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SUBJECT:

Revisions to
Publication 584
PennDOT Drainage Manual
March 2015 Edition, Change No. 1

INFORMATION AND SPECIAL INSTRUCTIONS:

Incorporate the attached revisions into the March 2015 Edition of Publication 584.

These new guidelines should be adopted on all new and existing designs as soon as practical without affecting any letting schedules, but no later than September 3, 2022.

CHAPTER 1 INTRODUCTION

-Updated description of Chapter 14.

CHAPTER 6 DATA COLLECTION

-Removed reference to Pub 70M, Guidelines for Design of Local Roads and Streets.

CHAPTER 8 OPEN CHANNELS

*Section 8.8.F (Channel Lining Design Procedure)

-Updated Table 8.11 Allowable Shear Stresses for Various Linings.

*Section 8.8.G (Vegetated Lining)

-Revised the methodology for calculating Manning's n for grass-lined channels.

*Section 8.8.H (Rolled Erosion Control Products)

-Removed description of ECM, ECMB, HV ECMB, TRM, and ECRM.

-Updated descriptions of Mulch Control Netting, Open Weave Textile, Erosion Control Blankets, and Turf Reinforced Mats.

-Updated Table 8.15 Temporary RECPs.

-Updated Table 8.16 Permanent RECPs to include Type 5.D, 5.E, and 5.F and to include the Unvegetated Permissible Shear Stress.

*Section 8.8.I (Rock (Riprap) and Concrete)

-Updated reference for Figure 8.18.

CHAPTER 12 EROSION AND SEDIMENT POLLUTION CONTROL

*Section 12.4 (Stabilization BMPs)

-Removed second sentence.

*Section 12.4.B (Rolled Erosion Control Products)

-Revised guidelines for installation of RECPs to include on all slopes 3H:1V and steeper, on areas within 50 feet of non-special protection waters, and within 100 feet of special protection waters.

-Added sentence about rocky soil causing problems for proper staking/stapling.

-Removed description of ECM, ECMB, HV ECMB, TRM, and ECRM.

-Updated Table 12.4 RECP Types for Slope Application to include maximum slope by V:H and added Type 5.A, 5.B, 5.C, 5.D, 5.E, and 5.F.

-Updated paragraph beneath original Table 12.4 to follow guidance from Pub 293 Chapter 6 when dealing with cut slopes in soil that are altered or steepened.

*APPENDIX B Recommended Notes for E&SPC Plans

-Removed Standard Notes.

-Added reference to Pub 14M Chapter 6.

CHAPTER 14 POST-CONSTRUCTION STORMWATER MANAGEMENT

*Section 14.0 (Introduction)

-Revised content of overview and background.

*Section 14.1 (Targeted Parameters for PCSM)

-Replaced previous title of Highway Specific Stormwater Issues.

-Revised content in first paragraph; removed second paragraph.

**Subsection 14.1.A (Runoff Rate and Volume)

-Replaced previous title of Increases in Runoff Rate and Volume.

-Revised content in each paragraph.

**Subsection 14.1.B (Constituents in Runoff)

-Replaced previous title of Winter Maintenance Materials.

-Added significant content.

**Subsection 14.1.C (Thermal Impact)

-Revised content in each paragraph.

*Section 14.2 (Risk Considerations)

-Replaced previous title of Policy. Removed Introduction (previous 14.2.A), Project Categories (previous 14.2.B), Act 167 Plans and Municipal Ordinances (previous 14.2.C), and Special Considerations (previous 14.2.E).

-Content moved from previous subsection 14.2.D Limitations.

-Updated wording in first paragraph.

*Section 14.2.A (Infiltration SCMs)

-Replaced previous title of Structural Infiltration BMPs (14.2.D.1).

-Updated wording in first and second paragraphs.

*Section 14.2.B (Karst Topography)

-Previously subsection 14.2.D.2.

-Revised content in each paragraph.

*Section 14.3 (Legal)

-Replaced content in this section.

*Section 14.4 (SCM Toolbox – Non-structural)

-Added first paragraph.

-Updated wording in subsections 14.4.A, 14.4.B, 14.4.C, and 14.4.D.

*Section 14.5 (SCM Toolbox – Structural)

-Added first paragraph.

-Updated wording in subsections 14.5.A, 14.5.B, and 14.5.C.

-Updated content in subsection 14.5.D Stream Restoration and Stabilization.

-Updated content in subsection 14.5.E Vegetated Swale.

-Updated content in subsection 14.5.F Bioretention.

-Updated content in subsection 14.5.G Vegetated Filter Strip.

-Updated content in subsection 14.5.H on Media Filter Drain.

-Updated content in subsection 14.5.I on Subsurface Infiltration Trench.

-Updated content in subsection 14.5.J on Infiltration Detention Basin.

-Updated content in subsection 14.5.K Infiltration Berm.

-Updated content in subsection 14.5.L on Wet Detention Basin.

-Updated content in subsection 14.5.M on Stormwater Wetland.

-Added subsection 14.5.N on Dry Detention Basins.

-Added subsection 14.5.O on Dry Ultra-Extended Detention Basins.

-Added subsection 14.5.P on Level Spreader / Flow Dispersion.

-Added subsection 14.5.Q on Regenerative Step Pool.

*Section 14.6 (SCMS Requiring Special Approval)

-Updated reference to Section 14.7 (formerly 14.8).

-Updated subsection 14.6.B on Pervious Pavement.

-Added subsection 14.6.C on Constructed Stormwater Filter.

-Added subsection 14.6.D on Manufactured Treatment Devices.

-Added subsection 14.6.E on Subsurface Detention Storage.

*Section 14.7 (Areas of Application)

-Added Table 14.1 Siting and Maintenance Constraints of SCMs.

*Section 14.8 (Storage Facilities)

-Updated subsection 14.8.F (Underground Storage).

*Section 14.18 (Maintenance)

-Added section on considering maintenance in design of SCMs.

*Section 14.19 (Soil and Infiltration Testing)

-Added section on Soil Characterization (14.19.B) and Infiltration Testing (14.19.C).

*Section 14.20 (PCSM Plan)

-Added brief section on the requirements for a PCSM Plan.

*Section 14.21 (PCSM on Non-Chapter 102 Permit Projects)

-Added section providing guidelines on satisfying PCSM requirements on projects that do not require a Ch. 102 permit but do require a Ch. 105 permit.

*Section 14.22 (Definitions)

-Added: Antidegradation Best Available Combination of Technologies (ABACT), Best Management Practices (BMPs), Combined Sewer Systems (CSS), Disturbed Area, Ephemeral Stream, Impervious Area, Municipal Separate Storm Sewer System (MS4), Non-Discharge

Alternative, Post Construction Stormwater Management (PCSM), Post-development, Riparian Buffer, Riparian Forest Buffer, Road Maintenance Activity (RMA), Special Protection Waters, Stormwater Control Measures (SCM), and Total Maximum Daily Load (TMDL).

-Updated: Surface Waters and Thermal Impacts.

Gender neutral language has been updated in all new and revised pages, i.e., Table of Contents, Chapter 1, Chapter 6, Chapter 8, Chapter 12, and Chapter 14.

Any comments or questions regarding the above revisions should be directed to the Highway Design and Technology Division, Bureau of Design and Delivery.

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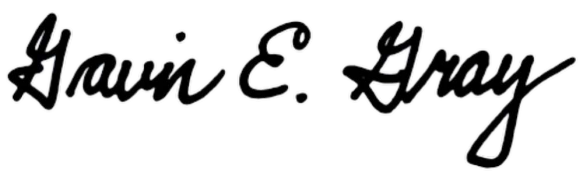
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PennDOT Drainage Manual

2015 Edition - Change #1





pennsylvania
DEPARTMENT OF TRANSPORTATION

PUB 584 (6-22)

PENNDOT DRAINAGE MANUAL

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CHAPTER 1

INTRODUCTION

1.0 INTRODUCTION

Drainage design is an essential element of highway design. It encompasses hydrology, hydraulics, erosion and sediment pollution control, permitting, and ultimately, providing facilities to collect and intercept surface runoff, remove or divert it from the roadway, and channel it to suitable locations where it can be managed and then safely discharged downstream of the roadway.

Drainage is a key factor in the development of all types of improvements on all classifications of highways, and in all phases of project development. For projects ranging from local roads in rural areas to busy, urban freeways, drainage must be considered in the development of preliminary and final design plans, specifications and estimates.

Hydrology, hydraulics and soil mechanics are the sciences generally applicable to the design of highway drainage. This manual may be revised from time to time to keep pace with the modern development of hydrologic, hydraulic and soils science. It should be noted that this manual serves as a general guide to design techniques and procedures; there is no intention to replace sound engineering judgment.

Controlled topography should be obtained, prior to construction, on the portion of projects involving significant floodplain encroachment or on those projects where there is a potential for significant additional flooding. This data would be of significant value during design and in evaluating flooding complaints.

The roadway drainage and waterway structures, as referred to above, include culverts, bridges, channel changes and longitudinal encroachments on waterways or floodplains.

This manual provides the engineer with general guidance and direction to evaluate the Department's drainage design options by addressing a broad range of issues related to drainage design. The manual has been developed to provide the designer with a basic working knowledge of hydrology and hydraulics complete with example problems. Careful analysis of existing site conditions, sound engineering judgment, and judicious application of the principles and procedures described or referenced in this manual is to result in highway drainage designs that are functional and cost effective.

1.1 BACKGROUND

The *PennDOT Drainage Manual* was developed using the American Association of State Highway and Transportation Officials (AASHTO) Task Force on Hydrology and Hydraulics produced *The Model Drainage Manual* (MDM) as the basis. The MDM was developed as part of the Task Force's continuing work to assist the Standing Committee on Highways, Subcommittee on Design, in developing guidelines and in formulating policy. The first edition of the MDM, which was published in 1991, was produced in cooperation with the FHWA Rural Technical Assistance Program and the Georgia Institute of Technology. The second edition of the manual, published in 1999, was produced in cooperation with the Transportation Research Board, National Cooperative Highway Research Program (NCHRP) and was developed using the International System of Units (SI).

The 2005 third edition of the MDM was used as the basis for the *PennDOT Drainage Manual* along with incorporating applicable sections from and coordination with Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10. Each chapter has been written based upon the MDM's generic design policies and state-of-the-practice design procedures. These design policies and procedures have been supplemented by PennDOT's policies and procedures.

1.2 CONTENTS OF PENNDOT DRAINAGE MANUAL

The *PennDOT Drainage Manual* currently contains fourteen chapters.

Due to a great number of variables that may be presented within a chapter, the next to last section of several of the chapters provides the nomenclature that is used for the specific chapter. The user is directed to these sections to assist with understanding the variables used within each chapter.

1. Chapter 1, *Introduction,* introduces the *PennDOT Drainage Manual*, how it was developed and introduces each of the chapters contained in the *PennDOT Drainage Manual*.

2. Chapter 4, *Documentation and Document Retention*, introduces the plans, reports, specifications and analyses that are to be developed during the design of a hydraulic facility. A system for organizing the documentation of hydraulic designs and reviews to provide a complete history of the design process is identified. Information that is to be included in the design files and on the construction plans is reviewed. This will result in a compilation and preservation of the design and related details and all pertinent information on which the design and decisions were based and is referred to as the District Project Files.

3. Chapter 6, *Data Collection*, introduces the types of data, along with possible sources, that will be required prior to conducting the hydrologic and hydraulic, stormwater management, highway drainage, or erosion and sediment pollution control engineering analysis. In addition, the types of data and data collection, items to be considered while performing site investigation, necessary geographic information system (GIS) and survey information to be collected to aid in performing analyses and associated data evaluation are discussed.

4. Chapter 7, *Hydrology*, introduces hydrology as dealing with the design of highway drainage facilities to convey specific predetermined discharges in order to avoid significant flood hazards. The chapter also discusses how to convey floods in excess of these discharges in a manner that minimizes the damage and hazard. The analysis of the peak rate of runoff, volume of runoff and time distribution of flow is fundamental to the design of drainage facilities as well as site investigation and interagency coordination. Peak discharges are used to design facilities such as storm drain systems, culverts and bridges. The entire discharge hydrograph is used for the design to control the volume of runoff, like detention storage facilities, or where flood routing through culverts is used. The chapter discusses in detail the methods that are in PennDOT's approved hydrology toolbox.

5. Chapter 8, *Open Channels*, introduces open channel hydraulics for stream channels, roadside channels or ditches, irrigation channels, and drainage ditches and the associated interrelationship of channels to all highway hydraulic structures. The principles of open channel flow hydraulics are applicable to all drainage facilities, including culverts. The chapter discusses the hydraulic design process for open channels and consists of establishing criteria, developing and evaluating alternatives, and selecting the alternative which best satisfies the established criteria.

6. Chapter 9, *Culverts*, introduces the hydraulic aspects of culvert design, construction and operation of culverts, and makes references to structural aspects only when they are related to the hydraulic design. Culverts are considered minor structures as compared with bridges, but they are of great importance for drainage and the integrity of the facility. The following important concepts for culvert design are discussed as part of the chapter: critical depth, crown, flow type, free outlet, improved inlet, invert, normal flow, slope (steep and mild), inlet and outlet control, and submerged flow.

7. Chapter 10, *Bridge Hydraulics*, introduces structures designed hydraulically as bridges considering the total crossing, including approach embankments and structures in the flood plain. Bridges are also important and expensive highway-hydraulic structures and are vulnerable to failure from flood-related causes. To minimize the risk of failure, the hydraulic requirements of stream crossings must be recognized and considered carefully. The design of bridges for hydraulic requirements includes an estimate of peak discharge (sometimes complete runoff hydrographs), comparing water surface profiles for existing and proposed conditions, consideration of potential stream stability problems, and consideration of scour potential. Hydrologic and hydraulic analyses determine whether the water surface profiles are or are not affected.

8. Chapter 11, *Surface Water Environment*, is broken into three sections regarding surface waters, ground water and infiltration, and water budget. The section on surface waters discusses the evaluation of the effects

that highway construction has on surface waters and the designer's role in determining the hydrology and hydraulic related highway effect and significance on sensitive surface waters. Only the simpler, more reliable and accepted engineering practices are discussed and should address 90% or more of the surface water environmental problems encountered. More complex practices are briefly noted, but only to provide background and suggest possible analytical courses of action.

The second section presented in this chapter is regarding groundwater and infiltration. Highway and bridge projects can affect groundwater in several ways. Some of these effects are from the groundwater being disrupted in a way that prevents or degrades its use. Retention or detention basins work as a stormwater management tools by storing the stormwater to control the peak flow and timing of stormwater flowing downstream. However, a portion of the stormwater which ordinarily infiltrates into the subsurface is retained and routed to a body of surface water resulting in reduced infiltration. Infiltration of stormwater runoff into the groundwater hydrologic flow regime can often be an appealing alternative to more traditional stormwater control methods such as detention basins and wet ponds in that it promotes groundwater recharge. The concept of groundwater recharge is an important consideration in areas where aquifers have been depleted due to pumping, or in more urbanized areas where impervious cover has greatly reduced the quantity at which surface water infiltrates into the ground. This reduction has a compound effect on the hydrologic system in that while reducing infiltration of surface water depletes underground aquifers, the base flow from groundwater movement that feeds streams is reduced and surface waters suffer the consequence in turn.

The third section presented in this chapter is regarding water budget. The water budget (water balance) is a hydrologic procedure that refers to the balance between the inflow of water from precipitation, snow melt, and groundwater, and the outflow of water by evapotranspiration, groundwater, and streamflow. The water budget may be used in a variety of scenarios, and is able to predict human impact on a wetland, watershed, and the hydrologic cycle to determine a management approach to sustaining the natural hydrologic conditions of a site. The water budget as discussed in this section is presented in a generic format which may be applied annually or for any period of time as long as the parameters input into the equations are representative for that length of time. This section also presents equations necessary to calculate the water budget, and potential sources for the various parameters used in the equations.

9. Chapter 12, *Erosion and Sediment Pollution Control*, introduces guidance on how to determine when Best Management Practices (BMPs) are needed, and which BMPs would best meet the regulatory requirements and design objectives. This guidance categorizes BMPs into three groups: Stabilization BMPs, General Erosion and Sediment Pollution Control (E&S) BMPs and In-Channel Erosion and Sediment Control BMPs. These BMPs are temporary controls used by construction site operators during the period of earth disturbance to control erosion and sedimentation during construction activities. The guidance provided gives an overview of the regulatory basis for these requirements; guides the reader through the steps of coordinating with the federal, state, and local regulatory authorities; explains the types of analyses that should be performed; and, describes a variety of BMPs including where and when they can/should be used and how they should be designed to achieve their optimal functionality.

10. Chapter 13, *Storm Drainage Systems*, introduces guidance on storm drain design and analysis. The aspects of storm drain design (e.g., system planning, pavement drainage, gutter flow calculations, inlet spacing, pipe sizing, hydraulic grade line calculations) are discussed. The design of a drainage system must address the needs of the traveling public and 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.

11. Chapter 14, *Post-Construction Stormwater Management*, introduces PennDOT's Antidegradation and Post-Construction Stormwater Management (PCSM) Policy. In addition, general design criteria for detention/retention storage basins, procedures for sizing basins, procedures for performing routing calculations, and design guidelines for post-construction stormwater management controls intended to manage stormwater after construction is complete are provided.

12. Chapter 19, *Design-Related Construction Considerations*, introduces the designer to key construction-related drainage concepts to be considered during design. The material is presented in order to familiarize the designer with other construction aspects of the design that affect the function, constructability, maintenance, overall cost and the sustainability of a drainage design. In addition, special considerations for the construction of bridges, culverts, open channels, stormwater management measures and storm drains are discussed.

CHAPTER 4

DOCUMENTATION AND DOCUMENT RETENTION

4.0 OVERVIEW

A. 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 needed for:

- The importance of public safety;
- Justification of expenditure of public funds;
- Future reference by designers (when improvements, changes or rehabilitations are made to the highway facilities);
- Information leading to the development of defense material for matters of litigation; and
- Public information requirements.

Frequently, it is necessary to refer to plans, reports, specifications and analyses (records) long after the actual construction has been completed. Documentation also allows 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 so that recurring damage can be avoided. Records management is the systematic control of recorded information from the time that information is created until its ultimate disposition either through its destruction or its transfer to archives for permanent preservation. PennDOT is responsible for making provisions for the continued retention of permanently valuable records (permanent records) in a human-readable format (paper or microfilm) even though they have been copied into an imaging system. PennDOT must retain the original copy of a permanently valuable record or generate a security microfilm copy to serve as a substitute. Microfilm copies must be created and maintained in conformance with applicable Commonwealth standards.

B. Purpose. This chapter identifies the documentation to be included in the design files and on the construction plans. Although PennDOT's documentation requirements for existing and proposed drainage facilities are similar, the documentation retained for existing facilities are often slightly different than that for proposed facilities, and these differences are discussed. This chapter focuses on the documentation of the findings obtained in using the other chapters of this manual, and thus designers need to be familiar with all the hydrologic, hydraulic, erosion and sediment pollution control, drainage, and stormwater design procedures associated with this manual. This chapter identifies PennDOT's system for organizing the documentation of hydraulic designs and reviews to provide as complete a history of the design process as 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 determined. Often, there is expressed the myth that avoiding documentation will prevent or limit litigation losses because it supposedly precludes providing the plaintiff with incriminating evidence. This is seldom, if ever, the case and clear, organized documentation provides a record of reasonable and prudent design analysis based on the best available technology. Thus, good documentation can provide the following:

- Protection for PennDOT by proving that reasonable and prudent actions were taken.
- Identification of the situation at the time of design.
- Documentation that rationally accepted procedures and analysis were used at the time of the design that were commensurate with the perceived site importance and flood hazard.
- Continuous site history to facilitate future reconstruction.
- The file data necessary to quickly evaluate any future site problems that might occur during the facility's service life.
- Expedient plan development by clearly providing the reasons and rationale for specific design decisions.
- Ensure any municipal agreements for future maintenance or ownership are negotiated and documented.

C. Documentation and District Project Files. The definition of hydrologic, hydraulic, drainage and stormwater documentation is the compilation and preservation of the design and related details and all pertinent information on which the design and decisions were based and is referred to as the District Project Files (DPFs). DPFs may include:

- The documentation for filing National Pollutant Discharge Elimination System (NPDES) applications.
- The documentation for Erosion and Sediment Pollution Control (E&SPC) Plans.
- The documentation for Preparedness, Prevention and Contingency (PPC) Plans.
- The documentation for Post-Construction Stormwater Management (PCSM) Plans.
- The documentation for the determination of a significant encroachment as per Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.1.A.5.
- The Location Hydraulic Studies (Preliminary Engineering Phase) as defined in Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.1.A.10 is to be included as part of the environmental review documents noted below.
- The Design Hydraulic Studies (Final Design Phase) as defined in Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.1.A.15.
- The Hydrologic and Hydraulic Reports as defined in Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.7.
- The documentation for Roadway Drainage Report as per Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.3.G.
- Environmental review documents pursuant to 23 CFR 771 and Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.1.A.12(e).
- Conditional Letter of map Revision (CLOMR) or Letter of map Revision (LOMR) as per Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.7.C.9 and Appendix C.
- Written agreements with the municipalities to maintain closed stormwater facilities that PennDOT installs during construction projects if applicable.
- Other supporting documentation such as drainage area and other maps, field survey information, source references, photographs, engineering calculations, analyses and computer files, plans, specifications, measured and other data and flood history including narratives from newspapers and individuals (e.g., highway maintenance personnel and local residents who witnessed or had knowledge of an unusual event).

D. Roadway Management System (RMS) and Location Reference System (LRS). The Roadway Management System (RMS) is PennDOT's primary means for defining and monitoring the State-owned highway network, maintaining an inventory of the roadway features, conditions, and characteristics, and providing decision-makers with the information that is necessary for funding, business planning, project design, and maintenance programming. The Location Reference System (LRS) provides a framework for which all RMS data can be tied to true roadway locations. Data stored and managed in RMS includes roadway geometry information, traffic information, pavement and shoulder history, maintenance history, municipal and legislative boundaries, intersections, roadside features, structure locations, railroad crossings information, pavement testing, condition survey information (including guide rail and drainage features), and posting/bonding information. One of the primary uses of RMS is the annual allocation of highway maintenance funds. For an introduction to the RMS/SR system please see the *LRS Introduction Manual* specified in Appendix 4A, *Documentation Quick Reference Guide*, Section 4A.0.L.

RMS data can be obtained from all PennDOT District offices and most county maintenance offices, which are linked to the RMS network. For further information about RMS or the SR system, the RMS coordinator in each Engineering District office should be contacted. One of the most important aspects of RMS is having a viewable representation of a State Road (SR). This is accomplished with a graphical diagram called a Straight Line Diagram (SLD). The Bureau of Maintenance and Operations produces an electronic version of the SLD for every state road, in every county, annually that can be downloaded from the link specified in Appendix 4A, *Documentation Quick Reference Guide*, Section 4A.0.L.

E. Phases. Hydraulic documentation needs to be developed for up to four different phases of the project development cycle. The four phases are preliminary engineering, final design, construction, and maintenance or operation.

It is important to prepare and maintain in a permanent file the as-built plans for every drainage structure to document subsurface foundation elements (e.g., footing types and elevations, pile types and (driven) tip elevations).

According the Commonwealth of Pennsylvania, Governors' Office State Records Management Manual, record series considered permanent by an agency should be identified by using "999" in the Retention for Agency field on the STD-56 and the retention fields on the STD-57. As the hydraulic facility design or investigation develops, additional information may be identified by the designer and incorporated into the project file at the designer's discretion. PennDOT also needs to record and maintain the agreements with local municipalities for operation/ownership of stormwater facilities.

1. Preliminary Engineering. Preliminary engineering documentation in the project file needs to include the following, if available, or within the budgetary restraints of the project:

- Location map and identification of the facility.
- Aerial photographs.
- Contour mapping.
- Watershed map or plan including:
  - Flow directions.
  - Watershed boundaries.
  - Watershed areas.
  - Natural storage areas.
- Surveyed data reduced to include:
  - Existing hydraulic facilities.
  - Existing controls.
  - Profiles (e.g., roadway, channel, driveways).
  - Cross sections (e.g., roadway, channels, faces of structures).
- Flood insurance studies and maps by FEMA.
- NRCS soils information.
- Hydrologic and hydraulic investigation/report.
- Site visit report(s) that may include:
  - Video recordings.
  - Audio recordings.
  - Photographs.
  - Interviews with local residents, municipal officials, and PennDOT maintenance personnel.
  - Written analysis of findings with sketches.
- Reports from other agencies (local, state or federal), PennDOT's personnel, and newspapers.
- Design notes/assumptions.
- Engineering cost estimates.
- Other preliminary documentation as described in Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.2.B and Publication 10C, Design Manual, Part 1C, *Transportation Engineering Procedures,* Chapter 3.

2. Final Design. During Final Design, the designer may need to contact the county maintenance office to determine if there are any concerns with what is being installed. In addition, if no maintenance agreement with another entity has been developed, the designer needs to determine whether the District's Maintenance forces are willing to maintain it. Design documentation in the project file needs to include all the information used to justify the design, including:

- Reports from other agencies.
- Hydrologic and hydraulic (H&H) report.
- Erosion and sediment pollution control plan and report.
- Completed water obstruction and encroachment permit.
- Post-construction stormwater management plan and report.
- Roadway drainage report.
- Preparedness prevention contingency plan approvals as described in Section 4.2.

Other final documentation as described in Publication 10C, Design Manual, Part 1C, *Transportation Engineering Procedures,* Chapter 4.

3. Construction. Construction documentation in the project file needs to include:

- Plans.
- Revisions.
- As-built plans and subsurface borings.
- Photographs.
- Publication 408, *Specifications*, Section 100 requirements.
- Publication 2, *Project Office Manual*, Part A, Sections 2 and 3 requirements.

4. Operation and Maintenance. Operation and/or maintenance documentation in the project file needs to include:

- Record of operation during flooding events, complaints and resolutions.
- Plans or descriptions of any repairs or modifications affected by maintenance forces.
- Periodic reports or reviews. Publication 23, *Maintenance Manual*, requirements for documentation.
- The municipal agreement for maintenance of closed stormwater facilities.

5. Post-Construction Documentation. Possibly the most valuable experience of any designer is gained by observing and analyzing the performance of a facility under field conditions. Hydraulic engineers are in a unique position in this regard because their designs are often tested by nature and may suffer damage from relatively small flow rates as well as from larger flows. Unfortunately, documentation rarely is available for hydraulic engineers to accomplish more than a qualitative analysis of performance.

Hydraulic engineers should take advantage of their unique opportunities to gain experience from tests provided by nature. The following documentation are examples of the types of information which are of value in reviewing and analyzing designs to assess the validity of practices, procedures, assumptions and decisions:

- Highwater elevations and flow rates.
- Ice and drift conditions.
- Erosion of approach overflow sections, embankments and spur dikes.
- Stream aggradation or degradation.
- Scour location, depth and extent.
- Performance of scour and erosion preventive measures.
- Meander and bend migration.
- Performance of stream bank protection and river training measures.
- Costs of maintenance, repair and corrective measures.

While the emphasis in this section is on the knowledge and experience that can be gained from field tests of designs, the above documentation also are useful in detecting potential or existing problems at the crossing which can be resolved by providing corrective measures.

4.1 DOCUMENTATION TYPES

A. Documents. Through the design and permitting process, the designer may need to prepare a number of documents. These documents include:

- National pollutant discharge elimination system (NPDES) permit application.
- Erosion and sediment pollution control (E&SPC) plan and narrative (Chapter 102).
- Roadway drainage report (RDR).
- Post-construction stormwater management (PCSM) plan and report.
- Preparedness prevention contingency (PPC) plan.

- Aids to Navigation (ATON) requirements according to Publication 13M, Design Manual, Part 2, *Highway Design,* Section 10.5.C.6.
- Water obstruction and encroachment permit (Waterway permit).
- Hydrology and hydraulics (H&H) report.
- FEMA conditional letter of map revision (CLOMR) and letter of map revision (LOMR).

Appendix 4A, *Documentation Quick Reference Guide* follows the outline of this section. It provides a concise summary of the requirements and where to find the application or information on the regulation or requirement. Superscripts on the required item below correspond to the number on the Quick Reference Guide. The following is a short discussion on what should be included in the project file documentation, and where to find the requirements for each.

B. National Pollutant Discharge Elimination System (NPDES) Permit Application. The same NPDES "Notice of Intent" (NOI) form is required for both the PAG-02 or Individual Permit submission. Links to the form and its instructions can be found in Appendix 4A, *Documentation Quick Reference Guide*. A sample NOI completed for PennDOT projects is found in Appendix 4B, *Sample Notice of Intent (NOI) Application Form*. Documentation required for general and individual permits can be found in Chapter 12, *Erosion and Sediment Pollution Control*, Section 12.3.H.

The Applicant Checklist is a list of required items for the NPDES permit application that is attached to the application form. The checklist can be used as a guide when compiling information for a permit application and must be completed and submitted along with the NOI form. There are separate Applicant Checklists for the General and Individual permits.

Copies of the Act 167 and/or floodplain letters with return receipts and responses must be included in the permit application package. The PNDI review receipts must also be included in the NPDES permit application, a sample of which is found in Appendix 4D, *Sample PNDI Project Environmental Review Receipt*.

Refer to Appendix 4A, *Documentation Quick Reference Guide* and Appendix 4B, *Sample Notice of Intent (NOI) Application Form* for additional references on NPDES documentation.

C. Erosion and Sediment Pollution Control (E&SPC) Plan and Narrative. The E&SPC Plan is to be prepared in accordance with Chapter 12, *Erosion and Sediment Pollution Control,* (particularly Table 12.1), Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 13, and the Publication 72M, *Roadway Construction Standards*.

An E&SPC Plan consists of a plan and a narrative and contains:

- The items listed in Publication 14M, Design Manual, Part 3, *Plans Presentation,* Chapter 6, Section 6.2.
- Drawings of standard E&SPC items from Publication 72M, *Roadway Construction Standards,* specifically RC-70M to RC-77M or from PA DEP's BMP Manual, if applicable.
- The narrative according to Chapter 12, *Erosion and Sediment Pollution Control*, Section 12.3.I, and Appendix 12C, *Recommended Standards for E&SPC Plans.*
- The E&SPC Plan according to PA Code Title 25, Chapter 102 § 102.4. *Erosion and Sediment Control Requirements* and Appendix 12C, *Recommended Standards for E&SPC Plans*.
- Recommended notes for E&SPC Plans as per Appendix 12B, *Recommended Notes for E&SPC Plans.*

For a detailed list of E&SPC related regulations that may apply, refer to Appendix 12A, *E&S Related Regulations.*

D. Roadway Drainage Report (RDR). A roadway drainage report contains a descriptive narrative, the hydrologic and hydraulic computations, and drainage plan sheets with drainage area delineations for roadway drainage structures (inlets, storm pipes, pipe culverts, ditches and swales, pavement base drains, curb flow, etc.).

The following general items are to be included in the RDR:

- Complete drainage area map(s).
- Design frequencies.
- Information concerning outfalls, existing storm drains and other design considerations.

- Computations for drainage areas, inlets and storm drains, including hydraulic grade lines.
- Standard computation sheet given in Chapter 13, *Storm Drainage Systems*, and Publication 13M, Design Manual, Part 2, *Highway Design*, Figure 10.3.8.

A recommended outline for the RDR is provided in Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Section 10.3.G.

The RDR, the PCSM, and the E&SPC reports have redundant information, such as project description narrative and design calculations for rock outlet protection. As a result, coordination of the three reports, specifically if separate subconsultants are working on the different phases of a project, is necessary.

The RDR is sent to and approved by the District Office with one copy of the submission and approval sent to the Central Office according to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.3.H.

References to aid in development of the RDR can be found in Appendix 4A, *Documentation Quick Reference Guide*.

E. Post-Construction Stormwater Management (PCSM) Plan. A PCSM Plan is required for all NPDES permit applications. The PCSM Plan identifies the proposed stormwater BMPs being used to manage and treat the stormwater discharges to protect water quality after construction. Preparation and implementation of the PCSM Plans is to be done in accordance with Chapter 14, *Post-Construction Stormwater Management* and the NPDES NOI checklist.

References to aid in development of the PCSM Plan can be found in Appendix 4A, *Documentation Quick Reference Guide*.

F. Preparedness, Prevention, & Contingency (PPC) Plan. A PPC Plan is required for any NPDES Application for Storm Water Discharge General Permits or Water Management Permits. Development of the PPC Plan is to be done in accordance with 25 PA Code Section 91.33 and 91.34.

For projects where the potential exists for causing accidental pollution of air, land, or water, or for causing endangerment of public health and safety through accidental release of toxic, hazardous, or other polluting materials, the NPDES permittee or co-permittee will need to develop a PPC Plan. In PennDOT's NPDES submission, the PPC Plan box (if it is believed an emergency or accidental spill during construction is possible) is to be checked. In addition, BMPs must be located on the PPC Plan for each identified area.

The PPC Plan is most often developed by the contractor after the project is let. In these cases, a special provision must be included in the construction bid documents and a statement needs to be provided in the E&S Plan general notes that the contractor is responsible for providing a PPC Plan. In rare cases, Districts may determine that for certain environmentally sensitive projects, the design consultant may prepare the PPC Plan for the project, prior to letting for inclusion in the bid documents.

No formal PPC Plan submission to an Agency is required, nor is formal approval from an agency required. However, a copy of the PPC Plan is to be filed in the District Project Files as well as be made available at the job site.

All members of the installation's organization for developing, implementing, and maintaining the PPC plan are to review the plan and be thoroughly familiar with its provisions.

References to aid in development of the PPC Plan can be found in Appendix 4A, *Documentation Quick Reference Guide*.

G. Water Obstruction and Encroachment Permit (Waterway Permit). Upon identification of the correct permit type, the JPA_2 system and PA DEP's requirements describe the documentation required for the identified permit. References and links for much of this information can be found in Appendix 4A, *Documentation Quick Reference Guide*.

The waterways permit flowchart in Appendix 3A, *Waterways Permitting Flow Chart*, is a guide to assist in selecting the appropriate waterways permit for a project. Even though the permit may not require the submission of all of the environmental data, most of the preliminary environmental research needs to be completed to ensure that the project site meets the conditions of the general permit. For instance, although the GP-7 (Minor Road Crossing) permit does not require the submission of FEMA maps, the FEMA maps need to be researched; if a FEMA delineated floodway exists at the project site, the GP-7 is not applicable.

A full JPA submission is to follow the JPA_2 system of reporting and includes:

- Application Interview.
- General Information Form (GIF) - The completed general information form and application properly signed sealed and witnessed (which includes the following):
  - Application fee.
  - Site information.
    - Name and location.
    - Written directions to the site.
    - Location map (copy of USGS quad map).
  - Project Information.
    - Name and description.
    - Time schedules/project milestones.
  - Act 14/67/68/127 letters (see Appendix 4B, *Sample Notice of Intent (NOI) Application Form*, for sample).
- Joint Permit Application.
- PNDI Search.
- Plans.
- Engineer Seal & Certification.
- Location Map.
- Project Description.
- Photographs with Map.
- Environmental Assessment (EA).
  - EA Introduction.
  - EA Form and signature.
  - Wetlands delineation and survey (where applicable).
  - Threatened & endangered species (T&E).
    - A search for threatened and endangered species (T&E) in the project area may be conducted using the PNDI Search on the Pennsylvania Natural Heritage Program web site.
    - Bog turtle screening is also required in the following counties: Adams, Berks, Bucks, Carbon (Aquashicola Creek Watershed only), Chester, Cumberland, Delaware, Lancaster, Lebanon, Lehigh, Monroe, Montgomery, Northampton, Schuylkill (Swatara Creek Watershed only), and York.
  - High quality (HQ) or exceptional value (EV) watershed designation.
  - Wild trout stream classification.
  - Aquatic habitat.
  - Project impacts.
  - Resource identification.
- Cultural Documentation.
  - Description of historic, cultural, archaeological resources at the site.
  - Cultural Resource Notices completed and mailed to the Pennsylvania Historical and Museum Commission (PHMC).
  - National Registry of Natural Landmarks designation (Chapter 36 in the Code of Federal Regulations (CFR) Part 62.2).
  - Wild or scenic rivers designation.
- E&SPC.
  - Approval letter from the Conservation District; or
  - NPDES permit.
- Hydrologic/Hydraulic (H&H) Report.
  - The H&H report (See Section 4.2.H).

  - FEMA information (hydrology, profiles, maps).
  - A stormwater and/or floodplain management analysis and / or consistency letter(s).
- H&H Input/Output Files.
  - HEC-RAS Input Files.
  - HEC-RAS Output files.
- Mitigation Plan.
- PA DEP Facility Detail.
- PennDOT Information and Files.
  - Application Log.

A Stormwater Consistency Letter is only needed when the project is in a watershed with an approved Act 167 Plan. A Floodplain Management Letter is only needed when the proposed waterway obstruction/encroachment is in a delineated FEMA floodway. Sample letters are included in Appendix 4C, *Sample Stormwater and Floodplain Management Consistency Letters*.

PennDOT requires that Joint Permit Applications be submitted electronically through the JPA_2 Expert System. The approved permit and all computations and back-up data are saved and stored electronically within the JPA_2 system. The link can be found in Appendix 4A, *Documentation Quick Reference Guide*.

In addition, reference Publication 15M, Design Manual, Part 4, *Structures,* Chapter 1, Section 1.9 for documentation requirements for bridges.

H. Hydrology and Hydraulics (H&H) Report. The location hydraulic studies are to be developed in conformance with and incorporate the requirements of 23 CFR Part 650.111, Paragraphs (a) through (f) which are repeated in Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.1.A.10 and 12. Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.7.B provides an ***abbreviated*** H&H Report outline and may be used for structure rehabilitation and replacement projects. Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.7.C provides an example of a ***full*** H&H report that is required for all new alignments, structure rehabilitation, and replacement projects that do not meet the requirements of an abbreviated H&H report.

Generally, the H&H report for the design hydraulic study includes a general description of the site, upstream and downstream structures, watershed, a risk assessment, alternatives analysis, mitigation plan (if necessary), photographs, hydrologic and hydraulic supporting computations, FEMA FIS information, line and grade and TS&L drawings, computer (HEC-RAS) files, a summary sheet, (Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Figure 10.7.1, *Sample Summary Data Sheet),* and QA/QC checklists. More specifically, the abbreviated and full H&H reports are to contain the information specified in Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Sections 10.7.B and 10.7.C respectively, and Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Appendix C.

For projects that have permits submitted electronically, the Final H&H Report is saved and stored electronically within the JPA_2 Expert System. For all other projects, the final H&H report is to be submitted to the District Project Manager (PM) in accordance with the Districts individual requirements and then be retained with the Bridge Files. A temporary conditions evaluation may be required where causeway/temporary stream crossings causes' water surface elevations to be beyond the normal stream lines.

Other references to hydraulic design of encroachments in floodplains can be found in Appendix 4A, *Documentation Quick Reference Guide*.

I. FEMA Conditional Letter of Map Revision (CLOMR) and Letter of Map Revision (LOMR). The CLOMR describes any eventual revisions to be made to the NFIP maps upon completion of the project. A map of the existing and proposed floodplain boundaries and elevations and any right-of-way impacts that may occur is to be included in the CLOMR.

The FEMA MT-2 application forms are applicable for CLOMR and LOMR requests and its web site location can be found in Appendix 4A, *Documentation Quick Reference Guide*.

Along with the FEMA MT-2 application forms, the request is to include the following supporting information:

- Transmittal letter (see example in Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Figure 10.7.2).
- Completed MT-2 application forms.
- Project narrative (optional).
- Hydrologic computations (if applicable) along with digital files of computer models used.
- Hydraulic computations (if applicable) along with digital files of computer models used.
- Certified topographic map with floodplain and floodway (if applicable) delineations.
- Annotated FEMA FIRM and/or FBFM to reflect changes due to project.
- Items required to satisfy any FEMA NFIP regulatory requirements.
- Payment information form along with payment.

J. Pump Stations. The following items are to be included in the documentation for pump station design:

- Drainage area to the pump.
- Inflow design hydrograph from drainage area to pump.
- Inflow mass curve.
- Maximum allowable headwater elevations and related probable damage.
- Sump dimensions.
- Stage-storage curve.
- Stage-pump discharge relation and sequence.
- Mass curve routing results.
- Pump sizes and operations.
- Discharge line and fittings sizing.
- Total dynamic head curves.
- Selected pump performance curves.
- Design report.

4.2 PROCEDURE

A. File Overview. Each District is to maintain the complete hydrologic and hydraulic design and analysis documentation for each waterway encroachment or crossing, storm drain or stormwater management system, and erosion and sedimentation control plan.

The documentation for each facility is prepared by a designer, and contains design/analysis data and information that influenced the facility design that may not appear in other project documentation. Once the design is prepared, regardless of whether it was developed by a PennDOT or consultant design, the design is to be reviewed and approved by PennDOT.

For specific documentation procedures for bridge opening selection, refer to the Chapter 10, *Hydraulics,* Section 10.14.

B. Practices. Following are PennDOT's practices related to documentation of hydrologic and hydraulic designs and analyses:

- 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, are to be compiled in the documentation.
- The designer is to document all design assumptions and selected criteria including the related decisions.
- The amount of documentation detail for each design or analysis is to be commensurate with the risk and the importance of the facility.
- Organization of the documentation is to be as concise and complete as practicable so that knowledgeable designers can understand years hence what was done by predecessors.
- Circumvent incriminating statements wherever possible by stating uncertainties in less than specific terms (e.g., the culvert *may* back water rather than the culvert *will* back water).

- Provide all related references in the documentation to include such items as published data and reports, memos and letters and interviews. Include dates and signatures where appropriate.
- Documentation is to include data and information from the conceptual stage of project development through service life to provide successors with all information.
- Documentation is to be organized to logically lead the reader from past history through the problem background, into the findings and through the performance.
- An executive summary at the beginning of the documentation will provide an outline of the documentation file to assist users in finding detailed information.
- A copy of the completed application forms for each documentation file as described in Section 4.2 are to be kept on file at the District.

The file reviewer needs to be aware that regulations, requirements, and forms are constantly being updated, and the designer is to ensure that he is following the latest documentation requirements.

C. Filing and Retention. Required permit application documents for GP-11 and full permits are electronically filed in the JPA_2 Expert System as specified in Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Sections 10.5.A and 10.5.B. All other General Permits and 9999 maintenance permits are to be maintained in the District files.

There are several types of electronic storage media available, such as scanning of project files and storage on compact discs, digital cameras where photographs can be stored electronically, computer-generated construction drawings, digital elevation models (DEMs) and Digital Terrain Models (DTMs), computer input and output files, e-mails, and other electronic files associated with a project.

Districts are to maintain the project documentation files including microfilm, microfiche, magnetic/electronic media, etc. where information can be accessed for use during construction, for defense of litigation and future replacement or extension. Bridge project files and non-bridge hydraulic structures are kept for 100 years or life of structure by the Districts in which they were built. Original plans, project correspondence files, construction modifications and inspection reports are the types of documentation usually kept with the bridge project files. Non-bridge hydraulic structure documentation files are to be retained for a period equal to the projected life span of the structure as indicated in Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Table 10.3.4.

H&H reports and waterway permits (e.g., Chapter 105, Section 404, General Permits) are retained with the bridge files. NPDES permits are often not related to bridge projects and, as such, are kept with the project design files. "Design Project Files" are kept for seven years after the work is complete on the project. An electronic copy of the "Design Project Files" is scanned into ECMS. Hydrologic/hydraulic documentation is to be retained in the project plans or other permanent location at least until the drainage facility is totally replaced or modified as a result of a new drainage study.

Only the documents that received approval need to be retained by the District Office. Draft documents leading up to the documents that received approvals need not be retained.

D. Scheduling. Documentation does not need to occur 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 is an ongoing process and part of each step in the hydrologic, hydraulic, drainage, stormwater and erosion and sedimentation pollution control analysis and design process. This increases 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. Accurate documentation is to be developed and provided during the following steps or phases of the plan development process:

- Surface water environmental (EIS) phase.
- Reconnaissance phase.
- Route location phase.
- Survey phase (drainage surveys).
- Preliminary engineering phase.
- Final design phase.
- Construction phase to include "as-built" plans.

- Operational and maintenance phase (continuous documentation is to occur over the structure's life cycle).

E. Responsibility. PennDOT's designer in consultation with the District Project Manager (PM) is responsible for determining what hydrologic analyses, hydraulic design and related information is documented during the plan development process. The designer, in conjunction with the charts and information contained in Appendix 4A, *Documentation Quick Reference Guide*, is to make a determination that complete documentation has been achieved as a result of the plan development process through final design.

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CHAPTER 4, APPENDIX A

DOCUMENTATION QUICK REFERENCE GUIDE

4A.0 INTRODUCTION

This guide is intended to provide a reference and web links to the documents, instructions, and applications for the various drainage related plans and reports required for PennDOT projects. The links were current at the time of publication, however, web sites oftentimes change URLs. Should the links become broken, enough key words have been provided in the description to allow the reader to search for the new link. Numbered items refer to the superscript in Chapter 4, *Documentation and Document Retention*.

