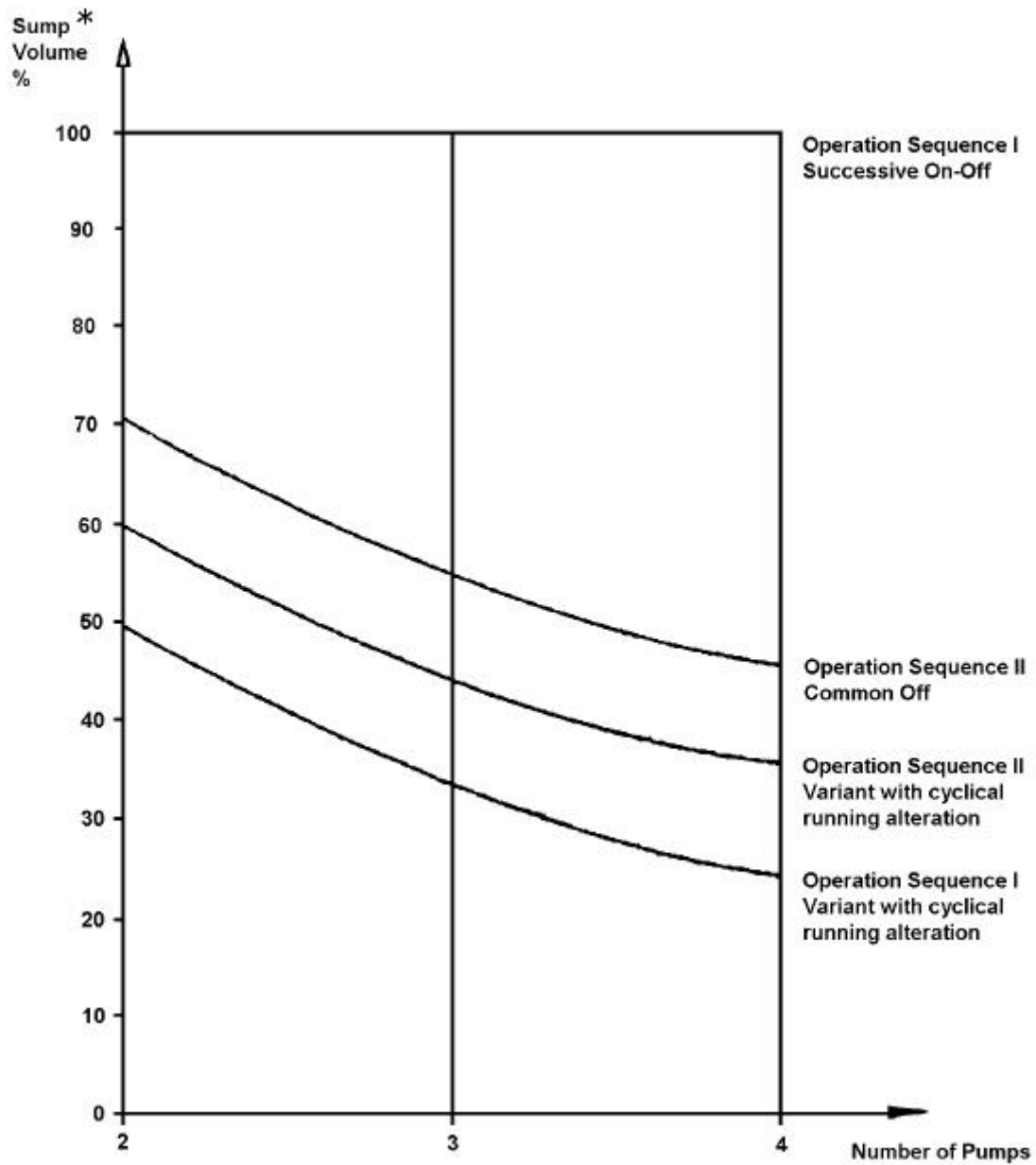


If Q_{in} is greater than the capacity of two pumps but less than three pumps, the pumps are operating 5/8 of their cycle time.



If the inflow is greater than the capacity of three pumps but less than the capacity of four pumps the pumps will operate 7/8 of their cycle time.

Figure 12-8 Pumps With Cyclical Running Alternation — A Variant Of Operation Sequence I



* 100% corresponds to the volume, which is received from the formula $V_t = (Q_p T_{min})/4N$

Figure 12-9 A Comparison Of The Pump Volume With And Without Cyclical Running Alteration

Pump "On" Elevations

These should be set at the elevations which satisfy the individual pump cycling volumes (V_x). Starting the pumps as soon as possible by incrementing these volumes successively above the lowest pump off elevation will maximize what storage is available within the wet well and the collection system. The depth required for each volume is computed as follows:

$$H_x = V_x / \text{plan area} \quad (12.3)$$

Allowable High Water Elevation

The allowable high water (AHW) elevation in the station should be set such that the water surface elevation at the lowest inlet in the collection system provides 0.3 to 0.6 m (1 to 2 ft) of freeboard below the roadway grate.

Clearances

Pump to pump, pump to backwall, and pump to sidewall clearances should be the minimum possible to minimize the potential for sedimentation problems. Consult manufacturer or a dimensioning guide. The pump inlet to floor clearance plus the pump submergence requirement constitutes the distance from the lowest pump "off" elevation to the wet well floor. The final elevation may have to be adjusted if the type of pump to be installed is different than anticipated.

Intake System Design

The primary function of the intake structure is to supply an even distribution of flow to the pumps. An uneven distribution may cause strong local currents resulting in reduced pump efficiency and undesirable operational characteristics. The ideal approach is a straight channel coming directly into the pump or suction pipe. Turns and obstructions are detrimental, since they may cause eddy currents and tend to initiate deep cored vortices. The inflow should be perpendicular to a line of pumps and water should not flow past one pump to get to another. Unusual circumstances will require a unique design of the intake structure to provide proper flow to the pumps.

12.5.10 Stormwater Pump Station Storage

The development of the wetwell design as discussed in Section 12.5.9 has general application when it is anticipated that most of the peak flow will be pumped. In that case, pump run time and cycling sequences are of great importance. In the case of many of the highway storm drain situations, it has been the practice to store substantial parts of the flow in order to minimize pumping requirements as well as outflow piping. The demands on the pumping system are different and thus additional considerations should be made.

The designer should recognize that a balance should be reached between pump rate and storage volume. This will require a trial and error procedure used in conjunction with an economic analysis. Pump stations are very costly and alternatives to minimize total costs need to be considered.

The principles discussed for minimum run time, pump cycling, etc. in the design of wet wells should also be considered in the case of larger storage volume development. However, it will be noted that differences exist as the volume of storage become larger. Typically, the concern for

meeting minimum run times and cycling time will be reduced because the volume of storage is sufficient to prevent these conditions from controlling the pump operation. The start and stop elevations will be of different magnitudes because of the volume represented by each increment of storage depth.

The approach used for the design of the pump station will be that associated with the development of an inflow mass curve. In this process, the designer will need to have an inflow hydrograph and a developed stage-storage relationship. Trial pumping systems will be imposed on the inflow mass curve to develop a mass curve routing diagram. The inflow hydrograph is a fixed design component while the storage and pumping discharge rates are variable. The designer may assign a pumping discharge rate based on downstream capacity considerations, limits imposed by local jurisdictions, etc. It is becoming a common requirement that post-development discharges not exceed pre-development discharges. This requirement can most often be met with a design that includes storage. With the inflow mass curve and an assigned pumping rate, the required storage can be determined by various trials of the routing procedure.

As the stormwater flows into the storage basin, it will accumulate until the first pump start elevation is reached. The first pump is activated and if the inflow rate is greater than the pump rate, the stormwater will continue to accumulate until the second pump start elevation is reached. As the inflow rate decreases, the pumps will shut off at their respective pump stop elevations.

These conditions are modeled in the mass curve diagram by establishing the point at which the cumulative flow curve has reached the storage volume associated with the first pump-start elevation. This storage volume is represented by the vertical distance between the cumulative flow curve and the base line. A vertical storage line is drawn at this point since it establishes the time at which the pump first starts.

The pump discharge line is drawn from the intersection of the vertical storage line and the base line upwards toward the right; the slope of this line is equal to the discharge rate of the pump. The pump discharge curve represents the cumulative discharge from the storage basin, while the vertical distance between the inflow mass curve and the pump discharge curve represents the amount of stormwater stored in the basin.

If the rate of inflow is greater than the pump capacity, the inflow mass curve and the pump discharge curve will continue to diverge until the volume of water in storage is equal to the storage associated with the second pump-start elevation. At this point the second pump starts, and the slope of the pump discharge line is increased to equal the combined pumping rates.

The procedure continues until peak storage conditions are reached. At some point on the inflow mass curve, the inflow rate will decrease, and the slope of the inflow mass curve will flatten. To determine the maximum amount of storage required, a line is drawn parallel to the pump discharge curve and tangent to the inflow mass curve. The vertical distance between the lines represents the maximum amount of storage required.

The routing procedure continues until the pump discharge curve intersects the inflow mass curve. At this point the storage basin has been completely emptied, and a pumping cycle has been completed. As the storm recedes, the pumps will cycle to discharge the remaining runoff.