A. NPDES Permit Application.

1. NOI. The PAG-2 Stormwater Discharges Associated with Construction Activities, NOI for Coverage under General or Individual Permit: www.elibrary.dep.state.pa.us/dsweb/View/Collection-9429.

2. Act 14 Letter. Notification must be given to each of the affected municipalities and counties with Act 14 letters. A sample Act 14 letter is included in Appendix 4C, *Sample Stormwater and Floodplain Management Consistency Letters*.

3. PNDI Form. Pennsylvania Natural Diversity Index (PNDI) Environmental Review through the Pennsylvania Natural Heritage Program: www.naturalheritage.state.pa.us/. A sample PNDI Environmental Review Receipt is included in Appendix 4D, *Sample PNDI Project Environmental Review Receipt*.

4. General Information Form (GIF). Department of Environmental Protection - General Information Form (GIF) – Authorization application: DOCUMENT NUMBER 8000-PM-IT0001 Rev 6/07/2002.
www.dep.state.pa.us/dep/subject/advcoun/oil_gas/2003/03_GIF_Form.pdf

5. Cultural Resource Notices. A basic search for registered historic places can be conducted through the National Register of Historic Places using their online National Register Information System at: www.nps.gov/nr/research. Application forms can be found at the Pennsylvania Historical and Museum Commission (PHMC) website:
http://www.portal.state.pa.us/portal/server.pt/community/pennsylvania_and_national_register_programs/3780

Completed Cultural Resource Notices should be mailed to the Pennsylvania Historical and Museum Commission (PHMC) at:

Bureau for Historic Preservation
Commonwealth Keystone Building
400 North Street
Harrisburg, PA 17120

6. Stormwater and Floodplain Management Consistency Letters. Samples are included in Appendix 4C, *Sample Stormwater and Floodplain Management Consistency Letters*.

7. Phased projects. Permit Guidelines for Phased NPDES Stormwater Discharges Associated with Construction Activity Permits, Chapter 102, *Erosion and Sediment Control Permits*, and Chapter 105, *Waterway Restoration Project Permits,* DOCUMENT NUMBER: 363-2134-013, March 29, 2003
www.elibrary.dep.state.pa.us/dsweb/Get/Document-48345/363-2134-013.pdf

8. General information on "NPDES Permits for Stormwater Discharges Associated with Construction Activities": www.elibrary.dep.state.pa.us/dsweb/View/Collection-10687.

B. Erosion and Sediment Pollution Control (E&SPC) Plan.

1. Chapter 102, *Erosion and Sediment Control:*
www.pacode.com/secure/data/025/chapter102/chap102toc.html

2. Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 13:
https://www.pa.gov/content/dam/copapwp-pagov/en/penndot/documents/public/pubsforms/publications/pub-13m/pub 13m title page.pdf

3. Publication 584, *PennDOT Drainage Manual*, Chapter 12, particularly Table 12.1:
https://www.pa.gov/content/dam/copapwp-pagov/en/penndot/documents/public/pubsforms/publications/pub-584/pub 584 title page.pdf

4. Publication 72M, *Roadway Construction Standards*. E&SPC controls are Series RC-70M through RC-77M:
https://www.pa.gov/content/dam/copapwp-pagov/en/penndot/documents/public/pubsforms/publications/pub-72m/72m_2010.pdf

5. Publication 14M, Design Manual, Part 3, *Plans Presentation*, Chapter 6:
https://www.pa.gov/content/dam/copapwp-pagov/en/penndot/documents/public/pubsforms/publications/pub-14m/pub 14m title page.pdf

C. Roadway Drainage Report (RDR).

1. Publication 584, Chapter 13, *Storm Drainage Systems*

2. Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.3.B:
https://www.pa.gov/content/dam/copapwp-pagov/en/penndot/documents/public/pubsforms/publications/pub-13m/pub 13m title page.pdf

D. Post Construction Stormwater Management (PCSM) Plan. Chapter 14, *Post-Construction Stormwater Management*

E. Preparedness, Prevention, & Contingency (PPC) Plan. Guidelines for the Development and Implementation of Environmental Emergency Response Plans DOCUMENT ID: 400-2200-001:
www.elibrary.dep.state.pa.us/dsweb/Get/Document-48522/400-2200-001.pdf

F. Water Obstruction and Encroachment Permits.

1. Joint Permit Application (JPA). Joint Permit Application for Pennsylvania Water Obstruction and Encroachment Permit and U.S. Army Corps of Engineers Section 404 permit:
www.elibrary.dep.state.pa.us/dsweb/View/Collection-9531

2. PA DEP permits search:
www.ahs.dep.pa.gov/eFACTSWeb/default.aspx

3. JPA_2 Expert System: www.dotdom1.state.pa.us/JPA2/jpahome.nsf

4. Environmental Assessment form: http://www.elibrary.dep.state.pa.us/dsweb/View/Collection-9534

5. Chapter 105. The Dam Safety and Waterway Management regulation (Title 25 of the PA Code):
www.pacode.com/secure/data/025/chapter105/chap105toc.html

6. Chapter 106. The Floodplain Management regulation (Title 25 of the PA Code):
www.pacode.com/secure/data/025/chapter106/chap106toc.html

7. Sections 401 and 404 (Clean Water Act): http://water.epa.gov/lawsregs/guidance/wetlands/sec401.cfm
http://water.epa.gov/lawsregs/guidance/wetlands/sec404.cfm

8. Section 10 of the Rivers and Harbors Act: www.epa.gov/OWOW/wetlands/regs/sect10.html

9. H&H Report. Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.7.B, Section 10.7.C, and Appendix C: https://www.pa.gov/content/dam/copapwp-pagov/en/penndot/documents/public/pubsforms/publications/pub-13m/pub 13m title page.pdf

10. Chapter 113. Floodplain Management regulation (Title 12 of PA Code): www.pacode.com/secure/data/012/chapter113/chap113toc.html

G. FEMA Conditional Letter of Map Revision (CLOMR) and Letter of Map Revision (LOMR).

1. FEMA Flood Hazard Mapping general page: https://www.fema.gov/national-flood-insurance-program-flood-hazard-mapping

2. FEMA MT-2 application forms applicable for CLOMR and LOMR requests: https://www.fema.gov/national-flood-insurance-program-flood-hazard-mapping/mt-1-application-forms-instructions.

3. FEMA documentation. FEMA research (Flood Insurance Studies (FIS's) and maps) can be conducted through the FEMA Map Service Center at:
msc.fema.gov/portal

4. Publication 13M**,** Design Manual, Part 2, *Highway Design,* Chapter 10, Appendix C:
https://www.pa.gov/content/dam/copapwp-pagov/en/penndot/documents/public/pubsforms/publications/pub-13m/pub 13m title page.pdf

H. Code of Federal Regulations Related to Special Flood Hazard Areas

1. CFR Title 44 – Emergency Management and Assistance, Part 65 – Identification and Mapping of Special Hazard Areas:

- Section 65.3 - Requirement to submit new technical data.
- Section 65.12 - Revision of flood insurance rate maps to reflect base flood elevations caused by proposed encroachments.

www.access.gpo.gov/nara/cfr/waisidx_02/44cfrv1_02.html

2. Coordination with FHWA - Location and Hydraulic Design of Encroachments on Flood Plains.

CFR Title 23 – Highways - Federal-Aid Policy Guide, 23 CFR 650A (OPI: HNG-31) Subchapter G - Engineering and Traffic Operations, *Part 650 - Bridges, Structures, and Hydraulics, Subpart A - Location and Hydraulic Design of Encroachments on Flood Plains*.
ecfr.gpoaccess.gov/cgi/t/text/text-idx?c=ecfr;sid=6426d25037301467da1f6ae3d98ec0cf;rgn=div5;view=text;node=23%3A1.0.1.7.28;idno=23;cc=ecfr#23:1.0.1.7.28.1

3. Coast Guard Coordination for Bridges over Navigable Water.

CFR Title 23 – Highways - Federal-Aid Policy Guide, 23 CFR 650A (OPI: HNG-31) Subchapter G - Engineering and Traffic Operations, *Part 650 - Bridges, Structures, and Hydraulics, Subpart H, Navigational Clearances for Bridges, Sections 805 through 807*.
ecfr.gpoaccess.gov/cgi/t/text/text-idx?c=ecfr;sid=6426d25037301467da1f6ae3d98ec0cf;rgn=div5;view=text;node=23%3A1.0.1.7.28;idno=23;cc=ecfr#23:1.0.1.7.28.1

4. USC Title 42 – The Public Health and Welfare, Chapter 50 – National Flood Insurance.
frwebgate.access.gpo.gov/cgi-bin/usc.cgi?ACTION=BROWSE&TITLE=42USCC50&PDFS=YES

I. Other Environmental Documentation References.

1. Wetlands. Preliminary wetland research can be conducted using the National Wetland Inventory (NWI) at: www.fws.gov/wetlands

2. Threatened and Endangered Species (T&E). Search the Pennsylvania Natural Diversity Index (PNDI) Environmental Review through the Pennsylvania Natural Heritage Program: www.naturalheritage.state.pa.us/. A sample PNDI Environmental Review Receipt is included in Appendix 4D, *Sample PNDI Project Environmental Review Receipt*.

3. High Quality (HQ) or Exceptional Value (EV) watershed designation. PA Code Title 25, Chapter 93. Chapter 93 also lists the Protected Water Uses, such as Trout Stocked Streams (TSF). This PA DEP publication can be accessed online at: www.pacode.com/secure/data/025/chapter93/chap93toc.html.

4. Wild Trout Streams. Class A wild trout streams are listed on the PA Fish and Boat Commission website at: www.fish.state.pa.us/classa98.htm. In addition, PA streams that support wild trout production are listed at: www.fish.state.pa.us/trout_repro.pdf.

J. Other Cultural Documentation References.

1. The National Registry of Natural Landmarks (Chapter 36 (Parks, Forests, and Public Property) in the Code of Federal Regulations (CFR) Part 62.2) can be accessed online at: www.gpoaccess.gov/cfr/.

2. Wild or Scenic Rivers in PA can be found on the Department of Conservation and Natural Resources (DCNR) website at: http://www.dcnr.state.pa.us/brc/conservation/rivers/scenicrivers/index.htm

K. Software:

1. US Army Corps of Engineers HEC Software downloads (HEC-RAS, HEC-2): www.hec.usace.army.mil/software

L. PennDOT Roadway Management System and Location Reference System:

1. *Location Reference System Introduction Manual*
https://www.dot.state.pa.us/public/Bureaus/BOMO/RM/RITS/LRS%20Introduction%20and%20Technical%20Manual.pdf

2. PennDOT Straight Line Diagrams
https://www.pa.gov/agencies/penndot/research-planning-and-innovation/roadway-management-testing/rms-annual-straight-line-diagrams.html

CHAPTER 4, APPENDIX B

SAMPLE NOTICE OF INTENT (NOI) APPLICATION FORM

4B.0 INTRODUCTION

An NPDES permit applies to earth disturbance activities that disturb five or more acres, or an earth disturbance on any portion, part, or during any stage of, a larger common plan of development or sale that involves five or more acres of earth disturbance, AND, earth disturbance activities with a point source discharge to surface waters of this commonwealth that disturb from one to less than five acres, or an earth disturbance on any portion, part, or during any stage of, including earth disturbance activities of less than one acre, that are part of a larger common plan of development or sale that involves one to less than five acres of disturbance with a point source discharge to surface waters of this Commonwealth.

This permit does not apply to road maintenance activities.

Construction activities which are not eligible for coverage under the General Permit as referenced in 25 PA Code Chapter 92, must utilize the Individual NPDES Permit Application for Stormwater Discharges Associated with Construction Activities. These activities include, but are not limited to, earth disturbance activities that are located in "special protection" watersheds (high quality, exceptional value, and exceptional value wetlands), or may affect existing water quality standards or threatened or endangered species and habitat, or have the potential for hazardous or toxic discharges.

The major components of a NPDES Permit Application includes:

- Erosion and Sediment (E&S) Control Plan.
- Pennsylvania Natural Diversity Inventory (PNDI) Search.
- Post Construction Stormwater Management (PCSM) Plan.
- Thermal Impact Analysis.
- Antidegradation Analysis.

Permit Applicants must fill out the Notice of Intent (NOI) for either the general or individual permit. A sample of how one should be completed is shown in Section 4B.1.

4B.1 SAMPLE NOTICE OF INTENT (NOI) FORM

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pennsylvania
DEPARTMENT OF ENVIRONMENTAL PROTECTION

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF ENVIRONMENTAL PROTECTION
BUREAU OF WATERSHED MANAGEMENT

OFFICIAL USE ONLY
ID # _____
Date Received _____

PERMIT APPLICATION
NOTICE OF INTENT FOR COVERAGE UNDER THE GENERAL (PAG-02) NPDES PERMIT
OR
APPLICATION FOR AN INDIVIDUAL NPDES PERMIT FOR STORMWATER DISCHARGES ASSOCIATED WITH CONSTRUCTION ACTIVITIES

PLEASE READ THE PERMIT SUMMARY SHEET AND INSTRUCTIONS PROVIDED IN THIS PERMIT APPLICATION PACKAGE BEFORE COMPLETING THIS FORM. COMPLETE THE ATTACHED CHECKLIST AND APPROPRIATE WORKSHEETS ATTACHED AFTER APPENDIX C OF THIS PERMIT APPLICATION. COMPLETE ALL APPLICABLE WORKSHEETS REFERENCED IN THE APPLICATION CHECKLIST.

PLEASE PRINT OR TYPE INFORMATION IN BLACK OR BLUE INK.

CHECK APPROPRIATE BOX	GENERAL ☐		INDIVIDUAL ☐ (Special Protection Waters)	
APPLICATION TYPE	NEW ☒	RENEWAL ☐	MAJOR MODIFICATION ☐	PHASED ☐

SECTION A. APPLICANT INFORMATION

Applicant's Last Name (District Executive)	First Name	MI	Phone FAX	
Email Address				
Organization Name or Registered Fictitious Name PennDOT District (#)-0			Phone FAX	
Mailing Address	City		State	ZIP + 4
			Employer ID (EIN)	
Co-Applicant's Last Name (if applicable)	First Name	MI	Phone FAX	
Email Address				
Organization Name or Registered Fictitious Name			Phone FAX	
Mailing Address	City		State	ZIP + 4

SECTION B. PROJECT INFORMATION AND SITE ANALYSIS

1. Project Name: SR (####) Section (###): SR (####) over Some Creek; etc.
2. Total Project Site (Acres): (R/W area between limits of work; includes temporary areas; NPDES permit boundary; round to 0.04 hectare (0.1 acre))
3. Total Disturbed Area (Acres): (all earth disturbances within the NPDES permit boundary)
4. Project Description
   Describe the primary activities: widening, new alignment, bridge replacement etc.

☐ Residential Subdivision	☐ Sewerage/Water System	☐ Private Road/Residence
☐ Commercial/Industrial	☒ Public Road	☐ Government Facility
☐ Utility Facility/Transmission	☐ Recreational	☐ Remediation/Restoration

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5. Project Location or Physical Address (if available):
SR (####) Section (###) near (name of nearest town)

6. County Municipality City Boro Twp
(list all within project limits) ☐ ☐ ☐
☐ ☐ ☐

7. Latitude: ____°/ ____'/ ____" Longitude: ____°/ ____'/ ____"
Collection Method: ☐ EMAP ☐ HGIS ☐ GISDR ☐ ITPMP ☐ GPS ☐ WAAS ☐ LORAN
Check the horizontal reference datum (or projection datum) employed in the collection method. EMAP and HGIS (PNDI) have known datum and do not require checking here. ☐ NAD27 ☐ NAD83 ☐ WGS84 (GEO84)
Enter the date of collection if the lat and long coordinates were derived from GPS, WAAS or LORAN. _____ mm _____ dd _____ yyyy

8. U.S.G.S. Quad Map Name (list all quad names covered by the project)

9. Existing and Previous Uses of the Land Proposed for Construction (use separate sheet if necessary):
Existing Land Uses: ☐ Agriculture ☐ Forest/Woodland ☐ Barren ☐ Urban ☐ Brownfield ☐ Other
Description: (typically highway)
Previous Land Uses: ☐ Agriculture ☐ Forest/Woodland ☐ Barren ☐ Urban ☐ Brownfield ☐ Other
Description: (typically highway)

10. **Site Analysis**

a. Describe how Natural Resources features on the site (Worksheets 2 and 3 referenced in the Pa. Stormwater BMP Manual) were considered in: Location and Design of the project, E & S Plan Design, PCSM Plan Design. (attach additional sheet if necessary) (example) Sensitive natural features, such as waterbodies, floodplains, riparian areas, wetlands, woodlands, natural drainage ways, and steep slopes, are identified on the E&SPC and PCSM plans. Disturbance of these features has been minimized as much as practicable.

b. Identify naturally occurring geologic formations or soil conditions that may have the potential to cause pollution during earth disturbance activities and include BMPs to avoid or minimize potential pollution and its impacts from the formation.
N/A

11. Potential Toxic or Hazardous Pollutants: (Submit the following data if soil contaminant, geology or past or present land use provides a potential for contaminated runoff from the project site) N/A ☐ Use additional sheets if necessary.

Pollutant	Concentration w/Units	Source	Sample Type	Date(s) / Number of Samples
(Consult the District environ. unit or consultant responsible for environ. site assessment.)				

12. **Fill Material**

Based on a cut/fill analysis of the project site, will the site need to import fill, export fill or will the site balance? Be sure to read the instructions before completing this section. Clean Fill can not be placed in or on waters of the Commonwealth.

Check the appropriate box

☐ Import fill – the Operator will, in most situations, be responsible to perform environmental due diligence and determine that all fill imported to the site meets the department's definition of clean fill. The plan designer must include a note on the drawings to identify the operator(s) responsibility and provide the definition of Clean Fill and Environmental Due Diligence.

☐ Export fill – the Applicant is responsible for performing environmental due diligence at the time this application was submitted to determine that any fill exported from the site will be certified as clean fill.

☐ Balance all cuts and fills with the amount of rock and soil available on the site.

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13. Estimated Timetable for Phased Projects Build Out (Complete for phased projects only)

Phase No. or Name	Proposed Type of Activity	Total Area	Disturbed Area	Start Date	End Date
n/a	Full build out - (X) years	(copy #2)	(copy #3)	mo/yr	mo/yr

14. Stormwater Discharges to nearest receiving stream (during construction). Check all that apply:

Waters of the Commonwealth ☐ Municipal Separate Storm Sewer ☐ Private Storm Sewer ☐ Non Surface Waters ☐

Impaired Waters According to Category 4 or 5 of PA Integrated Water Quality Monitoring and Assessment Report ☐ check eMap

If waters are impaired list type of impairment: (e.g., aquatic life)

Receiving Water/Watershed Name:	Chapter 93 Receiving Water Classification: (Designated use)	Existing Use (if different from the Designated use)
(name) Creek, UNT to (name) Creek etc.	CFW, WWF, TSF, HQ, EV, etc.	CWF, WWF, TSF, HQ, EV, etc.
Name of Municipal Storm Sewer Operator: (if municipal separate storm sewer is checked above)	Name of Private Storm Sewer Operator: (if private storm sewer is checked above)	Other: (including off-site discharges)

Will you meet CG-1? ☐ Yes ☒ No (CG-1 includes a 20% meadow condition; applying PennDOT Policy or Act 167)

If no, you may need to use worksheets 11 through 13.

SECTION C. E & S AND POST CONSTRUCTION STORMWATER MANAGEMENT (PCSM) PLAN

Note: For projects involving multiple watershed boundaries, please submit a complete, separate Section C for each additional watershed.

1. Provide a brief summary of proposed BMPs and their performance to manage E & S for the project. If E & S BMPs and their application do not follow the guidelines referenced in the Pa. Erosion and Sediment Pollution Control Program Manual, provide documentation to demonstrate peformance equivalent to, or better than, the BMPs in the Manual.

E & S BMPs

(bullet list of all E&S BMPs shown on the E&SPC Plan)

2. **PCSM Plan Information** - The PCSM Plan should be designed to maximize volume reduction technologies, eliminate (where possible) or minimize point source discharges to surface waters, preserve the integrity of stream channels, and protect the physical, biological and chemical qualities of the receiving surface water. **The DEP recommends the use of Control Guideline 1 (CG1) referenced in the Pa. Stormwater BMP Manual to achieve this goal.**

Design standards applied to develop the PCSM Plan. Check those that apply.

☐ Act 167 Plan - The attached PCSM plan is consistent with an applicable approved Act 167 Plan. **A letter of consistency from the Municipal or County Engineer should be provided with the application.** Complete and submit all applicable worksheets referenced in the application checklist as part of the permit application for each approved Act 167 Plan.

Complete the following table for all applicable approved Act 167 Stormwater Management Plans. (use additional sheets if necessary)

ACT 167 Plan Name	Date Adopted	Consistency Letter Included ☐ / Consistency Letter Pending ☐

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☐ ~~The attached PCSM plan is consistent with all applicable local stormwater management ordinances, including MS4 (NPDES Permit to Discharge Stormwater Through a Municipal Separate Storm Sewer System) ordinances.~~ ~~**A letter of consistency from the Municipal or County Engineer should be provided with the application.**~~ ~~Complete and submit all applicable worksheets referenced in the application checklist as part of the permit application.~~

~~Complete the following table for all applicable Municipalities. (use additional sheets if necessary)~~

~~Municipality Name~~	~~Ordinance Number~~	~~Consistency Letter Included~~ ☐
_____	_____	~~Consistency Letter Pending~~ ☐

The PCSM Plan must satisfy either subparagraph A, B **or** C below. Check those that apply.

A. ☐ Act 167 Plan approved on or after January 2005 – The attached PCSM Plan, in its entirety, is consistent with all requirements pertaining to rate, volume, and water quality from an Act 167 Stormwater Management Plan approved by DEP on or after January 2005.

B. ☐ The PCSM Plan meets the standard design criteria from the PA Stormwater BMP Manual.

OR

C. ☐ Alternative Design Standard – The attached PCSM plan was developed using approaches other than 102.8(g)(2). Demonstrate/explain in the space provided how this standard will be either more protective than what is required in 102.8(g)(2) or will maintain and protect existing water quality and existing and designated uses.

(use only the parts that apply) The post-construction stormwater management design for this project is consistent with PennDOT Antidegradation and Post Construction Stormwater Management Policy. As a PCSM Level (1-4) project, the design targets are as follows: (Level 1 - minimize disturbance), (Level 2 - capture 5.08 cm (2") of runoff impervious area contributing to existing BMPs/infiltrate 2.54 cm (1") from new impervious ares), (Level 3 and Level 4 - pre = post peak runoff rate for the 2-100 year events/pre = post runoff volume for the 2-year 24 hour event/ consistency with water quality standards).

Where it is not practicable or feasible to achieve the quantitative targets described above, BMPs have been implemented to maximize qualitative stormwater benefits (e.g. natural infiltration, shading, riparian, restoration, vegetative slope and channel protection, etc.) The net result is that the project will not affect existing or designated uses of the receiving waters, nor will it adversely affect water quality. Additionally, the design is consistent with the (release rate, volume control, water quality) provisions in the approved Act 167 Plan for (name) Creek, which was approved prior to January 2005.

3. **Riparian Buffers**

A. Will you be protecting, converting or establishing a riparian buffer or a riparian forest buffer as a part of this project?
☐ Yes ☐ No

B. If the regulations require a riparian buffer or riparian forest buffer and you are not providing one, please list the waiver provisions in the Chapter 102 regulations, Section 102.14(d)(2)(*i*)-(*vi*), that you are requesting and provide additional documentation to demonstrate reasonable alternatives for compliance with 102.14 requirements.

C. Will you be protecting, converting or establishing a voluntary riparian forest buffer as part of this project? ☐ Yes ☐ No

If yes you must include a Riparian Forest Buffer Management Plan as part of the PCSM plans.

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Note: Complete one table per receiving surface water (not necessarily per discharge point from ROW).

4. **Summary Table for Supporting Calculation and Measurement Data**

Please reference the Stormwater Methodology used (Numbers generated in this table should be consistent with worksheets 1-5.)

NRCS CN / Rational

	Pre-construction	Post Construction	Net Change
Design storm frequency 2 year Rainfall amount 24-hr P from PDT - inches IDF curves in DM2			
Impervious area (acres) include gravel shoulders	1 Per watershed	2	3 +/- 0.1 ac
Volume of stormwater runoff ☐ acre-feet or ☐ cubic feet without planned stormwater BMPs (check appropriate box)	4 Per watershed	5	6 +/- 0.10 ac-ft +/- 10ft^3
Volume of stormwater runoff ☐ acre-feet or ☐ cubic feet with planned stormwater BMPs (check appropriate box)		7	8 +/- 0.10 ac-ft +/- 10 ft^3
Stormwater peak discharge rate for the design frequency storm (cubic feet per second)	9 At the point where the discharge reaches the surface water	10	11 +/- 1 cfs

Box 1. **Pre-construction impervious area**: The total acres of impervious area on the project site before construction activities begin, based on land use for five years preceding the planned project.

Box 2. **Post construction impervious area**: The total acres of impervious area on the project site after construction activities have been completed.

Box 3. **Net change of impervious area**: The difference between the acres of impervious area listed in Box 1 and Box 2. Zero or negative values are acceptable.

Box 4. **Pre-construction stormwater runoff volume without planned BMPs**: The amount of stormwater runoff volume from the project site that would result from the design storm occurrence before construction activities begin, based on land use for five years preceding the project.

Box 5. **Post construction stormwater runoff volume without planned BMPs**: The amount of stormwater runoff volume from the project site that would result from the design storm occurrence after construction activities have finished assuming that no stormwater infiltration or retention BMPs have been installed.

Box 6. **Net change in stormwater volume without planned BMPs**: The difference between the amounts of stormwater runoff volume listed in Box 4 and Box 5.

Box 7. **Post construction stormwater runoff volume with planned BMPs**: The amount of stormwater runoff volume from the project site that would result from the design storm occurrence after construction activities have finished and the planned stormwater infiltration or retention BMPs have been installed.

Box 8. **Net change in stormwater runoff volume with planned BMPs**: The difference between the amounts of stormwater runoff volume listed in Box 4 and Box 7.

Box 9. **Pre-construction stormwater discharge rate**: The stormwater runoff discharge rate for the design frequency storm as determined by the land use for the past five years.

Box 10. **Post construction stormwater discharge rate**: The stormwater runoff discharge rate for the design frequency storm event after all planned stormwater BMPs are installed.

Box 11. **Net change stormwater discharge rate**: The difference between the stormwater runoff discharge rates listed in Box 9 and Box 10.

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5. **Summary Description of Post Construction Stormwater BMPs (consistent with the design or applicable worksheets)**
(underlined indicates primary SW benefit)
Key: **RC = Rate Control** **VC = Volume Control** **WQ = Water Quality**

In the lists below, check the BMPs identified in the PCSM Plan, and their function(s) using the above Key. More than one function may be checked for a BMP. List the stormwater volume and area of runoff to be treated by each BMP type. If any BMP in the PCSM Plan is not listed below, describe it in the space provided after "Other".

BMP	Function(s)	Volume of stormwater treated	Acres treated
☐ Wet ponds	☐ VC ☐ RC ☐ WQ	↑	↑
☐ Constructed wetlands	☐ VC ☐ RC ☐ WQ		
☐ ~~Retention basins~~	☐ VC ☐ RC ☐ WQ		
☐ ~~Detention basin~~	☐ VC ☐ RC ☐ WQ	Note that the volume treated by each BMP is simply the total volume of runoff that is routed to/ through the BMP for the indicated storm event (for purposes of the permit, will always be the 2-year 24-hour storm event).	
☐ Underground detention	☐ VC ☐ RC ☐ WQ		
☐ Dry Extended detention basin	☐ VC ☐ RC ☐ WQ		
☐ Sediment fore bay	☐ VC ☐ RC ☐ WQ		
☐ Infiltration trench	☐ VC ☐ RC ☐ WQ		This it the total area contributing runoff to the BMP.
☐ Infiltration Berm/Retentive Grading	☐ VC ☐ RC ☐ WQ		
☐ ~~Subsurface Infiltration bed~~	☐ VC ☐ RC ☐ WQ		
☐ Infiltration basin	☐ VC ☐ RC ☐ WQ		
☐ Pervious pavement (limited application)	☐ VC ☐ RC ☐ WQ		
☐ ~~Dry well/Seepage pit~~	☐ VC ☐ RC ☐ WQ		
☐ ~~Bio-infiltration areas~~	☐ VC ☐ RC ☐ WQ	↓	↓
☐ Rain gardens/Bio-retention	☐ VC ☐ RC ☐ WQ		
☐ Vegetated swales	☐ VC ☐ RC ☐ WQ		
☐ ~~Constructed filters~~	☐ VC ☐ RC ☐ WQ		
☐ Protect Sensitive & Special Value Features	☐ VC ☐ RC ☐ WQ	Use the volume "Credit" indicated in the DEP BMP Manual for each of these non-structural/ restoration BMPs.	Use the total area protected/restored (first 3 BMPs); for the other BMP use the drainage area directed away from a storm sewer.
☐ Protect/Convert/Establish Riparian buffers	☐ VC ☐ RC ☐ WQ		
☐ Restoration: Buffers/ Landscape/Floodplain	☐ VC ☐ RC ☐ WQ		
☐ Disconnection from storm sewers	☐ VC ☐ RC ☐ WQ		
☐ ~~Rooftop disconnection~~	☐ VC ☐ RC ☐ WQ		
☐ ~~Vegetated roofs~~	☐ VC ☐ RC ☐ WQ		
☐ ~~Runoff capture/Reuse~~	☐ VC ☐ RC ☐ WQ		
☐ Oil/grit separators (req. special approval)	☐ WQ		
☐ Water quality inserts/inlets (req. special approval)	☐ WQ		
☐ Street sweeping (confirm w/District Maint.)	☐ WQ		
☐ Other Bioslope / Ecology Embankment	☐ VC ☐ RC ☐ WQ		
☐ Other _____	☐ VC ☐ RC ☐ WQ		

6. **Off Site Discharge Analysis**

Does the project propose any off-site discharges to areas other than surface waters? ☐ **Yes** ☐ **No**

If yes, the applicant must have appropriate easement that provides the legal authority for this off-site discharge.

Applicant must provide a demonstration in both the E&S and PCSM plans that the discharge will not cause erosion, damage, or nuisance to off-site properties.

Note: This should be completed for point-source discharge (i.e., not sheet flow from pavements surface or embarkment) that are not directed into a swale, ditch, channel, storm sewer, etc.

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7. **Thermal Impacts Analysis**

Please explain how thermal impacts associated with this project were avoided, minimized, or mitigated.

The following strategies were used to avoid or mitigate potential thermal impacts:

(List all applicable strategies or BMPs to reduce or limit thermal degradation.)

8. Identify the critical stages of implementation of the PCSM plan for which a licensed professional or designee shall be present on site:

n/a

(Indicate something other than "not applicable" here only when a large amount of new impervious area discharges directly into a cold water, headwater stream, without first being conveyed through several hundred feet of either a vegetated swale or a buried storm sewer pip).

SECTION D. ANTIDEGRADATION ANALYSIS MODULE

This Section is to be completed for Special Protection Watershed Only. (HQ/EV and EV Wetlands)

PART 1 NON-DISCHARGE ALTERNATIVES EVALUATION

The applicant must consider and describe any and all non-discharge alternatives for the entire project area which are environmentally sound and will:

- **Minimize accelerated erosion and sedimentation during the earth disturbance activity**
- **Achieve no net change from pre-development to post-development volume, rate and concentration of pollutants in water quality**

E & S Plan	Official Use Only	PCSM Plan	Official Use Only
Check off the environmentally sound non-discharge Best Management Practices (BMPs) listed below to be used prior to, during, and after earth disturbance activities that have been incorporated into your E & S Plan based on your site analysis. For BMPs not checked, provide an explanation of why they were not utilized. (attach additional sheets if necessary)		Check off the environmentally sound non-discharge Best Management Practices (BMPs) listed below to be used after construction that have been incorporated into your PCSM Plan based on your site analysis. For BMPs not checked, provide an explanation of why they were not utilized. (attach additional sheets if necessary)	
Non-discharge BMPs ☐ Alternative Siting ☐ Alternative location ☐ Alternative configuration ☐ Alternative location of discharge ☐ Limited Disturbed Area ☐ Limiting Extent & Duration of Disturbance (Phasing, Sequencing) ☐ Riparian Buffers (150 ft min) ☐ Riparian Forest Buffer (150 ft min) ☐ Other _____		**Non-discharge BMPs** ☐ Alternative Siting ☐ Alternative location ☐ Alternative configuration ☐ Alternative location of discharge ☐ Low Impact Development (LID / BSD) ☐ Riparian Buffers (150 ft min) ☐ Riparian Forest Buffer (150 ft min) ☐ Infiltration ☐ Water Reuse ☐ Other _____	

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Part 2 Antidegradation Best Available Combination of Technologies (ABACT)

If the net change in stormwater discharge from or after construction is not fully managed by non-discharge BMPs, the applicant must utilize ABACT BMPs to manage the difference. The Applicant must specify whether the discharge will occur during construction, post-construction or both, and identify the technologies that will be used to ensure that the discharge will be a non-degrading discharge. ABACT BMPs include but are not limited to:

E & S Plan	Official Use Only	PCSM Plan	Official Use Only
☐ **Treatment BMPs:** ☐ Sediment basin with skimmer ☐ Sediment basin ratio of 4:1 or greater (flow length to basin width) ☐ Sediment basin with 4-7 day detention ☐ Flocculants ☐ **Land disposal:** ☐ Vegetated filters ☐ Riparian buffers <150ft. ☐ Riparian Forest Buffer <150ft. ☐ Immediate stabilization ☐ **Pollution prevention:** ☐ PPC Plans ☐ Street sweeping ☐ Channels, collectors and diversions lined with permanent vegetation, rock, geotextile or other non-erosive materials ☐ **Stormwater reuse technologies:** ☐ Sediment basin water for dust control ☐ Sediment basin water for irrigation ☐ **Other** _____		☐ **Treatment BMPs:** ☐ Infiltration Practices ☐ Wet ponds ☐ Created wetland treatment systems ☐ Vegetated swales ☐ Manufactured devices ☐ Bio-retention/infiltration ☐ Green Roofs ☐ **Land disposal:** ☐ Vegetated filters ☐ Riparian Buffers <150ft. ☐ Riparian Forest Buffer <150ft. ☐ Disconnection of roof drainage ☐ Bio-retention/bio-infiltration ☐ **Pollution prevention:** ☐ Street sweeping ☐ Nutrient, pesticide, herbicide or other chemical application plan alternatives ☐ PPC Plans ☐ Non-structural Practices ☐ Land Preservation ☐ Restoration BMPs ☐ **Stormwater reuse technologies:** ☐ Cisterns ☐ Rain barrels ☐ Dry hydrant with underground storage ☐ Spray/Drip Irrigation ☐ **Other** _____	
Are the ABACT BMPs selected sufficient to minimize E & S discharges to the extent that existing or designated surface water uses are protected? ☐ Yes ☐ No. If no, and the project is located in a HQ water, proceed to Part 3.		Are the ABACT BMPs selected sufficient to achieve no net change to the extent that existing or designated surface water uses are protected? ☐ Yes ☐ No. If no, and the project is located in a HQ water, proceed to Part 3.	

Part 3 Social or Economic Justification (SEJ) (for projects in high quality waters only)

If the applicant cannot demonstrate that the net change in discharge will protect the existing quality of the receiving surface waters, for projects in HQ waters, the applicant may pursue the SEJ process for demonstrating that lowering water quality is necessary to accommodate important economic or social development in the area in which the waters are located, in accordance with Chapter 10 of the Water Quality Antidegradation Implementation Guidance Manual, DEP Document ID No. 391-0300-002.

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SECTION E. CONSULTANT FOR THIS PROJECT

Last Name	First Name	MI
Doe	John	C

Title	Consulting Firm
Project Engineer	XYZ Engineering

Mailing Address: 123 Market St.

City	State	ZIP+4
Somewhere	PA	##### ####

Email: jdoe@xyzengineering.com

Phone ### #### Ext

FAX ### ####

SECTION F. COMPLIANCE HISTORY REVIEW

Is/was the applicant(s) in violation of any permits issued by DEP or any regulated activities within the past five years?

☐ Yes ☐ No Follow the directions below and then answer yes or no.

If yes, list each permit or project that is/was in violation and provide compliance status of the activity (use additional sheets to provide information on all permits).

Permit Program or Activity: Permit Number (if applicable):

Brief description of non-compliance:

Complete the following steps for Section F:

1 - Go to http://www.ahs2.dep.state.pa.us/eFactsWeb/default.aspx

2 - Click on "Inspection Search"

3 - Search for violations using all of PennDOT's Client IDs (see table on next page)

4 - Print violations within the last 5 years and attach to permit

Steps taken to achieve compliance	Date(s) Compliance Achieved

Current Compliance Status: ☐ In-Compliance ☐ In Non-Compliance

If the applicant is not in compliance with any permit requirement of DEP Regulations or regulated activity, provide a narrative description of how the applicant will achieve compliance with the permit requirement or activity, including the schedule for achieving compliance with appropriate milestones.

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SECTION G. PERMIT COORDINATION

Does the applicant (owner and/or operator) have, have pending, or require any other environmental permits for this project and any additional planning requirements?

☐ Yes ☐ No If yes, list each permit or approval, permit number, and description.

(List any waterway or other types of permits required for the project.)

Coordination Questions

1. Does the project involve any of the following: Placement of fill, excavation within or a placement of a structure located in, along, across, or projecting into a water course, floodway or body of water (including wetlands)?

   ☐ Yes ☐ No If yes, identify which authorization under Chapter 105 is applicable.

   ☐ Joint Permit ☐ General Permit ☐ Waiver

2. What is your 537 Plan status? Please note that 537 Plan approval is required prior to initiation of earth disturbance activity.

   n/a

3. Is your project associated with a Brownfield's Remediation? ☐ Yes ☐ No If yes, please indicate any coordination to date with the Environmental Cleanup Program (Act 2 or Superfund).

4. Are there any additional permits or approvals that may be required for this project? ☐ Yes ☐ No If yes, please list them.

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SECTION H. CERTIFICATION

Applicant Certification

I certify under penalty of law that this application and all related attachments were prepared by me or under my direction or supervision by qualified personnel to properly gather and evaluate the information submitted. Based on my own knowledge and on inquiry of the person or persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate and complete. The responsible official's signature also verifies that the activity is eligible to participate in the NPDES permit, and that BMP's, E&S Plan, PPC Plan, PCSM Plan, and other controls are being or will be, implemented to ensure that water quality standards and effluent limits are attained. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment or both for knowing violations pursuant to Section 309(c)(4) of the Clean Water Act and, 18 Pa. C.S. §§4903-4904.

Applicant	**Co-Applicant (if applicable)**
(District Executive)	
Print Name and Title of Person Signing	Print Name and Title of Person Signing
(###) ###-####	( )
Telephone Number of Person Signing	Telephone Number of Person Signing
SIGN HERE	
Signature of Applicant	Signature of Co-Applicant
mo/day/year	
Date Signed	Date Signed

Please note below the name, address and telephone number of the individual that should be contacted in the event additional information is required.

Name: (this can be either the consultant or someone at the District)

Address: _____

Telephone: ( ) _____ FAX: ( ) _____

Notarization:

Commonwealth of Pennsylvania

County of _____

Sworn to and Subscribed to Before Me This

_____ Day of _____, 20_____

NOTARY

SEAL

WITNESS

_____

Notary Public

My Commission Expires: _____

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Checklist

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF ENVIRONMENTAL PROTECTION
BUREAU OF WATERSHED MANAGEMENT

APPLICATION CHECKLIST
GENERAL NPDES PERMIT FOR STORMWATER DISCHARGES ASSOCIATED WITH CONSTRUCTION ACTIVITIES

Please check the following list to make sure that you have included all the required information. Place a check mark in the column provided for all items completed and/or provided. Failure to provide all of the requested information will delay the processing of the application and may result in the application being placed ON HOLD with NO ACTION, or being considered withdrawn and the application file closed.

THIS CHECKLIST MUST BE COMPLETED AND ENCLOSED WITH YOUR GENERAL PERMIT APPLICATION FORM

	✓CHECKLIST FOR NEW GENERAL NPDES PERMIT APPLICATION			Applicant Check √ If Included	Official Use Only
1.	Fully completed, properly signed and notarized Notice of Intent Form (1 original and 2 copies).			☐	☐
2.	Complete Erosion and Sediment Control Plans. (3 copies) Location: Drawings (D), Narrative (N).			☐	☐
	a. Written Narrative *(Must be labeled "E&S Plan" or "Erosion & Sediment Control Plan", be complete & legible, and be the final plan for construction)* Written Narrative Includes the following:	Location **N**	Page _____	☐	☐
	i. USGS map with outline of project site	Location **N**	Page _____	☐	☐
	ii. Soils information (including hydric soils) Types, depth, slope and locations of soils	Location **N**	Page _____	☐	☐
	iii. Physical characteristics and limitations of soils	Location **N**	Page _____	☐	☐
	iv. Supporting calculations to show anticipated peak flows for the design storms	Location **N**	Page _____	☐	☐
	v. Analysis of the impact that runoff from the project site will have on existing downstream watercourses resistance to erosion	Location **N**	Page _____	☐	☐
	vi. Provide supporting calculations, standard worksheet, and narrative description of the location for all proposed E&S Control BMPs used before, during and after earth disturbance including but not limited to the following:				
	A. Channels	Location **N**	Page _____	☐	☐
	B. Sediment Basins	Location **N**	Page _____	☐	☐
	C. Sediment Traps	Location **N**	Page _____	☐	☐
	D. Filter Fabric Fencing	Location **N**	Page _____	☐	☐
	E. Outlet Protection	Location **N**	Page _____	☐	☐
	F. Other BMPs (Specify) _____	Location **N**	Page _____	☐	☐

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Checklist

				Applicant Check √ If Included	Official Use Only
	G. Other BMPs (Specify) _____	Location **N**	Page _____	☐	☐
b.	Plan Drawings *(Must be labeled "E&S Plan" or "Erosion & Sediment Control Plan", be complete & legible, and be the final plan for construction)* Drawings include the following:	Location **D**	Page _____	☐	☐
	i. Legend for any symbols that may be used on the drawing	Location **D**	Page _____	☐	☐
	ii. Topographic Features including existing contours, improvements, streams, wetlands, watercourses, etc. and sufficient surrounding area	Location **D**	Page _____	☐	☐
	iii. Soil types and locations	Location **D**	Page _____	☐	☐
	iv. Construction techniques or special considerations to address soil limitations	Location **D**	Page _____	☐	☐
	v. Limits of project site, NPDES boundary	Location **D**	Page _____	☐	☐
	vi. Limits of earth disturbance	Location **D**	Page _____	☐	☐
	vii. Proposed alteration including proposed contours and proposed improvements	Location **D**	Page _____	☐	☐
	viii. Maximum during construction drainage areas to hydraulic BMPs	Location **D**	Page _____	☐	☐
	ix. Location of water which may receive runoff and receiving water classification pursuant to Chapter 93 and the "statewide existing use listing"	Location **D**	Page _____	☐	☐
	x. Standard Construction Details for all proposed E&S Control BMPs used before, during and after earth disturbance	Location **D**	Page _____	☐	☐
	xi. Location of BMPs showing final contours are identified	Location **D**	Page _____	☐	☐
	xii. Complete and site specific sequence of BMP installation and removal including activities planned to limit exposed areas	Location **D**	Page _____	☐	☐
	xiii. Procedures or Note requiring the proper recycling or disposal of waste materials associated with the project site	Location **D**	Page _____	☐	☐
	xiv. Maintenance Program including inspection schedule, sediment cleanout levels, repair parameters and time frames, and directions for sediment removal	Location **D**	Page _____	☐	☐
	xv. Note explaining responsibilities for fill materials including definition of environmental due diligence and clean fill	Location **D**	Page _____	☐	☐

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				Applicant Check √ If Included	Official Use Only
3.	Permit filing fee of $500 payable to the appropriate Clean Water Fund.			☐	☐
4.	Disturbed acre fee payable to the Commonwealth of Pennsylvania Clean Water Fund.			☐	☐
5.	Notifications to the local municipality and county governments that specify Acts 67 and 68 Coordination, and that the application is for a general NPDES stormwater permit authorizing the discharge of stormwater during construction activities. A "sample" notification letter is provided in Appendices B and C.			☐	☐
6.	Proof of receipt of municipal notifications; copies of certified mail receipts or acknowledgment letters from the local municipality and county government. (3 copies)			☐	☐
7.	The PNHP Review receipt for the project area. Include impact clearance letters if proof of agency coordination is required. (3 copies)			☐	☐
8.	Complete Post Construction Stormwater Management Plan. (3 copies) Location: Drawings (D), Narrative (N).			☐	☐
	a. Written Narrative *(Must be separate from E&S Plan and labeled "PCSM" or Post-Construction Stormwater Management"" and be the final plan for construction)* Written Narrative Includes the following:	Location **N**	Page _____	☐	☐
	i. Site Description & Analysis	Location **N**	Page _____	☐	☐
	ii. Soil types and descriptions (including hydric soils)	Location **N**	Page _____	☐	☐
	iii. Pre-development and post-development drainage area runoff calculations for each drainage area	Location **N**	Page _____	☐	☐
	iv. Routing Analysis to demonstrate peak control for the 2-, 10-, 50-, and 100-year/24-hour storm events *(Routing should consider the benefits of BMPs)*	Location **N**	Page _____	☐	☐
	v. Calculations for permanent stormwater BMPs *(including volume of water treated through BMPs)*	Location **N**	Page _____	☐	☐
	vi. Curve Numbers and/or land use coefficients	Location **N**	Page _____	☐	☐
	vii. Infiltration/Geotechnical report and soil infiltration test pit results	Location **N**	Page _____	☐	☐

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Checklist

				Applicant Check √ If Included	Official Use Only
	b. Additional Worksheets **Note: Worksheets 1 through 5 are required. Complete the following worksheets as applicable.**				
	i. Worksheet 6 – Small Site/Small Impervious Area Exception for peak rate Mitigation Calculations ***(If worksheet 6 is not applicable, rate control is required)***	Location **N**	Page _____	☐	☐
	ii. Worksheet 10 – Water Quality Compliance for Nitrate	Location **N**	Page _____	☐	☐
	iii. Worksheet 11 – BMPs for Pollution Prevention ***(Required if applicant is not meeting Nitrate requirements)***	Location **N**	Page _____	☐	☐
	iv. Worksheet 12 – Water Quality Analysis of Pollutant Loading from all Disturbed Areas ***(Required if applicant is not meeting Nitrate requirements)***	Location **N**	Page _____	☐	☐
	v. Worksheet 13 – Pollutant Reduction Through BMP Applications ***(Required if applicant is not meeting Nitrate requirements)***	Location **N**	Page _____	☐	☐
	c. Plans/Drawings *(Must be a stand alone separate plan from the E&S Plan and labeled "PCSM" or Post-Construction Stormwater Management"" and be the final plan for construction)*	Location **D**	Page _____	☐	☐
	i. Construction Details for permanent stormwater BMPs including permanent stabilization	Location **D**	Page _____	☐	☐
	ii. Location of BMPs showing final contours are identified	Location **D**	Page _____	☐	☐
	iii. Location of soil types are identified (including hydric soils)	Location **D**	Page _____	☐	☐
	iv. Location and depths of test pits / infiltration testing sites are identified (where applicable)	Location **D**	Page _____	☐	☐
	d. Ownership, Operations, and Maintenance Procedures *(Must be included on drawings)*	Location **D**	Page _____	☐	☐
	i. Applicant or entity (association, company, agency, etc.) listed as responsible party	Location **D**	Page _____	☐	☐

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				Applicant Check √ If Included	Official Use Only
	e. Riparian Forest Buffer Management Plan (If Applicable) Location: Drawings (D), Narrative (N).			☐	☐
	i. A Planting Plan for converted or newly established Riparian Forest Buffers.	Location **D**	Page _____	☐	☐
	ii. A maintenance schedule and measures for converted or newly established Riparian Forest Buffers to ensure growth and survival.	Location **N D**	Page _____	☐	☐
	iii. An inspection schedule and measures to ensure long-term maintenance and proper functioning of Riparian Forest Buffers.	Location **N**	Page _____	☐	☐
	f. Identification of critical stages of implementation of PCSM Plan for which a licensed professional or designee will be present on site.	Location: **N D**	Page _____	☐	☐
9.	Consistency letter from Municipal or County Engineer (where applicable)			☐	☐
10.	Appendix A Land Use Questions			☐	☐
11.	Complete Required Worksheets 1 – 5 (see attached worksheets **at the end of** the NPDES Permit Application Package)			☐	☐
12.	Checklist for Subsequent Phases (of permitted projects)				
	a. Estimated time frame for phased project build-out (update as necessary)			☐	☐
	b. Complete E & S Plans for specific phase (3 copies)			☐	☐
	c. New Section C and complete PCSM Plan for specific phase (3 copies)			☐	☐
	d. Consistency letter from municipal or county engineer (where applicable)			☐	☐
	CHECKLIST FOR GENERAL NPDES PERMIT RENEWALS ONLY			**Applicant Check √ If Included**	**Official Use Only**
1.	Administratively complete, signed, and notarized Notice of Intent Form, including items 1-8. (1 signed original and 2 copies of the NOI/application)			☐	☐

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APPLICATION CHECKLIST
NPDES **INDIVIDUAL** PERMIT FOR DISCHARGES OF STORMWATER
ASSOCIATED WITH CONSTRUCTION ACTIVITIES

Please check the following list to make sure that you have included all the required information. Place a check mark in the column provided for all items completed and/or provided. Failure to provide all of the requested information will delay the processing of the application and may result in the application being placed ON HOLD with NO ACTION, or being considered withdrawn and the application file closed.

THIS CHECKLIST MUST BE COMPLETED AND ENCLOSED WITH YOUR INDIVIDUAL PERMIT APPLICATION FORM

	CHECKLIST FOR NEW INDIVIDUAL NPDES STORMWATER PERMIT APPLICATION			Applicant Check √ If Included	Official Use Only
1.	Fully completed, properly signed and notarized Individual Permit Application (1 original and 2 copies).			☐	☐
2.	Fully completed General Information Form (GIF) (1 original and 2 copies)			☐	☐
3.	Complete Erosion and Sediment Control Plan (3 copies) Location: Drawings (D), Narrative (N).			☐	☐
	a. Written Narrative *(Must be labeled "E&S Plan" or "Erosion & Sediment Control Plan", be complete & legible, and be the final plan for construction)* Written Narrative Includes the following:	Location **N**	Page _____	☐	☐
	i. USGS map with outline of project site	Location **N**	Page _____	☐	☐
	ii. Soils information (including hydric soils) Types, depth, slope and locations of soils	Location **N**	Page _____	☐	☐
	iii. Physical characteristics and limitations of soils	Location **N**	Page _____	☐	☐
	iv. Supporting calculations to show anticipated peak flows for the design storms	Location **N**	Page _____	☐	☐
	v. Analysis of the impact that runoff from the project site will have on existing downstream watercourses resistance to erosion	Location **N**	Page _____	☐	☐
	vi. Provide supporting calculations, standard worksheets, and description of the location for all proposed E&S Control BMPs used before, during and after earth disturbance including but not limited to the following:				
	A. Channels	Location **N**	Page _____	☐	☐
	B. Sediment Basins	Location **N**	Page _____	☐	☐
	C. Sediment Traps	Location **N**	Page _____	☐	☐
	D. Filter Fabric Fencing	Location **N**	Page _____	☐	☐
	E. Outlet Protection	Location **N**	Page _____	☐	☐
	F. Other BMPs (Specify) _____	Location **N**	Page _____	☐	☐
	G. Other BMPs (Specify) _____	Location **N**	Page _____	☐	☐

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				Applicant Check √ If Included	Official Use Only
	b. Plan Drawings *(Must be labeled "E&S Plan" or "Erosion & Sediment Control Plan", be complete & legible, and be the final plan for construction)* Drawings include the following:	Location **D**	Page _____	☐	☐
	i. Legend for any symbols that may be used on the drawing	Location **D**	Page _____	☐	☐
	ii. Topographic Features including existing contours, improvements, streams, wetlands, watercourses, etc. and sufficient surrounding area	Location **D**	Page _____	☐	☐
	iii. Soil types and locations	Location **D**	Page _____	☐	☐
	iv. Construction techniques or special considerations to address soil limitations	Location **D**	Page _____	☐	☐
	v. Limits of project site, NPDES boundary	Location **D**	Page _____	☐	☐
	vi. Limits of earth disturbance	Location **D**	Page _____	☐	☐
	vii. Proposed alteration including proposed contours and proposed improvements	Location **D**	Page _____	☐	☐
	viii. Maximum during construction drainage areas to hydraulic BMPs	Location **D**	Page _____	☐	☐
	ix. Location of water which may receive runoff and receiving water classification pursuant to Chapter 93 and the "statewide existing use listing"	Location **D**	Page _____	☐	☐
	x. Standard Construction Details for all proposed E&S Control BMPs used before, during and after earth disturbance	Location **D**	Page _____	☐	☐
	xi. Location of BMPs showing final contours are identified	Location **D**	Page _____	☐	☐
	xii. Complete and site specific sequence of BMP installation and removal including activities planned to limit exposed areas	Location **D**	Page _____	☐	☐
	xiii. Procedures or Note requiring the proper recycling or disposal of waste materials associated with the project site	Location **D**	Page _____	☐	☐
	xiv. Maintenance Program including inspection schedule, sediment cleanout levels, repair parameters and time frames, and directions for sediment removal	Location **D**	Page _____	☐	☐
	xv. Note explaining responsibilities for fill materials including definition of environmental due diligence and clean fill	Location **D**	Page _____	☐	☐
4.	Permit filing fee of $1,500 payable to the appropriate Clean Water Fund.			☐	☐
5.	Disturbed acre fee payable to the Commonwealth of Pennsylvania Clean Water Fund.			☐	☐
6.	Notifications to the local municipality and county governments that specify Acts 67 and 68 Coordination, and that the application is for an individual NPDES stormwater permit authorizing the discharge of stormwater during construction activities. A "sample" notification letter is provided in Appendices B and C.			☐	☐

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				Applicant Check √ If Included	Official Use Only
7.	Proof of receipt of municipal notifications; copies of certified mail receipts or acknowledgment letters from the local municipality and county government. (3 copies)			☐	☐
8.	Copy of Cultural Resource Notice including PHMC reply or certified mail receipt (for projects disturbing ten acres or more). (3 copies)			☐	☐
9.	The PNHP Review receipt for the project area. Include impact clearance letters if proof of agency coordination is required. (3 copies)			☐	☐
10.	Complete Post Construction Stormwater Management Plan. (3 copies) Location: Drawings (D), Narrative (N).			☐	☐
	a. Written Narrative *(Must be a stand alone, separate plan from the E&S Plan and labeled "PCSM" or Post-Construction Stormwater Management"" and be the final plan for construction)* Written Narrative Includes the following:	Location **N**	Page _____	☐	☐
	i. Site Description & Analysis	Location **N**	Page _____	☐	☐
	ii. Soil types and descriptions (including hydric soils)	Location **N**	Page _____	☐	☐
	iii. Pre-development and post-development drainage area runoff calculations for each drainage area	Location **N**	Page _____	☐	☐
	iv. Routing Analysis to demonstrate peak control for the 2-, 10-, 50-, and 100-year/24-hour storm events *(Routing should consider the benefits of BMPs)*	Location **N**	Page _____	☐	☐
	v. Calculations for permanent stormwater BMPs *(including volume of water treated through BMPs)*	Location **N**	Page _____	☐	☐
	vi. Curve Numbers and/or land use coefficients	Location **N**	Page _____	☐	☐
	vii. Infiltration/Geotechnical report and soil infiltration test pit results	Location **N**	Page _____	☐	☐
	b. Additional Worksheets **Note:** Worksheets 1 through 5 are required. Complete and attach the following worksheets where applicable	Location **N**	Page _____	☐	☐
	i. Worksheet 6 – Small Site/Small Impervious Area Exception for peak rate Mitigation Calculations ***(If worksheet 6 is not applicable, rate control is required)***	Location **N**	Page _____	☐	☐
	ii. Worksheet 10 – Water Quality Compliance for Nitrate	Location **N**	Page _____	☐	☐
	iii. Worksheet 11 – BMPs for Pollution Prevention ***(Required if applicant is not meeting Nitrate requirements)***	Location **N**	Page _____	☐	☐

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				Applicant Check √ If Included	Official Use Only
	iv. Worksheet 12 – Water Quality Analysis of Pollutant Loading from all Disturbed Areas ***(Required if applicant is not meeting Nitrate requirements)***	Location **N**	Page _____	☐	☐
	v. Worksheet 13 – Pollutant Reduction Through BMP Applications ***(Required if applicant is not meeting Nitrate requirements)***	Location **N**	Page _____	☐	☐
	c. Plans/Drawings *(Must be separate from E&S Plan and labeled "PCSM" or Post-Construction Stormwater Management"" and be the final plan for construction)*	Location **D**	Page _____	☐	☐
	i. Standard Details for permanent stormwater BMPs including permanent stabilization	Location **D**	Page _____	☐	☐
	ii. Location of BMPs showing final contours are identified	Location **D**	Page _____	☐	☐
	iii. Location of soil types are identified (including hydric soils)	Location **D**	Page _____	☐	☐
	iv. Location and depths of test pits/infiltration testing sites are identified	Location **D**	Page _____	☐	☐
	d. Ownership, Operations, and Maintenance Procedures *(Must be included on drawings)*	Location **D**	Page _____	☐	☐
	i. Applicant or entity (association, company, agency, etc.) listed as responsible party	Location **D**	Page _____	☐	☐
	e. Riparian Forest Buffer Management Plan (If Applicable) Location: Drawings (D), Narrative (N).			☐	☐
	i. A Planting Plan for converted or newly established Riparian Forest Buffers.	Location **D**	Page _____	☐	☐
	ii. A maintenance schedule and measures for converted or newly established Riparian Forest Buffers to ensure growth and survival.	Location **N**	Page _____	☐	☐
	iii. An inspection schedule and measures to ensure long-term maintenance and proper functioning of Riparian Forest Buffers.	Location **N**	Page _____	☐	☐
	f. Identification of critical stages of implementation of PCSM Plan for which a licensed professional or designee will be present on site.	Location: **N D**	Page _____	☐	☐
11.	Consistency letter from Municipal or County Engineer (where applicable)			☐	☐
12.	Completed Required Worksheets 1 – 5 (attached at the end of the NPDES Permit Application Package)			☐	☐
13.	Checklist for Subsequent Phases (of permitted projects)				

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				Applicant Check √ If Included	Official Use Only
	a. Estimated time frame for phased project build-out (update as necessary)			☐	☐
	b. Complete E & S Plans for specific phase (3 copies)			☐	☐
	c. New Section C and complete PCSM Plan for specific phase (3 copies)			☐	☐
	d. Consistency letter from municipal or county engineer (were applicable)			☐	☐
	CHECKLIST FOR INDIVIDUAL NPDES PERMIT RENEWALS ONLY			**Applicant Check √ If Included**	**Official Use Only**
1.	Resubmit items 1 through 7, and 9 through 12. Note: Only one copy of the Erosion and Sediment Control Plan and Post Construction Stormwater Management Plan is required.			☐	☐

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APPENDIX A

Land Use Information Questions

Responses to the following questions are required to determine applicability of DEP's Land Use Policy for Permitting of Infrastructure and Facilities.

Note: Applicants are encouraged to submit copies of local zoning approvals with their authorization application.

LAND USE INFORMATION			
1.	Is there an adopted county or multi-county comprehensive plan?	Yes ☐	No ☐
2.	Is there an adopted municipal or multi-municipal comprehensive plan?	Yes ☐	No ☐
3.	Is there an adopted county-wide zoning ordinance, municipal zoning ordinance or joint municipal zoning ordinance?	Yes ☐	No ☐
If the applicant answers NO to either Question 1, 2, or 3, the provisions of the PA MPC are not applicable and the applicant does not need to respond to questions 4 and 5 below. *If the applicant answers YES to questions 1, 2 and 3, the applicant should respond to questions 4 and 5 below.*			
4.	Does the proposed project meet the provisions of the zoning ordinance or does the proposed project have zoning approval? *If zoning approval has been received, attach documentation.*	Yes ☐	No ☐
5.	Have you attached Municipal and County Land Use Letters for the project?	Yes ☐	No ☐

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APPENDIX B
SAMPLE COUNTY LAND USE LETTER*

**(This sample letter and form is provided for the convenience of the applicant and the County. It does not prohibit the applicant from using a different template nor does it prohibit the County from submitting a different form of response.)*

Date:

Dear County Planning Director:

Acts 67, 68 and 127, which amended the Municipalities Planning Code, direct state agencies to consider comprehensive plans and zoning ordinances when reviewing applications for permitting of facilities and infrastructure, and specify that state agencies may rely upon comprehensive plans and zoning ordinances under certain conditions as described in Sections 619.2 and 1105 of the Municipalities Planning Code. The Pennsylvania Department of Environmental Protection's Policy for Consideration of Local Comprehensive Plans and Zoning Ordinances in DEP Review of Permits for Facilities and Infrastructure (DEP's Land Use Policy) provides direction and guidance to DEP staff, permit applicants, and local and county governments for the implementation of Acts 67, 68 and 127 of 2000. This policy can be found at www.depweb.state.pa.us; keyword: Land Use.

In accordance with DEP's Land Use Policy, enclosed please find a County Land Use Letter that is to be submitted with our permit application to DEP for an NPDES Permit for Stormwater Discharges Associated with Construction Activities. Please complete the attached form and return within 30 days to:

Name of Applicant: _____

Address of Applicant: _____

Project Location: _____

Project Description: _____

_____

Please do not send this form to DEP, as we must include the County Land Use Letter with our permit application. If we do not receive a response from you **within 30 days**, we shall proceed to submit our permit application to DEP without the County Land Use Letter. If the County Land Use Letter is not submitted with our permit application, and we provide proof to DEP that we attempted to obtain it, DEP will assume there are no substantive land use conflicts and proceed with the normal application review process.

If you have any questions, please do not hesitate to contact me at (phone number and/or email).

Sincerely,

Attachment – Sample County Land Use Letter

cc: /county commissioners

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APPENDIX B
SAMPLE COUNTY LAND USE LETTER

Date: _____

To: _____ (Name of Applicant)

From: _____ County Planning Agency/Commission

Re: _____ (Name of DEP Permittee)

The County of _____ states that it:

_____ has adopted a county or multi-county comprehensive plan.
If yes, please provide date of adoption:

_____ has not adopted a county or multi-county comprehensive plan.

If applicable:

The above referenced project:

___ is consistent with the adopted county or multi-county comprehensive plan.
___ is not consistent with the adopted county or multi-county comprehensive plan.

Additional Comments (attach additional sheets if necessary):

_____

_____

_____

_____

Submitted By:

Name	
Title	
Contact Information (Address & Phone)	
Signature	
Date	

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APPENDIX C
SAMPLE MUNICIPAL LAND USE LETTER*

**(This sample letter and form is provided for the convenience of the applicant and the Municipality. It does not prohibit the applicant from using a different template nor does it prohibit the Municipality from submitting a different form of response.)*

Date:

Dear Municipal Secretary:

Acts 67, 68 and 127, which amended the Municipalities Planning Code, direct state agencies to consider comprehensive plans and zoning ordinances when reviewing applications for permitting of facilities and infrastructure, and specify that state agencies may rely upon comprehensive plans and zoning ordinances under certain conditions as described in Sections 619.2 and 1105 of the Municipalities Planning Code. The Pennsylvania Department of Environmental Protection's Policy for Consideration of Local Comprehensive Plans and Zoning Ordinances in DEP Review of Permits for Facilities and Infrastructure (DEP's Land Use Policy) provides direction and guidance to DEP staff, permit applicants, and local and county governments for the implementation of Acts 67, 68 and 127 of 2000. This policy can be found at www.depweb.state.pa.us; keyword: Land Use.

In accordance with DEP's Land Use Policy, enclosed please find a Municipal Land Use Letter that is to be submitted with our permit application to DEP for an NPDES Permit for Stormwater Discharges Associated with Construction Activities. Please complete the attached form and return within 30 days to:

Name of Applicant: _____

Address of Applicant: _____

Project Location: _____

Project Description: _____

_____

Please do not send this form to DEP, as we must include the Municipal Land Use Letter with our permit application. If we do not receive a response from you **within 30 days**, we shall proceed to submit our permit application to DEP without the Municipal Land Use Letter. If the Municipal Land Use Letter is not submitted with our permit application, and we provide proof to DEP that we attempted to obtain it, DEP will assume there are no substantive land use conflicts and proceed with the normal application review process.

If you have any questions, please do not hesitate to contact me at (phone number and/or email).

Sincerely,

Attachment – Sample County Land Use Letter

cc: /township supervisor chair

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APPENDIX C
SAMPLE MUNICIPAL LAND USE LETTER

Date: _____

To: _____ (Name of Applicant)

From: _____ Township/Borough/City

Re: _____ (Name of DEP Permittee)

The municipality of _____ states that it:
_____ has adopted a municipal or multi-municipal comprehensive plan.
If yes, please provide date of adoption:

_____ has not adopted a municipal or multi-municipal comprehensive plan.

The municipality of _____ states that it:
_____ has adopted a county zoning ordinance, or a municipal or joint-municipal zoning ordinance.
_____ has not adopted a county zoning ordinance, or a municipal or joint-municipal zoning ordinance.

If applicable:

The municipality of _____ states that its zoning ordinance is generally consistent with its municipal comprehensive plan and the county comprehensive plan.

The above referenced proposed project
_____ meets the provisions of the local zoning ordinance

If zoning approval is required for the project to proceed, the above referenced project:
_____ has received zoning approval.
_____ has not received zoning approval.

If the proposed project has not received zoning approval:

What is the status of the zoning request for the proposed project? (e.g., Special Exception Approval from the Zoning Hearing Board required, Conditional Use approval from the Governing Body required)

_____
_____
_____
_____
_____
_____

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Is there a legal challenge by the applicant with regard to zoning for the proposed project?

Name and Contact Information for Municipal Zoning Officer:

Additional Comments (attach additional sheets if necessary):

Submitted By:

Name	
Title	
Contact Information (Address & Phone)	
Signature	
Date	

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Worksheet 1. General Site Information

INSTRUCTIONS: Fill out Worksheet 1 for each watershed

Date:

Project Name:

Municipality:

County:

Total Area (acres):

Major River Basin:

http://www.pawaterplan.dep.state.pa.us/StateWaterPlan/docroot/default.aspx

Watershed:

Sub-Basin:

Nearest Surface Water(s) to Receive Runoff:

Chapter 93 – Designated Water Use:

http://www.pacode.com/secure/data/025/chapter93/chap93toc.html

Impaired according to Category 4 or 5 of the Integrated Water Quality Monitoring and Assessment Report? Yes ☐ No ☐

http://www.portal.state.pa.us/portal/server.pt/community/water_quality_standards/10556/integrated_water_quality_report_-_2010/682562

List Causes of Impairment:

Is there an established TMDL that applies: Yes ☐ No ☐

Total Maximum Daily Loads (TMDLS)

http://www.dep.state.pa.us/watermanagement_apps/tmdl/.

http://www.epa.gov/reg3wapd/tmdl/pa_tmdl/index.htm

Is project subject to, or part of:

Municipal Separate Storm Sewer System (MS4) Requirements? Yes ☐ No ☐

http://www.portal.state.pa.us/portal/server.pt/community/stormwater_management/10628/npdes_ms4%C2%A0information/669119

Existing or planned drinking water supply? Yes ☐ No ☐

If yes, distance from proposed discharge (miles):

Approved Act 167 Plan? Yes ☐ No ☐

http://www.portal.state.pa.us/portal/server.pt?open=514&objID=554325&mode=2

Existing River Conservation Plan? Yes ☐ No ☐

http://www.dcnr.state.pa.us/brc/rivers/riversconservation/registry/

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Worksheet 2. Sensitive Natural Resources

INSTRUCTIONS

1. Provide Sensitive Resources Map according to non-structural BMP 5.4.1 in Chapter 5. This map should identify wetlands, woodlands, natural drainage ways, steep slopes, and other sensitive natural areas.

2. Summarize the existing extent of each sensitive resource in the Existing Sensitive Resources Table (below, using Acres). If none present, insert 0.

3. Summarize Total Protected Area as defined under BMPs in Chapter 5.

4. Do not count any area twice. For example, an area that is both a floodplain and a wetland may only be considered once.

EXISTING NATURAL SENSITIVE RESOURCE	MAPPED? Yes/no/n/a	TOTAL AREA (Ac.)	PROTECTED AREA (Ac.)
Waterbodies			
Floodplains			
Riparian Areas			
Wetlands			
Woodlands			
Natural Drainage Ways			
Steep Slopes, 15% - 25%			
Steep Slopes, over 25%			
Other:			
Other:			
TOTAL EXISTING:			

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Worksheet 3. Nonstructural BMP Credits

PROTECTED AREA

1.1 Area of Protected Sensitive/Special Value Features (see WS 2) _____ **Ac.**

1.2 Area of Riparian Forest Buffer Protection _____ **Ac.**

3.1 Area of Minimum Disturbance/Reduced Grading _____ **Ac**

TOTAL _____ **Ac**

Site Area	Minus	Protected Area	=	Stormwater Management Area
	-		=	

This is the area that requires stormwater management

VOLUME CREDITS

3.1 Minimum Soil Compaction (See Chapter 8, page 22 – SW BMP Manual)

Lawn _____ ft^2 x 1/4" x 1/12 = _____ ft^3

Meadow _____ ft^2 x 1/3" x 1/12 = _____ ft^3

3.3 Protect Existing Trees (See Chapter 8, page 23 – SW BMP Manual)

For Trees within 100 feet of impervious area:

Tree Canopy _____ ft^2 x 1/2" x 1/12 = _____ ft^3

5.1 Disconnect Roof Leaders to Vegetated Areas (See Chapter 8 page 25 – SW BMP Manual)

For runoff directed to areas protected under 5.8.1 and 5.8.2

Roof Area _____ ft^2 x 1/3" x 1/12 = _____ ft^3

For all other disconnected roof areas

Roof Area _____ ft^2 x 1/4" x 1/12 = _____ ft^3

5.2 Disconnect Non-Roof impervious to Vegetated Areas (See Chapter 8, page 26 – SW BMP Manual)

For Runoff directed to areas protected under 5.8.1 and 5.8.2

Impervious Area _____ ft^2 x 1/3" x 1/12 = _____ ft^3

For all other disconnected roof areas

Impervious Area _____ ft^2 x 1/4" x 1/12 = _____ ft^3

TOTAL NON-STRUCTURAL VOLUME CREDIT* _____ ft

**For use on Worksheet 5*

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Worksheet 4. Change in Runoff Volume for 2-YR Storm Event

PROJECT: _____
Drainage Area: _____
2-Year Rainfall: _____ in

Total Site Area: _____ acres
Protected Site Area: _____ acres
Managed Area: _____ acres

Existing Conditions:

Cover Type/Condition	Soil Type	Area (sf)	Area (ac)	CN	S	Ia (0.2*S)	Q Runoff[1] (in)	Runoff Volume[2] (ft^3)
Woodland								
Meadow								
Impervious								
TOTAL:								

Developed Conditions

Cover Type/Condition	Soil Type	Area (sf)	Area (ac)	CN	S	Ia (0.2*S)	Q Runoff[1] (in)	Runoff Volume[2] (ft^3)
TOTAL:								

2-Year Volume Increase (ft3):

2-Year Volume Increase = Developed Conditions Runoff Volume – Existing Conditions Runoff Volume

1. Runoff (in) = $Q = (P-0.2S)^2 / (P+0.8S)$ where
   P = 2-Year Rainfall (in)
   S = (1000/ CN)-10
2. Runoff Volume (CF) = Q x Area x 1/12
   Q = Runoff (in)
   Area = Land use area (sq. ft)

Note: Runoff Volume must be calculated for EACH land use type/condition and HSGI.
The use of a weighted CN value for volume calculations is not acceptable.

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Worksheet 5. Structural BMP Volume Credits

PROJECT: _____
SUB-BASIN: _____

Required Control Volume (ft^3) – *from Worksheet 4*: _____

Non-structural Volume Credit (ft^3) – *from Worksheet 3*: - _____
(maximum is 25% of required volume)

Structural Volume Reqmt (ft^3) _____

(Required Control Volume minus Non-structural Credit)

	Proposed BMP	Area (ft^2)	Volume Reduction Permanently Removed (ft^3)
6.4.1	Porous Pavement		
6.4.2	Infiltration Basin		
6.4.3	Infiltration Bed		
6.4.4	Infiltration Trench		
6.4.5	Rain Garden/Bioretention		
6.4.6	Dry Well / Seepage Pit		
6.4.7	Constructed Filter		
6.4.8	Vegetated Swale		
6.4.9	Vegetated Filter Strip		
6.4.10	Berm		
6.5.1	Vegetated Roof		
6.5.2	Capture and Re-use		
6.6.1	Constructed Wetlands		
6.6.2	Wet Pond / Retention Basin		
6.7.1	Riparian Buffer/Riparian Forest Buffer Restoration		
6.7.2	Landscape Restoration / Reforestation		
6.7.3	Soil Amendment		
6.8.1	Level Spreader		
6.8.2	Special Storage Areas		
Other			

Total Structural Volume (ft^3): _____
Structural Volume Requirement (ft^3): _____
DIFFERENCE _____

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Worksheet 6 – Small Site/Small Impervious Area Exception For Peak Rate Mitigation Calculations

The following conditions must be met for exemption from peak rate analysis for small sites under CG-1:

- ☐ The 2-Year/24-Hour Runoff Volume increase must be met in BMPs designed in accordance with Manual Standards
- ☐ Total Site Impervious Area may not exceed 1 acre
- ☐ Maximum Development Area is 5 Acres
- ☐ Maximum site impervious cover is 50%
- ☐ No more than 25% Volume Control can be in Non-structural BMPs
- ☐ Infiltration BMPs must have an infiltration of at least 0.5 in/hr.

Site Area	Percent Impervious	Total Impervious
5 acre	20%	1 acre
2 acre	50%	1 acre
1 acre	50%	0.5 acre
0.5 acre	50%	0.25 acre

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Worksheet 10 – Water Quality Compliance for Nitrate

Does the site design incorporate the following BMPs to address nitrate pollution? A summary "yes" rating is achieved if at least 2 Primary BMPs for nitrate are provided across the site or 4 secondary BMPs for nitrate are provided across the site (or the equivalent) "provided across the site" is taken to mean the specifications for that BMP set forward in Sections 5 and 6 are satisfied.

	Yes	No
Primary BMPs for Nitrate:	☐	☐
NS BMP 5.4.2 – Protect/Conserve/Enhance Riparian Buffers	☐	☐
NS BMP 5.5.4 – Cluster Uses at Each Site	☐	☐
NS BMP 5.6.1 – Minimize Total Disturbed Area	☐	☐
NS BMP 5.6.3 – Re-Vegetate/Re-Forest Disturbed Areas (Native Species)	☐	☐
NS BMP 5.9.1 – Street Sweeping/Vacuuming	☐	☐
Structural BMP 6.7.1 – Riparian Buffer Restoration	☐	☐
Structural BMP 6.7.2 – Landscape Restoration	☐	☐
	☐	☐
Secondary BMPs for Nitrate:	☐	☐
NS BMP 5.4.1 – Protect Sensitive/Special Value Features	☐	☐
NS BMP 5.4.3 – Protect/Utilize Natural Drainage Features	☐	☐
NS BMP 5.6.2 – Minimize Soil Compaction	☐	☐
Structural BMP 6.4.5 – Rain Garden/Bioretention	☐	☐
Structural BMP 6.4.8 – Vegetated Swale	☐	☐
Structural BMP 6.4.9 – Vegetated Filter Strip	☐	☐
Structural BMP 6.6.1 – Constructed Wetland	☐	☐
Structural BMP 6.7.1 – Riparian Buffer Restoration	☐	☐
Structural BMP 6.7.2 – Landscape Restoration	☐	☐
Structural BMP 6.7.3 – Soils Amendment/Restoration	☐	☐

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Worksheet 11 – BMPs for Pollution Prevention

Does the site design incorporate the following BMPs to address nitrate pollution? A summary "yes" rating is achieved if at least 2 Primary BMPs are provided across the site. "Provided across the site" is taken to mean that the specifications for that BMP set forward in Chapters 5 and 6 are satisfied.

	Yes	No
BMPs for Pollution Prevention:	☐	☐
NS BMP 5.4.1 – Protect Sensitive/Special Value Features	☐	☐
NS BMP 5.4.2 – Protect/Conserve/Enhance Riparian Buffers	☐	☐
NS BMP 5.4.3 – Protect/Utilize Natural Flow Pathways in Overall Stormwater Planning and Design	☐	☐
NS BMP 5.5.1 – Cluster Uses at Each Site; Build on the Smallest Area Possible	☐	☐
NS BMP 5.6.1 – Minimize Total Disturbed Area - Grading	☐	☐
NS BMP 5.6.2 – Minimize Soil Compaction in Disturbed Areas	☐	☐
NS BMP 5.6.3 – Re-Vegetate/Re-Forest Disturbed Areas (Native Species)	☐	☐
NS BMP 5.7.1 – Reduce Street Imperviousness	☐	☐
NS BMP 5.7.2 – Reduce Parking Imperviousness	☐	☐
NS BMP 5.8.1 – Rooftop Disconnection	☐	☐
NS BMP 5.8.2 – Disconnection from Storm Sewers	☐	☐
NS BMP 5.9.15 – Street Sweeping	☐	☐
Structural BMP 6.7.1 – Riparian Buffer Restoration	☐	☐
Structural BMP 6.7.2 – Landscape Restoration	☐	☐
Structural BMP 6.7.3 – Soils Amendment and Restoration	☐	☐

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Worksheet 12 – Water Quality Analysis of Pollutant Loading from All Disturbed Areas

Total Site Area (AC)	
Total Disturbed Area (AC)	
Disturbed Area Controlled by BMPs (AC)	

Total Disturbed Areas:

		Pollutant					Pollutant Load		
	Land Cover Classification	TSS EMC (mg/l)	TP EMC (mg/l)	Nitrate-Nitrite EMC (mg/l as N)	Cover (Acres)	Runoff Volume (AF)	TSS** (LBS)	TP** (LBS)	NO_3 (LBS)
Pervious Surfaces	Forest	39	0.15	0.17					
	Meadow	47	0.19	0.3					
	Fertilized Planting Area	55	1.34	0.73					
	Native Planting Area	55	0.40	0.33					
	Lawn, Low-Input	180	0.40	0.44					
	Lawn, High-Input	180	2.22	1.46					
	Golf Course Fairway/Green	305	1.07	1.84					
	Grassed Athletic Field	200	1.07	1.01					
Impervious Surfaces	Rooftop	21	0.13	0.32					
	High Traffic Street/Highway	261	0.40	0.83					
	Medium Traffic Street	113	0.33	0.58					
	Low Traffic/Residential Street	86	0.36	0.47					
	Res. Driveway, Play Courts, etc.	60	0.46	0.47					
	High Traffic Parking Lot	120	0.39	0.60					
	Low Traffic Parking Lot	58	0.15	0.39					
						TOTAL LOAD			
						REQUIRED REUCTION (%)	85%	85%	50%
						REQUIRED REDUCTION (LBS)			

*Pollutant Load = [EMC, mg/l] X [Volume, AF] X [2.7, Unit Conversion]

**TSS and TP calculations only required for projects not meeting CG1/CG2 or not controlling less than 90% of the disturbed area

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Worksheet 13 – Pollutant Reduction Through BMP Applications*

*Fill this worksheet out for each BMP type with different pollutant removal efficiencies. Sum pollutant reduction achieved for all BMP types on final sheet.

BMP Type: _____

Disturbed Area Controlled by this BMPs (AC)

Disturbed Area Controlled by this BMPs:

		Pollutant					Pollutant Load**		
	Land Cover Classification	TSS EMC (mg/l)	TP EMC (mg/l)	Nitrate-Nitrite EMC (mg/l as N)	Cover (Acres)	Runoff Volume (AF)	TSS** (LBS)	TP** (LBS)	NO_3 (LBS)
Pervious Surfaces	Forest	39	0.15	0.17					
	Meadow	47	0.19	0.3					
	Fertilized Planting Area	55	1.34	0.73					
	Native Planting Area	55	0.40	0.33					
	Lawn, Low-Input	180	0.40	0.44					
	Lawn, High-Input	180	2.22	1.46					
	Golf Course Fairway/Green	305	1.07	1.84					
	Grassed Athletic Field	200	1.07	1.01					
Impervious Surfaces	Rooftop	21	0.13	0.32					
	High Traffic Street/Highway	261	0.40	0.83					
	Medium Traffic Street	113	0.33	0.58					
	Low Traffic/Residential Street	86	0.36	0.47					
	Res. Driveway, Play Courts, etc.	60	0.46	0.47					
	High Traffic Parking Lot	120	0.39	0.60					
	Low Traffic Parking Lot	58	0.15	0.39					
	TOTAL LOAD TO THIS BMP TYPE								
	POLLUTANT REMOVAL EFFICIENCIES FROM APPENDIX A. STORMWATER MANUAL (%)						85%	85%	50%
	POLLUTANT REDUCITON ACHIEVED BY THIS BMP TYPE (LBS)								

POLLUTANT REDUCTION ACHIEVED BY ALL BMP TYPES (LBS)			
REQUIRED REDUCTION from WS12 (LBS)			

*Pollutant Load = [EMC, mg/l] X [Volume, AF] X [2.7, Unit Conversion]

**TSS and TP calculations only required for projects not meeting CG1/CG2 or not controlling less than 90% of the disturbed area

CHAPTER 4, APPENDIX C

SAMPLE STORMWATER AND FLOODPLAIN MANAGEMENT CONSISTENCY LETTERS

4C.0 INTRODUCTION

Water obstructions and encroachments requiring PA DEP permits are often additionally regulated under a watershed-based or county-wide Stormwater Management Plan adopted under the Stormwater Management Act (32 P.S. §§ 680.1-680.17). If such a plan exists, an analysis of the proposed project's impact on the Stormwater Management Plan must be undertaken and reviewed by either the local municipality or the county in which the project is located. The analysis and a letter of review from the municipality or county must be included in the application package.

If the proposed water obstruction or encroachment is located within a floodway delineated on a FEMA map, an analysis of the project's impact on the floodway delineation and water surface profiles may be required. The analysis and a letter of review from the municipality must be included in the application package.

4C.1 STORMWATER CONSISTENCY LETTER

LETTER FROM MUNICIPALITY OR COUNTY TO APPLICANT FOR STORMWATER PLAN CONSISTENCY

Date_____

{Name of Project Manager}
PennDOT District ____
{Address}

Dear {Name of Project Manager}:

This letter is to support PennDOT's project to {describe project activities} on Section {insert section number} of State Route {insert route number}. All or a portion of the proposed project discharges into a watershed that contains a PA DEP approved Act 167 Stormwater Management Plan.

We concur that the proposed project is consistent with the standards in the {insert name of watershed} Act 167 Stormwater Management Plan, as it was approved by PA DEP in {insert year of approval}.

Sincerely,

_____

4C.2 FLOODPLAIN CONSISTENCY LETTER

LETTER FROM MUNICIPALITY OR COUNTY TO APPLICANT FOR FLOODPLAIN CONSISTENCY

Date_____

{Name of Project Manager}
PennDOT District ____
{Address}

Dear {Name of Project Manager}:

This letter is to support PennDOT's project to {describe project activities} on Section {insert section number} of State Route {insert route number}. A portion of the project encroaches on the floodway of a detailed FEMA flood study.

We concur that the proposed project is consistent with FEMA floodplain regulations related to projects encroaching on a delineated floodway.

Sincerely,

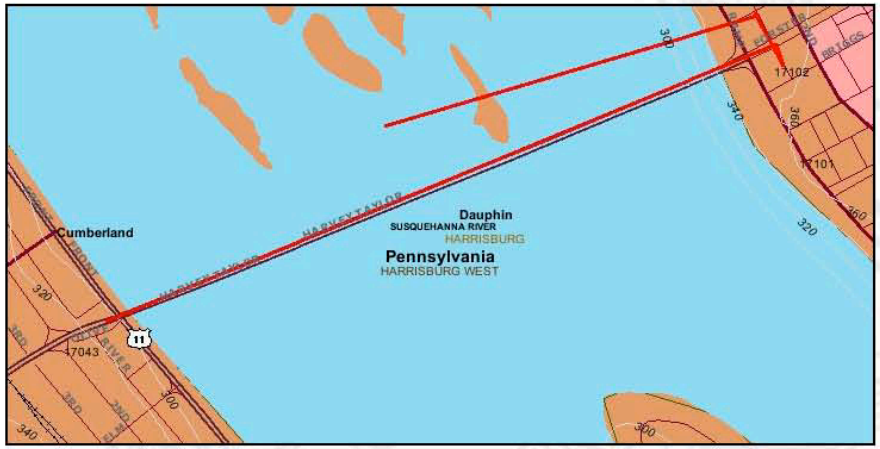
_____

CHAPTER 4, APPENDIX D

SAMPLE PNDI PROJECT ENVIRONMENTAL REVIEW RECEIPT

PNDI Project Environmental Review Receipt
Project Search ID: 20080605144073
Project Name: SWM/NPDES Course Example
Date: 6/5/2008 12:33:53 PM

Project Location



Project Name: SWM/NPDES Course Example
On Behalf Of: Self
Project Search ID: 20080605144073
Date: 6/5/2008 12:33:46 PM
# of Potential Impacts: 1
Jurisdictional Agency:
Pennsylvania Fish and Boat Commission
Project Category: Transportation,Bridge - demolition and/or construction (replacement) on existing alignment (Boring, piers, abutments, causeways, temporary stream crossings)
Project Location
Decimal Degrees: 40.26503 N, -76.88969 W
Degrees Minutes Seconds: 40° 15' 54.1" N, 76° 53' 22.9" W
Lambert: 307857.06841630, 462218.18264367 ft
ZIP Code: 17101,17102,17043
County: Cumberland,Dauphin
Township/Municipality: WORMLEYSBURG,HARRISBURG
USGS 7.5 Minute Quadrangle ID: 217
Quadrangle Name: HARRISBURG WEST
Project Length: 7622.5 feet

Location Accuracy

Project locations are assumed to be both precise and accurate for the purposes of environmental review. The creator/owner of the Project Review Receipt is solely responsible for the project location and thus the correctness of the Project Review Receipt content.

1 Potential Impacts

Under the Following Agencies' Jurisdiction: Pennsylvania Fish and Boat Commission

Pennsylvania State Programmatic General Permit (PASPGP)

Please note that regardless of PNDI search results, projects requiring a Chapter 105 DEP individual permit or GP 5, 6, 7, 8, 9 or 11 in certain counties (Adams, Berks, Bucks, Chester, Cumberland, Delaware, Franklin, Lancaster, Lebanon, Lehigh, Monroe, Montgomery, Northampton, Schuylkill and York) are required by DEP to comply with the bog turtle habitat screening requirements of the PASPGP.

APPLICANT INITIALS: _____

PNDI Project Environmental Review Receipt
Project Search ID: 20080605144073
Project Name: SWM/NPDES Course Example
Date: 6/5/2008 12:33:53 PM

Pennsylvania Natural Diversity Inventory (PNDI) records indicate there are potential impacts on special concern species and resources within the project area. If the project is pursued, the jurisdictional agency/agencies indicated require that the instructions below regarding potential impacts and/or avoidance measures be followed in their entirety.

These determinations were based on the project-specific information you provided, including the exact project location; the project type, description, and features; and any responses to questions that were generated during this search. If any of the information you provided does not accurately reflect this project, or if project plans change, DEP and the jurisdictional agencies require that another PNDI review be conducted.

This response represents the most up-to-date summary of the PNDI data files and is <u>good for one(1) year</u> from the date of this PNDI Project Environmental Review Receipt.

1 potential impact
The Applicant should MAIL/FAX a copy of this Project Environmental Review Receipt, a cover letter with project narrative, acreage to be impacted, how construction/maintenance activity is to be accomplished, township/municipality and county where project is located, and a USGS 7.5 minute quadrangle with project boundary and quad name marked on the map.

Natural Diversity Section
Pennsylvania Fish and Boat Commission
Division of Environmental Services
450 Robinson Lane
Bellefonte, PA 16823

Please mail only one (1) copy of the project review request. Do not email the project information. Allow 30 days for completion of the project review from the date of PFBC receipt of the project review request.

Based on the project-specific information you provided, no impacts to federally listed, proposed, or candidate species are anticipated. Therefore, no further consultation under the Endangered Species Act (87 Stat. 884, as amended; 16 U.S.C. 1531 *et seq.* is required with the U.S. Fish and Wildlife Service. Because no take of federally listed species is anticipated, none is authorized. For a list of species that could occur in your project area (but have not been documented in PNDI), please see the county lists of threatened, endangered, and candidate species. A field visit or survey may reveal previously undocumented populations of one or more threatened or endangered species with a project area. If it is determined that any federally listed species occur in your project area, the U.S. Fish and Wildlife Service requires that you initiate consultation to identify and resolve any conflicts. This response does not reflect potential Fish and Wildlife Service concerns under the Fish and Wildlife Coordination Act or other authorities.

DISCLAIMER

The PNDI environmental review website is a preliminary environmental screening tool. It is <u>not</u> a substitute for information obtained from a field survey of the project area conducted by a biologist. Such surveys may reveal previously undocumented populations of species of special concern. In addition, the PNDI only contains information about species occurrences that have actually been <u>reported</u> to the Pennsylvania Natural Heritage Program.