In developing the pump discharge curve, the designer should remember that the pump's performance curve is quite sensitive to changes in head and that the static head will fluctuate as the water level in the storage basin fluctuates. The designer should also recognize that the pump discharge rate represents an average pumping rate.

Example: Determine the required storage to reduce the peak flow of $0.62 \text{ m}^3/\text{s}$ to $0.40 \text{ m}^3/\text{s}$ as shown in Figure 12-10. Using the assumed storage pipe shown in Figure 12-11, the stage-storage curve in Figure 12-12, the stage discharge curve in Figure 12-13 and the inflow hydrograph in Figure 12-10, the storage can be determined.

The inflow mass curve is developed in Figure 12-14. Since $0.40 \text{ m}^3/\text{s}$ was to be pumped, it was assumed that two $0.2\text{-m}^3/\text{s}$ pumps would be used. The pumping conditions are as follows:

	<u>Pump-Start Elevation</u>	<u>Pump-Stop Elevation</u>
Pump No. 1 ($0.2 \text{ m}^3/\text{s}$)	0.61 (57.3)	0.0 (0)
Pump No. 2 ($0.2 \text{ m}^3/\text{s}$)	0.91 (119.7)	0.30 (16.9)

The numbers in parentheses are the storage volumes (m^3) associated with the respective elevations.

Figure 12-15 shows the plotting of the pump discharge curve on the inflow mass diagram. Note that the first pump is turned on at about hour 11.4 when a storage volume of 57 m^3 has accumulated. At about hour 11.5 pump number one has emptied the storage basin and the pump turns off. At about hour 11.7 the storage volume has again reached 57 m^3 and a pump is turned on. If an alternating start plan had been developed, this would be the second pump that would turn on at this point. If an alternating start plan had not been designed the first pump would again be started. At about hour 11.8 the volume in storage has increased to 120 m^3 which is associated with a turn on elevation of 0.91 m . Both pumps operate until about hour 12.4 when the volume in the storage basin has been essentially pumped out. The pumps will continue to start and stop until the hydrograph has receded and the inflow stops.

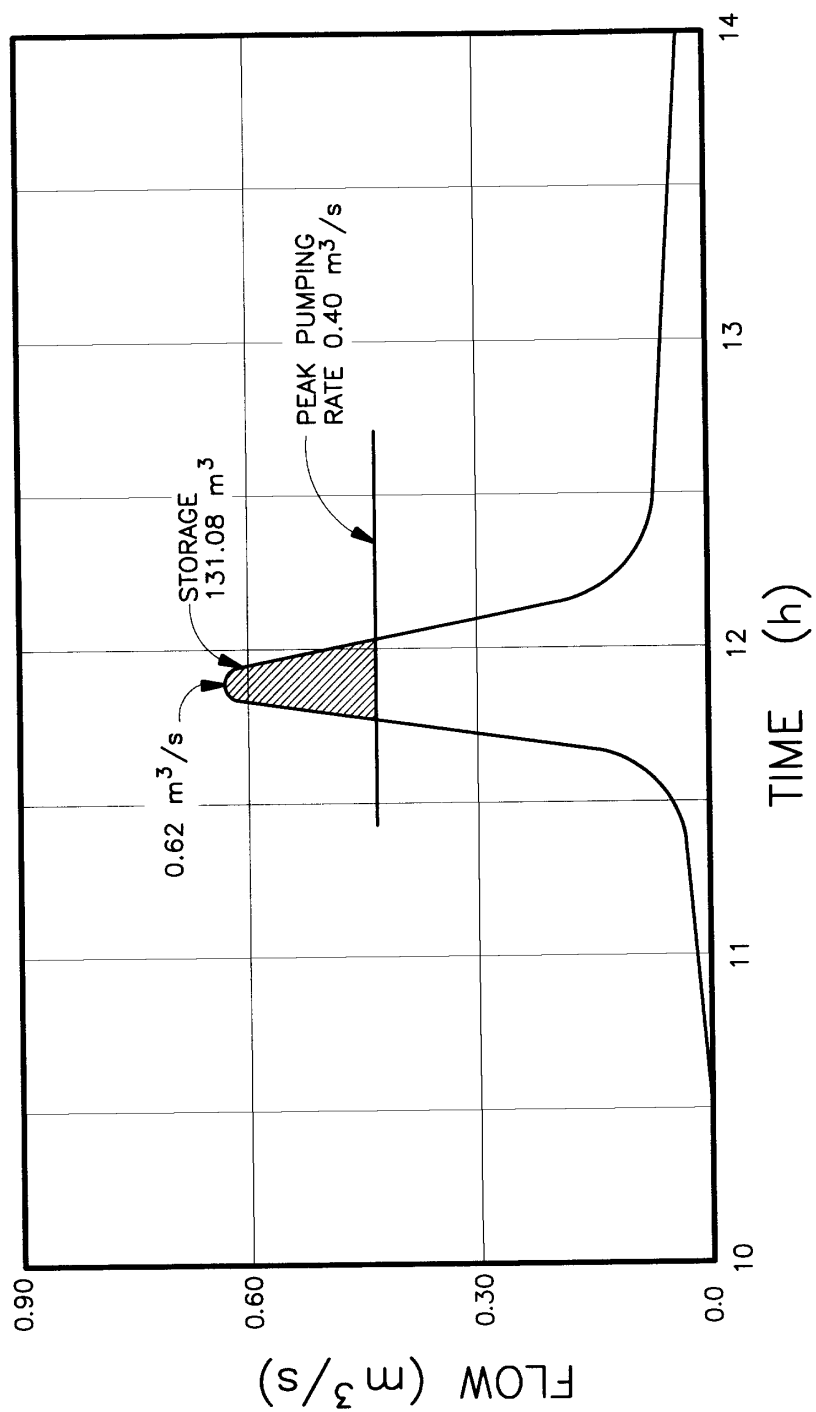


Figure 12-10 Estimating Required Storage

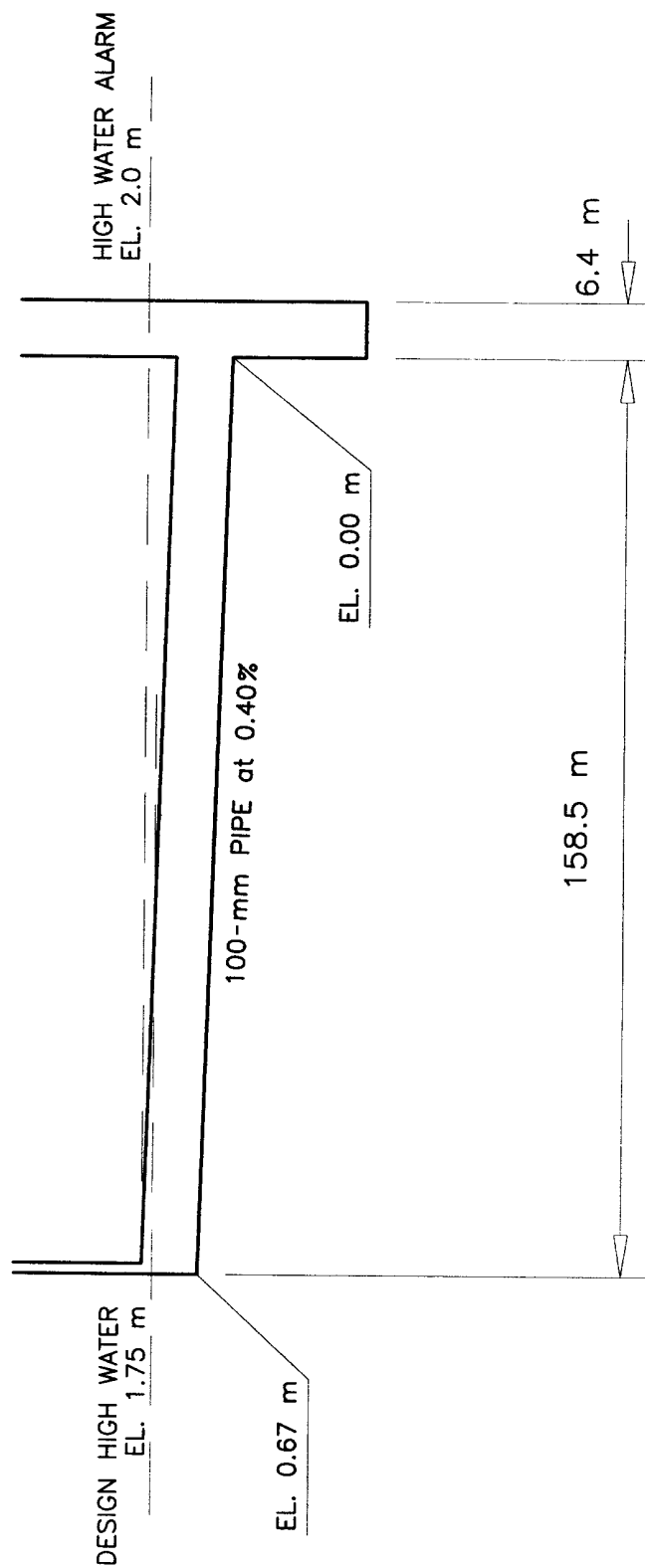


Figure 12-11 Storage Pipe Sketch

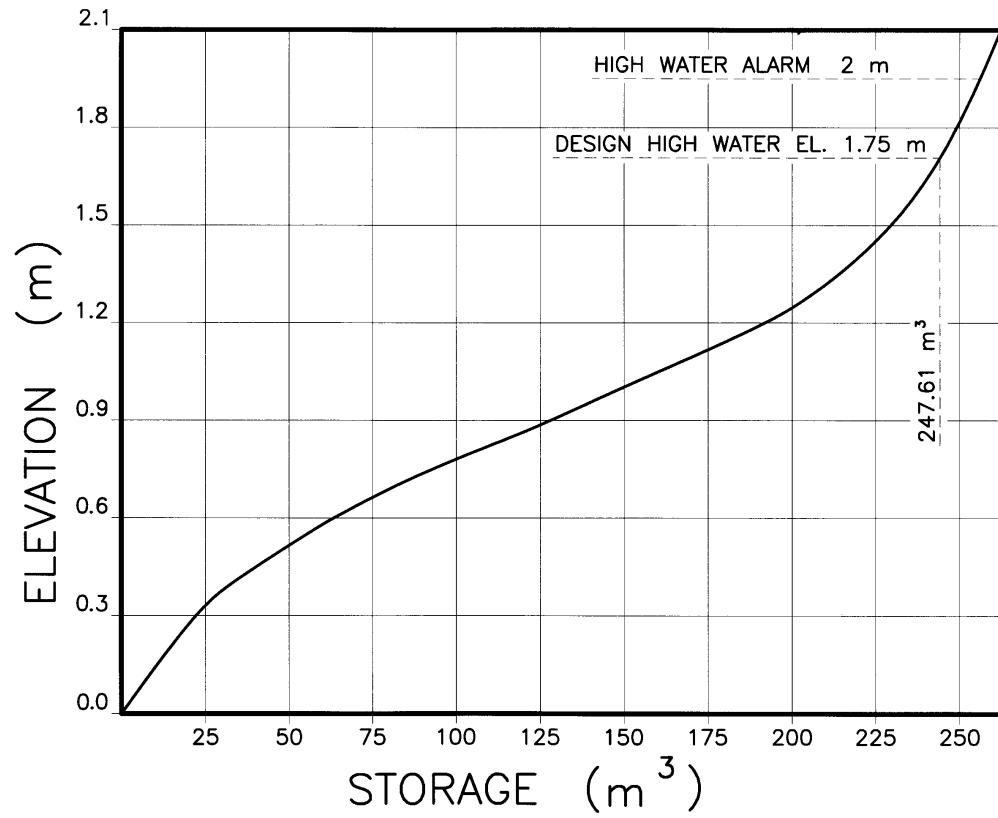


Figure 12-12 Stage-Storage Curve

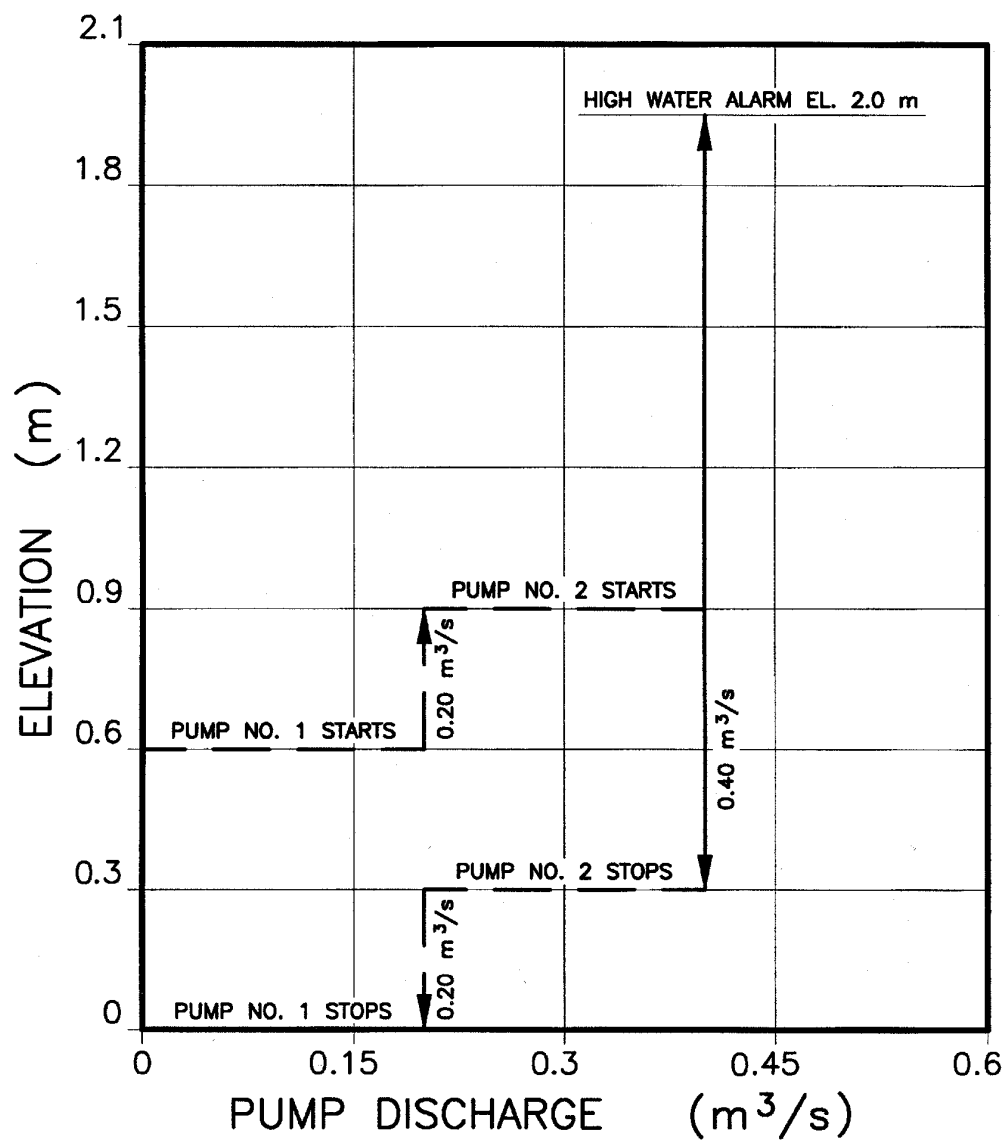


Figure 12-13 Stage-Discharge Curve

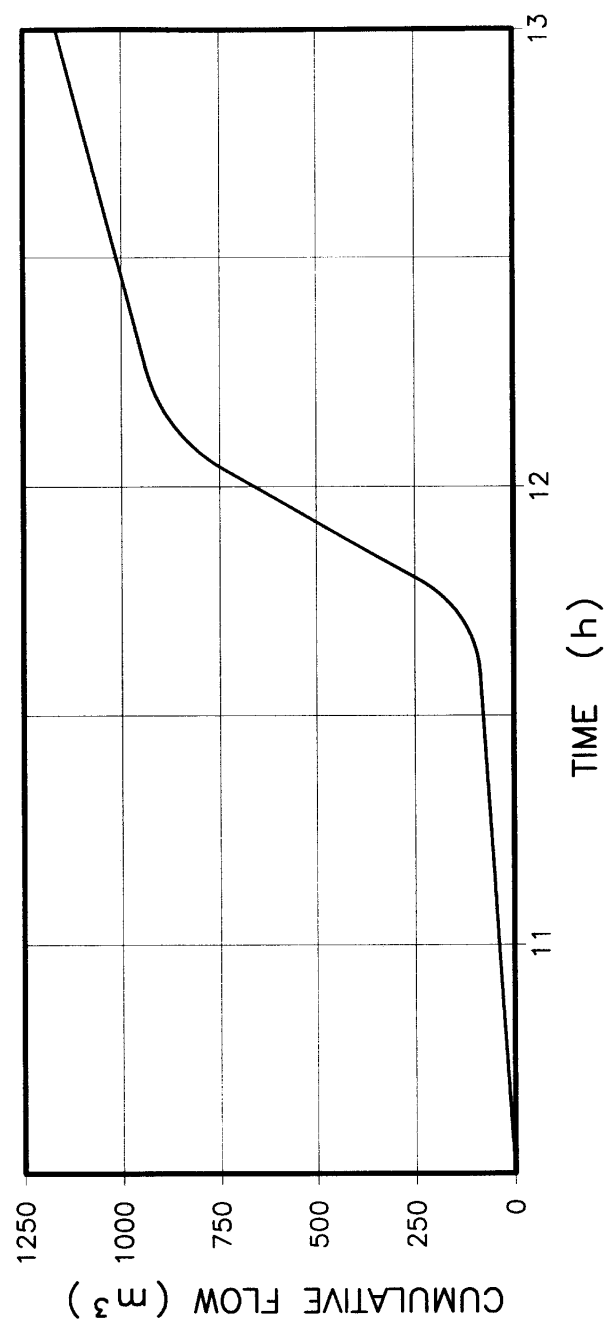


Figure 12-14 Development Of Inflow Mass Curve

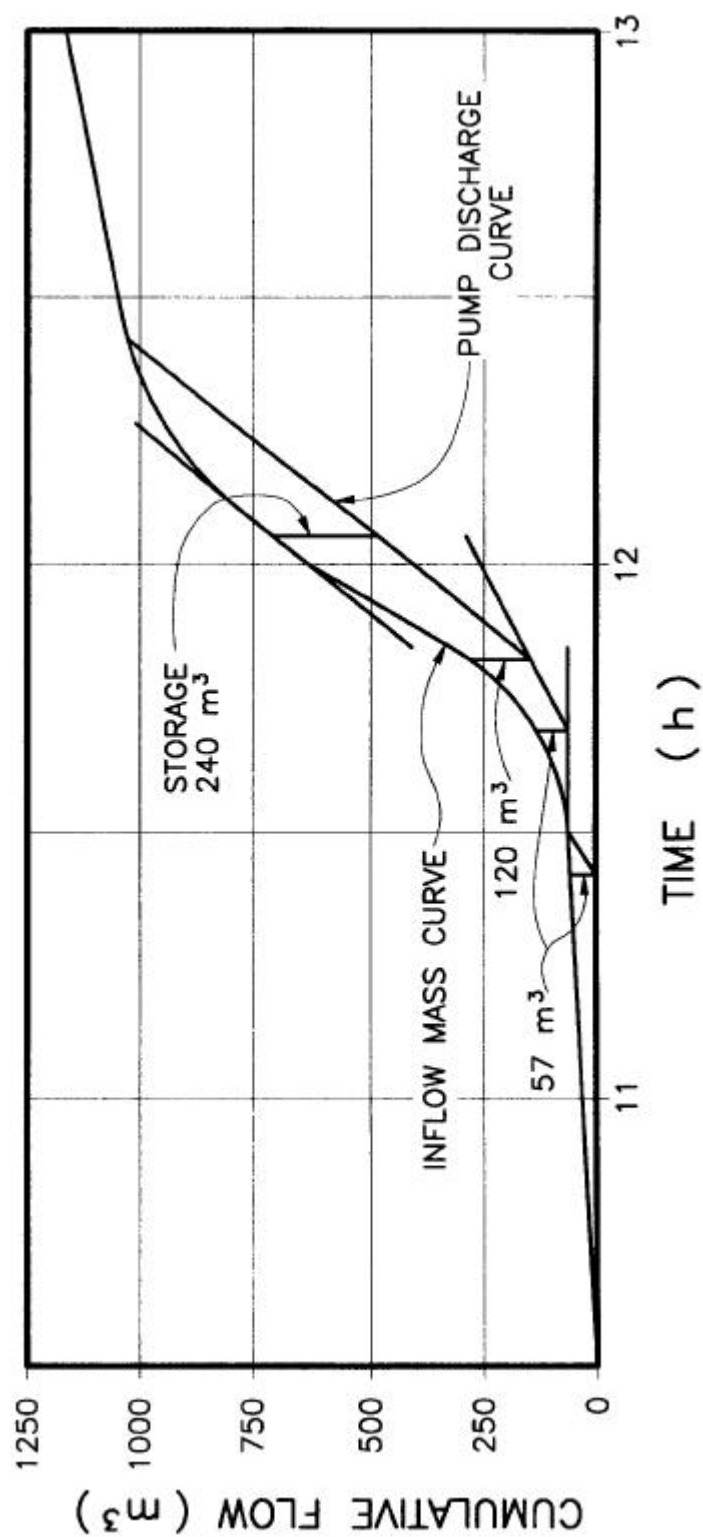


Figure 12-15 Mass Curve Routing Diagram

The shaded area between the curves (see Figure 12-16) represents stormwater that is going into storage. Pump cycling at the end of the storm has been omitted in order to simplify the illustration. When the stored volume remaining is equal to the volume (17 m^3) associated with the Pump No. 2 stop elevation (0.3 m), pump number 2 shuts off; Pump No. 1 shuts off when the storage pipe is emptied at Pump No. 1 stop elevation (0.0).

The maximum vertical distance between the pump discharge curve and the inflow mass hydrograph is 240 m^3 . This represents the maximum storage required for the reduction of the $0.62\text{-m}^3/\text{s}$ peak to $0.40 \text{ m}^3/\text{s}$ for the defined conditions, i.e., the storage volume as defined, the start elevations defined and the pump rates defined. The design is adequate since the available storage at the high water alarm is 249 m^3 . It should be noted that a reduction of the starting elevations would have reduced the required storage volume. The designer must make these adjustments on a trial basis until a satisfactory operating condition is developed. Other pumping rates could have also been plotted on the inflow mass curve to determine their performance. Once the mass inflow curve has been developed, it is a relatively easy process to try different pumping rates and different starting elevations until a satisfactory design is developed. It should be noted that number of starts per hour can be determined by looking at the plots on the inflow mass curve. The system should be designed so that the allowable number of starts per hour for the selected pump size is not exceeded.