TERMS OF USE

Upon signing into the PNDI environmental review website, and as a condition of using it, you agreed to certain terms of use. These are as follows:

The web site is intended solely for the purpose of screening projects for

PNDI Project Environmental Review Receipt
Project Search ID: 20080605144073
Project Name: SWM/NPDES Course Example
Date: 6/5/2008 12:33:53 PM

potential impacts on resources of special concern in accordance with the instructions provided on the web site. Use of the web site for any other purpose or in any other way is prohibited and subject to criminal prosecution under federal and state law, including but not limited to the following: Computer Fraud and Abuse Act of 1986, as amended, 18 U.S.C. Â§ 1030; Pennsylvania Crimes Code, Â§ 4911 (tampering with public records or information), Â§ 7611 (unlawful use of computer and other computer crimes), Â§ 7612 (disruption of service), Â§ 7613 (computer theft), Â§ 7614 (unlawful duplication), and Â§ 7615 (computer trespass).

The PNHP reserves the right at any time and without notice to modify or suspend the web site and to terminate or restrict access to it.

The terms of use may be revised from time to time. By continuing to use the web site after changes to the terms have been posted, the user has agreed to accept such changes.

This review is based on the project information that was entered. The jurisdictional agencies and DEP require that the review be redone if the project area, location, or the type of project changes. If additional information on species of special concern becomes available, this review may be reconsidered by the jurisdictional agency.

PRIVACY and SECURITY

This web site operates on a Commonwealth of Pennsylvania computer system. It maintains a record of each environmental review search result as well as contact information for the project applicant. These records are maintained for internal tracking purposes. Information collected in this application will be made available only to the jurisdictional agencies and to the Department of Environmental Protection, except if required for law enforcement purposesâ€"see paragraph below.

This system is monitored to ensure proper operation, to verify the functioning of applicable security features, and for other like purposes. Anyone using this system consents to such monitoring and is advised that if such monitoring reveals evidence of possible criminal activity, system personnel may provide the evidence to law enforcement officials. See Terms of Use.

In order for this project to be considered for subsequent review, a signed and initialed copy of this receipt is required by the agency or agencies indicated. DEP requires that a signed and initialed copy of this receipt, along with any required documentation from jurisdictional agencies concerning resolution of potential impacts, be submitted in applications for permits requiring PNDI review. See DEP PNDI policy at www.naturalheritage.state.pa.us or visit the following websites for further information.

Regional Offices
Http://www.dep.state.pa.us/dep/deputate/fieldops/map.pdf

District Mining Operations
Http://www.dep.state.pa.us/dep/deputate/minres/Districts/homepage/Default.htm

Oil and Gas Management
Http://www.dep.state.pa.us/dep/deputate/minres/OILGAS/CustomerNeeds.htm

Print this Project Review Receipt using your Internet browser's print function and keep it as a record of your search.

Signature: _____

Date: _____

Project applicant on whose behalf this search was conducted:

Page 3 of 4 **APPLICANT INITIALS: _____**

PNDI Project Environmental Review Receipt
Project Search ID: 20080605144073
Project Name: SWMNPDES Course Example
Date: 6/5/2008 12:33:53 PM

Division of Environmental Services
450 Robinson Lane
Bellefonte, PA 16823

APPLICANT

Contact Name: _____

Address: _____

City, State, Zip: _____

Phone: _____

Email: _____

PERSON CONDUCTING SEARCH (if not applicant)

Contact Name: _____

Address: _____

City, State, Zip: _____

Phone: _____

Email: _____

The following contact information is for the agencies involved in this Pennsylvania Natural Diversity Inventory environmental review process. Please read this entire receipt carefully as it contains instructions for how to contact these agencies for further review of this particular project.

Natural Diversity Section
Pennsylvania Fish and Boat Commission

Page 4 of 4 **APPLICANT INITIALS: _____**

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CHAPTER 6

DATA COLLECTION

6.0 OVERVIEW

A. Introduction. It is necessary to identify the types of data required prior to conducting the hydrologic and hydraulic, stormwater management, highway drainage, or erosion and sediment pollution control engineering analysis. The effort necessary for data collection and compilation is to be tailored to the importance of the project. Not all data discussed in this chapter is needed for every project. Appendix 6A, *Sources of Data and Data Access Quick Reference Guide*, provides a quick reference guide to types of data required for PennDOT projects and the web source to find them.

Data collection for a specific project is to be commensurate with the project scope and tailored to:

- Site conditions.
- Scope of the engineering analysis.
- Social, economic, environmental and archaeological requirements.
- Unique project requirements.
- Regulatory requirements.

Uniform or standardized survey requirements for all projects may prove uneconomical or data deficient for a specific project. Special instructions outlining data requirements may have to be provided to the survey party by the designer for unique sites.

B. Data Requirements. This chapter outlines the types of data that are normally required for drainage analysis and design, possible sources and other aspects of data collection. The following subjects are presented in this chapter:

- Types of data and data collection.
- Site investigation.
- Survey information.
- Data evaluation.
- Sources of data in the appendices.

6.1 TYPES OF DATA REQUIRED AND DATA COLLECTION

A. General. The designer is to compile data that is specific to the subject site. The major types of data that may be required are:

- Permit and environmental regulation requirements.
- Watershed characteristics.
- Stream-reach data (especially in the vicinity of the facility).
- Existing facility data.
- Other physical data in the general vicinity of the facility (e.g., utilities, easements).
- Hydrologic and meteorologic data (stream-flow and rainfall data related to maximum or historical peak and low-flow discharges and hydrographs applicable to the site).
- Existing land-cover data in the subject drainage area and in the general vicinity of the facility.
- Floodplain, environmental and archaeological data.
- Channel meander data.
- Aerial photographs.
- Photographs as described in Section 6.1.E and Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Sections 10.7.B. and 10.7.C.

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

Prior to initiating data collection, the designer should review the following data collection references, as they relate to the project:

- Preliminary data collection requirements for roadway drainage facilities per Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Section 10.2.B.
- Site investigation per Chapter 7, *Hydrology*, Section 7.0.E.
- Site-specific characteristics per Chapter 7, *Hydrology*, Section 7.1.K.
- Site flood history per Chapter 7, *Hydrology*, Section 7.3.D and Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Section 10.6.C.1.
- FEMA studies per Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Section 10.6.C.2.
- Data requirements for the slope conveyance method per Chapter 8, *Open Channels*, Section 8.4.C.
- Data requirements for the standard step-backwater method per Chapter 8, *Open Channels*, Section 8.5.C.
- Culvert site data collection per Chapter 9, *Culverts*, Section 9.1.C.
- Bridge site data collection per Chapter 10, *Bridge Hydraulics*, Section 10.3.
- Storm drain systems per Chapter 13, *Storm Drainage Systems*, Section 13.2.C.
- Stormwater best management practice (BMP) data needs per Chapter 14, *Post-Construction Stormwater Management*, Section 14.9.

B. Watershed Characteristics. Following is a brief description of the major data topics that relate to drainage facility analysis and design. Additional discussion is contained in Chapter 13, *Storm Drainage Systems*.

1. Watershed Characteristics.

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

- Direct field surveys with conventional surveying instruments.
- Use of State highway planning or survey maps or construction drawings; Use of aerial maps or aerial photographs that have been produced by United States Geological Survey (USGS), Pennsylvania Department of Conservation and Natural Resources (DCNR), Delaware Valley Regional Planning Commission (DVRPC) or other Federal, State or Local Agency.
- Light Detection And Ranging (LiDAR) data.
- Use of USGS topographic maps, aerial photographs, and digital elevation models.
- Other topographic maps of the drainage area may also be obtained from municipal and county entities and local developers.

In determining the size of the contributing drainage area, any subterranean flow or any areas outside the physical boundaries of the drainage area that have runoff diverted into the drainage area being analyzed are to be included in the total contributing drainage area. In addition, a determination needs to be made if flood waters are diverted out of the basin before reaching the site.

Field checks may be required to determine any changes in the contributing drainage area that does not show up on the mapped data such as:

- Terraces.
- Storage areas (lakes, reservoirs, sinks or wetlands).
- Debris barrier.
- Reclamation/flood-control structures.
- Irrigation or other diversions.
- Storm drainage systems.

b. Slopes. The slope of the stream and the average slope of the watershed (basin slope) are to be determined. Hydrologic and hydraulic procedures in other chapters of this manual are dependent on

watershed slopes and other factors. Watershed slopes can be determined in the Watershed Modeling System, Geographic Information System (GIS) software, or USGS StreamStats. Results determined from digital or electronic means have sometimes produced erroneous results; therefore, all results have to be reviewed to determine if they are reasonable.

2. Watershed Land Use / Land Cover. Within the watershed, the designer is to define and document the current land use and land cover. In particular, the location and degree of urbanization occurring within the watershed needs to be defined and documented, as well as the information source. Information on existing land cover or urbanization trends may be obtained from:

- Field review.
- Aerial photographs (conventional and infrared).
- Zoning maps and master plans.
- USGS national land cover dataset (NLCD) and other maps.
- Municipal, county and regional planning agencies.
- Tax assessors.
- LiDAR orthophotographs.
- Landsat (satellite) images.

Specific information about particular tracts of land can often be obtained from owners, developers, realtors and local residents. Exercise care in using data from these sources because their reliability may be questionable, and these sources may not be aware of future development within the watershed that might affect specific land uses.

Land-cover (what actually shows as covering the ground as from aerial photographs) is typically the layer used for hydrologic modeling. Land use, on the other hand, is sometimes classified by tax parcel, and although a tax parcel may be classified as "Industrial", a large percentage of the tract may actually have a forest or meadow land cover. Existing land-cover data for small watersheds can sometimes best be determined or verified from a field survey.

3. Ponds, Lakes, Reservoirs, Wetlands and Detention Basins. For the streams, rivers, ponds, lakes, reservoirs, wetlands and detention basins that affect or may be affected by the proposed structure or construction, the following list of data is to be secured. The designer is to determine if there is enough storage above normal pool in the pond, lake or reservoir to attenuate flood flows. If the inflow is essentially the same as the outflow, the following information need not be obtained. For those ponds, lakes or reservoirs that reduce the outflow, a hydrologic model such as HEC-1 or HEC-HMS may be required to model inflow versus outflow (see Chapter 7, *Hydrology*). For those structures that may influence the design of the PennDOT structure, the following data are essential in determining the expected hydrology and may be needed for regulatory permits:

- The boundary (perimeter) of the water body for the normal pool elevation.
- Elevations of normal and highwater for various frequencies.
- Detailed description of any natural or manmade spillway or outlet works including dimensions, elevations, coefficients, capacities and operational characteristics.
- Detailed descriptions of any emergency spillway works including dimensions, elevations, coefficients and capacities.
- Document descriptions of adjustable gates and water-control devices.
- Stage/storage/discharge (S/S/D) curves.
- Use of the water resource (stock water, fish, recreation, power, irrigation, municipal or industrial water supply).
- Existing conditions of the stream, river, pond, lake or wetlands as to turbidity and silt.
- Classification of waters in the Commonwealth.
- Riparian ownership(s) and riparian requirements according to 25 PA Code §102, *Erosion and Sediment Control*.

PA DEP Division of Dam Safety has data on regulated high hazard dams in the State. The high hazard dams, meaning that in the unlikely event of a structural failure, the loss of life or property in downstream communities would be possible, may influence the hydrology of a watershed.

4. Environmental Considerations. The need for environmental data in the engineering analysis and design stems from the need to investigate and mitigate possible impacts due to specific design configurations. Environmental data needs may be summarized as follows:

- Identify information necessary to define the environmental sensitivity of the facility's site relative to impacted surface waters (e.g., water use, water quality and standards, aquatic and riparian wildlife, biology, and wetlands information). Some of this information is available in the water-quality standards and criteria published by PA DEP (e.g., 25 PA Code §93, *Water Quality Standards*).
- Physical, chemical and biological data for many streams are also available from Federal water pollution control agencies such as Environmental Protection Agency (EPA), USGS and municipalities, conservation districts, county planning commissions, and industries that use surface waters as a source of water supply. In unique instances, a data collection program possibly lasting several years and tailored to the site may be required.
- The designer is to gather information necessary to determine the most environmentally compatible design (e.g., circulation patterns, sediment transport data, low flow fish passage). Data on circulation, water velocity, water quality and wetlands are available from the USGS, USACE, United States Fish and Wildlife Service (USFWS), universities, marine institutes and other State, Federal and local agencies and organizations.
- Information necessary to define the need for and design of mitigation measures is to be obtained, including fish characteristics (type, size, migratory habits), fish habitat (depth, cover, pool-riffle relationship), sediment analysis and water-use and quality standards. Fish and fish habitat information is available from PA DEP, PA Fish and Boat Commission, and other State and Federal fish and game agencies.
- Wetlands are unique in that they provide unique habitat and contain unique species and data needs can be identified through coordination with PA DEP, US Fish and Boat Commission, USACE, and USFWS, etc.

C. Geographic Information Systems (GIS) Data. GIS data may be used as a source of georeferenced watershed data required in hydrologic and hydraulic design decision making. For example, a GIS can be used to develop and store a database containing the land cover, soil type and digital elevation models (DEMs) or topography for a watershed. This database may then be used to produce the design hydrographs and an array of maps, graphs and tables needed to complete the hydrologic analysis.

The current state-of-the-art understanding of GIS data gathering in hydrologic models is available from numerous textbooks, including *Watersheds: Processes, Assessment and Management* (DeBarry, 2004) and *GIS Modules and Distributed Models of the Watershed* (DeBarry and Quimpo, et.al ASCE, 1999). Most of the digital data required for hydrologic and hydraulic modeling can be found on the Pennsylvania Spatial Data Access (PASDA) website or other website as listed in Appendix 6A, *Sources of Data and Data Access Quick Reference Guide*.

Several methods to use electronic data for hydraulic and hydrologic studies are available. Design of drainage systems can be accomplished using CADD software and electronic surface data. Hydrologic and hydraulic models can be developed using this data.

The types of data normally used by digital models are:

- Elevation data;
- Features (e.g., streams and roadways);
- Land use; and
- Soils and infiltration.

Some of the electronic data is readily available, though not always with the desired resolution. Elevation data is available from the USGS in DEM format. The data is normally available in Universal Transverse Mercator (UTM) coordinates and in 5 m to 90 m (15 ft to 300 ft) resolution, depending on the location. Natural Resources Conservation Service (NRCS) also maintains soil and land-use data basis in GIS formats in certain areas. Detailed hydraulic and hydrologic studies may require higher resolution elevation data than is normally available through USGS and NRCS. Higher resolution (LiDAR) elevation, orthoimagery and land cover data is available through PASDA for most of the

State, and its availability can be determined from the PASDA website as shown in Appendix 6A, *Sources of Data and Data Access Quick Reference Guide*.

D. National Flood Insurance Program. Many streams have been analyzed for local flood insurance studies (FIS). In some cases, the discharges and hydraulic models are available from FEMA. Even though these studies are a good source of data, their technical content is to be reviewed prior to using the data. Many of the studies are outdated and/or may not reflect changes that may have occurred in the study reach since its initial publication.

Procedures and where to obtain FEMA publications may be found in Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Sections 10.1.A.4, 10.6.C.2, 10.7.C.8.h, 10.7.C.9 and in Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Appendices A, C and F. Appendix 6A, *Sources of Data and Data Access Quick Reference Guide*, lists addresses and the FEMA website to access Flood Insurance Studies and maps.

E. Field Investigation - Site Characteristics. Site investigations made by the designer will enable them 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.

Several criteria are to be established before making the field visit. Any needed information that can be obtained from maps, from aerial photos or by telephone calls should be obtained before the site visit. The research prior to the field visit will aid in identifying the critical features to be investigated at the site. Known the critical features to research will also identify the equipment to take to the site to enable an effective field visit.

Stream classifications for Pennsylvania are shown in Table 6.1.

Table 6.1. Pennsylvania Stream Classifications.

Stream Classification	Code	Agency
Special Protection (High Quality or Exceptional Value)	HQ or EV	PA DEP
High Quality, Cold Water Fishery	HQ-CWF	PA DEP
Warm Water Fishery	WWF	PA DEP
Migratory Fishery Waters	MFW	PA DEP
Trout Stocked Fishery	TSF	PA DEP
Approved Trout Waters		PA F&B Commission
Class A Wild Trout Waters		PA F&B Commission
Special Regulation Areas		PA F&B Commission
Stream Sections that Support Natural Reproduction of Trout		PA F&B Commission
Wilderness Trout Streams		PA F&B Commission
Wild or Scenic River		PA DCNR
Perennial or Intermittent		USGS
Navigable Waters		USCG, USACE

Website references with more detail on each stream classification are found in Appendix 6A, *Sources of Data and Data Access Quick Reference Guide*. The designer is to determine how the stream is classified before going to the site, so the appropriate characteristics can be observed.

The actual visit to the project site is to be made before or in combination with the field survey and before the preliminary hydraulic design is started. This may be combined with the visit by others (e.g., the surveyor, roadway and structural designers, wetland scientists, environmental reviewers, PennDOT maintenance personnel, local officials). The designer may visit the site separately, due to interests that are different from the others and the time required to obtain the data as warranted below.

The designer is to inspect the site and its contributing watershed (if necessary) to determine the required field and/or aerial drainage survey data required for the hydraulic analysis and design. Factors that most often need to be confirmed by field inspection are:

- Selection of roughness coefficients (see below).
- Estimation of D_{50}.
- Evaluation of apparent flow direction and diversions.
- Flow concentration.
- Observation of land use and related flood hazards.
- High-water marks or profiles including the date of occurrence and related frequencies.
- Existing structure size and type.
- Existence of wetlands.
- Drift/debris characteristics.
- General ecological information on the drainage area and adjacent lands.
- Scour.
- Streambank erosion.

A complete understanding of the physical nature of the natural channel or stream reach is of prime importance to a good hydraulic design, particularly at the site of interest. 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 needs to be aware of plans for channel modifications and any other changes that might affect the facility design.

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 and meandering).
- Scour history/evidence of scour.
- Median bed-sediment size D_{50}.
- Bed and bank material identification.

All required information to apply for the environmental and waterway encroachment permits is to be compiled by the designer. Appendix 10A, *Field Checklist for the Preliminary Design Permit Coordination*, provides a field checklist for preliminary design permit coordination that is to be completed as part of the site inspection. At a minimum, photos looking upstream and downstream from the site and along the contemplated highway centerline in both directions are to be taken. Details of the streambed and banks as well as the structures in the vicinity both upstream and downstream are to also be photographed. Close-up photographs complete with a surveyors rod are beneficial to facilitate estimates of structure and parapet dimensions and the streambed gradation.

Other data to be obtained from the site investigation includes:

1. Roughness Coefficients. Estimate roughness coefficients, ordinarily in the form of Manning's *n* values, for the entire flood limits of the study area. A tabulation of Manning's *n* values with descriptions of their applications can be found in Chapter 8, *Open Channels*.

2. Acceptable Flood Levels. Acceptable flood levels are based upon Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Table 10.6.1. Development and property use adjacent to the proposed site, both upstream and downstream, may require consideration of different flood levels. Note the floor elevations of structures or fixtures. In the absence of upstream development, acceptable flood levels may be based on tailwater and freeboard requirements of the highway itself. In these instances, the presence of downstream development may determine appropriate overflow points when an overtopping design of the highway is considered.

3. Flood History. The history of past floods and their effect on existing structures are of exceptional value in making flood hazard evaluation studies, and they provide needed information for sizing structures. Information may be obtained from newspaper accounts, local residents, flood marks, consideration of local studies conducted for approved Act 167 plans, or other positive evidence of the height of historical floods. Changes in channel and

watershed conditions since the occurrence of the flood needs to be evaluated in relating historical floods to present conditions.

Recorded flood data are available from agencies such as:

- USACE.
- USGS.
- NRCS.
- FEMA.
- PEMA.
- PA DEP.
- PennDOT Maintenance.
- Local agencies.

4. Debris and Ice. To assist with the design of a structure, investigate the quantity and size of debris and ice carried or available for transport by a stream during flood events. In addition, determine the times of occurrence of debris and ice in relation to the occurrence of flood peaks, and consider the effect of backwater from debris and ice jams on recorded flood heights when using stream flow records.

5. Scour Potential. Scour potential is an important consideration relative to the stability of the structure over time. Determine the scour potential by a combination of the stability of the natural materials at the facility site, tractive shear force exerted by the stream and sediment transport characteristics of the stream. Data on natural materials can be obtained from tests at the site and is to follow Publication 15M, Design Manual, Part 4, *Structures*, Chapter 7.

Bed and bank material samples sufficient for classifying channel type, stability and gradations, and a geotechnical study to determine the substrata if scour studies are needed, are required.

6. Controls Affecting Design Criteria. Many controls affect the criteria applied to the final design of drainage structures including 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 to be considered during design. In addition, there may be downstream and upstream controls to be documented.

7. Downstream Control. Any lakes, ponds, or reservoirs, inline weirs or low-head dams, including their spillway elevations and design levels of operation, or other obstructions need to be noted because their effect on backwater and/or streambed aggradation may directly influence the proposed structure. In addition, any downstream confluence of two or more streams needs to be studied to determine the effects of backwater or streambed change resulting from that confluence.

8. Upstream Control. Note the upstream control of runoff in the watershed. 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 operational designs for these features from the dam owner or PA DEP Dam Safety. NRCS, USACE, consulting engineers and other reservoir sponsors, such as water companies, often have complete reports concerning the operational 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 irrigation facilities, debris jams, mud flows and highways or railroads.

For further information on accumulation of preliminary data refer to Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Section 10.2.B. and Publication 10C, Design Manual, Part 1C, *Transportation Engineering Procedures*, Chapter 3. The Hydrologic and Hydraulic Site Investigation Data Collection Form is located in Appendix 6B, *Hydraulic Site Investigation Forms*. For stream meander and instability data collection forms, refer to FHWA HEC-20, *Stream Stability at Highway Structure,* Appendix C.

F. Survey Information.

1. General. Complete and accurate survey information is necessary to develop a design that best serves the requirements of a site. The project manager in charge of the drainage survey is to have a general knowledge of drainage design and, as such, coordinate the data collection with the designer. The amount of survey data gathered is to be commensurate with the importance and cost of the proposed structure and the expected flood hazard as discussed in Section 6.2.B and Appendix 6C, *Hydraulic Survey Instructions*.

The data that can be obtained or verified are:

- Stream-reach data (e.g., channel and floodplain cross sections, thalweg profile, top of banks).
- 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.

Much of these data can only be obtained from an on-site survey. It is often much easier to interpret published sources of data after an on-site inspection. Final design of the hydraulic facility can only occur after a thorough study of the area and a complete collection of all required information. All pertinent data and facts gathered through the survey are to be documented as discussed in Chapter 4, *Documentation and Document Retention*.

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 easily and cost effectively obtained in many geographic areas. A supplemental field survey is required to provide data in areas obscured on the aerial photos (e.g., underwater and heavy vegetation).

To avoid repeat visits, complete as much data collection as possible during the initial survey. Thus, identify and tailor data needs to satisfy the requirements of the specific location and size of the project early in the project design phase. Coordination by the project manager with all subconsultants requiring drainage-related survey data before the initial field work is started to ensure the acquisition of sufficient, but not excessive, survey data.

2. Drainage Surveys. Survey requirements for small drainage facilities (e.g., small culverts) are less extensive than those for major facilities (e.g., bridges). However, the purpose of each survey is to provide an accurate picture of the conditions within the zone of hydraulic influence of the facility. The surveyor is to obtain storm drains, inlets and inlet types, curbs, invert elevations, top of grate elevations and types, slopes, pipe sizes and materials. In addition, reference the following publications:

- Publication 72M, *Roadway Construction Standards.*
- Publication 13M, Design Manual, Part 2, *Highway Design*.

3. Surveys for Hydraulic Studies. PennDOT's general guide for H&H field survey data required to be obtained by surveyors for proper HEC-RAS analyses is found in Appendix 6C, *Hydraulic Survey Instructions*. Data typically collected during the site investigation may also be found in:

- Chapter 7, *Hydrology*, Section 7.0.E.
- Publication 122M, *Surveying and Mapping Manual,* Part A, Section 6.2.
- Publication 73, *Drainage Condition Survey Field Manual.*

Generally, the information required includes:

a. Stream Profile. Obtain streambed profile data in accordance with Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Section 10.7.B.6.a.(5). Extend these data a minimum of 150 m (500 ft) upstream and 150 m (500 ft) downstream from the existing and proposed crossings. In some instances, profile information may be needed beyond the 150 m (500 ft) to determine the average slope and to encompass any proposed construction or aberrations. For instance, where the stream slope is extremely flat, or the channel very wide, additional downstream profile and cross section data may be required to properly model the subcritical backwater. In some instances, identification of "headcuts" that could migrate to the site under consideration is particularly important. Where there is a stream gage relatively close,

obtain the gage height, discharge, date and hour of the reading. See Appendix 6C, *Hydraulic Survey Instructions*, and the *HEC-RAS Hydraulic Reference Manual* for suggested modeling limits.

b. Stream Cross Sections. Obtain stream cross section data that represents the conditions at the structure site. In addition, obtain stream cross section data at other locations where stage - discharge and related calculations are necessary e.g., the proposed crossing, location of temporary measures, etc. See Appendix 6C, *Hydraulic Survey Instructions*, and the *HEC-RAS Hydraulic Reference Manual* for specific cross section location suggestions and requirements for bounding structure sections.

c. Existing Structures. Obtain the location, size, description, condition, observed flood stages and channel section relative to existing structures on the stream reach and near the site to determine their capacity and effect on the stream flow. Investigate any structures, downstream or upstream, that may cause backwater or retard stream flow. In addition, note the manner in which existing structures have been functioning with regard to such items as scour, overtopping, debris and ice passage, fish passage, etc. For bridges, this data includes span lengths, type of piers and substructure orientation, which usually can be obtained from existing structure plans but needs to be verified with field survey. The necessary culvert data includes size, inlet and outlet geometry, slope, end treatment, culvert material and flow-line profile.

"As-built" highway construction plans may be available to obtain required bridge and/or culvert data. Photographs and high-water 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.

Additional information may be required depending on the nature of the activity and the extent of regulatory involvement.

Other required data may be obtained as described in Publication 10C, Design Manual, Part 1C, *Transportation Engineering Procedures*, Chapter 4.

4. LiDAR Data. The Pennsylvania Department of Conservation and Natural Resources (DCNR) through the PAMAP program, collected high-resolution light detection and ranging (LiDAR) elevation data for the entire state. LiDAR data is used to produce various elevation data products including point based digital terrain models (DTM); grid-based digital elevation models (DEM), and contours. In addition to these "end user" products, LiDAR processing generates other data resources, including the raw point cloud, processed points, and breaklines.

LiDAR topographic data can be utilized for floodplain elevations for cross sections, and the data can be combined with detailed culvert or bridge and channel data to create a good base for hydraulic modeling.

The terrain data products include:

- Digital Elevation Model – a 1 m (3.2 ft) pixel equivalent raster GeoTIFF Digital Elevation Model (DEM). Each pixel in the DEM has an interpolated elevation value. Each DEM is approximately 37.5 MB in size. The data has a 1.4 m (4.6 ft) average point spacing (2 m (6.56 ft) maximum) with a bare earth surface vertical accuracy of 0.185 m (0.61 ft) RMSE. All data is in units of feet, NAD83 horizontal datum, Ellipsoid GRS80, NAVD88 vertical datum, and Geoid03. The data is used to produce 0.6 m (2 ft) contour data set.
- Contours – a three-dimensional (3D) shapefile of 0.6 m (2 ft) contours is provided for each tile. Each contour is attributed with its elevation value and whether it is an intermediate (0.6 m (2 ft) interval) or index (3 m (10 ft) interval) contour. The size of each shapefile varies.
- Breaklines – a 3D shapefile of breakline is provided for each tile. The breaklines are interpreted from the PAMAP orthophotos and the LiDAR data is used to provide elevation (z-values) to each line. The breaklines are used in the creation of the DEM and contours. Each breakline is attributed for feature type (e.g. edge of road); the feature types are documented in metadata. The size of each shapefile varies.

PAMAP data are organized into tiles, which do not have gaps or overlaps. Each tile represents 3048 m by 3048 m (10,000 ft by 10,000 ft) on the ground. The coordinate system for tiles in the northern half of the state (6679 tiles) is Pennsylvania State Plane North (datum: NAD83, units: feet); tiles in the southern half of the state (6736

tiles) are in Pennsylvania State Plane South as shown in Figure 6.1. A tile name is formed by concatenating the first four digits of the state plane northing and easting defining the block's northwest corner, the State identifier "PA", and the state plane zone designator "N" or "S" (e.g. 45001210PAS).

The PAMAP tile structure can be viewed and queried using PASDA's Pennsylvania Imagery Navigator application and the tile shapefiles can be downloaded from PASDA as indicated in Appendix 6A, *Sources of Data and Data Access Quick Reference Guide*.

Figure 6.1. PAMAP Tile Structure.



The DEMs are not designed specifically for large map scale hydrologic modeling. For instance, stream channels do not continue under roads in the DEMs (i.e., features such as culverts are not factored into the creation of the DEM). Therefore, the DEM data may need to be manipulated before use with hydrologic models. The LiDAR data, although accurate for overbank floodplain use, does not provide the accuracy around bridges or within streambanks. The normal approach is to develop topographic surveys for a project in these areas.

The LiDAR also produce orthoimagery. All imagery with the exception of the 2003 data is 1:2400 scale, 0.3 m (1 ft) pixel resolution, color data. The 2003 data is 0.6 m (2 ft) pixel resolution. The PAMAP imagery can be viewed using the prototype PAMAP Viewer found in Appendix 6A, *Sources of Data and Data Access Quick Reference Guide*.

In addition to actual field survey and LiDAR data, there are two basic methods to develop topographic surveys:

- Global Positioning System (GPS); and,
- Aerial photogrammetry.

Both basic methods to develop topographic surveys are discussed in more detail below.

G. Global Positioning System (GPS). Field data collection is normally accomplished using electronic survey equipment such as Total Station and GPS.

Using Total Station as a data collection tool, the engineer can develop topographic mapping directly from the field work, with little additional processing. This information can be directly used in certain highway or hydraulics software, saving time and resources in the tedious process of survey decoding and data entry. Digital Elevation Models (DEMs) or Digital Surface Models can be developed using the data collected using this method. Other feature data (e.g., flood limits, bank full indicators, vegetation markers, point bars, flow boundaries) can also be located by a

surveyor and automatically decoded along with the elevation data. The accuracy of this method can be very high and is dependent on the experience of the field personnel.

Hand-held GPS units that have sub-meter horizontal precision are available. Hand-held GPS based surveying is still less accurate because it depends on many factors such as accuracy of the unit, location of the survey reach and time of day. Vertical precision to collect elevation data is not sufficiently accurate around bridges for many design functions. However, this method makes a one-person survey crew possible with minimal training. GPS data can be obtained by a hydraulics engineer during a site visit. This facilitates rapid development of field data, especially location data, and quick office evaluations. For further information, reference Publication 122M, *Surveying and Mapping Manual*, Part A, Chapter 8.

H. Aerial Photogrammetry. Under this method, topographic mapping is developed using pictures of the ground taken from an aircraft or satellite. Ground controls are established using field survey methods and contours are developed.

Aircraft used for taking photographs can be fixed wing (airplane) or helicopter. Fixed wing still is the most economical method; however, helicopter based surveys offer low altitude flights, resulting in much higher accuracy. The pictures taken can also be used as data for hydraulic investigations and studies.

High-resolution satellite and multi-spectral imagery is available and may be substituted for other methods if necessary. Satellite imagery can be used to determine land uses. Because satellite data is stored for a period of time, multi-spectral satellite imagery can also be used to investigate flooding, actually after an event has occurred. Potentially, the technology can be used to develop "before-and-after" images and topography to investigate a flood event or other significant change in an area of interest and can be obtained through FEMA or USGS.

A new method of aerial topographic generation is using laser or radar beams from an aircraft carrying differential GPS. The laser-based method is called Light Detection and Ranging (LiDAR). LiDAR or radar generated data have the advantage of being inexpensive when compared to traditional photogrammetry. However, the accuracy is highly dependent on the technology available to the vendor in aerial equipment and available software to filter trees and other covered land areas. The LiDAR dataset also provides aerial imagery from which land use can be determined. The specifications for the LiDAR orthoimagery are:

- Dataset name: PAMAP Program High Resolution Color Orthophotography.
- Data currentness: Maintained on a 3 year cycle.
- Accuracy/Scale: For 2004-2006 images, 0.3 m (1 ft) pixel resolution, 1:2400 scale + 1.5 m (5 ft) horizontally at the 95% confidence level for true 0.3 m (1 ft) resolution). The horizontal accuracy standard follows the NSSDA-1998 standard. For 2003 images, 0.6 m (2 ft) pixel resolution, 1:2400 scale.
- Ground sample resolution: 0.3 m (1 ft).
- Horizontal datum: NAD 83.

For further information, reference Publication 122M, *Surveying and Mapping Manual,* Part B.

I. Channel Meander and Instability Data. Oftentimes, practicing engineers need to evaluate and determine bridge and other highway facility locations and sizes and ascertain the need for countermeasures considering the potential impacts of channel meander migration over the life of a bridge or highway river crossing. Rivers prone to channel migration may be spanned by static structures and paralleled by fixed highway alignments and appurtenances. Channel migration (alluvial river meander, platform deformation) is a major consideration in designing bridge crossings and other transportation facilities in affected areas; it causes the channel alignment and approach conditions present during construction to deteriorate as the upstream channel location changes. Channel migration can result in the following:

- Excess bridge pier and abutment scour,
- Threats to bridge approaches and other highway infrastructure,
- Worsened debris problems, and
- Obstructed conveyance through bridge openings.

Channel migration is typically an incremental process. On meandering streams, the problem at a bridge site may become apparent two or three decades after the bridge is constructed. Channel migration is often evident throughout large sections of a drainage basin; it is not localized in the vicinity of a bridge. It is a natural phenomenon that occurs in the absence of specific disturbances, but may be exacerbated by such basin-wide factors as land use changes, gravel mining, dam construction, and removal of vegetation. Remedial action such as constructing guide banks or installing bank protection becomes increasingly expensive or difficult as the channel migrates. A methodology was developed to evaluate the potential for channel movement and predict future channel migration with and without the installation of appropriate countermeasures.

The Transportation Research Board's (TRB's) National Cooperative Highway Research Program (NCHRP) Web Document 67, *Methodology for Predicting Channel Migration*, documents and presents the results of a study to develop a practical methodology to predict the rate and extent of channel migration in proximity to transportation facilities. The principal product of this research was NCHRP Report 533, *Handbook for Predicting Stream Meander Migration*, a stand-alone handbook for predicting stream meander migration using aerial photographs and maps. Use of historic aerial photos to identify and predict the direction and rate of lateral channel migration, can be an effective component of a channel stability analysis. A companion product to NCHRP Web Document 67 is NCHRP CD 49: *Archived River Meander Bend Database*, a four-CD-ROM set that contains a database of 141 meander sites containing 1,503 meander bends on 89 rivers in the United States.

Identifying stream instability problems at highway-stream crossings is an important consideration at many bridges and culverts. Guidelines for techniques for stream channel classification and reconnaissance, as well as rapid assessment methods for channel instability are summarized in FHWA, *Stream Stability at Highway Structures, Latest Edition*, Hydraulic Engineering Circular No. 20 (HEC-20). Qualitative and quantitative geomorphic and engineering techniques useful in stream channel stability analysis are presented in this publication. Specifically, HEC-20, Chapter 4 and HEC-20, Appendix C explain field reconnaissance and provides data collection forms respectively, in the case that stream stability data is required for a specific project.

6.2 SOURCES OF DATA

A. Objectives. These are:

- Identify possible sources of data.
- Rely on PennDOT experience as to which sources yield likely desired data.
- Use the guides in this chapter for data sources. Acquaint the designer with available data and PennDOT procedures for acquiring it.

B. Sources. 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 6A, *Sources of Data and Data Access Quick Reference Guide*. The following information is given for each data source on the list:

- Type of data;
- Agency;
- Address of source; and,
- Web site.

Appendix 6A, *Sources of Data and Data Access Quick Reference Guide*, provides a quick reference guide with internet links to quickly obtain data required.

6.3 DATA EVALUATION

A. Objective. Once the needed data have been collected, the next step is to compile it into a usable format. The designer determines whether the data contains inconsistencies or other unexplained anomalies that might lead to erroneous calculations or results. The main reason for analyzing the data is 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.

B. Evaluation. The designer is to carefully study the data for accuracy and reliability. Experience, knowledge and judgment are important parts of data evaluation. It is in this phase that reliable data such as historical data and that data obtained from measurements is separated from that which is less reliable. The data is to be evaluated by the designer for consistency and to identify any changes from perceived final data or as-built drawings.

In reviewing previous studies and old plans for types and sources of data, determine how the data were used and any indications of accuracy and reliability. Determine whether significant changes have occurred in the watershed and whether the historical data can be used. Data acquired from the publications of established sources such as USGS can usually be considered as valid and accurate. Evaluate and summarize basic data (e.g., streamflow data derived from non-published sources) before use. In addition, compare and resolve any inconsistencies found of maps, aerial photographs, Landsat images and land-use studies with one another and the results of the field survey. Consult general references to help define the hydrologic character of the site or region under study and to aid in the analysis and evaluation of data.

C. Sensitivity. Sensitivity studies may be used to evaluate data and establish the relative importance of specific data items to the final design. Sensitivity studies 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. For the more sensitive data items, more time and effort is justified to be spent on 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 effort of data collection and evaluation is to be commensurate with the importance and extent of the project and/or facility.

D. Accuracy of Data. In any engineering computations, it is important to understand the limitations of accuracy of the computations based on the accuracy of the input data. In step-backwater computations utilizing HEC-RAS, there are several factors that have significant effects on the accuracy of the results: accuracy of the survey data, correct layout of the cross-sections, spacing between cross sections, correct establishment of upstream and downstream study limits, and selection of roughness coefficients.

Most field surveys of channel and floodplain cross sections are recorded to an accuracy of 30 mm (0.1 ft). If the survey truly represents the cross sections of the reach of the stream being studied to a 30 mm (0.1 ft) accuracy, the greatest accuracy that would result from a step-backwater computation could be no more than 30 mm (0.1 ft). Any results expressed more precisely than 30 mm (0.1 ft) are simply due to the mathematics.

The accuracy of aerial survey technology for generating cross sectional coordinate data is governed by mapping industry standards. Cross sections obtained from contours of topographic maps developed by photogrammetric methods are generally not as accurate as those generated from field data collection methods. Aerial photography can supplement field survey cross sections. The use of aerial elevation survey technology permits additional coordinate points and cross sections to be obtained at small incremental cost, and the coordinate points may be formatted for direct input into commonly used water surface profile computer programs such as HEC-RAS.

For further information on determining the relationships between (1) survey technology and accuracy employed for determining stream cross sectional geometry, (2) degree of confidence in selecting Manning's roughness coefficients, and (3) the resulting accuracy of hydraulic computations, refer to the *USACE Publication Technical Paper No. 114* (USACE, 1986). This publication also presents methods of determining the upstream and downstream limits of data collection for a hydraulic study requiring a specified degree of accuracy. Computer software has been developed to perform the calculations for the various routines presented in these publications.

E. Data Merging. Merging of electronic surface data is common during highway design. Better data is usually collected within the highway area, while the data for the area outside the expected cut/fill lines is less precise. Because

watershed and flood limits fall well outside the highway cut/fill lines, hydraulics engineers need to collaborate the data that has multiple resolutions.

Electronic data is available in various forms differentiated by software products, type of data structure (DEMs and TINs), coordinate systems (UTM, State Plane, Latitude-Longitude), units (ft or m), resolution and datums. While merging data in different forms, take care to ensure proper conversion prior to merging. Standardizing all data to the most current and most accurate format is the best way to ensure compatibility. There are tools available to accomplish the data "translation."

A more serious issue in data merging is caused by differences in data resolution. For example, a digital surface model developed using a photogrammetric method is typically of a lower resolution compared to a surface model developed using a field data collection survey. When merging the data, elevation differences at the boundaries of the different data areas need to be carefully reconciled.

There is often a problem with artificial pits (sinks) and peaks due to the creation of DEMs and TINs. Evaluation of the data by the engineer is necessary to correct these inconsistencies. The designer is to reference Publication 122M, *Surveying and Mapping Manual*, Part A, Chapter 5.

6.4 REFERENCES

AASHTO, *Highway Drainage Guidelines,* Chapter 2, American Association of State Highway and Transportation Officials, Task Force on Hydrology and Hydraulics, Washington, DC, 2003.

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National Cooperative Highway Research Program (NCHRP) Report 533, *"Handbook for Predicting Stream Meander Migration,"* December 20, 2004.

PennDOT, Publication 584, *PennDOT Drainage Manual*, Current Edition.

Transportation Research Board, *Methodology for Predicting Channel Migration,* NCHRP Web-Only Document 67 (Project 24-16), August, 2004.

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CHAPTER 6, APPENDIX A

SOURCES OF DATA AND DATA ACCESS QUICK REFERENCE GUIDE

This appendix includes references, data source and web links to the data most commonly used in development of drainage, hydrologic and hydraulic, stormwater management and erosion and sediment pollution control analyses. As with all web links, they may be updated from time to time, but were active at the time of this publication. If the link is no longer active, the designer may search for the link, or begin with the agency web site.

6A.0 PRINCIPLE HYDROLOGIC DATA SOURCES

- METEOROLOGICAL DATA

  National Oceanic and Atmospheric Agency (NOAA)
  National Climatic Data Center
  37 Battery Park Avenue
  Federal Building
  Asheville, North Carolina 28801
  (704) 271-4800
  Fax: (704) 271-4876

- REGIONAL AND LOCAL FLOOD STUDIES

  FEMA
  500 "C" Street, SW
  State and Local Programs, Room 418
  Washington, D.C. 20472
  (Also refer to Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Appendix A for procedures)

  Or

  Federal Insurance and Mitigation Division Director
  FEMA Region III
  615 Chestnut Street, 6th Floor
  Philadelphia, PA 19106-4404

  PEMA
  Pennsylvania Office of Homeland Security
  2605 Interstate Drive, Suite 380
  Harrisburg PA 17110
  (717) 651-2715

  Or

  Chief, Floodplain Management Division
  Department of Community and Economic Development
  Keystone Building, 4th Floor
  Harrisburg, PA 17120

  Surveyed high-water marks and site visits by PennDOT or the designer.

6A.1 PRINCIPLE WATERSHED DATA SOURCES

- USGS maps ("Quad" sheets)
  U.S. Geological Survey
  Rocky Mountain Mapping Center
  Mail Stop 504
  Denver Federal Center
  Denver, Colorado 80225
  (303) 236-5829

  Or

  Pennsylvania Spatial Data Access (PASDA), www.pasda.psu.edu/

- EROS AERIAL PHOTOGRAPHS
  U.S. Geological Survey
  EROS Data Center
  Sioux Falls, South Dakota 57198
  (605) 594-6151

- NRCS Soils Maps, websoilsurvey.nrcs.usda.gov/app/HomePage.htm

- Site visits by PennDOT.

- USGS StreamStats Data Site, water.usgs.gov/osw/streamstats/pennsylvania.html

- USGS Stream Gage Data, waterdata.usgs.gov/pa/nwis/sw

- State and local maps, geologic maps and aerial photos: Pennsylvania Spatial Data Access (PASDA), www.pasda.psu.edu/

- Approved county-wide or watershed stormwater management plans (Act 167 plans) and associated studies.

6A.2 PRINCIPLE REGULATORY DATA SOURCES

- FEDERAL FLOODPLAIN DELINEATIONS AND STUDIES

  Federal Emergency Management Agency
  Flood Map Distribution Center
  6930 (A-F) San Tomas Road
  Baltimore, Maryland 21227-6227
  (800) 358-9616

- FHWA DESIGN CRITERIA AND PRACTICES

  Federal Highway Administration
  U.S. Department of Transportation
  400 Seventh Street SW
  Washington, D.C. 20590

- FEDERAL REGISTERS

  Superintendent of Documents
  U.S. Printing Office
  Washington, D.C. 20402
  (202) 783-3238

- USACE Section 404 Permit Program (see Section 6A.4, *Principle Environmental Data Sources*).

- U.S. Coast Guard (USCG) (see Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Section 10.8).

- U.S. Environmental Protection Agency (USEPA) (see Section 6A.3, *Principle Environmental Data Sources*).

- State Department of Environmental Protection (DEP) (see Section 6A.3, *Principle Environmental Data Sources*).

- State Floodplain Delineation and Studies: Bureau of Waterways Engineering/PA DEP, (717) 783-1754.

6A.3 PRINCIPLE ENVIRONMENTAL DATA SOURCES

- USEPA data and studies, www.epa.gov/

- USACE data and studies (see Publication 13M, Design Manual Part 2, *Highway Design*, Chapter 10, Section 10.9.G).

  USGS water quality data, waterdata.usgs.gov/nwis

- State water quality data, www.depweb.state.pa.us – Go to "Water Topics."

- Environmental statements prepared by other Federal, State and local agencies and private parties.

- Environmental data from others.

6A.4 OTHER DATA SOURCES

- Regional U.S. Environmental Protection Agency (USEPA)
  EPA Region 3 Regional Office
  1650 Arch Street
  Philadelphia, PA 19103-2029
  (800) 438-2474

- Regional U.S. Federal Emergency Management Agency (FEMA)
  Federal Emergency Management Agency
  615 Chestnut Street
  Office Independence Mall, Sixth Floor
  Philadelphia, PA 19106-4404
  (215) 931-5608

- Regional and State U.S. Fish and Wildlife Service (USFWS)
  Ecological Services Field Office
  Northeast Region (5)
  315 South Allen Street, Suite 322
  State College, PA 16801-4850
  (814) 234-4090
  Fax: (814) 234-0748

- Regional and State U.S. Forest Service (USFS)
  USDA Forest Service
  Northeastern Area
  Office of the Director
  11 Campus Blvd., Suite 200
  Newtown Square, PA 19073
  (610) 557-4103

- Regional and State U.S. Natural Resources Conservation Service (NRCS)
  Pennsylvania State Office
  Office Credit Union Place, Suite 340
  Harrisburg, PA 17110-2993
  (717) 237-2100
  Fax: (717) 237-2238

- Regional and State U.S. Army Corps of Engineers (USACE). See Publication 13M, Design Manual Part 2, *Highway Design*, Chapter 10, Section 10.9.G.

- Regional U.S. Coast Guard (USCG). See Publication 13M, Design Manual Part 2, *Highway Design*, Chapter 10, Section 10.8.B.

- Regional and State U.S. Geological Survey (USGS)
  Pennsylvania Water Science Center
  215 Limekiln Road
  New Cumberland, PA 17070
  (717) 730-6900
  Fax: (717) 730-6997

- National Spatial Data Infrastructure Partnership Office (Liaison for: PA)
  U.S. Geological Survey (USGS)
  Pennsylvania Geological Survey
  3240 Schoolhouse Road
  Middletown, PA 17057-3534
  (717) 702-2027

- Federal Highway Administration (FHWA)
  Pennsylvania Division Office
  228 Walnut Street, Room 508
  Harrisburg, PA 17101-1720

- Municipal governments, local ordinances and Master Plans.

- Any river basin compacts, commissions, committees and authorities.

- Private citizens.

- Private industry.

Table 6A.1. Data Access Quick Reference Guide.

Data Source or Layer	GIS, Map or Other data	Originating Agency	Web Source Agency / Site	Product Organized by	Website[1]
Elevation Data					
• USGS topographic quadrangles - Digital Raster Graphic (DRG's)	G	USGS	PASDA	Quadrangle	www.pasda.psu.edu/
• Digital Elevation Models (DEMs) - 10 m	G	USGS	PASDA	Quadrangle	www.pasda.psu.edu/
• Digital Elevation Models (DEMs) - 30 m	G	USGS	PASDA	Quadrangle	nationalmap.gov/viewer.html
LiDAR Data					
• Tile index	G	PADCNR	PASDA	Tiles	www.pasda.psu.edu/
• Tile shapefiles	G	PADCNR	PASDA	Tiles	www.pasda.psu.edu/
• TINs	G	PADCNR	PASDA	Tiles	www.pasda.psu.edu/
• Contours	G	PADCNR	PASDA	Tiles	www.pasda.psu.edu/
Imagery					
• Aerial photographs	G	PADCNR	PASDA	Tiles	www.pasda.psu.edu/
• PA MAP Imagery Viewer	G	PADCNR	PASDA	Tiles	http://maps.psiee.psu.edu/ImageryNavigator/
• Digital Orthophoto Quarter-Quadrangle (DOQQ)	G	USGS	PASDA	Quadrangle	www.pasda.psu.edu/
Land Use/Land Cover/Geology					
• USGS Land Use / Land Cover (LULC)	G	USGS	WebGIS	Seamless	www.webgis.com
• USGS Land Use / Land Cover (LULC), 1992	G	USGS	USGS	Seamless	nationalmap.gov/landcover.html
• PAMAP Land Cover for Pennsylvania, 2005	G	PADCNR	PASDA	Seamless	www.pasda.psu.edu/
• Pennsylvania karst	G	PSU	PASDA	Seamless	www.pasda.psu.edu/pub/pasda/compendium/
Soils					
• NRCS Soils	G	USDA	NRCS	County	http://www.nrcs.usda.gov/wps/portal/nrcs/site/soils/home/
• NRCS Soils	G	USDA	PSU	County	mcdc.cas.psu.edu/status.htm
• NRCS Soils	G	USDA	PASDA	County	www.pasda.psu.edu/pub/pasda/soils/

[1] Note: Web sites are continually changing. If the link is no longer active, the designer may search for the link, or begin with the agency web site.

Table 6A.1. Data Access Quick Reference Guide (continued).

Data Source or Layer	GIS, Map or Other data	Originating Agency	Web Source Agency / Site	Product Organized by	Website[1]
Flood data					
• FEMA Flood Insurance Studies	M	FEMA	FEMA	Municipality/ County	www.fema.gov/floodplain-management/flood-insurance-study
• DFIRM (Flood Insurance Rate Map)[2]	G	FEMA	PASDA	County	www.pasda.psu.edu/default.asp
• DFIRM (Flood Insurance Rate Map)[2]	G	FEMA	FEMA	County	www.fema.gov/floodplain-management/flood-insurance-rate-map-firm
Stream data					
• Stream gage data	O	USGS	USGS	Gage	waterdata.usgs.gov/pa/nwis/sw
• Stream designated water uses and water quality criteria	O	PADEP	DEP	Stream	www.pacode.com/secure/data/025/chapter93/chap93toc.html
• Stream designated water uses and water quality criteria - updated list	O	PADEP	PADEP	Stream	www.depweb.state.pa.us/watersupply/cwp/view.asp?a=1261&q=449172
• Stream designated water uses and water quality criteria	G	PADEP	PADEP	Stream	www.pasda.psu.edu
• Trout water classifications	O	PAF&B	PAF&B	State	www.fish.state.pa.us/waters_trout.htm
• Wild or scenic river	M	PADCNR	PADCNR	State	www.dcnr.state.pa.us/brc/conservation/rivers/scenicrivers/index.htm

[2] Not available for all Counties.

Table 6A.1. Data Access Quick Reference Guide (continued).

Data Source or Layer	GIS, Map or Other data	Originating Agency	Web Source Agency / Site	Product Organized by	Website[1]
Environmental/Cultural					
• PA Natural Diversity Index (PNDI)	O	PNHP	PNHP	State	www.naturalheritage.state.pa.us/
• Threatened and endangered species	O	PAGC	PAGC	State	www.portal.state.pa.us/portal/server.pt?open=514&objID=621014&mode=2
• National Wetland Inventory (NWI)	G,M	USF&WS	USF&WS	Quadrangle	www.fws.gov/wetlands/data/index.html
• Cultural Resources Notices	O	NPS	NPS	State	www.nps.gov/nr/research/

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CHAPTER 6, APPENDIX B

HYDRAULIC SITE INVESTIGATION FORM

6B.0 HYDRAULIC SITE INVESTIGATION FORM

I. GENERAL PROJECT DATA

1. Project Number: _____
2. County(ies): _____ 3. Municipality(ies): _____
4. Road Name: _____ 5. SR/Section, Seg./Off.: _____
6. Stream Name: _____
7. Project Description: [ ] Superstructure replacement [ ] Total replacement [ ] Stream realignment [ ] Other: _____
8. Survey Source: [ ] Field [ ] Aerial [ ] LiDAR [ ] Others: _____
9. Date Survey Received: _____ from: _____
10. Site Inspected By: _____ (name) and _____ (name) on _____ (date)

II. FIELD INVESTIGATION

Complete all relevant information below.

Bridge

1. Type: [ ] Conc Spread Box [ ] Conc Adjacent Box [ ] Conc Bulb-Tee [ ] Conc Other [ ] Steel I-beam [ ] Steel Other [ ] Steel Truss [ ] Stone Arch [ ] Other _____
2. No. of Spans: [ ] 1 [ ] 2 [ ] Other _____ 3. Abutment Type: [ ] Vertical [ ] Sloping [ ] n/a
4. Abutments Aligned w/ Flow: [ ] Yes [ ] No [ ] n/a
5. No. of Piers: [ ] 0 [ ] 1 [ ] 2 [ ] Other _____ 6. Aligned w/ Flow: [ ] Yes [ ] No [ ] n/a
7. Pier Width[s]: [ ] n/a [ ] Tapered [ ] Variable Width (top and bottom) [ ] _____ (ft)
8. Pier Nose Shape: [ ] Triangular, Angle _____ [ ] Semi-circular [ ] Elliptical [ ] Other _____ [ ] n/a

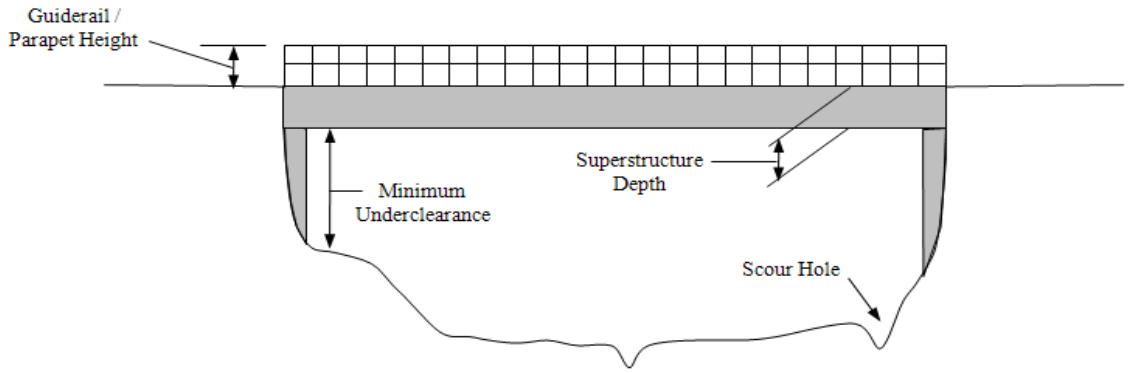
Culvert

9. Type: [ ] Conc Box [ ] Conc Slab [ ] Conc Arch [ ] Conc Pipe [ ] Metal Arch [ ] Metal Elliptical [ ] Metal Pipe [ ] Other _____
10. Span, Bottom Width, or Diameter: [ ] Could not measure [ ] _____ (ft), measured [ ] US [ ] DS
11. Headwall to Headwall Length: [ ] Could not measure [ ] _____ (ft)
12. Open Bottom: [ ] Yes [ ] No [ ] Could not determine
13. Special Features: (check all that apply) [ ] Depressed [ ] Fish Baffles [ ] Energy Dissipator

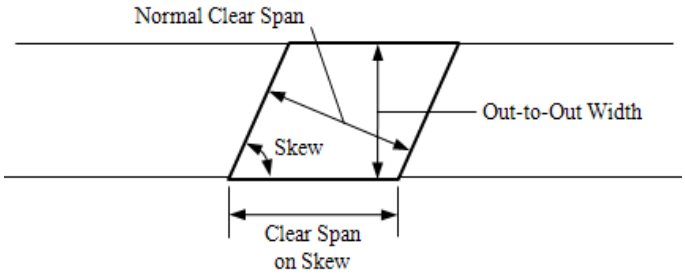
General

14. Historical High Water Marks Obtained: [ ] No [ ] Yes, (description) _____
15. Evidence of Recent High Water Event: [ ] No [ ] Trash/Debris Line [ ] Erosion [ ] Overbank Ponding [ ] Other _____

16. Observed Debris Accumulation: [ ] No [ ] Pier [ ] Superstructure
17. Potential for Debris Accumulation: [ ] Yes [ ] No
18. Potential for Ice Jam: [ ] Yes [ ] No
19. Minimum Underclearance: (see figure below) [ ] Could not measure [ ] _____ (ft), measured [ ] US [ ] DS
20. Parapet or Guide Rail Type: [ ] Conc Barrier [ ] Conc Pigeon Hole [ ] Conc Other [ ] Curb w/Guide Rail [ ] Guide Rail [ ] Stone Masonry [ ] Other
21. Curb/Sidewalk: [ ] No [ ] Yes, height _____ (ft)
22. Parapet/Guide Rail Height: [ ] Could not measure [ ] _____ (ft)
23. Parapet/Guide Rail a Significant Obstruction to Overtopping Flow: [ ] Yes [ ] No
24. Superstructure Depth: (see figure below) [ ] Could not measure [ ] _____ (ft)



25. Out-to-Out Structure Width: [ ] Could not measure [ ] _____ (ft), measured on [ ] Center [ ] Perpendicular
26. Normal Clear Span Length(s): [ ] Could not measure [ ] _____ (ft)
27. Clear Span Length(s) on Skew: [ ] Could not measure [ ] _____ (ft)
28. Structure Skewed to Roadway: [ ] Yes [ ] No 29. Estimated Skew: _____ degrees

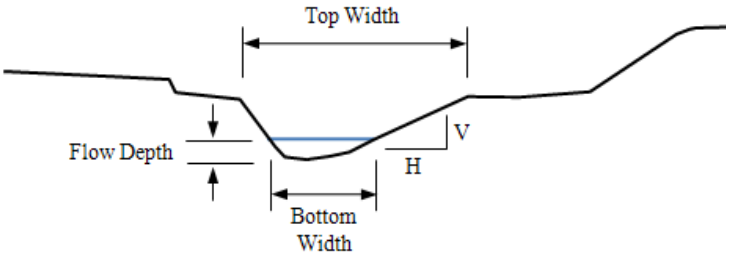


Bed Material	Particle Diameter
Clay and silt	< 0.002 in
Sand	0.002 - 0.08 in
Gravel	0.08 - 2.5 in
Cobbles	2.5 - 10 in

30. Stream Bed Material: (check all that apply) [ ] Bedrock [ ] Cobbles [ ] Gravel [ ] Sand [ ] Silt [ ] Clay [ ] Could not see
31. Estimated D_{50}: _____ (in) 32. Method: [ ] Visual [ ] Gravelometer
33. Estimated Manning *n* Value – Upstream: LOB _____ Channel _____ ROB _____
34. Estimated Manning *n* Value – Downstream: LOB _____ Channel _____ ROB _____

Channels	
Description	n
Clean, straight, no rifts or deep pools	0.030
Same as above, but more stones and weeds	0.035
Clean, winding, some pools and shoals	0.040
Same as above, but some weeds and stones	0.045
Same as above, but more stones	0.050
Sluggish reaches, weedy, deep pools	0.070
Floodways with heavy stands of timber/brush	0.100

Floodplains			
Description	n	Description	n
Grass field	0.030-0.035	Light brush/trees	0.050-0.060
Cultivated – no crop	0.030	Medium/dense brush	0.070-0.100
Cultivated – crops	0.035-0.040	Heavy trees	0.100-0.120



35. Observed Flow Depth: _____ (ft) 36. Average Channel Depth: _____ (ft)
37. Channel Side Slope: Left _____ Right _____ (H:V)
38. Average Channel Top Width: [ ] Could not measure [ ] _____ (ft), measured [ ] US [ ] DS
39. Average Channel Bottom Width: [ ] Could not measure [ ] _____ (ft), measured [ ] US [ ] DS
40. Upstream Channel Bottom Widths (for baffle design): [ ] n/a [ ] _____ (ft), _____ (ft), _____(ft)
41. Downstream Channel Bottom Widths (for baffle design): [ ] n/a [ ] _____ (ft), _____ (ft), _____(ft)
42. Downstream Flow Control: [ ] No [ ] Yes, Type: [ ] Dam [ ] Bridge [ ] Other _____
43. Upstream Flow Control: [ ] No [ ] Yes, Type: [ ] Dam [ ] Bridge [ ] Other _____
44. Evidence of Channel Instability: [ ] No [ ] Yes, because of observed [ ] Undercutting [ ] Bank Sloughing [ ] Braiding [ ] Meandering [ ] Other _____
45. Evidence of Structure Scour: [ ] No [ ] Yes, approximately _____ (ft) deep at _____
46. Evidence of Sediment Accumulation? [ ] No [ ] Yes, at _____
47. Permanent Structures in the 100-Year Floodplain: [ ] No [ ] Yes, at _____
48. Survey Appears Correct: [ ] Do not have survey yet [ ] Yes [ ] No, apparent errors are: _____

_____

III. LOCAL TESTIMONY

Obtained During Site Investigation: [ ] No [ ] Yes

Name: _____ Phone #: _____
Address: _____
Notes: _____
_____

Name: _____ Phone #: _____
Address: _____
Notes: _____
_____

IV. PHOTO DOCUMENTATION

Number(s)

[ ] Upstream channel (from structure) _____

[ ] Upstream LOB (and all property or structures in the floodplain) _____

[ ] Upstream ROB (and all property or structures in the floodplain) _____

[ ] Downstream ROB (and all property or structures in the floodplain) _____

[ ] Downstream LOB (and all property or structures in the floodplain) _____

[ ] Downstream channel (from structure) _____

[ ] Downstream face of structure (looking upstream) _____

[ ] Upstream face of structure (looking downstream) _____

[ ] Parapets or guide rails (with surveyors rod or tape measure for scale) _____

[ ] Left roadway approach _____

[ ] Right roadway approach _____

[ ] Channel bed material with scale _____

[ ] Evidence of channel instability _____

[ ] Evidence of scour _____

[ ] Existing structure inlet/outlet _____

[ ] Upstream structure (looking from downstream) _____

[ ] Upstream structure (looking from upstream) _____

[ ] Downstream structure (looking from downstream) _____

[ ] Downstream structure (looking from upstream) _____

[ ] Other _____ _____

[ ] Other _____ _____

[ ] Other _____ _____

[ ] Other _____ _____

[ ] Other _____ _____

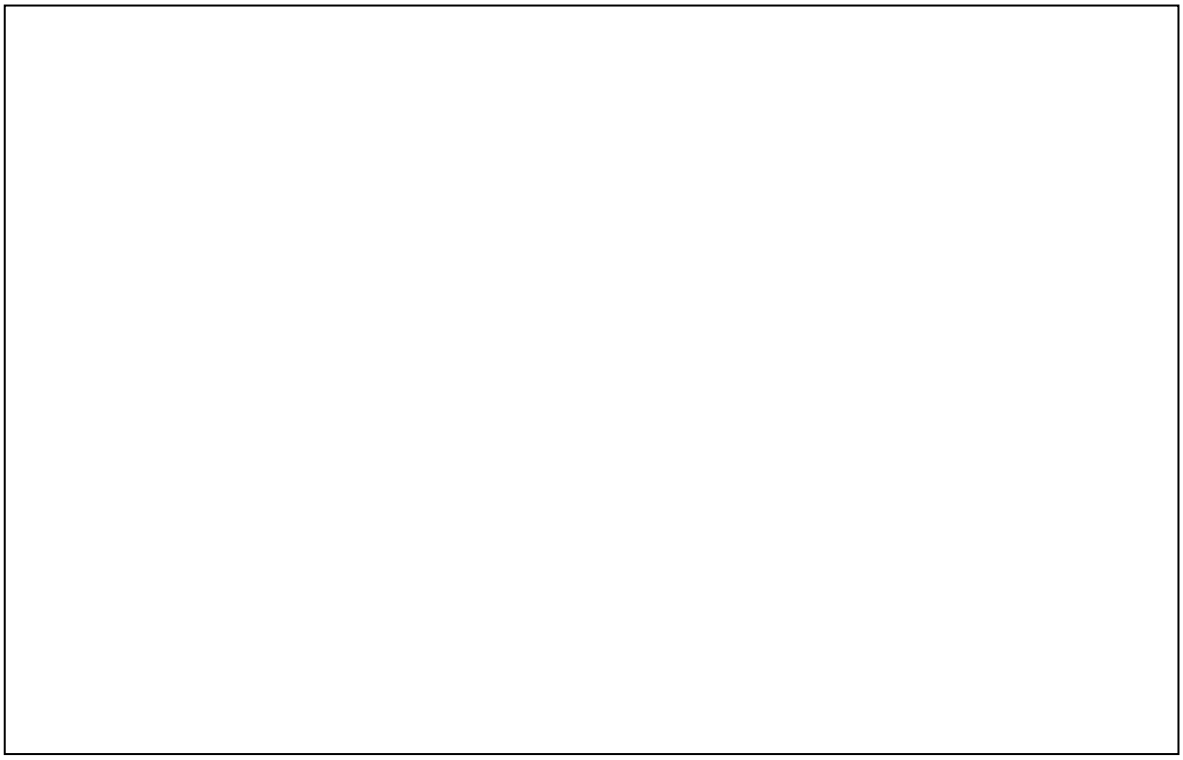
[ ] Other _____ _____

[ ] Other _____ _____

V. SKETCHES



Plan View



Elevation View

VII. NOTES

CHAPTER 6, APPENDIX C

HYDRAULIC SURVEY INSTRUCTIONS

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HYDRAULIC SURVEY FORM

Project: _____

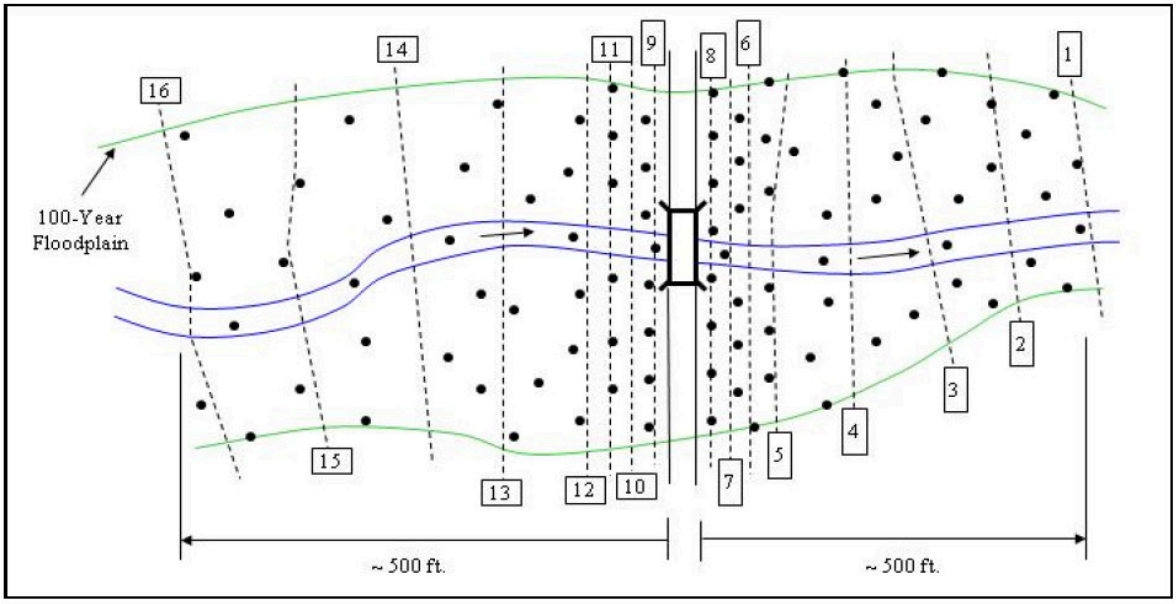
Stream Name: _____ Date: _____

County: _____ Job Number: _____

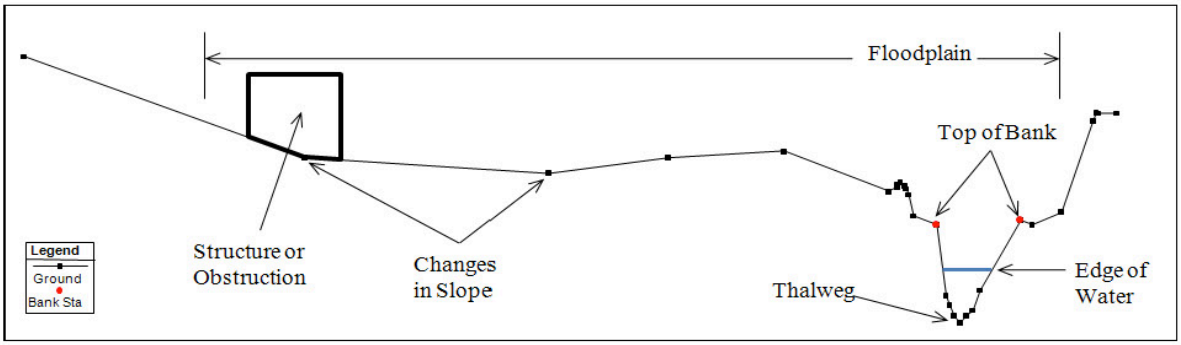
Obtaining complete and accurate stream and structure survey data is a critical step in the development of a hydraulic model. It is important for the engineer tasked with the hydraulic modeling to be involved in the selection of hydraulic cross section locations. This form is to be used by the engineer to communicate survey needs to the surveyor.

PART 1 - BASIC STREAM AND FLOODPLAIN SURVEY

The figure below depicts a typical hydraulic survey scenario. The survey must extend a minimum of 500 feet upstream and downstream of the structure, as required by DM2, Chapter 10, Section 10.7.B.6.a.5.
For most culvert and small bridge projects, a topographic survey sufficient to create a surface within the limits of the study is recommended.



In general, elevations should be obtained at the top and bottom of stream banks, edge of water, the thalweg, and breaks in slope in the overbanks (floodplain) area. A typical stream cross section is shown below.



HYDRAULIC SURVEY FORM

Project: _____

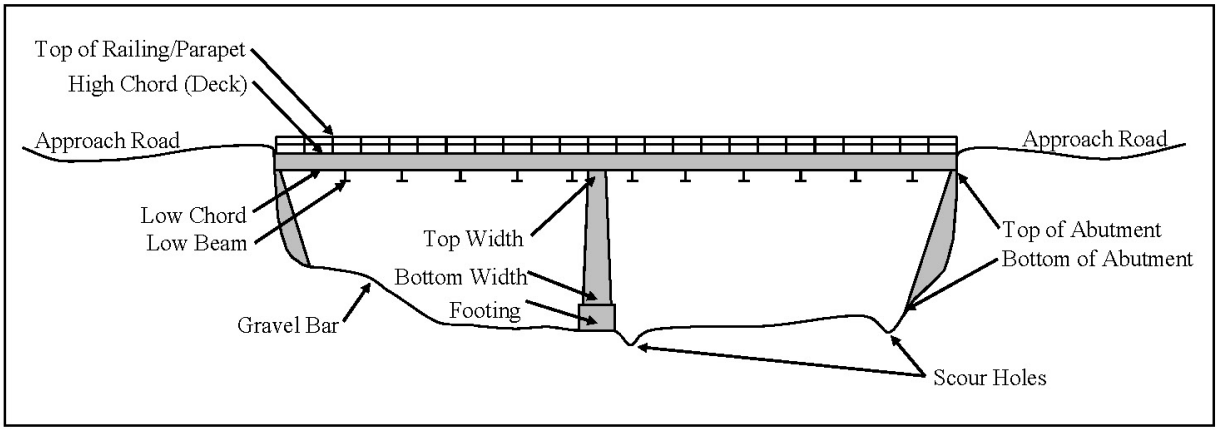
Stream Name: _____ Date: _____

County: _____ Job Number: _____

In addition to the stream channel and floodplain, the survey should include all of the applicable items listed in Parts 1A and 1B.

1A - STRUCTURE AND ROADWAY FEATURES

- Approach roads (centerline, edge of pavement or shoulder)
- Bridge centerline and edge of deck (if superelevated) elevations
- Low chord elevations of superstructure (US and DS, L and R)
- Low beam elevations (if applicable) (US and DS, L and R)
- Top of railing and parapet
- Abutments (top and bottom corners, clear distance between abutments, US and DS, L and R)
- Piers (footings, shape, width)
- Scour holes (location, approximate width and depth)
- Gravel bars, beaver dams, etc. (location, approximate width and height)
- Other _____



1B - OTHER FEATURES

- Other structures/obstructions (within survey limits)
- Changes in terrain/channel shape
- Gravel bars
- Meanders (sharp bends)
- Tributaries (section US/DS of intersection)
- Dams, spillways (top and bottom elevations)
- High water marks
- Stream gage locations
- Culverts (size, type, invert elevation)
- Bank protection
- Levees, walls
- Other _____

HYDRAULIC SURVEY FORM

Project: _____

Stream Name: _____ Date: _____

County: _____ Job Number: _____

PART 2 - SPECIAL REQUIREMENTS

Some projects require more than the minimum 500 feet of channel/floodplain data upstream and downstream of the site. Examples of scenarios that may require more than outlined in Part 1 include:

- Most "rivers"
- Streams with very mild longitudinal grades
- Projects located in a detailed FEMA study area (Zone AE)
- Sites where a significant flow control is at or near the 500-foot mark

Check the items below that apply to the project.

☐ Additional cross sections (outside of the area in Part 1) are needed.
→ See the instructions in Part 3.

☐ LiDAR data is available in this area to complement floodplain data.
→ Note that the survey must be tied to the State Plane Coordinate System.
LiDAR data can downloaded from the PASDA website (http://www.pasda.psu.edu/).

2A - EXTENDED CROSS SECTION SURVEY

Sketch or insert a figure below depicting the locations of all hydraulic cross sections required that are located outside of the 500-foot offsets from the site. An annotated aerial photograph, USGS map, or FEMA FIRM (if applicable) is preferred.

HYDRAULIC SURVEY FORM

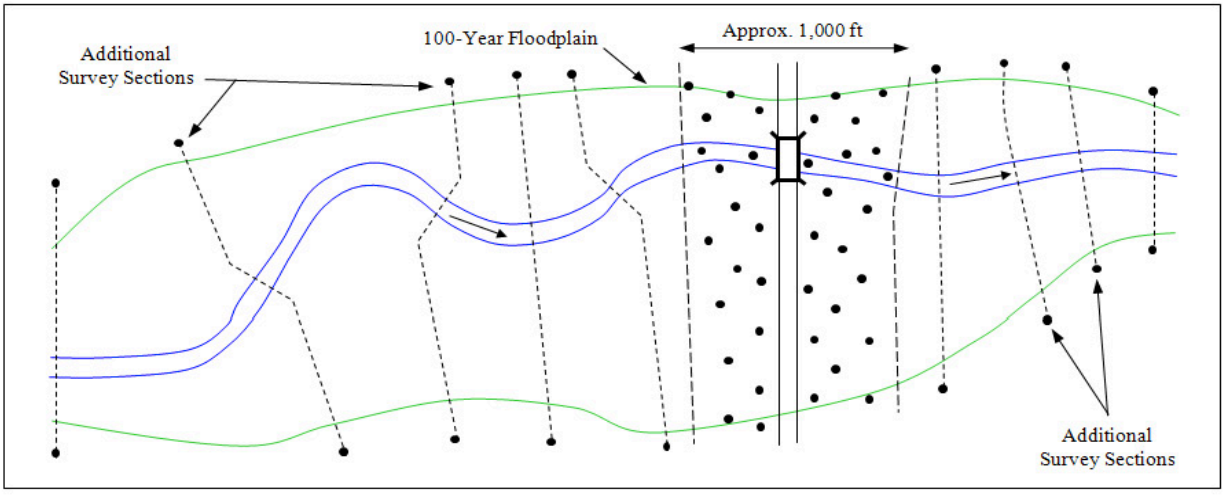
Project: _____

Stream Name: _____ Date: _____

County: _____ Job Number: _____

2B - GENERAL GUIDELINES FOR OBTAINING EXTENDED SURVEY

- The surveyed area should extend just beyond the 100-year floodplain, unless LiDAR is available.
- Cross sections should be perpendicular to the direction of flow. The direction of flow in the channel may be different than the direction of flow in the overbanks.
- Cross sections should not overlap.
- Cross sections should be perpendicular to the low flow channel and the direction of flow in the floodplain. Where the channel meanders through the floodplain, broken or dog-legged sections may be necessary. See the following sketch.
- Survey all of the applicable features listed in Parts 1A and 1B.



CHAPTER 7

HYDROLOGY

7.0 INTRODUCTION TO HYDROLOGY

A. General - Hydrology. Hydrology is generally defined as a science dealing with the interrelationship between 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 usually are considered in terms of peak runoff or discharge in cubic feet or meters per second and hydrographs as discharge per time. Peak discharge is used to design facilities such as storm drain systems, culverts and bridges. For systems that are designed to control the volume of runoff, like detention storage facilities, or where flood routing through culverts is used, the entire discharge hydrograph will be of interest.

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 structures that either are undersized and cause more drainage problems or oversized and cost more than necessary. On the other hand, one must realize that any hydrologic analysis is only an approximation. Although some hydrologic analysis is necessary for all highway drainage facilities, the extent of such studies should be commensurate with the hazards associated with the facilities and with other concerns, including economic, engineering, social, and environmental factors.

Since hydrologic science is not exact, it is possible that different hydrologic methods developed for determining flood runoff may produce different results for a particular situation. Sound engineering judgment must be exercised to select the proper method or methods to be applied. In some instances, certain federal, state, or local agencies may require that specific hydrologic methods be used for computing the runoff.

While performing the hydrologic analysis and hydraulic design of highway drainage facilities, the hydraulic engineer should recognize potential environmental problems that would impact the specific design of a structure. This area of concern should be evaluated early in the design process.

Considerations for hydrologic analysis and several of the most widely used hydrologic methods are outlined in this chapter. The omission of other hydrologic methods from this manual does not preclude their use; however, the designer should ensure the method chosen is appropriate for local conditions and acceptable to PennDOT.

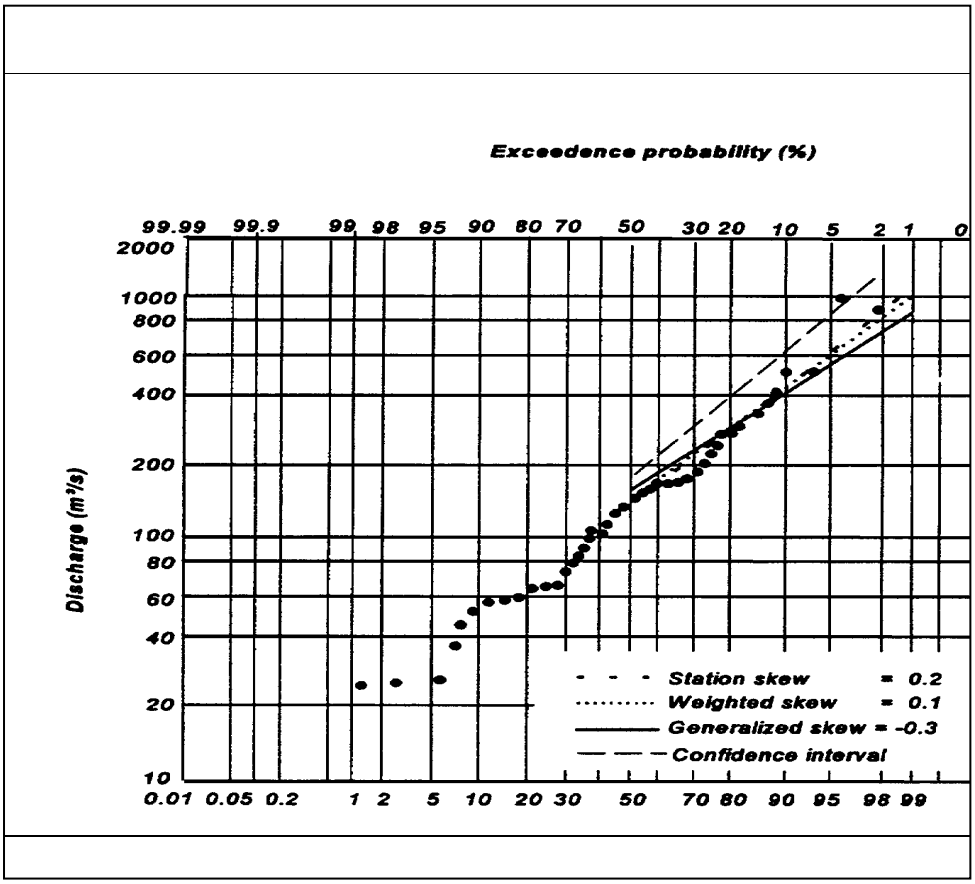
B. Peak Discharge versus Frequency Relations. Highway drainage facilities are designed to convey specific predetermined discharges in order to avoid significant flood hazards. Provisions also are made to convey floods in excess of these discharges in a manner that minimizes the damage and hazard. These discharges are often referred to as peak discharges because they occur at the peak of the stream's flood hydrograph. These flood discharge magnitudes are a function of their expected frequency of occurrence, which in turn relates to the magnitude of the potential damage and hazard.

The highway designer's chief interest in hydrology rests in estimating runoff and peak discharges for application in the design of highway drainage facilities. The highway drainage designer is particularly interested in the development of a flood magnitude versus frequency relationship. A flood frequency relation is a tabulation of peak discharges versus their probability of occurrence or exceedance. Peak discharges and probabilities of flooding will be discussed in this chapter.

A typical flood frequency curve is illustrated in Figure 7.1. In this figure the discharge is plotted on the ordinate (y-axis) on a logarithmic scale and the probability of occurrence or exceedance is expressed in terms of return interval and plotted on a probability scale on the abscissa (x-axis).

Also of interest is the performance of highway drainage facilities during the frequently occurring low flood flow periods. Because low flood flows do occur frequently, the potential exists for lesser amounts of flood damage to occur more frequently. It is entirely possible but not desirable to design a drainage facility to convey a large, infrequently occurring flood with an acceptable amount of flood plain damage only to find that the accumulation of damage from frequently occurring floods is intolerable.

Figure 7.1 Typical Flood Frequency Curve



C. Flood Hydrographs. Besides the peak discharges, the hydraulic engineer may be interested in the flood volume and time distribution of runoff. A flood hydrograph is a plot or tabulation of discharge with respect to time. Flood hydrographs can be used to route floods through culverts, flood storage structures, and other highway facilities. By accounting for the stored flood volume, the hydraulic engineer often can expect lower flood peak discharges and smaller required drainage facilities than would be expected without considering storage volume. Flood hydrographs are also useful for estimating design values of inundation times of flow over roadways as well as pollutant and sediment transport analysis.

D. Unit Hydrograph. Sherman, Snyder, and Clark developed the theory of unit hydrographs as a tool to estimate a flood hydrograph for any rainfall event. A unit hydrograph represents the response of a watershed to a unit rainfall excess having a specific duration. Excess rainfall is defined as the total rainfall minus the hydrologic abstractions (losses) and is equal to the direct runoff. For PennDOT practice, the unit is 1 mm (1 in). That is, the volume associated with an excess rainfall of 1 mm (1 in) distributed over the entire contributing area. Therefore, a unit hydrograph is a hydrograph of the runoff resulting from a hypothetical storm that has a specified duration, e.g., 1 hr, and that produces a response runoff hydrograph, resulting from the unit depth of excess rainfall over the drainage area. For example, when a unit hydrograph is shown with units of cubic meters per second, it is implied that the ordinates are cubic meters per second per millimeter of direct runoff.

The response of a watershed to rainfall is considered to be a linear process. This has two implications that are useful to the designer; the concepts of proportionality and superposition apply. For example, the runoff hydrograph resulting from a two-unit pulse of rainfall of specific duration would have ordinates that are twice as large as those resulting from one-unit pulse of rainfall of the same duration. Also, the hydrograph resulting from the sequence of two one-unit pulses or rainfall can be found by the superposition of two one-unit hydrographs. Thus, if a unit hydrograph for a watershed is known or can be determined, the flood hydrograph resulting from any measured or design rainfall can be determined using these two principles. Unit hydrograph applications are discussed later in this chapter.

E. Site Investigation. Every problem is unique, and reliance upon strict application of a standardized procedure is risky without due appreciation of the characteristics of the particular site. A field survey or site investigation always should be conducted except for the most preliminary analysis.

The need for a field survey to collect and appraise site-specific physical characteristics, as well as hydrologic and hydraulic data, cannot be overstated. Most complaints relating to highway drainage facilities result from changes to existing hydrologic and hydraulic characteristics. In order to minimize the potential for valid complaints, complete data reflecting existing drainage characteristics should be gathered and considered during design.

Typical data which should be collected during field surveys include the following:

- Highwater marks.
- Performance assessments of existing and nearby drainage structures.
- Assessment of stream stability and scour potential.
- Location and nature of important physical and cultural features which could affect or be affected by the proposed structure.
- Significant differences in land use from those indicated on available topographic maps.
- Other equally important and necessary items of information which could not be obtained from other sources, such as man-made features that affect the hydrology and hydraulics.
- Local residents, local landowners, and local or PennDOT highway maintenance personnel should be consulted.

The individual responsible for the drainage aspects of a field survey should have a general knowledge of drainage design. Field surveys should be well planned and a systematic approach should be employed to maximize efficiency and reduce wasted effort. Data collected should be well documented with written reports and photographs.

F. Interagency Coordination. Since many levels of government plan, design, and construct highway and water resource projects and because these projects often affect each other, interagency coordination is desirable and necessary. In addition, agencies can share data and experiences within project areas to assist in the completion of accurate well-coordinated hydrologic analyses.

7.1 FACTORS AFFECTING FLOODS

A. Rainfall versus Runoff Quantity/Volume. Runoff Quantity or Volume from a watershed is influenced by two main factors: precipitation and hydrologic abstractions (losses). A discussion of these factors follows.

1. Precipitation. Precipitation in Pennsylvania is represented by rainfall, snow, sleet, and hail. Rainfall occurring within a watershed can vary both temporally and spatially; however, for determinations using the Rational Method, EFH-2, and TR-55, rainfall rates are assumed spatially constant within the watershed.

2. Hydrologic Abstractions. Generally, the entire volume of rainfall occurring on a watershed does not appear as runoff. Losses, known as hydrologic abstractions, tend to reduce the volume of water entering into the stream system. Numerous losses have been accounted for in the runoff process. However, for the typical highway drainage problem, only six abstractions are commonly considered. They are:

a. Depression storage. The precipitation which is stored permanently in inescapable depressions within the watershed. It is a function of land use, ground cover and general topography.

b. Detention storage. The precipitation which is stored temporarily in the flow of streams, channels, and even reservoirs in the watershed. It is a function of the general drainage network of streams, channels, ponds, etc., in the watershed.

c. Interception. The precipitation which serves to first "wet" the physical features of the watershed (e.g., leaves, rooftops, pavements). It is mostly a function of watershed characteristics.

d. Evaporation. The precipitation which returns to the atmosphere as water vapor by the process of evaporation from water concentrations. It is mostly a function of climatological factors, but it is associated with exposed areas of water surfaces.

e. Transpiration. The precipitation which returns to the atmosphere as water vapor and which is generated by a natural process of vegetation foliage. It is a function of ground cover and vegetation.

f. Infiltration. Quantitatively, the most significant abstraction from rainfall before it becomes runoff is infiltration. The term infiltration has been used with diverse meanings, sometimes as a synonym of "percolation." However, for the purpose of highway hydrology, it may be termed as the phenomenon of water penetration from the surface of the ground into the subsurface soil. Actual infiltration (the passage of water through the soil surface into the soil) and percolation (the movement of water within the soil) are closely related, with the lesser of the two governing the abstraction of rainfall through infiltration.

Infiltration often begins at a high rate and decreases, often exponentially, to a much lower and more or less constant rate as the rain continues. The maximum rate at which a soil, in a given condition, can absorb water is called its "infiltration capacity."

Relative minimum infiltration capacities for three broad soil groups are provided for illustration purposes only:

Soil Group	Infiltration Capacity mm/hr (in/hr)
Sandy, Open-Structured	13 (0.51) - 25 (0.98)
Loam	2.5 (0.098) - 13 (0.51)
Clay, Dense-Structured	0.25 (0.0098) - 2.5 (0.098)

By definition, the surface runoff produced by a given storm is equal to that portion of the rainfall which is not removed through depression storage, detention storage, interception, evaporation, transpiration and infiltration. For most practical rainfall situations, evaporation and transpiration are negligible and therefore are usually neglected. If an estimate (i.e., initial abstraction) can be made for the depression and detention storage and the interception, only the rainfall, infiltration and runoff need to be determined. If the rainfall intensity is greater than the infiltration capacity for the duration of the rainfall, then surface runoff can be computed.

Infiltration capacity is influenced by many factors including soil type, moisture content, organic matter, vegetative cover, and the time of the year. Antecedent precipitation, such as high-intensity rains of short duration coming after a dry period, significantly affects soil infiltration capacity; the soil may be water repellent and, therefore, produce more runoff. It is noteworthy that for most soils, the infiltration capacity curve ultimately reaches a substantially constant infiltration capacity rate after a relatively short period, ordinarily 30-45 minutes.

Methods and procedures to estimate infiltration can be grouped into two categories: (1) when coincident data for rainfall and streamflow are available for the watershed of interest, and (2) where no data for streamflows are available for the watershed of interest. A detailed description given for the first case includes "the Phi-Index Method," "Horton's Equation," "Green and Ampt Equation," and other infiltration capacity formulae such as Kostiakov's formula, Philip's corresponding formulae and Holtan's formula. For the second case, the U.S. National Resource Conservation Service (NRCS) Method is cited exclusively. At times it may be necessary to actually measure the infiltration capacity of the soil(s) in the field. Basically, two major types of infiltrometers are used: "Flooding-Type Infiltrometers" and "Rainfall Simulation Infiltrometers."

The infiltration concept frequently has been used in mathematical models for predicting flood runoff from a relatively small drainage area, especially in a developed watershed.

The specific consideration of each of these abstractions is not usually explicit in the many hydrologic methods that are available. Usually, consideration of some or all of them is implicit in the methods.