The designer now has a complete design that allows the problem to be studied in-depth. The peak rate of runoff has been reduced from $0.62 \text{ m}^3/\text{s}$, the inflow hydrograph peak, to $0.40 \text{ m}^3/\text{s}$, the maximum pump discharge rate. A reduction of 46.5% is accomplished by providing for 240 m^3 of storage. This is only one possible design option. The designer may wish to reduce the pumping rate further by providing more storage, and additional combinations of pump discharge and storage can be considered.

To aid the designer in visualizing what is happening during the peak design period in this process, the pump discharge curve can be superimposed on the design inflow hydrograph as shown in Figure 12-16.

12.6 Design Procedure

12.6.1 Introduction

The following is a systematic procedure which integrates the hydraulic design variables involved in sump design. It incorporates the above recommended design criteria and yields the required number and capacity of pumps as well as the wet well and storage dimensions. The final dimensions can be adjusted as required to accommodate non-hydraulic considerations such as maintenance. Though the recommended station is a wet-pit, this procedure can be adapted for use in designing dry-pit stations.

Theoretically an infinite number of designs are possible for a given site. Therefore, to initiate design, constraints must be evaluated and a trial design formulated to meet these constraints. Then by routing the inflow hydrograph through the trial pump station its adequacy can be evaluated.

The hydraulic analysis of a pump station involves the interrelationship of 3 components:

- the inflow hydrograph
- the storage capacity of the wet well and the outside storage
- the discharge rate of the pumping system

The inflow hydrograph is determined by the physical factors of the watershed and regional climatological factors. The discharge of the pump station is often controlled by local regulations or physical factors. Therefore, the main objective in pump station design is to store enough inflow (volume of water under the inflow hydrograph) to allow station discharge to meet specified limits. Even if there are no physical limitations to pump station discharge, storage should always be considered since storage permits use of smaller and/or fewer pumps.

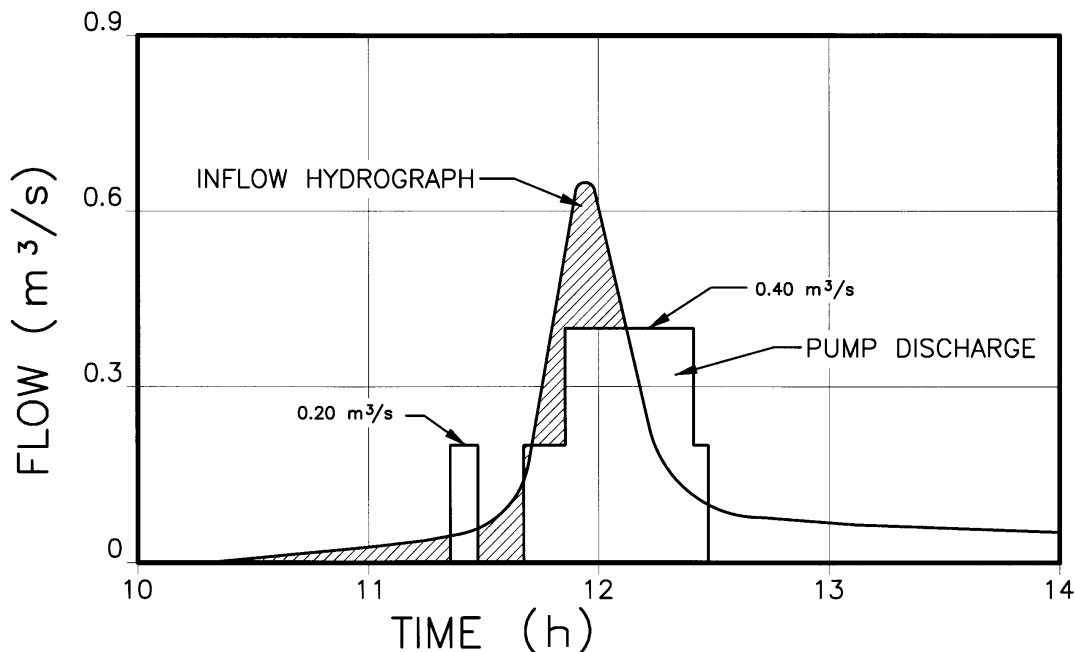


Figure 12-16 Pump Discharge

12.6.2 Pump Station Design

The procedure for pump station design is illustrated in the following 10 steps (metric units).