B. Drainage Area. A drainage basin commonly is surrounded by a readily discernible topographic divide, which is a line of separation that divides the precipitation that falls on two adjoining basins so that the ensuing runoff is directed into one or the other channel system. The size or area of the drainage basin is considered to be the area that

contributes to the surface runoff and is bounded by all or portions of the topographic divide. The size of a drainage basin is an important parameter with respect to the response of the basin to rainfall.

Determining the size of the drainage area that contributes to flow at the drainage structure site is a basic step in a hydrologic analysis. The drainage area, usually expressed in hectares, or km^2, (acres or square miles) is determined from field surveys, topographic maps, aerial photographs, digital elevation maps, or a combination of these items.

Topographic maps are valuable aids in obtaining the size of drainage areas. The most commonly used topographic maps are those of the U.S. Geological Survey (USGS), www.usgs.gov. Information concerning these can be obtained from the Earth Science Information Center, USGS, 507 National Center, Reston, Virginia 20192.

Field inspection of the drainage area, especially for small basins, is very desirable since topographic maps are not always current. Although the contour maps may show many areas as contributing to the runoff, a field inspection may show natural or man-made depressions such as gravel pits, or natural sinks which may intercept a portion of the runoff from the drainage area. There also may be subtle topographic features that divert runoff from one watershed into another or indistinct divides not apparent on topographic maps. Once the boundaries of the contributing areas have been established, they should be delineated on a base map and the areas determined. For urban areas, a local agency's sewer and storm water maps may be a valuable source of drainage boundary information.

Diversions and area changes due to urbanization and other development inevitably occur. The designer should try to identify or otherwise anticipate such circumstances. This is often difficult but the effort should be made.

Watershed area information often can be found in the following resources:

- Topographic maps.
- County and other local maps.
- Drainage boundary surveys.
- Reference to prior plans.
- Master drainage plans.
- Local drainage district plans.
- Plans for surrounding development.

C. Shape Factor. The shape, or outline form, of a drainage basin affects the rate at which water is supplied to the main stream as it proceeds along its course from the runoff source to the site of interest. Long, narrow watersheds generally have lower peaks than fan- or pear-shaped watersheds, other characteristics being equal. It has been observed that while long, narrow watersheds may have lower runoff rates when storm direction is across the watershed, rates would be higher if a storm moves longitudinally down the basin axis. Some hydrologic methods either explicitly or implicitly accommodate watershed shape; others may not. If a drainage area is unusually bulbous in shape or extremely narrow, the designer should consider using a hydrologic method that explicitly accommodates watershed shape. The Snyder synthetic unit hydrograph initially was developed to account for shape.

D. Slope. The slope of a drainage basin has an important but rather complex relation to infiltration, surface runoff, soil moisture, and groundwater contribution to stream flow. It is one of the major factors controlling the time of overland flow and concentration of rainfall in stream channels. It is of direct importance in relation to flood magnitude. Basin slopes usually are estimated from contours on topographic maps or may be determined by a field survey. This parameter is important in that steeper basins yield a quicker response time, whereas flat basins reflect a slower response time. Long response time will lower flood peaks, while a short response time will increase the peak discharges.

E. Land Use. Since human activities can change basin runoff characteristics, land use studies are necessary to define present and future conditions, particularly with regard to the degree of urbanization or other changes that might affect runoff that may take place within the drainage basin during the expected service life of a project. Information concerning land use trends may be obtained from local, state and federal agencies and planning studies.

There are several interrelated but separable effects of land use changes on the hydrology of an area. Among these are changes in peak discharge characteristics, changes in volume of runoff, changes in quality of water, and changes in other hydrologic characteristics. Of land use changes affecting the hydrology of an area, urbanization appears to be a dominant factor.

The effect of urbanization on peak discharges depends upon such factors as the amount of the area made impervious, the changes made in the drainage pattern through the installation of storm drains, the modification of surface channels, and, with frequently occurring storms, depression storage. Alteration of the land use of a watershed changes its response to precipitation. The most common effects are reduced infiltration and decreased travel time, which can result in higher peak discharges. Infiltration and depression effects normally are most apparent on the more frequent storms – up to the 15- to 20-year recurrence interval. Above this threshold the amount of infiltration is generally small compared to the total amount of precipitation. The volume of runoff is likewise increased primarily by reduced infiltration. Although urbanization tends to increase peak discharges and volume of runoff, there are some instances where these may be reduced by the application of storm water management techniques such as the installation of impoundment facilities. However, such techniques, applied at various sites within a watershed, may not achieve the intended reduction in runoff without a coordinated basin-wide management plan. The potential and actual effects of storm water management must be considered.

Urbanization and rural watershed practices have a significant effect on the hydrology of small watersheds, but they generally do not have a great effect on large watersheds because the percentage of the total watershed that is changed is likely to be small; this is particularly important in showing that the relative effect of highways is likely to be small.

To obtain a true picture of the relative effects of urbanization at a particular location, the peak discharge should be calculated and compared for the drainage area in its natural state and after urbanization has taken place. Such measurements are seldom practical and require several years of investigation. It often becomes necessary to estimate the magnitude and frequency of peak discharges through modeling of runoff using measurable watershed characteristics.

Factors subject to change with general variations in land use include:

- Permeable and impermeable areas.
- Vegetation.
- Minor topographic features.
- Drainage systems.

All of these factors usually are influential to the rate and volume of runoff which may be expected from a watershed. Therefore, current land use and future potential land use should be considered carefully in the development of the parameters of any runoff hydrograph.

F. Soil and Geology. Soil type generally has an important effect on flood runoff, principally in its effect on infiltration. The effect of soil type often varies with the magnitude and intensity of rainfall. As with effects of urbanization, the effect of soil type decreases as flood recurrence interval increases. The condition of soil at the time of precipitation can change the amount of runoff, especially the flood peaks. If the ground is frozen or saturated with moisture, most of the precipitation will result in runoff.

The basic make-up of underlying rock formations and other geophysical factors such as glacial and river deposits, faults, limestone sinks, and playa lakes can be quite significant in affecting runoff in some areas. Stream flow records are an integrated effect of many factors and the study of such records often indicates the effect of surface soils and geology of the area on floods.

Regions underlain by soluble rock formations, especially limestone, often have characteristics of "karst" topography, which produce little surface runoff. In these areas, the runoff usually enters the ground through sinkholes and pursues its course to an outlet through a system of underground passages. In determining the runoff from basins containing karst topography, it may be appropriate to exclude all karst areas, for they may not contribute toward flood runoff. Another approach to reduce the runoff in karst topography would be to apply a reduction factor such as the procedure outlined in PSU-IV.

The Natural Resource Conservation Service (NRCS) is an excellent repository for information about soils in the state of Pennsylvania. Information for the NRCS is available at www.nrcs.usda.gov. Several hydrologic procedures described herein may require specific data concerning the soil type.

G. Storage Area – Volume. Storage within a drainage basin may be interception storage, which is the rainfall intercepted by vegetation that consequently never becomes runoff; depression storage, which is the rainfall lost in filling small depressions in the ground surface; storage in transit in overland or channel flow; or storage in ponds, lakes, or swamps. Storage may also occur in flood-control or other reservoirs, and in surface mining areas. The effect of storage on the quantity and rate of flood runoff can be quite significant in some instances.

In some areas, interception and depression storage may not be important in highway engineering and may conservatively be ignored in rural design. However, depression storage can be important in urban drainage design. Because of the complex parameters involved in the determination of the storage for overland or channel flow and its limited applicability, this type of storage usually is not considered as a reduction factor in the flood runoff computations relative to highway drainage structures unless its impact would significantly affect the magnitude of the design flow. It is more commonly considered in the design of urban storm drains.

The effect of flood-control reservoirs in changing downstream conditions should be considered in evaluating flood peaks and river stages for design of highway structures. Often, helpful data can be obtained from the operator or the owner of the reservoir project.

H. Slope and Orientation of the Basin. Although slope affects the rainfall-runoff relationship principally because of an increase in the velocity of overland flow, thereby shortening the period of infiltration and producing a greater concentration of surface runoff in the stream channels, a secondary influence resulting from the general direction of the resultant slope, or orientation of basin, should be recognized. This factor affects the transpiration and evaporation losses because of its influence on the amount of heat received from the sun. Also, the direction of the resultant slope to the north or the south affects the time of melting of accumulated snows. If the general slope is to the south, each successive snowfall may soon melt and either infiltrate into the ground or produce surface runoff. On the other hand, if the slope is to the north, these snows may accumulate throughout the winter and remain on the ground until late spring when they may be removed by a heavy rain, thus producing a potential for a high flood peak.

The amount of flood runoff can be affected by the orientation of the basin with respect to the direction of storm movement. A storm traversing a drainage basin in the direction of stream flow would produce a higher flood peak and a shorter period of surface runoff than would otherwise occur. On the other hand, a storm traversing the outlet first and traveling upstream would have the opposite effect.

I. Influence of Channel and Floodplain Geometry. Surface and subsurface runoff are collected and conveyed by stream channels. The natural or altered condition of these channels and floodplains can materially affect the volume and rate of runoff; therefore, these conditions sometimes are considered in the hydrologic analysis.

Some streams have well-defined channels; others have relatively small, low flow channels and wide floodplains. Some streams have numerous tributaries, while others have one main watercourse receiving runoff from overland flow. The sinuosity of channels affects channel storage and the progression of peak discharges. The effect of the stream network often varies with flood magnitude.

Channel cross section can affect flood discharges. Channel storage, especially in channels with extremely wide vegetated floodplains, can be very significant and can reduce discharges considerably. This effect is an integral although transparent component in some flood forecasting methods that have a statistical base such as the various practices of the USGS. Where floodplain storage is not integral with a flood forecasting method, it would be necessary to use a flood routing model such as the U.S. Army Corps of Engineers' HEC-1 Flood Hydrograph Package and HEC-RAS (River Analysis System) Computer Programs. The flood would be predicted at a point where floodplain storage was not significant, and then routed to the point of interest.

J. Stream and Drainage Densities. The stream density or stream frequency of a drainage basin may be expressed by relating the number of streams to the area drained. The stream density may be expressed as the number of streams per unit area of the drainage area. The inverse form, namely the area per stream, might also be used as a measure of stream density. In some cases, the stream density does not provide a true measure of drainage efficiency. However, it usually does reflect the potential of the magnitude of flood runoff. Generally, the larger the value of the stream density, the higher the peak and total volume of runoff will be.

Drainage density is expressed as the length of stream per unit of the drainage area. Drainage density varies inversely as the length of overland flow and therefore provides at least an indication of the drainage efficiency of the basin, which in turn affects the quantity of flood runoff.

K. Site-Specific Characteristics. Site-specific characteristics include both natural and artificial controls which determine the relation of stage to discharge and regulate the flow. Natural control of stream flow may occur due to channel constrictions, gravel bars, rock outcrops, aggradation and degradation, and ice and debris jams. Tidal fluctuation also determines the relation of stage to discharge. Sometimes channel roughness is a control. Artificial controls include dams, floodwater retarding structures, diversion dams, grade-control structures, irrigation distribution systems, and recreational and water-use facilities. Channel modification may also affect the stage-discharge relationship. Usually information concerning these structures or facilities can be obtained from the agency responsible for their operation and maintenance. The hydrologic analysis should determine the degree or effect of such controls upon flood flow.

L. Aggradation and Degradation. The water surface profile of a stream or river will be affected through a reach where deposition or erosion (aggradation or degradation) occurs. This also affects the validity of using historical highwater marks to define present conditions. Aggradation (the deposition of sediment) may lessen the channel capacity, increase flood heights, and cause overflow at a lower discharge, while degradation (the erosion of stream bed material) may increase channel capacity, thereby reducing the effect of floodplain attenuation and resulting in a higher flood peak downstream. Although difficult to determine quantitatively, the effect of present and future aggradation or degradation may be assessed when designing a highway at or near a stream, so that a design can be provided to accommodate these phenomena.

Although channel aggradation or degradation may occur naturally in the river system, this phenomenon happens frequently as a result of man-made activities. Activities which will induce the aggradation or degradation may include, for example, water diversions from the river system, water diversions to the river system, construction of reservoirs, flood control works, cutoffs, levees, channelization, navigation works, the mining of sand and gravel, and changes in land use.

M. Ice and Debris. The quantity and size of ice and debris carried by a stream should be considered in the design of drainage structures. The times of occurrence of ice or debris in relation to the occurrence of flood peaks should be determined. The effect of backwater from ice or debris jams on recorded flood heights should be considered in using streamflow records. The location of the constriction or other obstacle-causing jams, whether at the site of the structure under study or downstream, should be investigated and the feasibility of correcting the problem considered. Ice or debris jams may form below the control, backing up the water, shifting the control, and completely or partially destroying the stage-discharge relationship. Ice also may form at the control, entirely changing the relationship between gage height and discharge. A false measurement may be obtained in these cases for rating a highwater mark to estimate a historical flood discharge.

N. Seasonal and Progressive Changes in Vegetation. A realistic evaluation of the conveyance or carrying capacity of a floodplain requires consideration of both seasonal and progressive changes in vegetation. A reach of floodplain may have an appreciably lower stage for a given discharge in late winter or early spring than for the same discharge during the height of a growing season. The difference between a row crop such as corn being planted normal or parallel with the flood flow direction can, during the later part of the growing season, cause a nearly 50% difference in the floodplain conveyance. Such differences must be considered in selecting the friction or roughness factor in the conveyance equation.

Aside from a marked effect on conveyance, summer vegetation including weeds, leaves on trees and crops increases temporary floodplain storage and infiltration, which tends to change the basin response time, and, as a result, alter the quantity of flood runoff.

References for estimating friction or roughness factors are *Open Channel Hydraulics* (Chow, 1970), *Roughness Characteristics of Natural Channels* (Barnes, 1967), and *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains* (Arcement & Schneider, 1989).

O. Channel Modifications. Channel modifications may range from small alterations, such as localized dredging or minor channel-straightening, to large-scale channel improvements or major installation of flood control levees.

Channel improvements include any type of work designed to improve the carrying capacity of the stream, for example: changes in alignment, dredging, cutoffs, overbank clearing, and removal of obstructions.

By lowering the stage corresponding to a given flow, channel improvements will modify the storage relationship downward in the reach adjacent to and upstream from the improvements. This reduces the natural attenuation and thus increases flood peaks downstream. Likewise, one effect of a levee system is to impede normal attenuation and thus make flood peaks downstream from the levee system higher than they were before its construction.

It is to be noted that short channel modifications such as those commonly caused by highway constrictions usually are not considered to affect flood flows. Storm drainage systems generally reduce infiltration and decrease travel time, which results in significantly higher peak rates of runoff.

P. Future Conditions. Changes in watershed characteristics directly affect runoff. A reasonable service life of a designed facility is expected. Therefore, the estimate of design flooding should be based upon runoff influences within the time of the anticipated service life of the facility.

In general, estimates for future land use and watershed characteristics within some future range should be considered. It is difficult to predict the future but the designer should try to make an effort at such a prediction, especially with regard to watershed characteristics. Information on potential future characteristics of the watershed can often be provided by the following groups:

- Land owners.
- Developers.
- Realtors.
- Local, state, and federal officials.
- Planners.

The designer should consider items such as changes in vegetative cover, surface permeability, and constructed drainage systems in estimating future characteristics of the watershed.

Q. Climate. Climate changes usually occur over extremely long periods of time such that it is not usually reasonable to consider potential climatic changes during the anticipated life span of the facility.

7.2 DESIGN FREQUENCY

A. The Concept of Frequency. As with other natural phenomena, occurrence of flooding is governed by chance. Chance of flooding is described by a statistical analysis of flooding history in the particular watershed of interest. Since it is not economically feasible to design a structure for the maximum possible runoff from a watershed, the designer must choose a return interval appropriate for the structure. The expected return interval for a given flood is defined as the reciprocal of the probability or chance that the flood will be equaled or exceeded in a given year. For example, if a flood has a 20% chance of being equaled or exceeded each year, over a long period of time, the flood will be equaled or exceeded on an average of once every five years. This is called the return period or recurrence interval (RI). Thus exceedance probability equals 100/RI. Table 7.1 lists the probability of occurrence for the standard design return.

Table 7.1 Return Interval Terms

Return Interval (years)	Probability (%)
2	50
5	20
10	10
25	4
50	2
100	1
500	.2

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

The establishment of a design return interval implies that the facility is at risk of failure or loss, causing damage under the circumstances of larger floods. For this reason, it is important that the assignment of a design return interval be supported by consideration and assessment of those possible risks.

The designer ultimately must establish physical design criteria with respect to the selected design return interval. Lesser floods should not violate those design criteria. On the other hand, larger floods probably will exceed the established design criteria. This is the "risk" taken by the designer and PennDOT.

The designer should first develop alternative solutions which satisfy design considerations to varying degrees. After evaluating each alternative, the designer should select the design which best satisfies the requirements of the structure. It may be beneficial to consider the design return intervals of other structures along the same highway corridor to ensure that the new structure is compatible with the rest of the roadway and also to consider the probability of any part or link of roadway being cut off due to flooding.

B. Design Storms. A traditional approach to establishing a return interval for design of a drainage facility is by use of reference tables in which specific ranges of design frequencies are established for different facility types. Table 7.2 presents suggested return intervals for PennDOT bridges, culverts and cross pipes. Inundation of the travel way dictates the level of traffic service provided by a waterway facility. The travel way overtopping flood level identifies the upper limit of serviceability, and it provides one of the important definitions of the term "design flood."

Table 7.2 Suggested Design Return Intervals (years)

Design Flood Selection Guidelines		
Functional Classification	**Maximum Exceedance Probability (%)**	**Minimum Return Period (years)**
Interstate and Limited Access Highways	2	50
Principal Arterial System	2	50
Minor Arterial System	4	25
Rural Collector System, Major	4	25
Other Collector System	10	10
Local Road and Street	10	10

Use of a return period smaller than listed in Table 7.2 must be justified in the hydraulic design analysis for the project and kept with the project's files. Such justification must be based on consideration of the following kinds of factors:

- Comparison to adjacent roadway sections.
- Class of the highway.
- Potential flood hazard to property.
- Magnitude of damage and risk associated with larger flood events.
- Considerations involving right-of-way limitations and constraints imposed by adjacent land use or development.
- Limitations based on the project's scope and budget.
- Existing and future level of service.
- Future development.

The design flood is assumed to result from the storm with the same return period; therefore, the terms "design flood" and "design storm" have the same meaning.

For most highway design projects, it is important to extend the hydrologic and hydraulic analysis to include consideration of floods of magnitudes other than the design floods listed in Table 7.2. These additional floods are referred to collectively as "check floods" and they may include the following kinds of floods:

- The regulatory environmental design flood as specified in 25 PA Code §105.161 (c), §105.191, §105.201, and elsewhere.
- Storms and floods which may be required according to Stormwater Management Plans adopted according to 1978 Act 167 (the Stormwater Management Act).
- The 100-year flood for evaluation of impacts on FEMA's floodplain mapping.
- The overtopping flood or greatest flood which must be passed, as discussed in the Federal Aid Policy Guide, Subchapter G, 23 CFR Parts 650.115 and 650.117.
- The specific superflood discussed in Publication 15M, Design Manual, Part 4, *Structures*, PP Section 7.2.3 for evaluation of the potential effects of scour and evaluation of foundation stability.
- The Probable Maximum Precipitation, Probable Maximum Storm, or the Probable Maximum Flood for projects involving a high risk factor resulting from such considerations as the volume of the impounded water and potential risk of life.

In establishing a design frequency for a drainage facility, the designer (and the Department) takes a risk. The risk is that a flood may occur which is too large for the structure to accommodate. The risk results from the fact that only limited public funding is available for the drainage facility. If funds were unlimited, no risk would be necessary. Use of Table 7.2 only implies but does not quantify the level of risk. For many projects, the potential risks

associated with design by frequency selection may be determined to be small such that no further appraisal of risk is necessary. However, if deviation from the suggested design frequencies is contemplated or the potential risks could be significant, the designer should perform a risk assessment. The extent of the assessment should be consistent with the value and importance of the facility (e.g., the consequences). HEC-17, *The Design of Encroachments on Flood Plains Using Risk Analysis* (FHWA, 1980) and *Hydraulic Design of Bridges with Risk Analysis* (FHWA, 1980) are suggested references.

C. Design by Cost Optimization Using Risk Assessment. The objective of cost optimization is to choose a design return interval which results in a facility that satisfies all the design requirements with the lowest total cost. Structures with lower design return intervals generally have lower construction costs but higher maintenance, repair and replacement costs. A large structure with a high design return interval may have a much larger construction cost yet require less maintenance or repair than a smaller low design return interval structure. The larger structure may last through several lifetimes of the smaller structure. In addition, potential costs of interruption to traffic and other damage may be higher for the smaller structure. The optimal design is one that balances capital costs with operational costs and claims to produce the lowest total cost.

Capital costs are those associated with the direct construction of a facility and can be readily estimated. Generally, the higher the design return interval, the higher the capital cost. Operational costs are those associated with maintenance and repair to the facility and costs of any damage incurred by the facility. For the hydraulic design of drainage structures, the primary concern is the potential for flood damage.

Risk is defined as the consequences associated with the probability of flooding. For low return interval designs, the probability of flood-related damages and losses may be higher than that associated with higher return interval designs. A risk assessment involves qualitatively evaluating the levels of risk for selected design alternatives.

The publication, HEC-17, *Design of Encroachments on Flood Plains Using Risk Analysis* (FHWA, 1980), provides more extensive detail on this subject. The fact that the example included in HEC-17 refers to a bridge does not imply that risk analysis should be limited to bridges; the same approach is valid for the design of most drainage facilities.

Design by cost optimization using risk analysis can be largely subjective and data requirements often are much more extensive than design by return interval selection. The following list provides some examples of situations in which cost optimization may be appropriate:

- Off-system bridge replacements where the existing facility has lower capacity than the suggested design return interval for given hydrologic conditions. Usually, off-system bridges are replaced for reasons other than hydraulic. A risk analysis would help to justify whether a structure larger than the existing structure is needed.
- Where there is a need to determine whether cost of exceeding the 50-year design return interval for a flood plain crossing is justifiable. This may be particularly relevant if 100-year FEMA mapping is involved.
- Justify any design which does fall within the design frequencies suggested in Table 7.2.
- Drainage facility for a type that is not addressed in Table 7.2.
- Roadway improvements required; existing drainage facilities in good condition but do not meet suggested design frequency.
- Any situation in which the potential risks of damage are high or questionable.

D. Check-Flood Frequencies. Most flood events are of smaller magnitude than the design flood while a few are of greater magnitude. From the standpoint of facility utilization, the designer should strive toward a facility which will operate efficiently for lesser floods, adequately for the design flood, and acceptably for greater floods. For these reasons, it often is important to consider floods of other magnitudes. To define the peak flows for frequencies other than the design frequency, the designer can use the approach of developing a general flood-return interval relation for the subject site.

For some drainage facilities, including storm drain systems, the impact of the 100-year flood event should be evaluated depending upon the risk. In some cases, the designer should evaluate a flood event larger than the 100-year flood (super-flood) to ensure the safety of the drainage structure and downstream development. A 500-year flood analysis is required for checking the design of bridge foundations against potential scour failure.

There may be instances when the potential for loss of life, disruption of essential services, or excessive economic damages justify a design discharge equal to the Probable Maximum Flood (PMF). The development of the PMF is based upon these steps of estimation:

1. Probable Maximum Precipitation (PMP) from National Weather Service data and generalized charts.

2. Probable Maximum Storm based upon a spatial distribution of the PMP as governed by the following:
   - Shape.
   - Orientation.
   - Movement.
   - Storm area.
   - A temporal distribution of precipitation during the storm.

3. PMF based upon hydrograph methods - FHWA Federal Aid Policy 23 CFR 650.

E. Frequencies of Coincidental Occurrence. Where the outfall of a system enters as a tributary of a larger drainage basin, the stage-discharge characteristics of the outfall may depend on the state of the main drainage basin and the probability of flooding events in the two systems may be independent. This is especially common in storm drain systems. For example, a small storm drain system designed for a 5-year return interval discharge may outfall into a major channel associated with a much larger watershed. The two independent events affecting the design are the storm occurring on the small storm drain system and the storm contributing to discharge in the larger watershed.

The simultaneous occurrence of two independent events is defined as the product of the probability of the occurrence of each of the individual events. In other words, if the events are independent, the probability of 5-year events occurring on the storm drain and the larger watershed simultaneously is 0.2 * 2 = 0.04 or 4%. This is equivalent to a 25-year frequency.

In ordinary hydrologic circumstances, particularly with adjacent watersheds, flood events are not entirely independent. Table 7.3 presents suggested return interval combinations for coincidental occurrence. Each design contains two combinations of frequencies; for instance, a 5-year design with watersheds of one square kilometer (1,000,000 m^2), or 0.386 square miles (10,800,000 ft^2) and one hectare (10,000 m^2, i.e., 100:1), or 2.47 acres (1580 ft^2, i.e., 100:1) can employ either of the following:

- A 2-year design on the main stream and a 5-year design on the tributary.
- A 5-year design on the main stream and a 2-year design on the tributary.

A structure required that satisfies both return interval combinations is assumed to meet the 5-year design objective.

Example:

Determine the 25-year design discharge to be used for a bridge located immediately downstream of the confluence of two streams. Stream A has a contributing drainage area of 100 acres. Stream B has a drainage area of 1,000 acres. The discharges are as follows:

Return Period	Stream A	Stream B
10 yr	300 cfs	2700 cfs
25 yr	450 cfs	3600 cfs

Using the drainage area ratio of 10:1, the table indicates that the 10-, and 25-year event should be combined to determine the combination that governs.

10 yr (A) + 25 yr (B) = 3,900 cfs when routed to structure
10 yr (B) + 25 yr (A) = 3,150 cfs when routed to structure

Use the greater of the two calculated numbers to satisfy the design. Note that this number is smaller than that predicted if 25 yr (A) + 25 yr (B) = 4,050 cfs.

F. Rainfall versus Flood Frequency. Drainage structures are designed based on some flood frequency. However, certain hydrologic procedures use rainfall and rainfall return interval as the basic input, with the basic assumption that the flood return interval and the rainfall return interval are the same. Depending on antecedent soil moisture conditions and other hydrologic parameters, this may or may not be true. However, for the design of hydraulic structures this is a reasonable assumption.

Table 7.3 Frequencies for Coincidental Occurrence

Area Ratio	2-year design		5-year design	
	main stream	**tributary**	**main stream**	**tributary**
10,000:1	1	2	1	5
	2	1	5	1
1,000:1	1	2	2	5
	2	1	5	2
100:1	2	2	2	5
	2	2	5	2
10:1	2	2	5	5
	2	2	5	5
1:1	2	2	5	5
	2	2	5	5
	10-year design		**25-year design**	
	main stream	**tributary**	**main stream**	**tributary**
10,000:1	1	10	2	25
	10	1	25	2
1,000:1	2	10	5	25
	10	2	25	5
100:1	5	10	10	25
	10	5	25	10
10:1	10	10	10	25
	10	10	25	10
1:1	10	10	25	25
	10	10	25	25
	50-year design		**100-year design**	
	main stream	**tributary**	**main stream**	**tributary**
10,000:1	2	50	2	100
	50	2	100	2
1,000:1	5	50	10	100
	50	5	100	10
100:1	10	50	25	100
	50	10	100	25
10:1	25	50	50	100
	50	25	100	50
1:1	50	50	100	100
	50	50	100	100

7.3 HYDROLOGIC METHOD SELECTION

A. Overview of Hydrologic Method Selection. Estimating peak discharges of various intervals is one of the most common engineering challenges faced by facility designers. The problem can be divided into two general categories:

- Gaged sites - Limitations on the use of gaged sites is discussed in Section 7.10.C.
- Ungaged sites - The site is not near a gaging station and no stream flow record is available. This situation is very common.

This chapter addresses hydrologic procedures that can be used for both categories. This section provides some guidance on selection of hydrologic methods. In general, results from several methods should be compared. The designer should use the discharge that appears to best reflect local project conditions. Averaging of results of several methods is not suggested. In addition, the designer should document reasons supporting the selection of the results.

B. Peak Flow Rates versus Hydrographs. Generally, the estimation of peak discharge is adequate for design of conveyance capacity of systems such as storm drains, open channels, culverts and bridges. However, if the design necessitates flood routing (e.g., through storage basins, complex conveyance networks, and pump stations), a flood hydrograph is required.

C. Hydrologic Procedures. Countless hydrologic methods are available for estimating peak discharges and runoff hydrographs. The following sections list selected hydrologic methodologies. These methodologies, when properly selected and applied in engineering analyses, will be acceptable to PennDOT, and are preferable over equivalent alternative methodologies. It is not PennDOT's intention to replace the use of sound engineering judgment when any unlisted methodology is determined to be superior; however, use of unlisted methodologies should be coordinated carefully with PennDOT at the earliest possible opportunity during the project development process.

The level of accuracy required for a specific hydrologic analysis is a matter of engineering judgment that generally depends on the specific characteristics of each individual project. Factors that tend to control the final accuracy of hydrologic studies include the selection of analytical methods or models and the level of effort invested in data collection and in the application of the method or model. Such factors generally are determined during the early phases of a highway project.

With respect to selection of methods or models, this document offers general guidelines regarding selection criteria; however, final decisions regarding the suitability of a particular method for a particular project must be determined by engineering judgment on a case-by-case basis.

D. Site Flood History. Discrepancies between any numerical methodology such as HEC-1, TR-55, the rational formula, or a regression method, and the site history should be considered very carefully and explained very thoroughly.

Site information such as the performance of an existing structure or the local history of flooding provide important data for corroborating the results of the numerical models used in the hydrologic study, especially if one or more peak water-surface elevations can be determined. When the return periods for a high-water event can be estimated, that event can be used to perform a partial calibration of the hydrologic model.

When known events are smaller than the design event, considerable care is required because the estimate of the design peak flow is an extrapolation from the observed historical data. If the watershed has undergone development, construction of flood control structures, reforestation, or other changes since the observed historical events, or if future changes are anticipated, care must be taken to ensure that these factors are accounted for appropriately in the hydrologic study.

E. FEMA Studies. If a project site lies within a Federal Emergency Management Agency (FEMA) study area, review the flood discharges reported in the FEMA study and incorporate into the hydrologic analysis. Verify FEMA's flood discharges by an appropriate hydrologic methodology when hydrologic conditions in the basin differ from the conditions described in the FEMA study.

For more information about FEMA, contact their web site at www.fema.gov/about/. If FEMA studies are to be used for sizing the waterway structure, then care should be taken to avoid misuse of the information. It should be remembered that the underlying purpose of a FEMA study is very different from a study to design an opening for a waterway structure; therefore, different levels of detail and different methodologies may be appropriate for these two types of studies. Often the FEMA study can be used as a relative comparison, comparing the present condition with the proposed.

The hydrologic methods or models used in FEMA studies must be evaluated according to current design guidance and current modeling practices for sizing highway waterway structures. For example, the peak flows in a number of Pennsylvania FEMA studies were based on PSU-III. Use of these peak flows would not be acceptable to the Department because PSU-III was superceded by PSU-IV in 1981. In this case, the results of the FEMA study shall be included in the engineering analysis for comparative purposes. Discrepancies between the results of the FEMA study and the methods selected for the project design shall be carefully analyzed and documented.

In all cases, when the site lies within a Federal Insurance Rate Map (FIRM) detailed study area, use of a 100-year discharge that differs from FEMA's for design of a waterway structure requires careful, documented justification that fulfills the review criteria of both the Pennsylvania Department of Environmental Protection (PA DEP) and FEMA.

F. 1978 Act 167 Stormwater Management Studies. If the subject drainage basin is part of a Storm Water Management Plan pursuant to 1978 Act 167, review the flood discharges reported in the plan and incorporate into the hydrologic analysis. Verify the flood discharges reported in a storm water management plan by an independent source or methodology if actual conditions in the basin or if map data have changed so that the two studies are based on different hydrologic information. Explain thoroughly the use of flood discharges for design of a waterway structure that differ significantly from the Act 167 discharges. PA DEP's Erosion and Sediment Control Manual, the model Storm Water Management Ordinances for the 1978 Act 167 program, and guidance associated with 25 PA CODE § 102 and § 105 may contain specific limits to the upper limit of applicability of various runoff models. The reader should check the regulations and regulatory guidance for more restrictive requirements.

For more information concerning the PA DEP contact their web site at www.depweb.state.pa.us. It should be remembered that the underlying purpose of an Act 167 study is very different from a study to design an opening for a waterway structure; therefore, different levels of detail and different methodologies may be appropriate for these two types of studies.

G. Hydrologic Methods and Models. Hydrologic methods are used to determine the various flow rates for waterway structures: design flood, overtopping flood, Q100, Q500, flood-of-record, probable maximum flood, etc. The hydrologic methods and models included in the PennDOT standard H&H toolbox are listed below. Brief statements on the use of the methods are included, as well as information on when the methods should not be used. If a design project does not meet the criteria for using a specific listed method and the method is selected anyway, then justification must be provided for making that selection.

For methods that require rainfall depths, such as the rational formula, TR-55, HEC-1, and HEC-HMS, the values should be obtained from Appendix 7A, *Field Manual for Pennsylvania Design Rainfall Intensity Charts from NOAA Atlas 14 Version 3 Data*. Generally, it may be assumed that rainfall of a given return period produces a flood of the same return period. See Section 7.2.B for specific guidance on selecting magnitudes of design storms.

PA DEP has specific limitations to the various hydrologic methods and models that can be used within the state. Also, guidance on the use and limitations of many of the listed methods and models is found in various sections of HDS-2, *Highway Hydrology* (FHWA, 1996), *The Model Drainage Manual* (AASHTO, 2005), and the *Highway Drainage Guidelines* (AASHTO, 2003).

H. Analysis of Stream Gage Records. If a design flow rate is being computed for a location on the same "main stem" of a stream within 0.5 to 1.5 times the gage's basin area, stream gage records shall be used to compute design or flood discharges. The hydrologic analysis for the gage should follow the suggestions of Bulletin 17B, *Guidelines for Determining Flood Frequency* (U.S. Water Resources Council, 1982). The statistical analysis described in Bulletin 17B often is referred to simply as the "WRC" method. Generally, a WRC analysis of a gage record takes precedence over all other hydrologic methods. When a gage record is of short duration, or poor quality, or the

results are judged to be inconsistent with field observations or sound engineering judgment, then the analysis of the gage record should be supplemented with other methods.

The validity of a gage record should be demonstrated following procedures outlined in Bulletin 17B. Gage records should contain at least 10 years of consecutive peak flow data and they should span at least one wet year and one dry year. If the runoff characteristics of a watershed are changing due to urbanization, for example, then a portion of the record will not be valid. If an invalid portion of a record is used, the results will be biased.

The USGS's computer program PEAKFQ performs a standard WRC analysis. The PEAKFQ program and instructions for its use are available from the USGS. The peak flow values computed from a gage record can be transposed from one location to another if $0.5 \leq \frac{A_g}{A_{Watershed}} \leq 1.5$ by scaling the discharge by a ratio of the drainage areas raised to an exponent. The Peak flow values can be transposed with the following equation:

(Equation 7.1)

$$\frac{Q}{Q_g} = \left(\frac{A}{A_g}\right)^b$$

where:

Q	=	peak discharge, m^3/s (cfs)
A	=	basin area above project, ha (ac)
Q_g	=	WRC peak discharge at gage, m^3/s (cfs)
A_g	=	area of gaged basin, ha (ac)
b	=	drainage area characteristic coefficient from Table 3 in the SIR 2008-5102 report, *Regression Equations for Estimating Flood Flows at Selected Recurrence Intervals for Ungaged Streams in Pennsylvania,* (USGS, 2008), for the basin's Flood Flow Region and recurrence interval.

I. Rational Method. The rational method (rational formula) is the suggested hydrologic method for estimating peak flows for drainage areas up to 80 hectares (200 acres) in size. Use of the rational formula on larger drainage areas above this limit requires the use of sound engineering judgment to ensure that reasonable results are obtained. The rational method may be used with caution up to 100 hectares (250 acres) with specific approval by qualified District personnel (typically the District H&H Coordinator).

The hydrologic assumptions underlying the rational formula include constant and uniform rainfall over the entire basin with a duration equal to the time of concentration (t_c). If a basin has more than one main drainage channel, if the basin is divided so that hydrologic properties are significantly different in one section versus another, if $t_c > 60$ minutes, or if storage is an important factor, then the rational method is not appropriate.

For typical roadway drainage problems where all of the conditions discussed in the preceding paragraph are met, the rational method should be applied. When replacing pipes less than 750 mm (30 in) in diameter, the time of concentration (t_c) may be reduced to 5 minutes to compute the design discharge.

J. Regression Methods. Regional regression equations provide estimates of peak flows at ungaged sites. They are comparatively easy-to-use and they provide relatively reliable and consistent findings when applied by different hydraulic engineers. The three methods listed below, USGS SIR 08-5102, USGS WRIR 00-4189, and PSU-IV, are statistical methods that quantify general regional relationships between peak flows, or other runoff variables, and a watershed's physiographic, hydrologic and meteorologic characteristics. PSU-IV is suggested only as a comparison method unless detailed site history justifies the flows developed using PSU-IV. USGS WRIR 00-4189 has been replaced by the USGS SIR 08-5102 method, but in a few specific scenarios, the USGS WRIR 00-4189 method may be considered as an acceptable method as further discussed in Section 7.3.J.4.

The regression processes used to derive the equations in USGS SIR 08-5102, USGS WRIR 00-4189, and PSU-IV tend to smooth the effects of the independent variables on computed peak flow estimates. For basins with atypical hydrologic characteristics, this smoothing effect can be a problem. Atypical characteristics may include steep slopes in the watershed, watersheds with higher length to width ratios, etc. To ensure quality in these hydrologic estimates, it may be prudent to consider comparing the results of USGS SIR 08-5102, USGS WRIR 00-4189, and PSU-IV with

the site's history of flooding and/or the results of physically based hydrologic models such as HEC-1. By analyzing and explaining any differences in result from the various methods included in the study, the confidence in the final peak flow estimates can be improved.

1. USGS SIR 08-5102. *Regression Equations for Estimating Flood Flows at Selected Recurrence Intervals for Ungaged Streams in Pennsylvania, Scientific Investigations Report (SIR) 08-5102* (USGS, 2008a) is a regression analysis of streamflow data for Pennsylvania drainage basins ranging in size from approximately 259 ha (1.0 mi^2) up to approximately 5200 km^2 (2000 mi^2). USGS SIR 08-5102 was first incorporated into the USGS's National Stream Statistics (NSS) program Version 4.0.b (USGS, 2008b). The program includes regression equations for estimating a typical flood hydrograph for a given recurrence interval as well as other stream low flow statistics. Although the NSS computer program has been incorporated into the Environmental Modeling Research Laboratory's (EMRL) Watershed Modeling System (WMS), WMS 8.1, 8.2 and 8.3 contain NSS version 4.0, which uses the USGS WRIR 00-4189 regression equations. If you are using WMS version 8.3 or older, the hydrologic data must be exported from WMS to use in either a standalone NSS program Version 4.0.b or newer or use the equations from the USGS SIR 08-5102 publication.

2. USGS WRIR 00-4189. *Techniques for Estimating Magnitude and Frequency of Peak Flows for Pennsylvania Streams, Water-Resources Investigations Report 00-4189,* (USGS, 2000) is a regression analysis of stream flow data for Pennsylvania drainage basins ranging in size from approximately 390 hectares (1.5 mi^2) up to approximately 5,200 km^2 (2,000 mi^2). USGS WRIR 00-4189 rarely is used for basins larger than 650 km^2 (250 mi^2) because a gaging station normally is available within 0.5 to 1.5 times the basin area.

The equations presented for Region B should not be used if the stream drains a basin with more than 5% urban development. The USGS WRIR 00-4189 regressions equations should not be used for areas less than 390 hectares (1.5 mi^2), for streams that drain extensively mined areas or for stream reaches immediately below flood-control reservoirs.

USGS WRIR 00-4189 has been incorporated into the National Flood Frequency (NFF) computer program and the USGS urban regression equations have been included to accommodate developed areas. NFF is available from the USGS's web site for surface water software. NFF has been superseded with the National Stream Statistics (NSS) program which includes the USGS SIR 08-5102 equations as described above.

3. PSU-IV. PSU-IV is a Pennsylvania regional regression method that is based on the Log Pearson III equation. Generally, the PSU-IV regression equations are considered valid for basins from 390 hectares (1.5 mi^2) to 390 km^2 (150 mi^2). PSU-IV is considered to produce good estimates for peak flows in this range; however the PSU-IV model requires caution when being applied to urbanized areas. In urbanized areas in Region 1, the default urban adjustment factor should NOT be applied.

The use of PSU-IV analyses in all urban areas should be based on sound engineering judgment. Urbanized watersheds larger than 39 km^2 (15 mi^2) require consideration of Stankowski's Urbanization factors in lieu of the default urbanization method in the PSU-IV program or USGS Urban Regression Equation.

Since USGS WRIR 00-4189 is based on data through 1997 and USGS SIR 08-5102 is based on data through 2006, PSU-IV is considered only as a comparison method. PSU-IV can be used to compare estimates from other methods, but should not be used as the final hydrologic method for flow selection unless there is site flooding history that justifies its use. PSU-IV may be used as a comparative method only.

4. Summary of Regression Performance. The two most recent USGS methods (USGS WRIR 00-4189 and USGS SIR 08-5102) were evaluated by PennDOT. In general the USGS SIR 08-5102 method will be the most applicable regression method, but there are some areas of the state that one of the older regression methods may be considered. When compared to observed values (i.e., stream gage analyses), severe under- and over-predictions occur using both USGS methods, and there is no real physiographic or basin characteristic pattern to the results. The USGS regression flows for watersheds with smaller drainage areas (<13 km^2 (<5 mi^2)) also have inconsistent results. The watersheds where USGS SIR 08-5102 highly over- or under-predicted the gage value are listed in Table 7.4 and shown in Figures 7.2 and 7.3. The observed values were calculated using a Log Pearson III analysis with a weighted skew coefficient, while the weighted values were calculated to minimize period of record bias by computing a predicted flood frequency discharge weighted average of the

observed, as well as the USGS SIR 08-5102 computed, using the period of record of the station and the equivalent period of record for the regression equation (USGS, 2008a).

The following procedure is recommended to determine the most appropriate design flows if regression methods are applicable.

5. Recommended Methodology for use of Regression Equations. The USGS SIR 08-5102 regression equations generally perform adequately in predicting flows; however, there are instances where flows may be unrepresentatively high or low in comparison to what one would expect from a stream gage analysis. Therefore, the following set of guidelines should be followed when watershed characteristics are within the limitations of the USGS SIR 08-5102 regression equations.

a. Determine the watershed in which the site is located.

b. Determine if the site may be substantially affected by upstream regulation, using USGS SIR 08-5102, Appendix 3 as a guide.

c. Determine if the site is in a watershed where USGS SIR 08-5102 may significantly over or under-predict the design event calculated from gage data by referring to Table 7.4 and Figures 7.2 and 7.3.

d. Perform calculations using the USGS SIR 08-5102 method to estimate flows. Evaluate the predicted flows in the hydraulic model and compare to local flood history and engineering judgment.

e. When the project site is located in one of the two precaution areas mentioned below and the results from USGS SIR 08-5102 are not consistent with the local flood history, it may be necessary to consider other hydrologic methods including the USGS WRIR 00-4189 method or regional gage comparison. Note for the gages/watersheds listed in Table 7.4 and shown in Figures 7.2 and 7.3, consideration should also be given to the number of years of record and quality of data at the gaging station for which the regression flows are being compared.

Precautions for use of the USGS SIR 08-5102 method as a result of PennDOT's evaluation include:

a. USGS SIR 08-5102 results in the watersheds identified in Table 7.4 and Figures 7.2 and 7.3 were particularly different from the gage data and should be closely evaluated for applicability. Some of these watersheds have shown to severely under-predict flows as compared to gage data within the watershed.

b. Sites with drainage areas less than <13 km^2 (<5 mi^2) for both the USGS SIR 08-5102 and USGS WRIR 00-4189 methods.