Step 1 Inflow to Pump Station

Develop inflow hydrograph to the pump station using the procedures presented in Chapter 6, Hydrology.

Step 2 Estimate Pumping Rate, Volume of Storage and Number of Pumps

Because of the complex relationship between the variables of pumping rates, storage and pump on-off settings, a trial and error approach is usually necessary for estimating the pumping rates and storage required for a balanced design. A wide range of combinations will produce an adequate design. The goal is to develop an economic balance between volume and pumping capacity.

Some approximation of all three parameters is necessary to produce the first trial design. One approach is shown in Figure 12-6. In this approach, the peak pumping rate is assigned and a horizontal line representing the peak rate is drawn across the top of the hydrograph. The shaded area above the peak pumping rate represents an estimated volume of storage required above the last pump turn on point. This area is measured to give an estimated starting size for the storage facility. Once an estimated storage volume is determined, a storage facility can be estimated. The shape, size, depth, etc., can be established to match the site and a stage-storage relationship can be developed.

The total pumping rate may be set by stormwater management limitations, capacity of the receiving system, the desirable pump size, or available storage. Two pumps would be the minimum number of pumps required. However, as many as five pumps may be needed in the case of a continuously depressed highway situation. Size and thus numbers of pumps may be controlled by physical constraints such as portable standby power as discussed in Section 12.5.3.

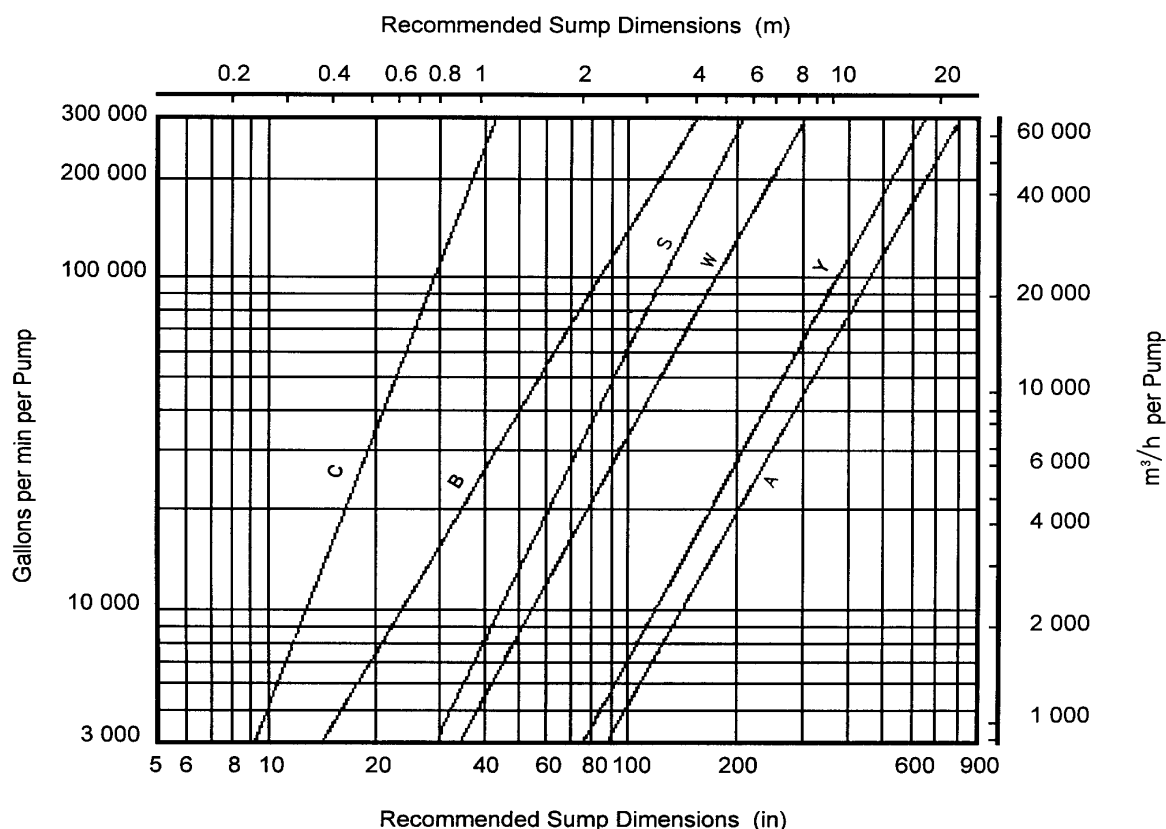
Step 3 Design High Water Level

The highest permissible water level must be set as 0.3 to 0.6 m (1 to 2 ft.) below the finished pavement surface at the lowest pavement inlet. The lower the elevation the more conservative the design.

At the design inflow, some head loss will occur through the pipes and appurtenances leading to the pump station. Therefore a hydraulic gradient will be established and the maximum permissible water elevation at the station will be the elevation of the hydraulic gradient. This gradient will be very flat for most wet well designs with exterior storage because of the unrestricted flow into the wet well.

Step 4 Determine Pump Pit Dimensions

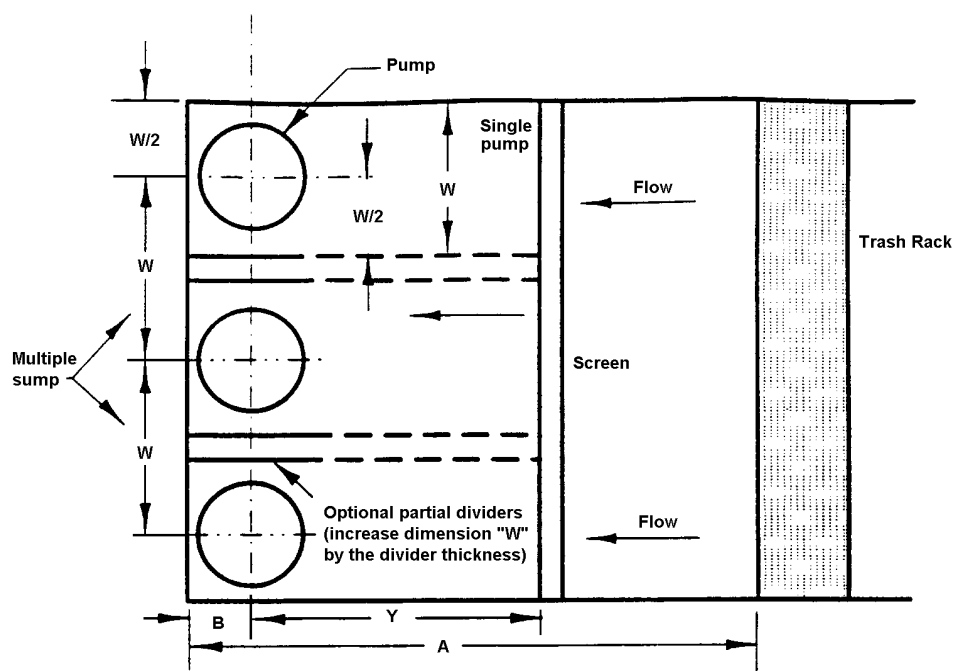
Determine the minimum required plan dimensions for the pump station from manufacturer's literature or from dimensioning guides such as those provided by the Hydraulic Institute, see Figures 12-17 and 12-18. The dimensions are usually determined by locating the selected number of pumps on a floor plan keeping in mind the guidance given in Section 12.5.9 for clearances and intake system design. Keep in mind the need for clearances around electrical panels and other associated equipment that will be housed in the pump station building.



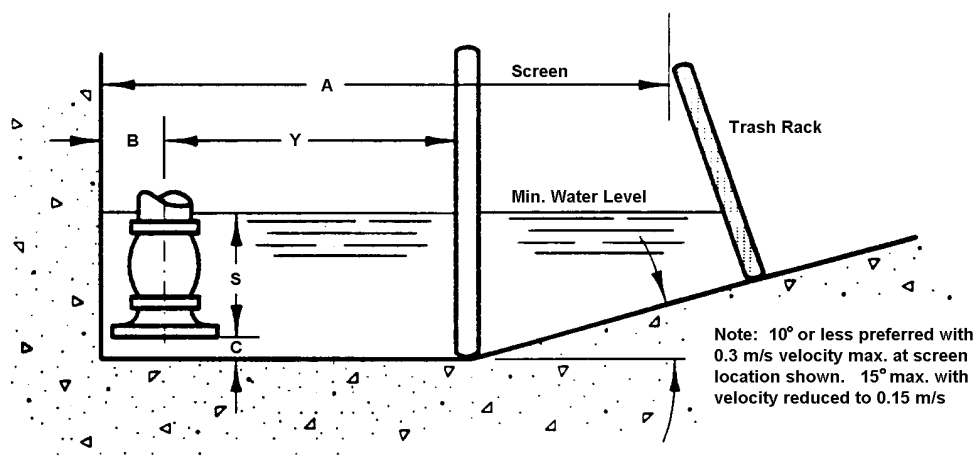
The above graph is reproduced from Hydraulic Institute Standards.