Table 7.4 Gages with the Highest Over- or Under-Predicted USGS SIR 08-5102 Values and Their Watershed Characteristics

Percent Difference from the Weighted Values	Percent Difference from the Observed Values	USGS Streamflow Station Number	Station Name	Drainage Area (mi^2)	Last Year of Record	Years of Record*	Mean Elevation	Percent Carbonate	Percent Storage	Percent Urban	Region
Highest Over-predictions											
89	167	1449360	Pohopohco Creek Kresgeville.	49.9	2006	40	1060	0	1.93	7.3	1
35	155	1480800	E. Br. Brandywine Ck. at Downingtown, PA	81.6	1968	11	531	6.72	2.03	9.03	2
81	130	1441000	McMichael Creek near Stroudsburg, PA	65.3	1955	28	952	0.23	3.34	3.17	1
68	118	1447680	Tunkhannock Creek near Long Pond, Pa.	20	2006	40	1,880	0	21.24	0.74	1
68	112	1438300	Vandermark Creek at Milford, Pa.	5.09	2006	45	1,060	0	0.75	5.31	1
Lowest Under-predictions											
-61	-71	03008000	Newell Creek near Port Allegany, Pa.	7.79	1978	19	1840	0	0	0.1	3
-64	-70	03049800	Little Pine Creek near Etna, Pa.	5.78	2005	42	1100	0	0	26	4
-57	-68	01638900	White Run near Gettysburg, Pa.	12.5	1980	20	553	0	2.31	8.64	4
-55	-68	01473100	Zacharias Creek near SkipPack, Pa.	7.29	1980	21	322	0	0.06	9.65	2
-53	-67	01479820	Red Clay Creek near Kennett Square, Pa.	28.33	2005	18	371	11.1	1.24	17.6	2
-55	-65	03021410	West Branch French Creek near Lowville, Pa.	52.3	1994	20	1470	0	5.25	1.12	3
-57	-64	01564512	Aughwick Creek near Shirleysburg, Pa.	296	2005	16	1,170	7.03	0.47	0.71	3
-49	-63	01578400	Bowery Run near Quarryville, Pa.	5.98	1981	19	655	25.4	0.98	0.02	2
-50	-63	01553600	EB Chillisquaque Creek near Washingtonville, Pa.	51.5	2005	26	735	6.65	1.14	0.23	1
-45	-62	03100000	Shenango River near Turnersville, Pa.	152	1922	11	1090	0	22.61	5.56	3
-48	-58	01473120	Skippack Creek near Collegeville, Pa.	53.7	1994	29	289	0	0.16	24.7	2
-40	-55	01429500	Dyberry Creek near Honesdale, Pa.	64.6	2005	16	1470	0	3.57	0.21	1
-50	-55	01567500	Bixler Run near Loysville, Pa.	15	2005	52	907	0	0.91	0.23	3
-38	-54	03029000	Clarion River at Ridgway, Pa.	303	2004	12	1910	0	0.93	2.71	3
-44	-54	01516350	Tioga River near Mansfield, Pa.	153	2005	34	1870	0	0.24	1.45	1
-42	-53	01546000	North Bald Eagle Creek at Milesburg, Pa.	119	1934	18	1400	7.47	0.32	0.72	3
-45	-53	01428750	West Branch Lackawaxen River near Aldenville, Pa.	40.6	2006	32	1750	0	1.95	0.28	1
-38	-53	01475530	Cobbs Cr at U.S. Hghwy No. 1 at Philadelphia, Pa.	4.78	2005	18	319	0	0.42	87.7	2
-48	-53	01447500	Lehigh River at Stoddartsville, Pa.	91.7	2006	65	1830	0	14.04	7.47	1
-35	-51	03104760	Harthegig Run near Greenfield, Pa.	2.26	1980	12	1260	0	4.12	2.08	3
-46	-51	03026500	Sevenmile Run near Rasselas, Pa.	7.84	2005	54	2070	0	0	0.16	3

* Years of record used in the USGS SIR 08-5102 analysis primarily consists of systematic period of record and may include historic peak year.

Figure 7.2. Watersheds and Gage Numbers with the Highest Over-Prediction from the Observed and Weighted Gage Flows.

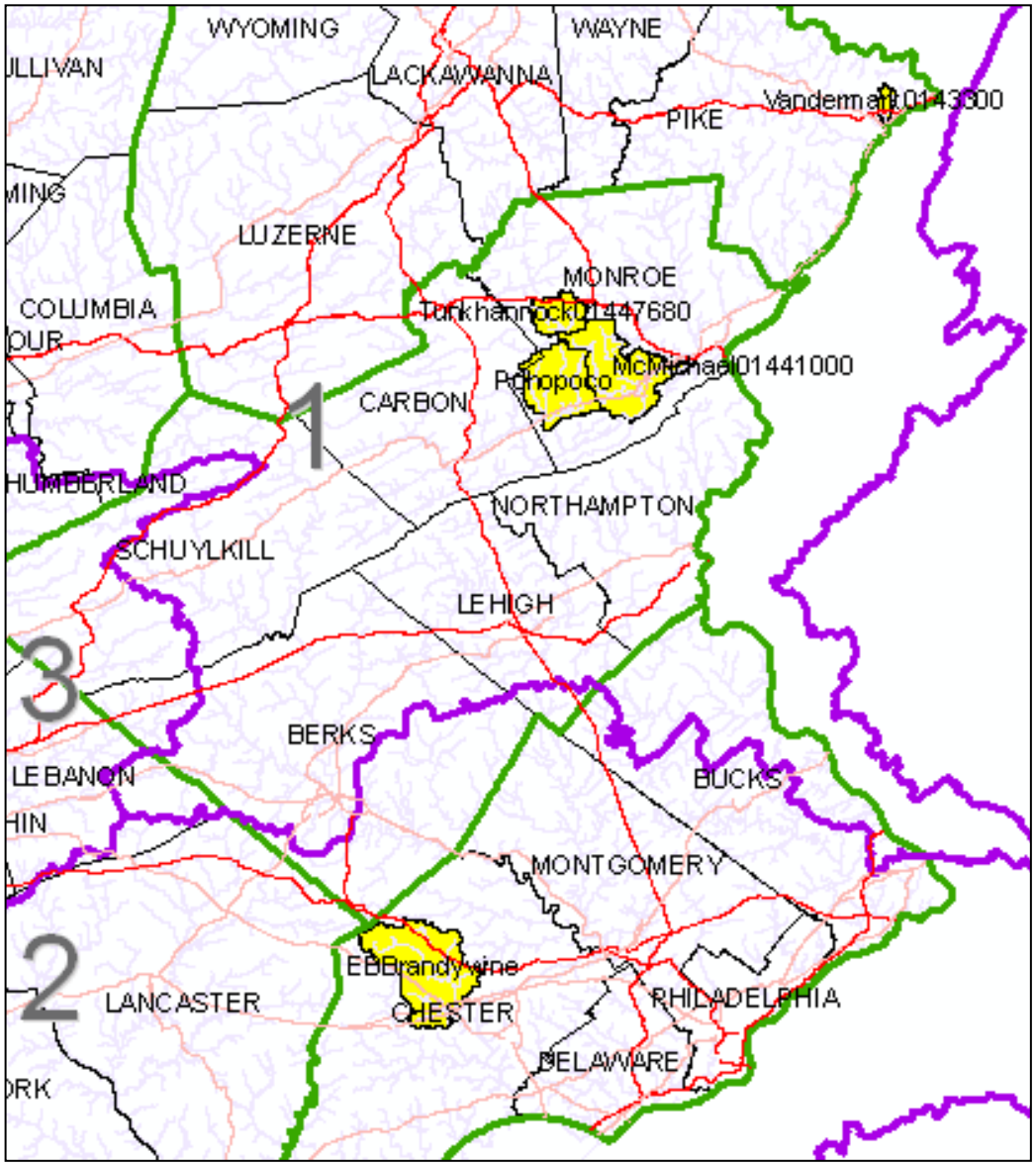
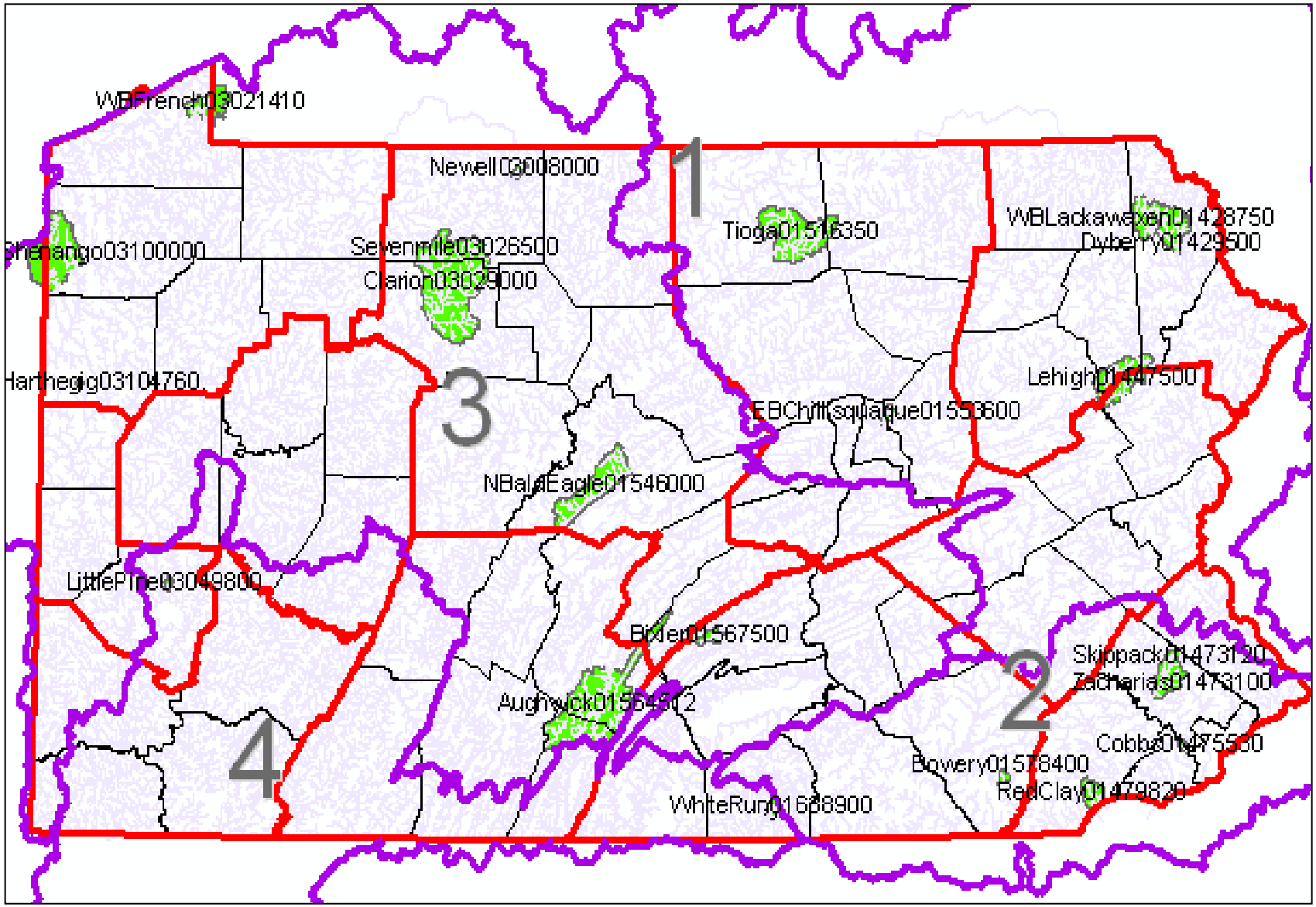


Figure 7.3. Watersheds and Gage Numbers with the Highest Under-Prediction from the Observed and Weighted Gage Flows.



K. HEC-1 & HEC-HMS. HEC-1 & HEC-HMS are generalized hydrologic simulation models that can be used with basins of almost any size and complexity. Use of the Engineering Computing Graphics Laboratory's (ECGL's) Watershed Modeling System (WMS) as an interface to HEC-1 is suggested since it systematizes the computation of the physiographic and hydrologic parameters required by HEC-1. When WMS is used for its graphical user interface to HEC-1, a practical upper limit in the vicinity of 300 to 400 km^2 (100 to 150 mi^2) is suggested.

HEC-1 assumes that the rainfall is spatially uniform over each sub-basin modeled. Soil Conservation Service (SCS) rainfall time distributions, loss methods, dimensionless unit hydrographs, and the lag equations often are used; however, careful consideration must be given to the assumptions and limitations underlying these methods.

The SCS developed the curve number method (i.e. the SCS lag equation) for basin sizes less than 800 hectares (2000 acres, 3.1 mi^2; NRCS, 1985). The upper limit on basin area for the SCS Loss Method (i.e., Runoff Curve Number) is not well established; however, ~5160 hectares or 52 km^2 (1290 acres, 20 mi^2) has been suggested. These limitations may be overcome by subdivision of the watershed and appropriate routing.

WMS uses USGS Digital Elevation Modules (DEMs) to compute basin geometric parameters. Relatively large data sets are required for, and produced by, WMS for basins spanning more than 8 or 10 DEMs. This may result in a serious degradation of computer performance and may require "overnight" computer runs. The use of HEC-1 by itself may be prudent in these cases.

When using WMS in urban areas, care must be taken to ensure that storm water management facilities are incorporated correctly into the model. This will require additional careful modeling and routing.

L. WinTR-55 and TR-55. Technical Release 55 (TR-55), *Urban Hydrology for Small Watersheds* (NRCS, 1986) provides a graphical method for computing peak discharges of drainage basins with areas ranging from 4.0 hectares (10 acres) up to 800 hectares (2000 acres, 3.1 mi^2).

TR-55 is a segmental method (i.e., flow time is computed by adding the times for the overland, shallow concentrated, and channel segments). TR-55 considers hydrologic parameters such as slope, roughness, losses, rainfall intensity, soil type, land use, and time. Although TR-55 has fewer assumptions than the rational formula, it also assumes that rainfall is uniform over the entire basin. Some hydrologists have stated that TR-55 tends to produce conservatively high estimates of peak flows. TR-55 should be used with caution when structure sizing is highly sensitive to the computed peak flow values.

This method must meet the following conditions:

- Basin drained by a single main channel or by multiple channels with times of concentration (t_c) within 10% of each other.
- t_c between 0.1 and 10 hours.
- Storage in the drainage area is $\leq$5% of the basin area and does not affect the time of concentration (i.e., storage in the drainage area is not on the main channel in which the t_c is being computed and is less than or equal to 5% of the drainage area).
- The watershed can be accurately represented by a single composite curve number.

The "Graphical Method" module in the TR-55 computer program and the TR-55 model in WMS are equivalent to the manual graphical method described in the TR-55 report. TR-55 includes a procedure for computing a "rough" synthetic hydrograph, which can be used to size small ponds; however, TR-55 should not be used for flood routing, which would include both channel routing and storage routing.

Refer to the TR-55 report for a complete discussion of limitations and assumptions of this methodology.

M. EFH-2. EFH-2 determines peak discharge by procedures discussed in *Engineering Field Handbook*, Chapter 2 (NRCS, 1985). EFH-2 is available online at http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/quality/?cid=stelprdb1042921. This method is applicable to rural watershed between 0.4047 and 809.4 hectares (1 and 2000 acres), and must meet the following conditions:

- Hydraulic length is between 60.0 and 7,900.8 meters (200 and 26,000 feet).

- Average slope is between 0.5 and 64%.
- Valley or reservoir routing is not required.
- Watershed can be represented accurately by a single composite curve number between 40 and 98.
- Urban land uses comprise no more than 10% of the basin.

Refer to the EFH-2 Manual for complete discussion of the methodology and its limitations.

7.4 TIME OF CONCENTRATION

A. Overview of Time of Concentration. Several common hydrologic methods require an estimation of the time of concentration. This section provides guidance on ways to estimate time of concentration for use in procedures such as the Rational Method and the SCS runoff curve number methods.

The time of concentration (t_c) is the time at which all of the watershed begins to contribute to runoff; this is calculated as the time taken for runoff to flow from the most hydraulically remote point of the drainage area to the point under investigation. Various methods can be used to estimate the time of concentration of a watershed. When selecting a method to use in design, it is important to select a method that is appropriate for the flow path. Some methods were developed as lumped parameter methods in that they characterize the time for the entire watershed: the SCS lag equation (Equation 7.2) is an example. Other methods are intended for one segment of the principal flow path and produce a flow velocity that can be used with the length of the segment to compute a travel time (L/V = time). With this method, the time of concentration equals the sum of the travel times on each segment of the principal flow path.

Generally, it is reasonable to consider three components of flow which can characterize the progression of runoff along a travel path: sheet flow (overland flow), shallow concentrated flow, and concentrated channel flow or conduit flow. Sheet flow, which is defined mathematically in Section 7.4.C., occurs in the upper reaches of the 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 (small depressions that flow intermittently, only when it rains) and swales. Equation 7.3 typically is used to compute the time for this portion of the overland flow. Concentrated flow is runoff that occurs in rills and swales and has depths on the order of 40 to 100 mm (1.58 to 3.94 in). Velocities in open channels usually are determined using the Manning equation to solve for the velocity. There may be a number of possible paths to consider in determining the longest travel time. The designer should identify the flow path along which the longest travel time is likely to occur. This can be a trial-and-error process

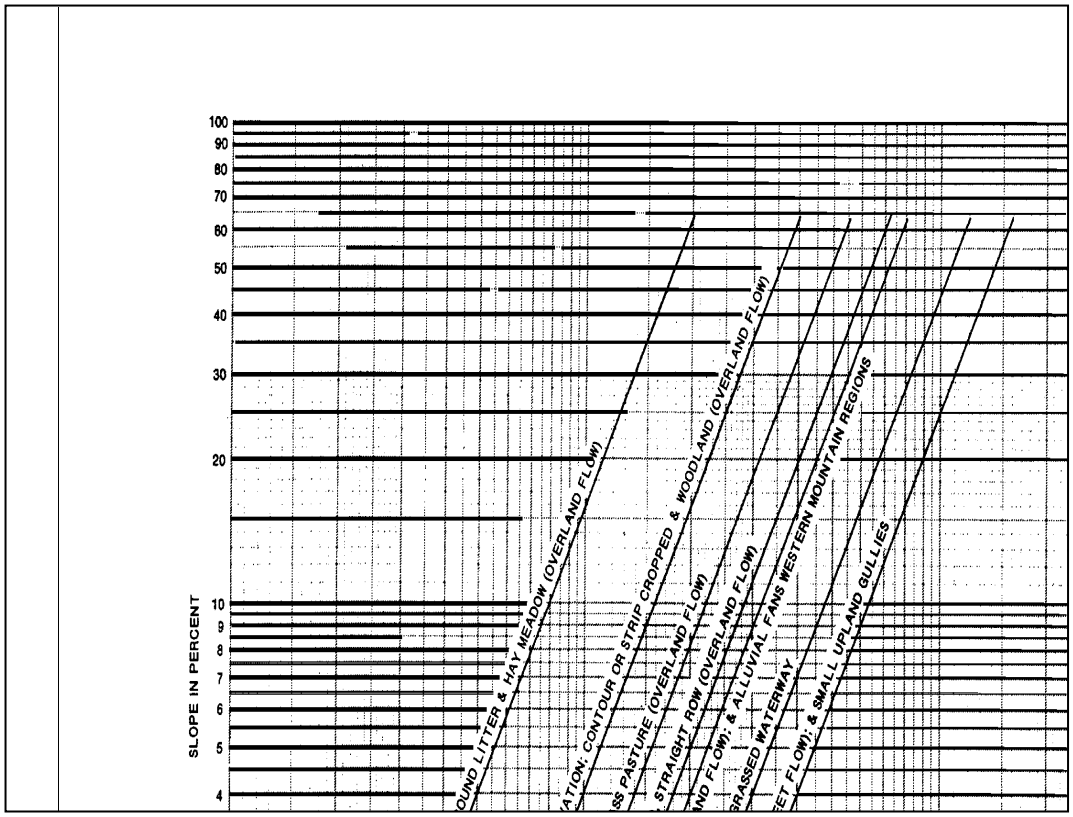
B. Overland Flow. One way to estimate the overland flow time is to use Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Table 10.2.2 to estimate overland flow velocity for a chosen path length. The path length divided by the velocity yields a travel time. A second method is to use the SCS time of concentration equation developed by Mockus in 1961. Equation 7.2 is intended to be the total travel time for the watershed and is considered a lumped parameter equation.

(Equation 7.2)

$$t_c = \frac{1.67(L)^{0.8}(1000/CN - 9)^{0.7}}{KS^{0.5}}$$

where: t_c = time of concentration, hr
L = longest hydraulic length from the point of interest to the top of the ridge, m (ft)
CN = SCS runoff curve number, as described in Section 7.4.A
K = 2949 (metric), 1900 (U.S. Customary)
S = average slope of the watershed in percent

Figure 7.4 Velocities for Upland Method of Estimating Time of Concentration, t_c
(Adapted from National Engineering Handbook, Volume 4 (NEH4), Figure 15.2)



C. Segmental Method.

1. Sheet-flow travel time is a shallow mass of runoff on a plane surface with the depth uniform across the sloping surface. Such flow occurs over relatively short distances, rarely more than ~46 meters (150 feet) (FHWA, 2002). Sheet flow rates are commonly estimated using a version of the kinematic wave equation.

(Equation 7.3)

$$t_t = \frac{K}{i^{0.4}}\left(\frac{nL}{\sqrt{s}}\right)^{0.6}$$

where: t_t = sheet flow travel time, min
K = 6.92 for Metric units or 0.933 for U.S. Customary Units
i = rainfall intensity, mm/hr (in/hr)
s = surface slope of the flow path, m/m, (ft/ft)
n = Manning's n-value (see Table 7.5 for example Manning's n-values)
L = longest hydraulic length, m (ft)

Since i depends on T_c and T_c is not initially know, the computation of T_c is an iterative process. An initial estimate of T_c is assumed and used to obtain i from the PDT-IDF curve for the site. The T_c is then computed from Equation 7.3 and used to check the initial value of T_c. If they are not the same, the process is repeated until two successive T_c estimates are the same.

Sheet flow can also be determined using Manning's kinematic solution specified in TR-55.

2. The velocity method can be used to estimate travel times for sheet flow, pipe flow, or channel flow. It is based on the travel time for a flow segment as a function of the length and the velocity.

(Equation 7.4)

$$T_t = \frac{L}{60V}$$

where, L and V have units of meters (feet) and meters/second (feet/second), respectively.

The time of concentration is then equal to the sum of the travel times.

(Equation 7.5)

$$t_c = \sum_{i=1}^{k} T_{ti} = \sum_{i=1}^{k} \left(\frac{L_i}{60V_i} \right)$$

For design conditions that do not involve complex drainage conditions, the designer may use Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Table 10.2.2 to estimate the velocities for the various segments.

For shallow concentrated flows - After short distances, sheet flow tends to concentrate in rills and gullies of increasing proportions. Such flow usually is referred to as a shallow concentrated flow. The velocity can be estimated with the figures described in this section, or using the relationship between velocity and slope as given below:

(Equation 7.6)

$$V = \alpha k S_p^{0.5}$$

where: V = velocity, m/s (ft/s)
α = 1.0 (metric), 3.28 (U.S. Customary)
k = intercept coefficient (see Table 7.6)
S_p = slope in percent of the channel or flow

D. Conduit Flow and Open Channel Flow. Pipe or open channel flow time can be estimated from the hydraulic properties of the conduit or channel. Generally it is reasonable to assume uniform flow and employ Manning's Equation with the following considerations. When assuming uniform flow conditions the engineer is assuming that the geometry and slope of the conduit and channel do not vary significantly and that the depth of flow does not vary within the conveyance system (conduit or open channel). If this is not the case, then the methods discussed in the Chapter 8, *Open Channels* and Chapter 9, *Culverts* should be used.

For open channel flow, consider the uniform flow velocity based on bank full flow conditions. That is, the main channel is flowing full without flow in the overbanks. This assumption avoids the significant iteration associated with other methods which employ rainfall intensity or discharges (since rainfall intensity and discharge are dependent on time of concentration).

For conduit flow, in a proposed storm drain system, compute the velocity at uniform depth based on the computed discharge at the upstream stream run. Otherwise, if the conduit is in existence, determine full capacity flow in the conduit and determine the velocity at capacity flow. It may be necessary to compare this velocity later with the velocity calculated during conduit analysis. If there is a significant difference and the conduit is a relatively large component of the total travel path, re-compute the time of concentration using the latter velocity estimate.

The coefficients listed are specified on the basis of engineering judgment and do not necessarily reflect the views of research institutions, manufacturers or other governmental agencies.

Table 7.5 Roughness Coefficients n-values for Manning's Equation (Pipes and Pavements)

Description	Manning's n-value
Polyvinyl Chloride (PVC) with smooth Inner Walls	0.010
Corrugated High-Density Polyethylene (HDPE) with Smooth Inner Walls	0.012
Corrugated High-Density Polyethylene (HDPE) with Corrugated Inner Walls	0.015
Concrete Pipe	0.012
Smooth-lined Corrugated Metal Pipe	0.012
Corrugated Plastic Pipe	0.024
Annular Corrugated Steel And Aluminum Alloy Pipe (Plain or polymer coated)	
68 mm × 13 mm (2 2/3 in × 1/2 in) Corrugations	0.024
75 mm × 25 mm (3 in × 1 in) Corrugations	0.027
125 mm × 25 mm (5 in × 1 in) Corrugations	0.025
150 mm × 50 mm (6 in × 2 in) Corrugations	0.033
Helically Corrugated Steel And Aluminum Alloy Pipe (Plain or polymer coated) 75 mm × 25 mm (3 in × 1 in), 125 mm × 25 mm (5 in × 1 in), or 150 mm × 50 mm (6 in × 2 in) Corrugations	0.024
Helically Corrugated Steel And Aluminum Alloy Pipe (Plain or polymer coated) 68 mm × 13 mm (2 2/3 in × 1/2 in) Corrugations	
a. Lower Coefficients*	
450 mm (18 in) Diameter	0.014
600 mm (24 in) Diameter	0.016
900 mm (36 in) Diameter	0.019
1200 mm (48 in) Diameter	0.020
1500 mm (60 in) Diameter or larger	0.021
b. Higher Coefficients**	0.024
Annular or Helically Corrugated Steel or Aluminum Alloy Pipe Arches or Other Non-Circular Metal Conduit (Plain or Polymer coated)	0.024
Vitrified Clay Pipe	0.012
Ductile Iron Pipe	0.013
Asphalt Pavement	0.015
Concrete Pavement	0.014
Grass Medians	0.050
Grass – Residential	0.030
Earth	0.020
Gravel	0.030
Rock	0.035
Cultivated Areas	0.030 - 0.050
Dense Brush	0.070 - 0.140
Heavy Timber (Little undergrowth)	0.100 - 0.150
Heavy Timber (with underbrush)	0.40
Streams:	
a. Some Grass And Weeds (Little or no brush)	0.030 - 0.035
b. Dense Growth of Weeds	0.035 - 0.050
c. Some Weeds (Heavy brush on banks)	0.050 - 0.070

Notes:

* Use the lower coefficient if any one of the following conditions apply:

a. A storm pipe longer than 20 diameters, which directly or indirectly connects to an inlet or manhole, located in swales adjacent to shoulders in cut areas, shoulders in cut areas or depressed medians.

b. A storm pipe which is specially designed to perform under pressure.

** Use the higher coefficient if any one of the following conditions apply:

a. A storm pipe which directly or indirectly connects to an inlet or manhole located in highway pavement sections or adjacent to curb or concrete median barrier.

b. A storm pipe which is shorter than 20 diameters long.

c. A storm pipe which is partly lined helically corrugated metal pipe.

In considering each factor more critical, judgment is necessary if it is kept in mind that any condition that causes turbulence and retards flow results in a greater Manning's n-value.

Outlet velocity for bituminous paved invert shall be determined based on a 25% reduction in Manning's roughness coefficient, Manning's n-value.

Table 7.6 Intercept Coefficients for Velocity versus Slope Relationship

Land cover (flow regime)	k
Forest with heavy ground litter; hay meadow (overland flow)	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)	0.152
Short grass pasture (overland flow)	0.213
Cultivated straight row (overland flow)	0.274
Nearly bare and untilled (overland flow)	0.305
Grassed waterway (shallow concentrated flow)	0.457
Unpaved (shallow concentrated flow)	0.491
Paved area; small upland gullies (shallow concentrated flow)	0.619

E. Other Methods for Estimating Travel Time. Other published methods may be used at the designer's discretion subject to the documented limitations of the methods.

F. Procedure to Estimate Time of Concentration

1. Divide the flow path into reach lengths along which flow conditions remain reasonably consistent. For natural drainage areas, the upper reaches usually will experience overland or sheet flow which transitions into shallow concentrated flow. If the area is large enough, the lower reaches will experience concentrated flow in swales, ditches, creeks and rivers.

2. For each identified reach length, make an estimate of the travel time and add the individual times to determine the total time. Use a method of estimating time that is appropriate for the flow conditions (see above). If the method results in a velocity, compute the time for component using Equation 7.7.

(Equation 7.7)

$$t_n = \frac{L_n}{60 v_n}$$

where: t_n = travel time over nth reach, min
L_n = length of nth reach along flow path, m (ft)
v_n = estimated flow velocity for nth reach, m/s (ft/s)

The total time is:

(Equation 7.8)

$$T = \sum_{n=1}^{m} t_n$$

where: T = total time along flow path, min
m = number of reaches in flow path
n = reach number

3. Choose an alternate flow path and repeat steps (1) and (2) as necessary.

4. Select the path which results in the longest time. This is the time of concentration (t_c), i.e., t_c = T (max).

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 was considered. In these cases, adjustments can be made to the drainage area and time of concentration by disregarding those areas where flow time is too long to add to the peak discharge. Sometimes it is necessary to estimate several different contributing areas and associated times of concentration to determine the design flow that is critical for a particular application.

When designing a drainage system, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. The land may be graded so that swales and streets will intercept the flow and change the time of concentration. Care should be exercised in selecting overland flow paths.

7.5 PEAK DISCHARGE USING THE RATIONAL METHOD

A. Introduction to the Rational Method. The Rational Method was first introduced in 1889. Though it often is considered simplistic, it still is deemed to be an appropriate method for estimating peak discharges for small drainage areas of up to 80 hectare (200 acres) in which there is no significant flood storage.

B. Assumptions and Applicability of the Rational Method. The Rational Method incorporates the following assumptions:

- The rate of runoff resulting from any constant rainfall intensity is at a maximum when the duration of rainfall is as long as or longer than the time of concentration. That is, if the rainfall intensity is constant, the entire drainage area contributes to the peak discharge when the time of concentration has elapsed. This assumption becomes less valid as the drainage area increases. 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. Additionally, rainfall intensities usually vary during a storm. In semi-arid and arid regions, storm cells are relatively small with extreme intensity variations.

- The frequency of peak discharge is the same as the frequency 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, mostly 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.

- The rainfall intensity is uniformly distributed over the entire drainage area. In reality, rainfall intensity varies spatially and temporally during a storm. For small areas, the assumption of uniform distribution is reasonable. However, as the drainage area increases, it becomes more likely that the rainfall intensity will vary significantly both in space and time.

- 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, accumulated volume of rainfall and antecedent moisture conditions. Thus, the consideration necessary for application of the Rational Method involves the selection of a coefficient that is appropriate for the storm magnitude, soil type, and land use conditions.

By limiting the application of the Rational Method to 80 hectares (200 acres), these assumptions are more likely to be reasonable.

Modern drainage practice often includes detention of urban storm runoff to reduce the peak rate of runoff downstream and to improve quality of the storm water runoff. The Rational Method severely limits the evaluation of design alternatives available in urban (and in some instances, rural) drainage design because of its inability to accommodate the presence of storage in the drainage area. When accommodation of any appreciable storage features in the drainage basin is required, runoff hydrograph methods should be employed.

C. The Rational Method 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 Equation 7.9.

(Equation 7.9)

$$Q = \frac{CC_f IA}{K}$$

where: Q = maximum rate of runoff, m^3/s (cfs)
C = runoff coefficient as outlined in Section 7.5.E.
C_f = runoff coefficient adjustment factor due to return interval
I = average rainfall intensity, mm/hr (in/hr) as outlined in Section 7.5.D.
A = drainage area, ha (ac)
K = 360 (metric), 1 (U.S. Customary)

D. Rainfall Intensity. The rainfall intensity (I) is the average rainfall rate in centimeters per hour (cm/hr) or inches per hour (in/hr) for a specific rainfall duration and a selected frequency. The duration is assumed to be equal to the time of concentration. For drainage areas in Pennsylvania, the rainfall intensity may be determined from the appropriate PDT-Intensity-Duration-Frequency, PDT-IDF, relationships found in Appendix 7A, *Field Manual for Pennsylvania Design Rainfall Intensity Charts From NOAA Atlas 14 Version 3 Data*. In the Appendix are maps of the different regions depending on the average recurrence interval (ARI) and storm duration. It was found that shorter duration storms had completely different orographic patterns than the longer duration storms, therefore the use of several maps.

Rainfall IDF curves for each of the regions are also shown in Appendix 7A, *Field Manual for Pennsylvania Design Rainfall Intensity Charts From NOAA Atlas 14 Version 3 Data*. As rainfall duration tends towards zero, the rainfall intensity increases. Since the rainfall intensity-duration relationship is calculated by assuming that the duration is equal to the time of concentration, small areas with exceedingly short times of concentration could result in design rainfall intensities that are unrealistically high. To minimize the likelihood that this will occur, the designer should use a minimum time of concentration of 5 minutes. As the duration tends to infinity, the design rainfall tends toward zero. Usually, the area limitation of 80 hectares (200 acres) should result in design rainfall intensities that are not unrealistically low. However, if the estimated time of concentration is extremely long, such as may occur in extremely flat areas, it may be necessary to consider an upper threshold of time (i.e., $t_c \leq 60$ minutes) or use a different hydrologic method.

There may be instances in which alternate methods of determining rainfall intensity are desired, especially for coordination with other agencies. The designer should take care to ensure that any alternate methods are applicable.

E. Runoff Coefficient. The selection of the runoff coefficient (C) tends to be somewhat subjective and varies with the topography, land use, vegetative cover, soil type, and moisture content of the soil at the time the rainfall-producing runoff occurs. In selecting the runoff coefficient, consideration should be given to the future characteristics of the watershed. If land use varies within a watershed, watershed segments must be considered individually and a weighted runoff coefficient, C_w, value can be calculated.

Table 7.7 lists suggested ranges of C values for various categories of ground cover. This table is typical of design guides found in civil engineering texts dealing with hydrology. The designer assigns a C value based on what is seen in the watershed with reference to the table.

An alternate, systematic approach for developing the runoff coefficient is shown in Table 7.7(a). In this table, which is applicable to rural watersheds only, the watershed is addressed as a series of aspects. For each of four aspects, a

systematic assignment of a runoff coefficient "component" is made. Using Equation 7.10, the four assigned components are then added together to form an overall runoff coefficient for the specific watershed segment.

(Equation 7.10)

$$C = C_r + C_i + C_v + C_s$$

Runoff coefficients, listed in Table 7.7 and Table 7.7(a), and others are applicable for storms of 2-year, 5-year, and 10-year return periods. Higher frequency storms will require modifying the runoff coefficient because infiltration and other abstractions have a proportionally smaller effect on runoff. The designer should adjust the runoff coefficient by the factor C_f as indicated in Table 7.8. Generally, the product of C and C_f should not exceed 1.0.

Table 7.7 Runoff Factors for the Rational Equation

TYPE OF DRAINAGE AREA OR SURFACE	RUNOFF FACTOR "C"	
	MINIMUM	MAXIMUM
Pavement, concrete or bituminous concrete	0.75	0.95
Pavement, bituminous macadam or surface-treated gravel	0.65	0.80
Pavement, gravel, macadam, etc.	0.25	0.60
Sandy soil, cultivated or light growth	0.15	0.30
Sandy soil, woods or heavy brush	0.15	0.30
Gravel, bare or light growth	0.20	0.40
Gravel, woods or heavy brush	0.15	0.35
Clay soil, bare or light growth	0.35	0.75
Clay soil, woods or heavy growth	0.25	0.60
City business sections	0.60	0.80
Dense residential sections	0.50	0.70
Suburban, normal residential areas	0.35	0.60
Rural areas, parks, golf courses	0.15	0.30

NOTES

1. Higher values are applicable to denser soils and steep slopes.
2. Consideration should be given to future land use changes in the drainage area in selecting the "C" factor.
3. For drainage area containing several different types of ground cover, a weighted value of "C" factor shall be used.
4. In special situations where sinkholes, stripped abandoned mines, etc. exist, careful evaluation shall be given to the selection of a suitable runoff factor with consideration given to possible reclamation of the land in the future.

Table 7.7(a) Runoff Coefficient for Rural Watersheds

	Slope			
	extreme	**high**	**normal**	**low**
Relief (C_r)	**0.28-0.35** steep, rugged terrain with average slopes above 30%	**0.20-0.28** hilly, with average slopes of 10-30%	**0.14-0.20** rolling, with average slopes of 5-10%	**0.08-0.14** relatively flat land, with average slopes of 0-5%
Soil Infiltration (C_i)	**0.12-0.16** no effective soil cover either rock or thin soil mangle of negligible infiltration capacity	**0.08-0.12** slow to take up water, clay or shallow loam soils of low infiltration capacity or poorly drained	**0.06-0.08** normal; well drained light or medium textured soils, sandy loams	**0.04-0.06** deep sand or other soil that takes up water readily, very light well drained soils
Vegetative Cover (C_v)	**0.12-0.16** no effective plant cover, bare or very sparse cover	**0.08-0.12** poor to fair; clean cultivation, crops or poor natural cover, less than 20% of drainage area over good cover	**0.06-0.08** fair to good; about 50% of area in good grassland or woodland, not more than 50% of area in cultivated crops	**0.04-0.06** good to excellent; about 90% of drainage area in good grassland, woodland, or equivalent cover
Surface (C_s)	**0.10-0.12** negligible; surface depression few and shallow; drainageways steep and small, no marshes	**0.08-0.10** well defined system of small drainageways; no ponds or marshes	**0.06-0.08** normal; considerable surface depression storage lakes and ponds and marshes	**0.04-0.06** much surface storage, drainage system not sharply defined; large flood plain storage or large number of ponds or marshes
The total runoff coefficient based on the four runoff components is: $C = C_r + C_i + C_v + C_s$				

Table 7.8 Runoff Coefficient Adjustment Factors for Rational Method

Recurrence Interval (years)	**C_f**
25	1.1
50	1.2
100	1.25

From *The Model Drainage Manual* (AASHTO, 2005)

The rational formula now becomes Equation 7.11:

(Equation 7.11)

$$Q = \frac{CC_f IA}{K}$$

F. Procedure for Rational Method. The general procedure for estimating the peak discharge for a watershed using the Rational Method is as follows:

1. Determine the watershed area in hectares (acres).

2. With consideration for future characteristics of the watershed, determine the time of concentration as defined in Section 7.4.

3. Assure consistency with the assumptions and limitations for application of the Rational Method.

4. According to the locality in Pennsylvania and the design frequency, extract the rainfall intensity from the appropriate IDF curve.

5. With consideration for future characteristics of the watershed, select or develop appropriate runoff coefficients for the watershed. Where the watershed comprises more than one characteristic, C values for each area segment must be estimated individually. A weighted C value then may be estimated using Equation 7.12.

(Equation 7.12)

$$C = \frac{\sum_{n=1}^{m} C_n A_n}{\sum_{n=1}^{m} A_n}$$

where: C = weighted runoff coefficient
n = nth sub-area
m = number of sub-areas
C_n = runoff coefficient for nth sub-area
A_n = nth sub-area size, ha (ac)

The runoff coefficient is dimensionless.

6. Using Equation 7.11, calculate the peak discharge for the watershed for the desired frequency. For frequencies of 2, 5, and 10 years, use $C_f = 1$. For frequencies 25-year and greater, use C_f from Table 7.8.

G. Example Problem - Rational Method.

1. Problem Statement. The following is an example problem which illustrates the application of the Rational Method to estimate peak discharges. Preliminary estimates of the maximum rate of runoff are needed at the inlet to a culvert for a 10-year and 100-year return period.

2. Site Data. From a topographic map and field survey, the area of the drainage basin upstream from the point in question is found to be 35 hectares (86.5 ac). In addition, the following data were measured:

- Length of sheet flow = 45 m (147.6 ft)
- Length of main basin channel = 700 m (2296 ft)
- Slope of channel = 0.018 m/m (ft/ft) = 1.8%
- Hydraulic radius = 0.6 m (1.97 ft)
- Average overland slope = 2.0%
- Estimated Roughness
- coeff. (n) of channel = 0.090

No shallow-concentrated flow.

3. Determine Runoff Coefficients and n-values

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

- Suburban (average conditions) 80%
- Rural, slope 2% 20%

For the residential single-family area:

- Using Table 7.7, the runoff coefficient was estimated for suburban land use:
  C = 0.45
- Using Table 7.5, the Manning's n-value for grass was estimated
  n = 0.30

For the rural area:

- Using Table 7.7(a), the runoff coefficient for average conditions was estimated assuming a slope of 0.5%:
  C = 0.225
- Using Table 7.5, the Manning's n-value was estimated for woods with light underbrush:
  n = 0.400

	1	2	3
Land Use	**Percent of Total Land Area**	**Runoff Coefficient**	**Product (Area * Runoff Coefficient)**
Residential single-family	80	0.45	36.0
Undeveloped	20	0.225	4.5
Total	100		40.5
Weight Runoff Coefficient			0.405

4. Determine Time-of-Concentration (T_c)

Sheet flow - Sheet flow occurs at the head of the basin for the first 150 feet in the residential single-family area.

Note: In the following computations, the i value in the kinematic wave equation is adjusted to the depth of water available as runoff (i.e. C times C_f times the intensity from the PDT-IDF curves). Since i depends on T_c and T_c is not initially know, the computation of T_c is an iterative process. An initial estimate of T_c is assumed and used to obtain i from the PDT-IDF curve for the site. The T_c is then computed from equation 7.3 and used to check the initial value of T_c. If they are not the same, the process is repeated until two successive T_c estimates are the same. For the purpose of this example, the first assumption will be assumed to be the correct one and the iterative process will not be performed.

10-year storm

$$Tc = \frac{6.92}{i^{0.4}}\left[\frac{nL}{\sqrt{S}}\right]^{0.6} = \frac{6.92}{(43.2*1.0*0.22)^{0.4}}\left[\frac{0.30*45}{\sqrt{0.02}}\right]^{0.6} = 43.4\,\text{minutes}$$

100-year storm

$$Tc = \frac{6.92}{i^{0.4}}\left[\frac{nL}{\sqrt{S}}\right]^{0.6} = \frac{6.92}{(71.5*1.25*0.22)^{0.4}}\left[\frac{0.30*45}{\sqrt{0.02}}\right]^{0.6} = 39.1\,\text{minutes}$$

Channel Flow

Velocity

$$V = \frac{1}{n}R^{2/3}S^{1/2} = \frac{1}{0.090}0.6^{2/3}0.018^{1/2} = 1.06\,\text{mps}$$

Travel time

$$Tch = \frac{L}{V * 60} = \frac{700}{1.06} = 11.0 \text{ minutes}$$

Total Time of Concentration (Ts + Tch)

10-year: 43.4 + 11.0 = 63.3 minutes (use 63)
100-year: 39.1 + 11.0 = 50.1 minutes (use 50)

5. Determine the rainfall region map. From Appendix 7A, Table 7A.1, determine what the appropriate Rainfall Region Map to be used based on Storm Duration and Frequency.

10-year: 63.3 minutes (use 63) - Map A
100-year: 50.1 minutes (use 50) - Map C

6. Determine the rainfall region. Locate the area of interest on the Pennsylvania Map on the Map that was determined in step 1 (Appendix 7A, Map A – Figure 7A.1, Map C – Figure 7A.3)

10-year: 43.4 + 11.0 = 63.3 minutes (use 63) - Map A – Region 3
100-year: 39.1 + 11.0 = 50.1 minutes (use 50) - Map C – Region 3

7. Determine Intensity (i)

A. From the PDT-IDF Curves, determine the Rainfall Intensity in Metric for Region 3 (Appendix 7A, Figure 7A.11(a)).

10 - year = Duration = 63 mins. Region 3 – i = 4.45 cm/hr = 44.5 mm/hr
100- year = Duration = 50 mins. Region 3 – i = 7.37 cm/hr = 73.7 mm/hr

B. From the PDT-IDF Curves, determine the Rainfall Intensity in U.S. Customary (Appendix 7A, Figure 7.11(b)).

10 - year = Duration = 63 mins. Region 3 – i = 1.75 in/hr
100 - year = Duration = 50 mins. Region 3 – i = 2.9 in/hr

8. Determine the Peak Runoff (Q)

A. Determine Peak Runoff in Metric.

10-year

$$\text{i(63 minutes)} = 44.5 \text{ mm / hr}$$

$$Q = \frac{CiA}{360} = \frac{0.29 * 44.5 * 35}{360} = 1.25 \frac{m^3}{s}$$

100-year

$$\text{i(50 minutes)} = 73.7 \text{ mm / h}$$

$$Q = \frac{C_f CiA}{360} = \frac{1.25 * 0.29 * 73.7 * 35}{360} = 2.60 \frac{m^3}{s}$$

B. Determine Peak Runoff in U.S. Customary.

10-year

$$\text{i (63 minutes)} = 1.75 \text{ in / h}$$

$$Q = CiA = 0.29 * 1.75(in/hr) * 86.5(acres) = 43.90 cfs$$

100-year

$$\text{i (50 minutes)} = 2.9 \text{ in / h}$$

$$Q = C_f CiA = 1.25 * 0.29 * 2.9(in/hr) * 86.5(acres) = 90.93 cfs$$

7.6 NRCS RUNOFF CURVE NUMBER METHODS

A. Introduction to NRCS Methods. The Department uses several runoff determination techniques. A graphical peak discharge method and the dimensionless unit hydrograph method were developed by the U.S. Department of Agriculture, Natural Resource Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS). TR-55 and EFH-2 also are used by the Department. The graphical peak discharge method and the dimensionless unit hydrograph method will be discussed here.