- A = Minimum distance from trash rack to backwall (length of pump pit).
- B = Maximum distance from centerline of pump to backwall
- C = Average dimension from underside of bell to bottom of pit
- S = Minimum dimension from minimum water level to underside of bell
- W = Minimum center-to-center spacing of pumps
- Y = Minimum distance to pump centerline from downstream end of any obstruction in sump (obstruction must be streamlined)

Figure 12-17 Recommended Pump Pit Dimensions



Plan View



Elevation View

Figure 12-18 Sump Dimensions, Plan and Elevation View, Wet Pit Type Pumps

Step 5 Stage-Storage Relationship

Routing procedures require that a stage-storage relationship be developed. This is accomplished by calculating the available volume of water for storage at uniform vertical intervals.

Having roughly estimated the volume of storage required and trial pumping rate by the approximate methods described in the preceding sections, the configuration and elevations of the storage chamber can be initially set. Knowing this geometry, the volume of water stored can be calculated for its respective depth. In addition to the wet-pit, storage will also be provided by the inflow pipes and exterior storage if the elevation of water in the wet-pit is above the inflow invert. If the storage pipe is circular, the volume can be calculated using the ungula of a cone formula as discussed in Figure 12-19. Figures 12-19, 12-20 and 12-21 give examples of the calculation and plotting of the storage in a circular pipe and a circular wet pit. A similar procedure would be followed for other storage configurations. Volume in a storage chamber can be calculated below various elevations by formulas depending on the shape of the chamber. A storage vs. elevation curve can then be plotted and storage below any elevation can readily be obtained.

Ungula Volume: $V_3 = H(0.67a^3 \pm cB) / (r \pm c)$

If base is greater than a semicircle, use + sign. If base is less than a semicircle, use - sign.

Where: H = length of ungula, m
 r = radius of base, m
 B = area of base, m²

May use King & Brater table, for determining area B of the cross section of a circular conduit flowing part full.

Let (Depth of water)/(Diameter of channel) = D/d and C_a = the tabulated value.
 Then $B = C_a d^2$

D/d	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0013	0.0037	0.0069	0.0105	0.0417	0.0192	0.0242	0.0294	0.0350
0.1	0.0409	0.0470	0.0534	0.0600	0.0668	0.0739	0.0811	0.0885	0.0961	0.1039
0.2	0.1118	0.1199	0.1281	0.1365	0.1449	0.1535	0.1623	0.1711	0.1800	0.1890
0.3	0.1982	0.2074	0.2167	0.2260	0.2355	0.2450	0.2546	0.2642	0.2739	0.2836
0.4	0.2934	0.3032	0.3130	0.3229	0.3328	0.3428	0.3527	0.3627	0.3727	0.3827
0.5	0.393	0.403	0.413	0.423	0.433	0.443	0.453	0.462	0.472	0.482
0.6	0.492	0.502	0.512	0.521	0.531	0.540	0.550	0.559	0.569	0.578
0.7	0.587	0.596	0.605	0.614	0.623	0.632	0.640	0.649	0.657	0.666
0.8	0.674	0.681	0.689	0.697	0.704	0.712	0.719	0.725	0.732	0.738
0.9	0.745	0.750	0.756	0.761	0.766	0.771	0.775	0.779	0.782	0.784

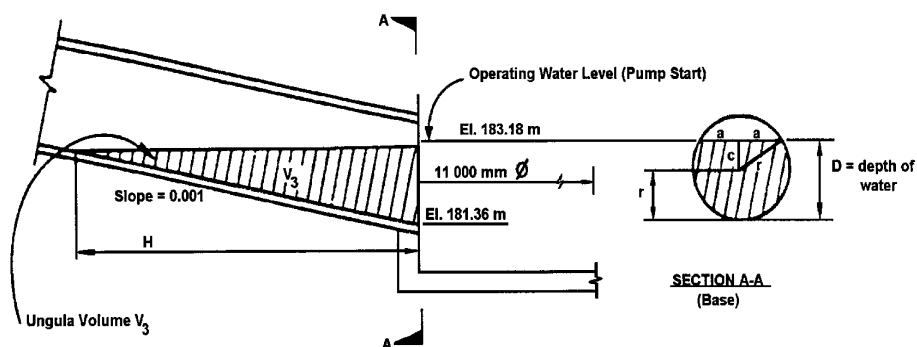


Figure 12-19 Storage in Ungula

Source: FHWA IP-82-17, Vol. 1

Stage-Storage Tabulation
1200 mm Pipe at 0.40%, 6400 mm Diameter Wet Well

Elevation (m)	Pipe (m ³)	Wet Well (m ³)	Total (m ³)
0.00	0.00	0.00	0.00
0.15	1.27	4.90	6.17
0.30	7.11	9.80	16.91
0.46	19.03	14.70	33.73
0.61	37.75	19.60	57.35
0.76	62.67	24.52	87.19
0.91	90.25	29.42	119.67
1.07	118.02	34.32	152.34
1.22	143.62	39.22	182.84
1.37	163.47	44.15	207.62
1.52	176.41	49.04	225.45
1.68	183.15	53.94	237.09
1.83	185.02	58.84	243.86
1.98	185.02	63.74	248.76
2.13	185.02	68.64	253.66

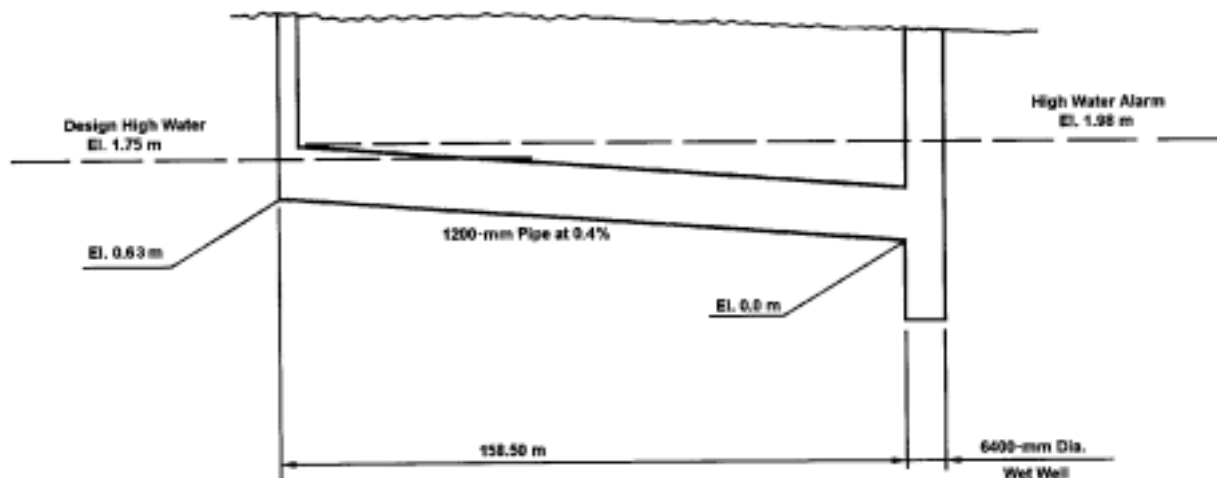


Figure 12-20 Storage Pipe Sketch

Source: FHWA IP-82-17, Vol. 1

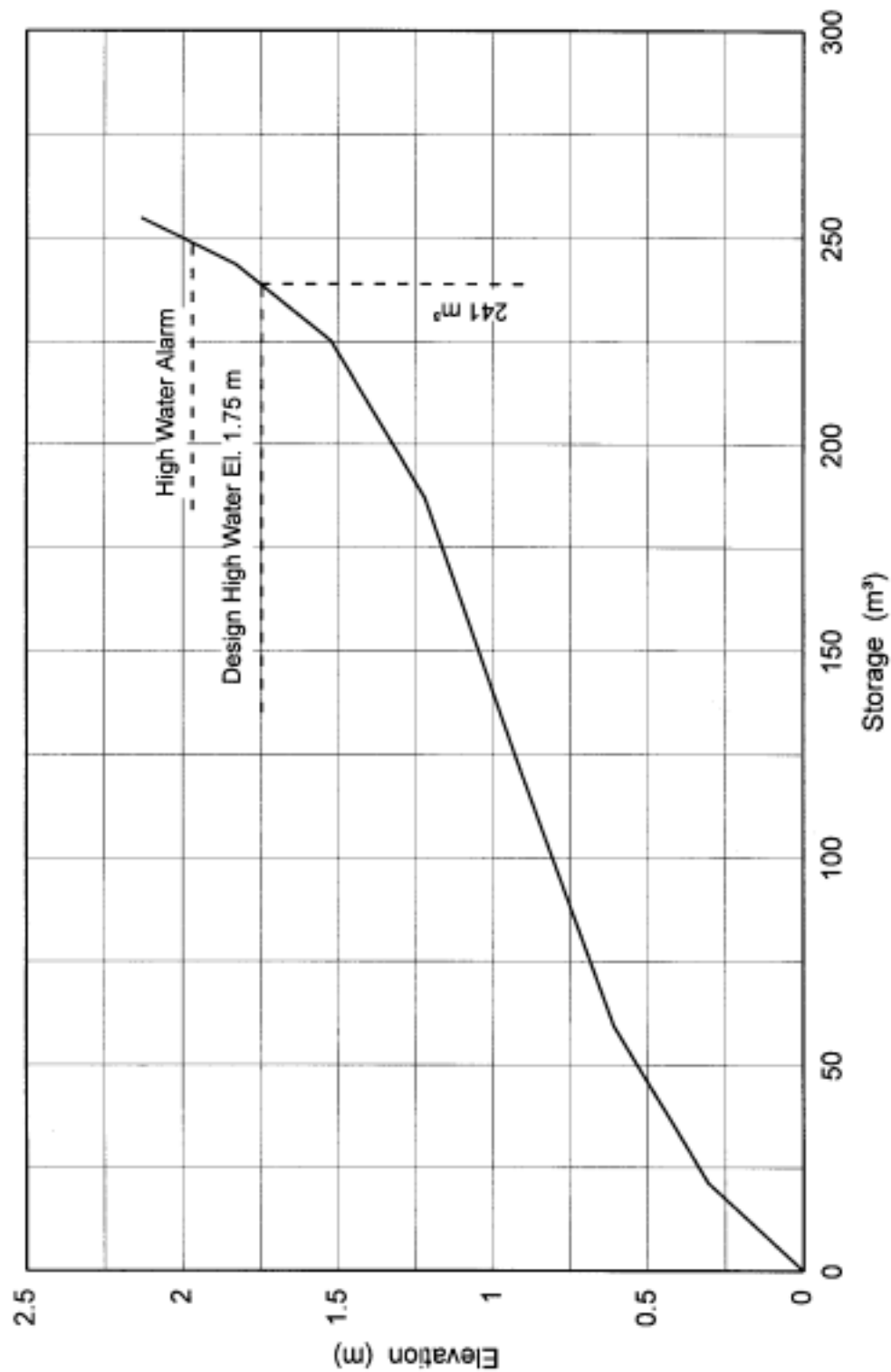


Figure 12-21 Stage-Storage Curve

Source: FHWA IP-82-17, Vol. 1

Step 6 Pump Cycling and Usable Storage

One of the basic parameters addressed initially was that the proper number of pumps must be selected to deliver the design Q . Also, the correct elevations must be chosen to turn each pump on and off. Otherwise, rapid cycling (frequent starting and stopping of pumps) may occur causing undue wear and possible damage to the pumps.

Before discussing pump cycling calculations, operation of a pump station will be described. Initially, the water level in the storage basin will rise at a rate depending on the rate of the inflow and physical geometry of the storage basin. When the water level reaches the stage designated as the first pump start elevation, the pump will be activated and discharge water from storage at its designated pumping rate. If this rate exceeds the rate of inflow, the water level will drop until it reaches the first pump stop elevation. With the pump stopped, the basin begins to refill and the cycle is repeated. This scenario illustrates that the cycling time will be lengthened by increasing the amount of storage between pump on and off elevations. This volume of storage between first pump on and off elevations is termed usable volume. In theory, the minimum cycle time allowable to reduce wear on the pumps will occur when the inflow to the usable storage volume is one-half the pump capacity. Assuming this condition, cycling time can be related to usable volume.

For a given pump with a capacity Q_p , cycling will be a maximum (least time between starts) when the inflow Q_i to the usable storage is one-half the pump capacity. The proof is as follows:

t = Time between starts

t = Time to Empty + Time to Fill usable storage volume V_t

$$t = V_t / (Q_p - Q_i) + V_t / Q_i \quad \text{When } Q_i = Q_p / 2, \text{ m}^3/\text{sec}, (\text{ft}^3/\text{sec}) \quad t = 4V_t / Q_p, \text{ sec} \quad (12.4)$$

$$\text{or } t \text{ in minutes } t = 4V_t / 60Q_p = V_t / 15Q_p$$

Generally, the minimum allowable cycling time, t , is designated by the pump manufacturer based on electric motor size. In general, the larger the motor, the larger is the starting current required, the larger the damaging heating effect and the greater the cycling time required. The pump manufacturer should always be consulted for allowable cycling time during the final design phase of project development.

However, the following limits may be used for estimating allowable cycle time during preliminary design:

Motor kW	Cycling Time (t), min
0 - 11	5.0
15 - 22	6.5
26 - 45	8.0
48 - 75	10.0
112 - 149	13.0

Knowing the pumping rate and minimum cycling time, the minimum necessary allowable storage, V , to achieve this time can be calculated by:

$$V = 15 Q_p t \quad (12.5)$$

Having selected the trial wet-pit dimensions, the pumping range, Δh , can then be calculated. The pumping range represents the vertical height between pump start and pump stop elevations. Usually, the first pump stop elevation is controlled by the minimum recommended bell submergence criteria specified by the pump manufacturer or the minimum water level, H , specified in the design. The first pump start elevation will be a distance, Δh , above H .

When the only storage provided is in the wet pit, the pumping range can be calculated by dividing the allowable storage volume by the wet pit area.

$$\Delta h = V / \text{wet pit area} \quad (12.6)$$

When larger volumes of storage are available, the initial pump start elevations can be selected from the stage-storage curve. Since the first pump turned on should typically have the ability to empty the storage facility, its turn off elevation would be the bottom of the storage basin. The minimum allowable storage would be calculated by the equation $V=15 Q_p t$. The elevation associated with this volume in the stage-storage curve would be the lowest turn-on elevation that should be allowed for the starting point of the first pump. The second and subsequent pump start elevations will be determined by plotting the pump performance on the mass inflow curve.

This distance between pump starts may be in the range of 0.3 to 1 m (1 to 3 ft) for stations with a small amount of storage and 0.07 to 0.15 m (0.2 ft to 0.5 ft) for larger storage situations.

Step 7 Trial Pumps And Pump Station Piping

The designer must select a specific pump in order to establish the size of the discharge piping that will be needed. This is done by using information either previously developed or established. Though the designer will not typically specify the manufacturer or a specific pump, he/she must study various manufacturers' literature in order to establish reasonable relationships between total dynamic head, discharge, efficiency and energy requirements. This study will also give the designer a good indication of discharge piping needed since pumps that produce the desired results will have a specific discharge pipe size.

Any point on an individual performance curve identifies the performance of a pump for a specific Total Dynamic Head (TDH) that exists in the system. It also identifies the horse power required and the efficiency of operation of the pump. It can be seen that for either an increase or decrease in TDH, the efficiency is reduced as the performance moves away from the eye of the performance curve. It should also be noted that as the TDH increases, the horse power requirement also increases. The designer must make certain that the motor specified is adequate over the full range of TDHs that will exist. It is desirable that the design point be as close to the eye as possible, or else to the left of the eye rather than to the

right of or above it. The range of the pump performance should not extend into areas where substantially reduced efficiencies exist.

Step 8 Total Dynamic Head

Total Dynamic Head is the sum of the static head, velocity head and various head losses in the pump discharge system due to friction. Knowing the range of water levels in the storage pit and having a trial pump pit design with discharge pipe lengths and diameters and appurtenances such as elbows and valves designated, total dynamic head for the discharge system can be calculated. To summarize the Total Dynamic Head (TDH) is equal to:

$$TDH = H_s + H_f + H_v + H_p$$

Where: H_s = static head or height through which the water must be raised, m (ft)

H_f = loss due to friction in the pipe, m (ft)

H_v = velocity head, m (ft)

H_p = loss due to friction in water passing through the pump valves, fittings and other items, m (ft)

The Manning's formula expressed as follows is generally used for discharge lines.

$$H_f = [L(Q \times n)^2] / [(A \times R)^{2/3}] \quad (12.7)$$

Where: Q = discharge, m^3/s (ft^3/s)

L = length of pipe, m (ft)

n = Manning's roughness value

A = cross sectional area of discharge pipe, m^2 (ft^2)

R = hydraulic radius of discharge pipe, m (R = diameter/4 for line running full)

Friction losses can also be computed by the Darcy Formula. This requires computation of the relative roughness of the pipe, the Reynold's number and the friction factor. The Hydraulic Institute and others have produced line loss tables and charts that make determination of losses quite easy and accurate. The tables and charts have been developed for a variety of pipe materials and are recommended for use in determining line and fitting losses for the discharge side of the pumping system.

Consult the Storm Drainage System Chapter for methods of determining head loss in various components of the system. Standard textbooks and manufacturers' catalogs should also be consulted.

Step 9 Pump Design Point

Using methods described in the previous step, the Total Dynamic Head of the outlet system can be calculated for a specific static head and various discharges. These TDHs are then plotted vs. discharge. This plot is called a system head curve. A system head curve is a graphical representation of total dynamic head plotted against discharge Q for the entire pumping and discharge system. The required design point of a pump can be established after the pump curve is superimposed to give a visual representation of both system and pump. As usually drawn, the system head curve starts from a low point on the Y-ordinate representing the static head at zero discharge. It then rises to the right as the discharge and the friction losses increase. A design point can be selected on the system head curve and a pump can be selected to match that point. The usual pump curve is the reverse of the system head curve so the point of intersection is clearly identifiable. System head curves are often drawn for several different static heads, representing low, design and maximum water levels in the sump. One, two or more pump curves can be plotted over the system head curves and conditions examined. If a change of discharge line size is contemplated, a new system head curve for the changed size (and changed head loss) is easily constructed. Figure 12-22 shows the same system with one pump and then with two identical pumps delivering into the same system. Note the increased head loss and reduced capacity of each pump when both are operating. This condition will only exist when a common discharge pipe is shared by both pumps. In highway design, it is common practice to provide individual discharge lines for each pump.

Therefore, the additional loss from a shared pipe is not experienced. It should be noted that the pump will always operate at the intersection of the system curve and the pump curve.

Each pump considered will have a unique performance curve that has been developed by the manufacturer. More precisely, a family of curves is shown for each pump, because any pump can be fitted with various size impellers. These performance curves are the basis for the pump curve plotted in the system head curves discussed above. The designer must have specific information on the pumps available in order to be able to specify pumps needed for the pump station. A study of pump performance curves should be made by all designers.

Any point on an individual performance curve identifies the performance of a pump for a specific Total Dynamic Head (TDH) that exists in the system. It also identifies the power required and the efficiency of operation of the pump. It can be seen that for either an increase or decrease in TDH, the efficiency is reduced as the performance moves away from the eye of the performance curve. It should also be noted that as the TDH increases, the power requirement also increases. The designer must make certain that the motor specified is adequate over the full range of TDHs that will exist. It is desirable that the design point be as close to the eye as possible, or else to the left of the eye rather than to the right of or above it. The range of the pump performance should not extend into the areas where substantially reduced efficiencies exist.

It is necessary that the designer correlate the design point discussed above with an elevation at about the mid-point of the pumping range. By doing this, the pump will work both above and below the TDH for the design point and will thus operate in the best efficiency range.

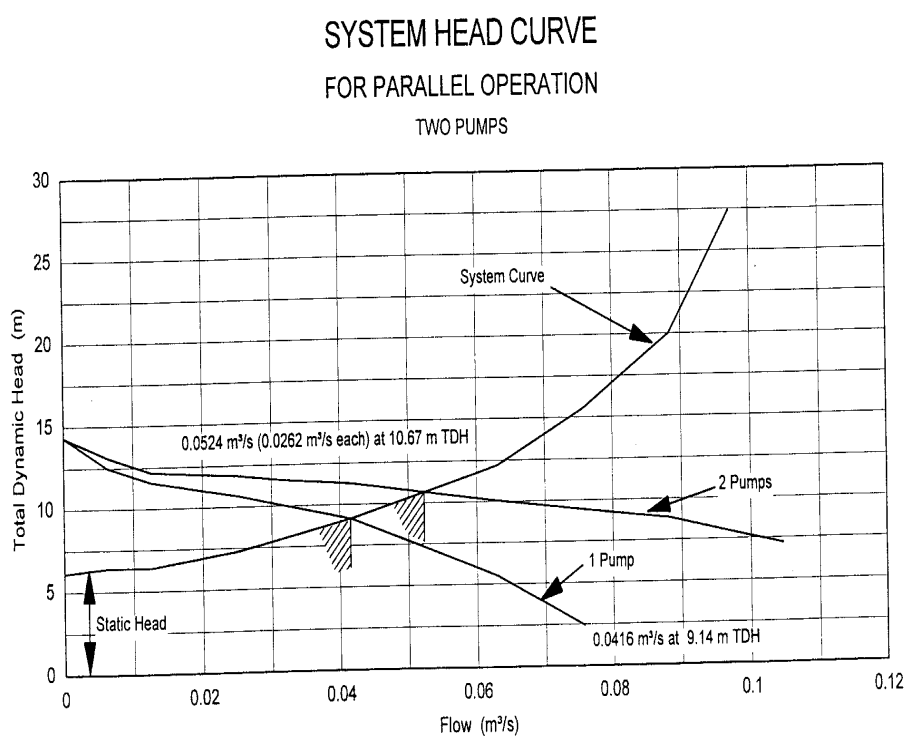
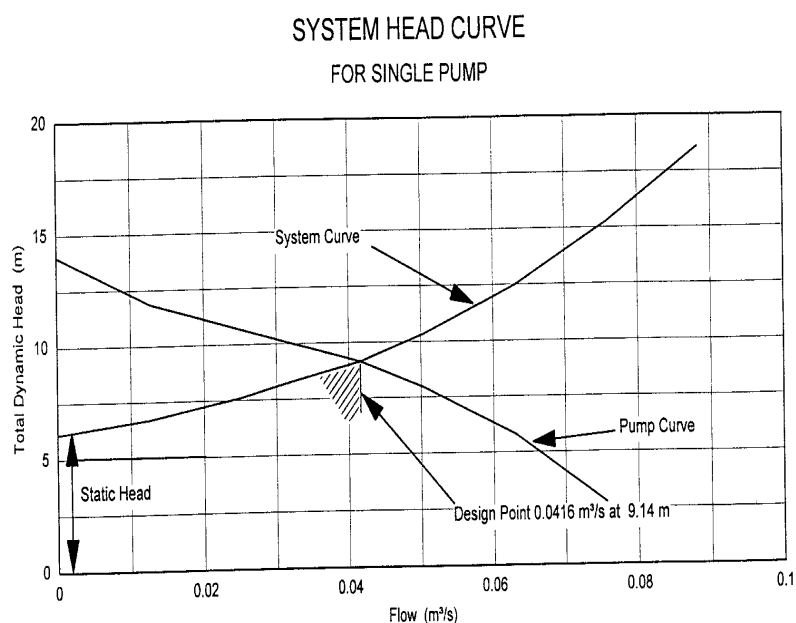


Figure 12-22 System Head Curves

Source: FHWA IP-82-17, Vol. 1

Step 10 Power Requirements

To select the proper size of pump motor, compute the energy required to raise the water from its lowest level in the pump pit to its point of discharge. This is best described by analyzing pump efficiency. Pump efficiency is defined as the ratio of pump energy output to the energy input applied to the pump. The energy input to the pump is the same as the driver's output and is called brake kilowatts.

$$\text{Efficiency, } e = \text{pump output/brake kilowatts} = Q\gamma H/1000 \text{ brake kW} \quad (12.8)$$

Where: Q = pump capacity, m^3/s (ft^3/s)
 γ = specific weight of liquid ($1000 \text{ kg}/\text{m}^3 \times 9.81 \text{ m}/\text{s}^2$ for cold water) ($62.42 \text{ lb}/\text{ft}^3$)
 H = head, m (ft)

Efficiency can be broken down into partial efficiencies — hydraulic, mechanical, etc. The efficiency as described above, however, is a gross efficiency used for the comparison of centrifugal pumps. The designer should study pump performance curves from several manufacturers to determine appropriate efficiency ranges. A minimum acceptable efficiency should be specified by the designer for each performance point specified.

To compute the energy required to drive a pump, assume that the pump will operate at 80% efficiency. The above equation can then be solved for brake kilowatts.

Step 11 Mass Curve Routing

The procedures described thus far will provide all the necessary dimensions, cycle times, appurtenances, etc. to design the pump station. A flood event can be simulated by routing the design inflow hydrograph through the pump station by methods described in Section 12.5.10. In this way, the performance of the pump station can be observed at each hydrograph time increment and pump station design evaluated. Then, if necessary, the design can be "fine-tuned."

12.6.3 Documentation

The following items shall be included in the documentation file. (See Chapter 1, Section 1.6). The intent is not to limit data to only those items listed, but rather establish a minimum requirement consistent with the pump station design procedures as outlined in this chapter. If circumstances are such that the pump design is prepared other than the normal procedures or is governed by factors other than recommended in this chapter, a narrative summary detailing the design basis shall appear with the other data.

The following items should be included in the documentation file:

- inflow design hydrograph from drainage area to pump
- flood frequency curve for the attenuated peak discharge
- maximum allowable headwater elevations and related probable damage
- starting sequence and elevations
- sump dimensions

- available storage amounts
- pump sizes and operations
- pump calculations and design report
- line storage and pit storage capacity
- maintenance plan

12.7 Philosophy

Typical pump station design procedures seen in the literature do not represent most highway stormwater pump station situations. Many stormwater management plans limit the post development discharge to that which existed prior to the development. In order to meet this requirement, it is often necessary to provide storage in the system. Traditional pump design procedures have not considered this storage volume and are thus oriented toward only wet well volumes. These designs are required to pump higher rates with limited storage volumes and thus start-stop and cycling relationships are very critical and can consume considerable design effort.

The mass inflow curve procedure discussed in this document is commonly used when significant storage is provided outside of the wet well. The plotting of the performance curve on the mass inflow diagram gives the designer a good graphical tool for determining storage requirements. The procedure also makes it easy to visualize pump start/stop and run times. In the event that a pump failure should occur, the designer can also evaluate the storage requirement and thus the flooding or inundation that could occur.

References

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