The techniques require basic data similar to that used in the Rational Method. However, the NRCS approach is more sophisticated in that it considers the time distribution of rainfall, initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. The NRCS methods produce the direct runoff for a storm, either real or fabricated, by subtracting infiltration and other losses from the total rainfall using a method sometimes termed the Runoff Curve Number Method.

The primary input variables for the NRCS methods are:

- Drainage area size (A) in square kilometers (square miles).
- Time of concentration (t_c) in hours.
- Weighted runoff curve number (RCN).
- Rainfall distribution PDT-IDF Curves.
- Total design rainfall (P) in millimeters (in).

The procedures presented here are derived from the *National Engineering Handbook,* Section 4 (NRCS, 1985) and TR-55, *Urban Hydrology for Small Watersheds* (NRCS, 1986).

B. NRCS Rainfall-Runoff Equation. Equation 7.13 represents a relationship between accumulated rainfall and accumulated runoff. This was derived by the NRCS from experimental plots for numerous soils and vegetative cover conditions. Data for land treatment measures, such as contouring and terracing, from experimental watersheds were included.

(Equation 7.13)

$$R = \frac{(P - I_a)^2}{(P - I_a) + S}$$

where: R = accumulated direct runoff, mm (in)
P = accumulated rainfall (potential maximum runoff), mm (in)
I_a = initial abstraction including surface storage, interception, and infiltration prior to runoff, mm (in)
S = potential maximum retention, mm (in)

The potential maximum retention (S) may be computed as the following:

(Equation 7.14)

Metric:

$$S = \frac{25400}{RCN} - 254$$

U.S. Customary:

$$S = \frac{1000}{RCN} - 10$$

where: RCN = Runoff Curve Number as described in Section 7.6.F.

Equation 7.14 is valid if S < (P - R). Equation 7.14 was developed mainly for small watersheds from recorded storm data that included total rainfall amount in a calendar day, but not its distribution with respect to time. Therefore, this method is appropriate for estimating direct runoff from 24-hour or 1-day storm rainfall. Generally, I_a may be estimated as the following:

(Equation 7.15)

$$I_a = 0.2S$$

which, when substituted in Equation 7.14, gives

(Equation 7.16)

$$R = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

C. Accumulated Rainfall (P). For PennDOT highway drainage design purposes, the accumulated rainfall may be developed from the PDT-IDF Curves for the various regions in the State.

D. Rainfall Distribution. The NRCS Type II Rainfall Distribution is commonly used for design studies (TR-55).

E. Soil Groups. Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The NRCS divides soils into four hydrologic soil groups based on infiltration rates: Groups A, B, C, and D.

1. Group A - Soils having a low runoff potential due to high infiltration rates even when saturated, 8.6 to 11.4 mm/hr (0.34 to 0.45 in/hr). These soils consist primarily of deep sands, deep loess, and aggregated silts.

2. Group B - Soils having a moderately low runoff potential due to moderate infiltration rates when saturated, 3.8 mm/hr to 8.6 mm/hr (0.15 to 0.34 in/hr). These soils consist primarily of moderately deep to deep moderately well to well drained soils with moderately fine to moderately coarse textures (shallow loess, sandy loam).

3. Group C - Soils having a moderately high runoff potential due to slow infiltration rates, 1.3 mm/hr to 3.8 mm/hr (0.052 to 0.15 in/hr if saturated). These soils consist primarily of soils in which a layer near the surface impedes the downward movement of water or soils with moderately fine to fine texture (clay loams, shallow sandy loams, soils low in organic content, and soils usually high in clay).

4. Group D - Soils having a high runoff potential due to very slow infiltration rates 1.3 mm/hr (0.052 in/hr). These soils consist primarily of clays with the following characteristics:

- High swelling potential.
- Soils with permanently high water tables.
- Soils with a claypan or clay layer at or near the surface.
- Shallow soils over nearly impervious parent material (soils that swell significantly when wet, heavy plastic clay, and certain saline soils).

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected.

F. Runoff Curve Number (RCN). Rainfall infiltration losses primarily are dependent on soil characteristics and land use (surface cover). The NRCS method uses a combination of soil conditions and land use to assign runoff factors known as runoff curve numbers. These represent the runoff potential of an area when the soil is not frozen. The higher the RCN, the higher the runoff potential. Tables 7.9 through 7.12 provide an extensive list of suggested Runoff Curve Numbers. The RCN values assume medial antecedent moisture condition.

(Equation 7.17)

$$RCN(I) = \frac{4.2RCN(II)}{10 - 0.058RCN(II)}$$

(Equation 7.18)

$$RCN(III) = \frac{23RCN(II)}{10 + 0.13RCN(II)}$$

If necessary, the designer should adjust the RCN for wet or dry antecedent moisture conditions. A 5-day period is used as the minimum for estimating antecedent moisture conditions. Antecedent soil moisture conditions also vary during a storm; heavy rain falling on a dry soil can change the soil moisture condition from dry to average to wet during the storm period. Equation 7.17 adjusts values for expected dry soil conditions (antecedent moisture

condition I). Use Equation 7.18 to accommodate wet soils (antecedent moisture condition III). Table 7.13 assists the determination of which moisture condition applies.

Table 7.9 Runoff Curve Numbers for Urban Areas

Cover type and hydrologic condition	Average percent impervious area	A	B	C	D
Fully developed urban areas (vegetation established)					
open space (lawns, parks, golf courses, cemeteries,etc.) Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover 50% to 75%)		39	61	74	80
Impervious areas: Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads: Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right of way)		76	85	89	91
Dirt (including right of way)		72	82	87	89
Western desert urban areas: Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

- Values in table are for average runoff condition and $I_a = 0.2S$.
- The average percent impervious area shown was used to develop the composite RCN's. Other assumptions are: (1) impervious areas are directly connected to the drainage system, (2) impervious areas have a RCN of 98, and (3) pervious areas are considered equivalent to open space in good hydrologic condition.

Table 7.10 Runoff Curve Numbers for Cultivated Agricultural Land

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

- Values in table are for average runoff condition and $I_a = 0.2S$.
- Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
- Hydrologic condition is based on a combination of factors that affect infiltration and runoff, including (1) density and canopy of vegetative areas, (2) amount of year-round cover, (3) amount of grass or closed-seeded legumes in rotation, (4) percent of residue cover on land surface (good > 20%), and (5) degree of roughness.
- Poor: Factors impair infiltration and tend to increase runoff.
- Good: Factors encourage average and better-than-average infiltration and tend to decrease runoff.

Table 7.11 Runoff Curve Numbers for Other Agricultural Lands

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

- Values in table are for average runoff condition and $I_a = 0.2S$.
- Pasture: Poor: < 50% ground cover or heavily grazed with no mulch
  Fair: 50 to 75% ground cover and not heavily grazed
  Good: > 75% ground cover and lightly or only occasionally grazed
- Meadow: Poor: < 50% ground cover
  Fair: 50 to 75% ground cover
  Good: >75% ground cover
- Woods/grass: RCNs shown were computed for areas with 50% grass (pasture cover). Other combinations of conditions may be computed from RCNs for woods and pasture.
- Woods: Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning
  Fair: Woods grazed but not burned, and some forest litter covers the soil
  Good: Woods protected from grazing, litter and brush adequately cover soil

Table 7.12 Runoff Curve Numbers for Arid and Semi-Arid Rangelands

Cover type	Hydrologic condition	A	B	C	D
Herbaceous -- mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen -- mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper -- pinyon, juniper, or both; grass understory					
	Poor		75	85	89
	Fair		58	73	80
Sagebrush with grass understory	Good		41	61	71
	Poor		67	80	85
	Fair		51	63	70
Desert shrub -- major plants include saltbush, greasewood, creosote-bush, blackbrush, brusage, palo verde, mesquite, and cactus	Good		35	47	55
	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

- Values in table are for average runoff condition and $I_a = 0.2S$.
- Hydrologic Condition: Poor: < 30% ground cover (litter, grass, and brush overstory)
  Fair: 30% to 70% ground cover
  Good: > 70% ground cover
- Curve numbers for group A have been developed only for desert shrub

Table 7.13 Rainfall Groups for Antecedent Soil Moisture Conditions During Growing and Dormant Seasons

Antecedent Condition	Conditions Description	Growing Season five-day Antecedent Rainfall	Dormant Season five-day Antecedent Rainfall
Dry AMC I	An optimum condition of watershed soils, where soils are dry but not to the wilting point, and when satisfactory plowing or cultivation takes place.	Less than 1.39 in (35 mm)	Less than 0.48 in (12 mm)
Average AMC II	The average case for annual floods	1.39 to 2.18 in (35 to 55 mm)	0.48 to 1.11 in (12 to 28 mm)
Wet AMC III	When a heavy rainfall, or light rainfall and low temperatures, have occurred during the five-days previous to a given storm	Over 2.18 in (55 mm)	Over 1.11 in (28 mm)

G. Procedure for NRCS Graphical Peak Discharge (TR-55). This method of peak discharge determination can be used for relatively homogeneous watersheds with a maximum time of concentration of 10 hours (600 minutes). In a similar fashion to the Rational Method, if soils and land use vary, the watershed should be subdivided. Precipitation amounts and distribution can be taken from the PDT-IDF curves for the appropriate regions. The method should not be used for runoff amounts of less than 38 mm (1.50 in) and runoff curve numbers of less than 60. Additionally, the range of curve numbers should be small (say 20%) to reasonably conform with the assumption of homogeneity. The following steps outline the method.

1. Determine the drainage area (A).

2. Determine the soil classification based on runoff potential (Type A, B, C, or D) as described in Section 7.5.E. One approach for a general classification is to determine the soil name and type from NRCS soil maps or reports.

3. Determine the antecedent soil moisture conditions (AMC) as discussed in Section 7.5.F.

4. Classify the hydrologic condition of the soil cover (good, fair or poor). Refer to the footnotes on Tables 7.9 through 7.12.

5. Use Tables 7.9 through 7.13 to determine the Runoff Curve Number (RCN) for the particular soil classification for an AMC II. If appropriate, adjust for AMC I or AMC III using Equations 7.17 and 7.18, respectively. If necessary, determine a weighted value by dividing the sum of the products of the sub-area sizes and RCNs by the total area. This process is similar to the weighting of runoff coefficients in the Rational Method. However, the runoff factors are not directly related.

6. Estimate the watershed time of concentration in hours (t_c).

7. Determine the potential maximum storage (S) using Equation 7.14.

8. Determine the initial abstraction (I_a) using Equation 7.15. These are the losses that occur before runoff begins and include depression storage, interception and infiltration. If I_a is greater than P, it is possible that the rainfall event would not produce runoff (which would be unusual for design frequencies). The abstraction equation may need modification or an alternate means of estimating this value may be necessary, although no specific research has been performed to determine such adjustments.

9. Based on the design frequency and a 24-hour duration storm, determine the total rainfall (P) for the location of the watershed.

10. Determine the accumulated direct runoff (R) using Equation 7.16. This value, when multiplied by the watershed area, will indicate the total volume of the rainfall that appears as runoff.

11. Refer to Equation 7.19 and Table 7.15 with the relevant distribution type from Step 9 to determine the unit peak discharge (q_u) using time of concentration (t_c) and the ratio I_a/P.

(Equation 7.19)

$$q_u = \left(10^{C_0 - 3.36609}\right) \times \left(t_c^{C_1 + C_2 \log T_c}\right)$$

where: q_u = unit peak discharge, $m^3/s/km^2/mm$ (cfs/mi^2/in)
t_c = time of concentration, hr
C_0, C_1, C_2 = coefficients determined from Table 7.14

Table 7.14 Coefficients for Equation 7.19

Rainfall Type	I_a/P	C0	C1	C2
II	0.1	2.5532	-0.6151	-0.164
	0.3	2.4653	-0.6226	-0.1166
	0.35	2.419	-0.6159	-0.0882
	0.4	2.3641	-0.5986	-0.0562
	0.45	2.2924	-0.5701	-0.0228
	0.5	2.2028	-0.516	-0.0126
III	0.1	2.4732	-0.5185	-0.1708
	0.3	2.3963	-0.512	-0.1325
	0.35	2.3548	-0.4974	-0.1199
	0.4	2.3073	-0.4654	-0.1109
	0.45	2.2488	-0.4131	-0.1151
	0.5	2.1777	-0.368	-0.0953

12. With reference to Table 7.15, determine the pond adjustment factor (F). This adjustment is to account for pond or swamp areas within the watershed which do not interfere with the time of concentration flow path.

NOTE: This factor is not intended to replace a hydrograph routing technique where considerable detention storage is present (typically, with surface area of ponding in excess of 5% of the watershed area).

13. Compute the peak discharge (Q) from Equation 7.20.

(Equation 7.20)

$$Q = q_u ARF$$

where: Q = peak discharge, m^3/s (cfs)
q_u = unit peak discharge, $m^3/s/km^2/mm$ ($cfs/mi^2/in$) from Step 11
A = drainage area, km^2 (mi^2) from Step 1
R = runoff volume, mm (in) from Step 10
F = ponding factor from Step 12

Table 7.15 Ponding Adjustment Factor

% Ponded/swamp area	Factor (F)
0	1
0.2	0.97
1	0.87
3	0.75
5	0.72

14. Repeat Steps 10 through 13 for other desired frequencies.

A detailed description of this method appears in *Urban Hydrology for Small Watersheds* (NRCS, 1986).

H. NRCS (SCS) Dimensionless Unit Hydrograph. In many instances, for highway drainage design, peak discharge methods will suffice for runoff estimation. However, the estimation of runoff hydrographs may be necessary for situations such as detention pond design, reservoir routing, or channel routing, especially for larger areas and those in which watershed conditions cannot be considered homogeneous. Many hydrograph methods are

available and not specifically excluded for use by the Department. However, the NRCS Dimensionless Hydrograph Method is incorporated here due to its relative ease of use as well as the relative ease with which variables can be determined.

A unit hydrograph represents the time distribution of flow resulting from one millimeter of direct runoff occurring over the watershed in a specified time. The NRCS dimensionless unit hydrograph is plotted in terms of the ratio of time over time to peak. A curvilinear dimensionless unit hydrograph is shown in Figure 7.5. For hand computations, a triangular hydrograph is considered reasonable, as shown in Figure 7.6.

The triangular hydrograph is a practical representation of excess runoff with only one rise, one peak, and one recession. Its geometric makeup can be easily described mathematically, which makes it very useful in the processes of estimating discharge rates. The NRCS developed Equation 7.21 to estimate the peak rate of discharge for an increment of runoff.

Figure 7.5 NRCS Dimensionless Curvilinear Unit Hydrograph

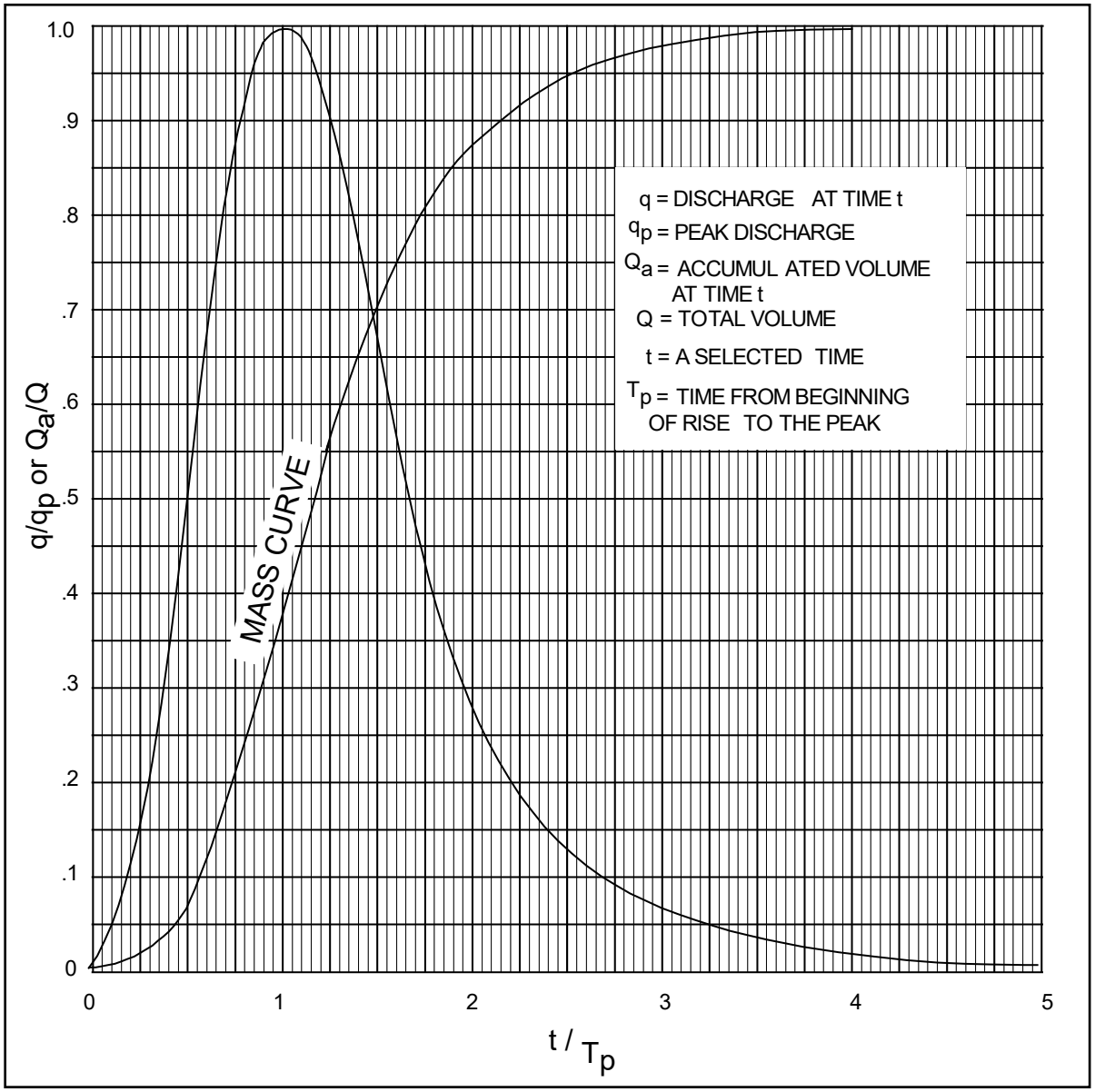
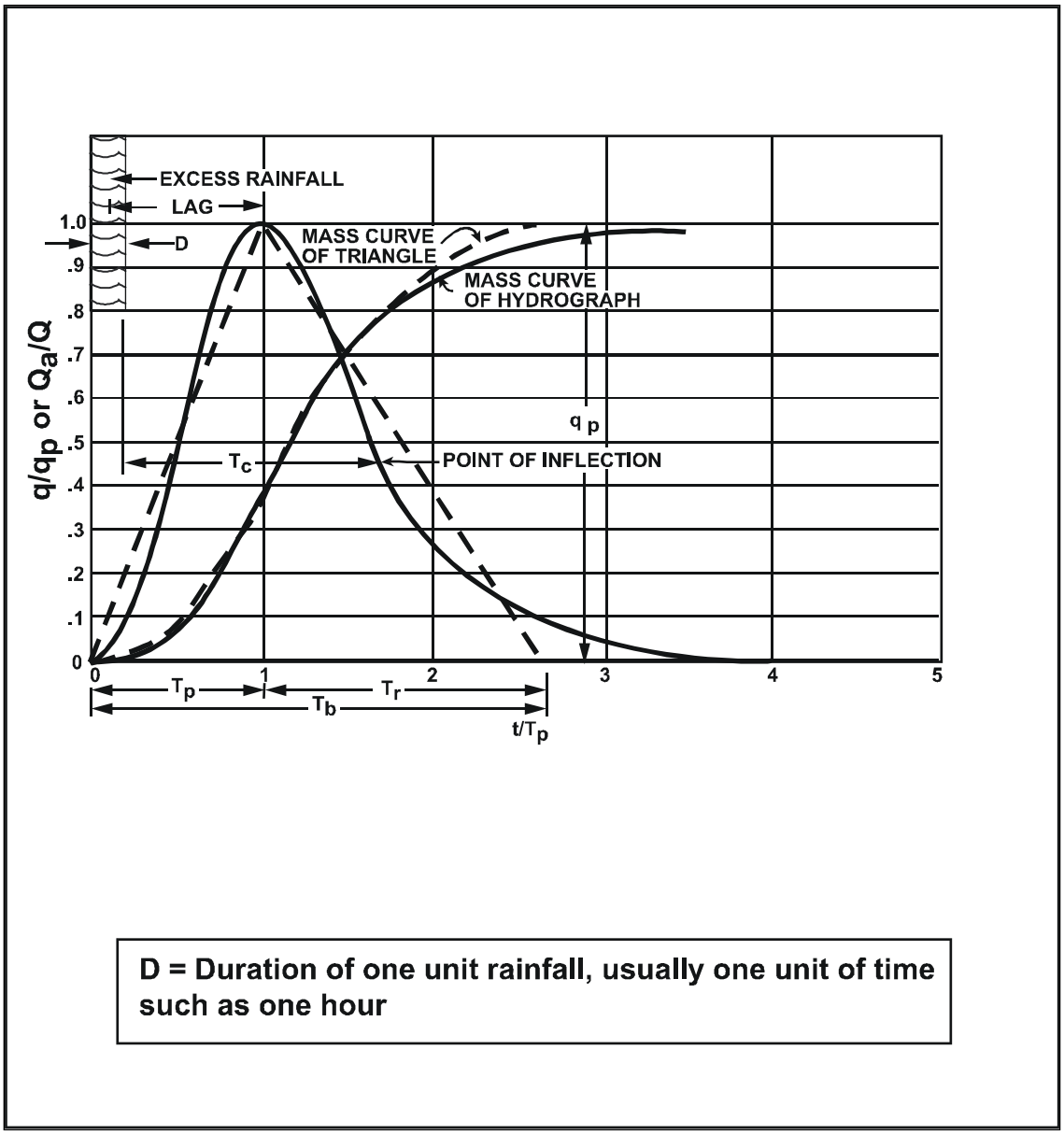


Figure 7.6 Triangular Unit Hydrograph



(Equation 7.21)

$$q_p = \frac{KAq}{T_p}$$

where: q_p = peak rate of discharge, m^3/s (cfs)
A = area, km^2 (mi^2)
q = depth of storm runoff during time interval = 1 mm (1 in) for unit hydrograph
T_p = time to peak runoff (hours), which is estimated using Equations 7.22 and 7.23
K = 0.208 for metric units and 484 for U.S. Customary Units

(Equation 7.22)

$$T_{lag} = 0.6T_c$$

(Equation 7.23)

$$T_p = \frac{d}{2} + T_{lag}$$

where: t_c = time of concentration, hr
d = duration of unit excess rainfall, hr

Equation 7.24 provides an estimate of the duration of unit excess rainfall (d).

(Equation 7.24)

$$d = 0.133\ t_c$$

Equation 7.21 can be used to estimate the peak discharge for the unit hydrograph. The shape of the unit hydrograph then can be derived with reference to Figure 7.5. The constant 0.208, or peak rate factor, is valid for the NRCS dimensionless unit hydrograph. Any change in the dimensionless unit hydrograph reflecting a change in the percent of volume under the rising side would cause a corresponding change in the shape factor associated with the triangular hydrograph and therefore a change in the constant 0.208.

More detail on the NRCS dimensionless hydrograph method is provided in the *National Engineering Handbook*, Volume 4 (NRCS, 1985).

Dimensionless unit hydrograph characteristics vary with the size, shape, and slope of the tributary drainage area. The most significant characteristics affecting the dimensionless hydrograph shape are the basin lag and the peak discharge (q_p) for a given rainfall (see Figure 7.6). Basin lag is the time from the center of mass of rainfall excess to the hydrograph peak.

Steep slopes and compact shapes tend to make lag time short and peaks high.

Flat slopes and elongated shapes tend to make lag time long and peaks low.

I. Procedure for Determining Flood Hydrographs Using the NRCS Dimensionless Unit Hydrograph. The following outline is for design discharges and assumes the area or sub-area is reasonably homogeneous (i.e., the unit hydrograph parameters, hydrologic abstractions, and the rainfall do not vary spatially and are considered constant). That is, the designer has subdivided the watershed into homogeneous areas where these values can be assumed constant. The procedure results in a hydrograph from the direct uncontrolled area. If the watershed has been subdivided, it might be necessary to perform hydrograph channel routing, storage routing, and hydrograph superposition to determine the hydrograph at the outlet of the watershed. Storage routing and channel routing procedures are presented in Sections 7.8 and 7.9.

1. Determine the following parameters:

a. Drainage area or sub-area size, A (km^2 or mi^2)

b. Time of concentration, t_c (hrs) as discussed in Section 7.5.F.

c. Weighted runoff curve number (RCN) as discussed in Section 7.5.F.

d. Total rainfall, P (mm or in), for design and check flood frequencies. Use the 24-hour precipitation.

2. Determine the unit hydrograph variables.

a. Duration of excess rainfall (runoff), d, using Equation 7.23. For convenience, d may be rounded such that the actual duration of precipitation is a whole number times d. For example, if d calculates to be 0.332 hours, for 24-hour precipitation, then 24/0.332 = 72.29; use 72 in which case, d = 24/72 = 0.333 hours (20 mins).

b. Time to peak of unit hydrograph (UH), T_p, using Equation 7.22.

c. Compute the peak runoff ordinate, q_p, for the UH, using Equation 7.21 and q = 1 mm or 1 in.

d. Using the dimensionless hydrograph (curvilinear, as appears in Figure 7.5, or triangular, as appears in Figure 7.6), develop a table of the unit hydrograph ordinates using time step increments d. That is, at each time n x d, where n is the time step, determine the time ratio (t/T_p).

(Equation 7.25)

$$\left(\frac{t}{T_p}\right) = \frac{nd}{T_p}$$

e. Find the discharge ratio (q/Q_p) at this time ratio and calculate the discharge at this time step as the following equation:

(Equation 7.26)

$$q = q_p\left(\frac{q}{Q_p}\right)$$

f. Use the following to check the area under the resulting hydrograph:

(Equation 7.27)

$$\frac{3.6d \times \sum(hydrograph\ \ ordinates)}{A}$$

where A = area of the watershed, km^2 (mi^2)

The result should be 1.0, reflecting the 1 mm (1 in) of runoff from the entire drainage area. Rounding of the unit duration, d, and the likelihood that T_p will not be an integer multiple of d will often result in an area slightly higher or lower than 1. If so, adjust all the ordinates proportionally until the resulting area is 1.

3. Develop a runoff (excess rainfall) table.

a. Develop a table of accumulated rainfall, P, for the appropriate distribution type and using a time increment of d hours: for each time step (n x d hours), determine the fraction of total rainfall and multiply it by the 24-hour precipitation. (A plot of the resulting values represents a rainfall hyetograph.)

b. On the same table, calculate the accumulated runoff, R, using the estimated RCN and Equations 7.22 and 7.24. If, for any time interval, $P - 0.2S < 0$, then R = 0.

c. Calculate the increased runoff for each time step as the difference between the current accumulated runoff and the accumulated runoff from the previous time step.

4. Compute the hydrographs resulting from each increment of runoff by multiplying the ordinates of the unit hydrograph by the increment of runoff using the same time step, d. This will result in as many hydrographs as there are increments of runoff, each of which should be displaced by the duration time (d) from the previous hydrograph.

5. At each time step, sum all the runoff values to yield the composite runoff hydrograph. This process step is often termed convolution. This is the resulting hydrograph for the watershed or sub-area. It may serve as an inflow hydrograph for channel or storage routing procedures.

Other superimposition or tabular methods may be used for the convolution process. However, it is expected that computer spreadsheets or programs will be used for large computations and the basic theory remains the same.

For complex watersheds, it often is necessary to subdivide the area, develop runoff hydrographs for each sub-area, and perform combinations of flood routing and channel routing. If the appraisal of the effect of storage is required, such as for detention pond design, the resulting hydrographs may be applied to flood-routing techniques such as appear in Section 7.8. Channel routing is discussed in Section 7.9.

7.7 DESIGN RAINFALL HYETOGRAPH METHODS

A. Use of the Rainfall Hyetograph. A rainfall hyetograph is a graphical representation of the variation of rainfall depth or intensity with time. Rainfall-runoff hydrograph methods require a description of this variation. It is possible to use actual rain gauge data in rainfall-runoff models if the data are recorded using a small enough time period (such as 15-minute increments). Oftentimes, such data are not readily available.

For design, the use of a single measured rainfall event without consideration of other events is not practical since storms vary considerably from event to event and no probability of occurrence would have been established. Two methods discussed here, the NRCS 24-hour Rainfall Distributions and the Balanced Storm Method, are statistically based and appropriate for design.

B. NRCS 24-Hour Rainfall Distributions. The NRCS 24-hour distributions are suggested for the design of highway stream crossings (culverts and bridges) where the designer considers it appropriate to employ runoff hydrograph methods. The distributions (Types II and III for Pennsylvania) are presented in Sections 7.8.D. and 7.8.C., respectively. A tabular representation of Types II and III appears in Table 7.16.

Table 7.16 represents the cumulative fraction of rainfall with respect to time. The following steps outline the procedure to develop the rainfall hyetograph from the cumulative fraction data.

1. Determine the 24-hour rainfall (P24) for the desired frequency.

2. Establish a time increment (it is useful to use the same time increment as that to be used for hydrograph generation).

3. Establish a table of time and the fraction of rainfall to total 24-hour rainfall by interpolation of Table 7.16 or Figure 7.7 for the appropriate distribution type.

4. Multiply the cumulative fractions by the total 24-hour rainfall to get the cumulative rainfall.

5. Determine the incremental rainfall for each time period by subtracting the cumulative rainfall at the previous time step from the current time step.

A plot of the resulting incremental rainfall versus times represents the rainfall hyetograph.

Table 7.16 NRCS 24-Hour Rainfall Distributions

Time (t) hours	Fraction of 24-hour rainfall	
	Type II	Type III
0	0.000	0.000
2	0.022	0.020
4	0.048	0.043
6	0.080	0.072
7	0.098	0.089
8	0.120	0.115
8.5	0.133	0.130
9	0.147	0.148
9.5	0.163	0.167
9.75	0.172	0.178
10	0.181	0.189
10.5	0.204	0.216
11	0.235	0.250
11.5	0.283	0.298
11.75	0.357	0.339
12	0.663	0.500
12.5	0.735	0.702
13	0.772	0.751
13.5	0.799	0.785
14	0.820	0.811
16	0.880	0.886
20	0.952	0.957
24	1.000	1.000

C. Example of Hyetograph Developed from NRCS 24-Hour Rainfall Distributions (Metric Units). The following is an example of a rainfall hyetograph for a 25-year, 24-hour storm, a 1-hour time increment is used.

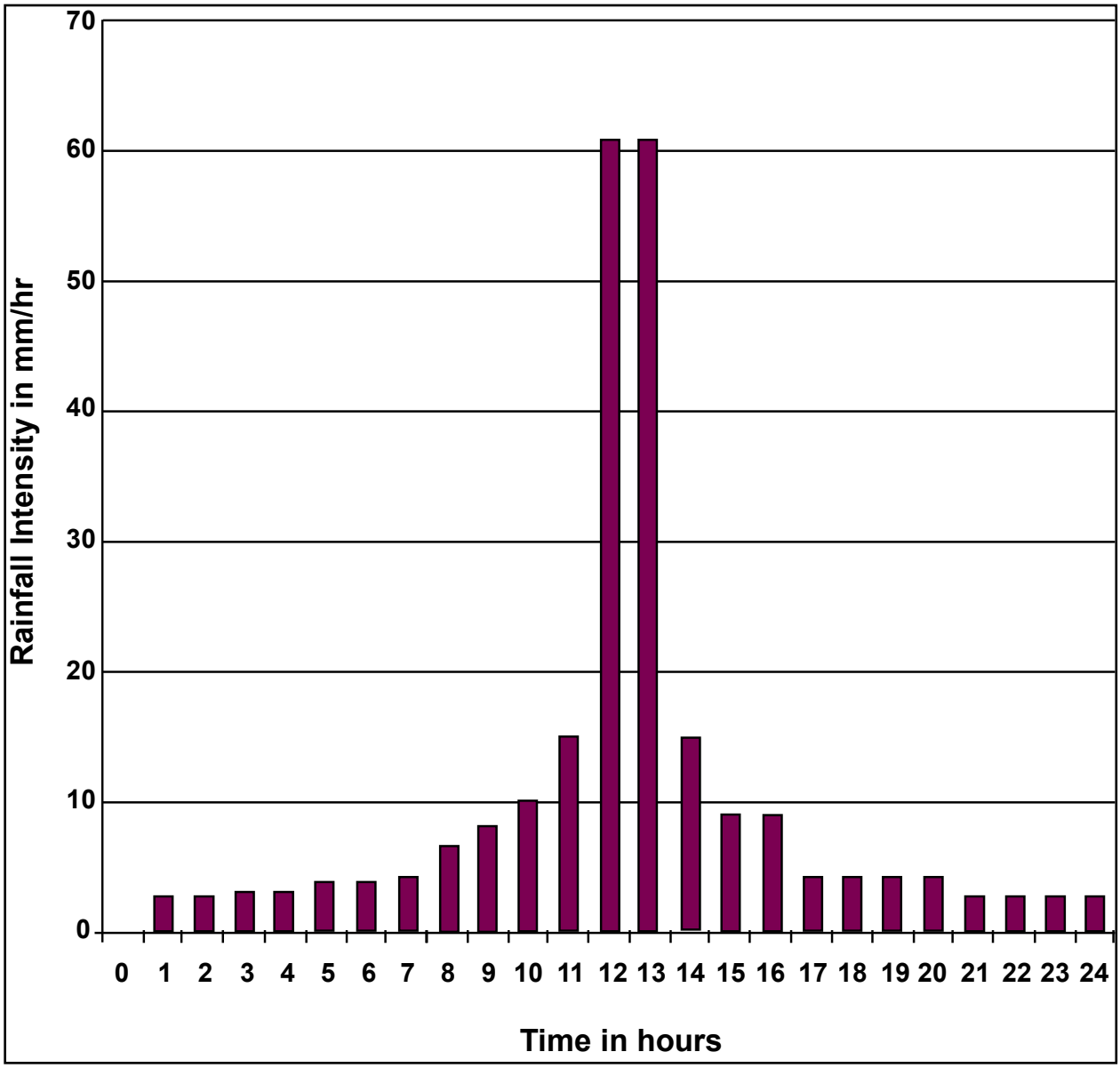
Total precipitation = 244 mm
Distribution type = Type III

Table 7.17 presents the calculations. Figure 7.7 shows the resulting hyetograph.

Table 7.17 Example of Incremental Rainfall Tabulation (Metric)

Time (hours)	Cum. Fraction (Pt/P24)	Cum. Rain Pt (mm)	Incr. Rain (mm)
0	0.00	0.00	0.00
1	0.01	2.44	2.44
2	0.02	4.88	2.44
3	0.0315	7.69	2.81
4	0.043	10.49	2.81
5	0.575	14.03	3.54
6	0.072	17.57	3.54
7	0.089	21.72	4.15
8	0.115	28.06	6.34
9	0.148	36.11	8.05
10	0.189	46.12	10.00
11	0.25	61.00	14.88
12	0.5	122.00	61.00
13	0.751	183.24	61.24
14	0.811	197.88	14.64
15	0.8485	207.03	9.15
16	0.886	216.18	9.15
17	0.90375	220.52	4.33
18	0.9215	224.85	4.33
19	0.93925	229.18	4.33
20	0.957	233.51	4.33
21	0.96775	236.13	2.62
22	0.9785	238.75	2.62
23	0.98925	241.38	2.62
24	1	244.00	2.62

Figure 7.7 Example of Rainfall Hyetograph (Metric)



For time = 1 hour,

- The cumulative fraction is determined by interpolation of Table 7.16:
  $P_1/P_{24} = 0 + (1 - 0)/(2 - 0) \times (0.02 - 0) = 0.01$.
- The cumulative rainfall is the product of the cumulative fraction and the total 24-hour rainfall:
  $P_1 = 0.01 \times 244 = 2.44$ mm.
- The incremental rainfall is the difference between the current and preceding cumulative rainfall values:
  2.44 - 0 = 2.44 mm.

Repeating the procedure for each time period yields the complete hyetograph ordinates.

D. Example of Hyetograph Developed from NRCS 24-Hour Rainfall Distributions (U.S. Customary Units). The following is an example of a rainfall hyetograph for a 100-year, 24-hour storm, a 1-hour time increment is used.

Total precipitation = 4.60 in
Distribution type = Type II

Table 7.18 presents the calculations. Figure 7.8 shows the resulting hyetograph.

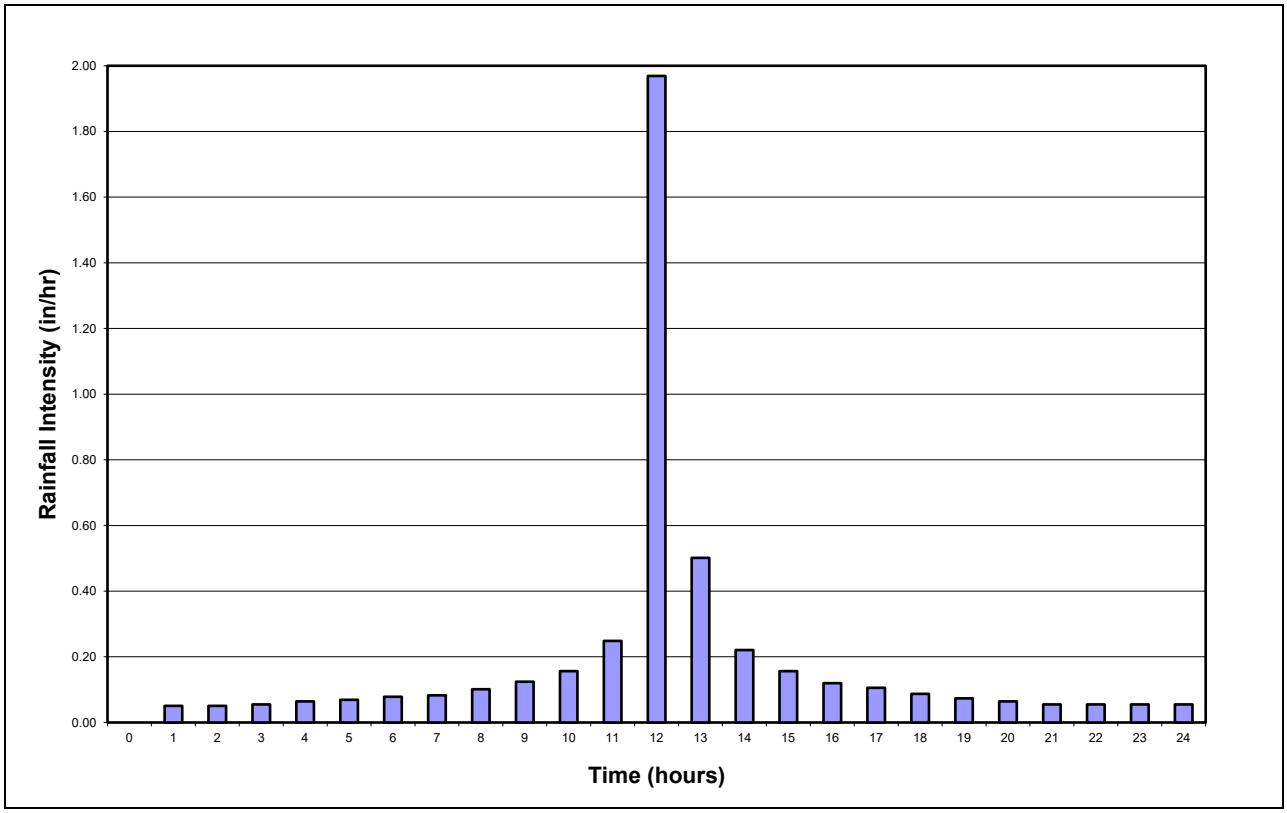
Being a Type II distribution, there is a difference in the cumulative fraction from the previous example. The cumulative fraction for a Type II distribution is shown in Table 7.18. The cumulative rain for each time increment is found by multiplying the total precipitation by the cumulative fraction, as shown in Table 7.18.

Determining the difference between consecutive cumulative rain depths will result in the incremental precipitation depths.

Table 7.18 Example of Incremental Rainfall Tabulation (U.S. Customary)

Time (hour)	Cum. Fraction (P_t/P_{24})	Cum. Rain (in)	Incr. Rain (in)
0	0.00	0.00	0.00
1	0.01	0.05	0.05
2	0.02	0.10	0.05
3	0.03	0.16	0.06
4	0.05	0.22	0.06
5	0.06	0.29	0.07
6	0.08	0.37	0.08
7	0.10	0.45	0.08
8	0.12	0.55	0.10
9	0.15	0.68	0.12
10	0.18	0.83	0.16
11	0.24	1.08	0.25
12	0.66	3.05	1.97
13	0.77	3.55	0.50
14	0.82	3.77	0.22
15	0.85	3.93	0.16
16	0.88	4.05	0.12
17	0.90	4.15	0.11
18	0.92	4.24	0.09
19	0.94	4.31	0.07
20	0.95	4.38	0.06
21	0.96	4.43	0.06
22	0.98	4.49	0.06
23	0.99	4.54	0.06
24	1.00	4.60	0.06

Figure 7.8 Example of Rainfall Hyetograph (U.S. Customary)



7.8 STORAGE ROUTING OF FLOOD HYDROGRAPHS

A. Flood Routing. Most design situations for Department drainage facilities are accomplished under the general category of steady flow. Steady flow implies no change of discharge with respect to time.

As a flood hydrograph approaches and passes through a reservoir or detention facility, the characteristics of unsteady flow become significant. An accounting of inflow and outflow rates per flood time and water storage characteristics must be made. This accounting is accomplished by routing a flood hydrograph through the storage facility.

Reservoir or detention pond storage routing is applicable when outflow depends only upon the volume of flood storage. Storage routing techniques are used to do the following:

- Determine peak discharges from watersheds containing reservoirs, flood water detention basins and other flow retardation structures.
- Determine overtopping flood magnitudes.
- Evaluate the duration of traffic interruption due to roadway overtopping.

B. Hydrograph Routing Methods. There are several analytical and graphical methods for routing flood hydrographs through reservoirs or other detention facilities. All of the methods require reliable descriptions of three items:

- An inflow runoff hydrograph for the subject flood.
- The storage capacity versus water elevation within the retention facility or reservoir.
- The performance characteristics of outlet facilities associated with the operation of the reservoir.

By definition, when inflow and outflow from a reservoir (or any type of storage facility) are equal, there is a steady-state condition. If the inflow exceeds the outflow, the additional discharge is stored in the system. Conversely, when the outflow exceeds the inflow, water is taken from storage.

The basic reservoir routing equation is expressed as:

Average Inflow - Average Outflow = Rate of Change in Storage

In numerical form, this statement of flow continuity can be written in the form of Equation 7.28.

(Equation 7.28)

$$\frac{I_t + I_{t+1}}{2} - \frac{O_t + O_{t+1}}{2} = \frac{S_{t+1} - S_t}{\Delta T}$$

where: I_t = inflow at time step number t
I_{t+1} = inflow at time step number t + 1
O_t = outflow at time step number t
O_{t+1} = outflow at time step number t + 1
S_t = storage in the reservoir at time step number t
S_{t+1} = storage in the reservoir at time step number t + 1
ΔT = the time increment
t = time step number

Several routing methods may be useful in specific instances, but the suggested method for the Department is the storage-indication method. This is sometimes referred to as the level pool method of routing.

C. Storage-Indication Routing Method. Of the many methods for routing storage floods through reservoirs, the Storage-Indication Method is a relatively simple procedure suitable for most highway drainage applications. This method is described as follows.

Since the outflow discharge (O) is a function of storage alone, it is convenient to rewrite the routing equation (Equation 7.28) in the following form:

(Equation 7.29)

$$\frac{2S_{t+1}}{\Delta T} + O_{t+1} = I_t + I_{t+1} + \frac{2S_t}{\Delta T} - O_t$$

The use of the Storage-Indication Method requires that the relationships between stage storage and discharge be determined. This information is in addition to a description of the inflow hydrograph.

1. Stage-Storage Relation. The stage-storage relation is simply the volume of water held by the reservoir or storage facility as a function of the water surface elevation or depth. This information often is available from the reservoir sponsor or owner. Where the stage-storage relation is not available, it may be necessary to develop one by successive calculations of storage vs. associated stages in the storage facility.

2. Stage-Outflow Relation. The stage-outflow relation is based on the association of the reservoir stage (head) and the resulting outflow from the storage facility. This description of performance characteristics may be the following:

- Ratings of the primary and/or emergency spillway of a reservoir.
- Pump flow characteristics in a pump station.
- Hydraulic performance curve of a culvert or bridge on a highway.
- Hydraulic performance curve of a weir and orifice outlet of a detention pond.

The stage-outflow relation of the outlet works of a reservoir often is available through the reservoir sponsor or owner. In some cases, it may have to be developed by the highway designer.

3. Storage-Outflow Relation. With stage-storage and stage-outflow relations established, storage and outflow can be related at each stage. The relationship is described in the form of Equation 7.30.

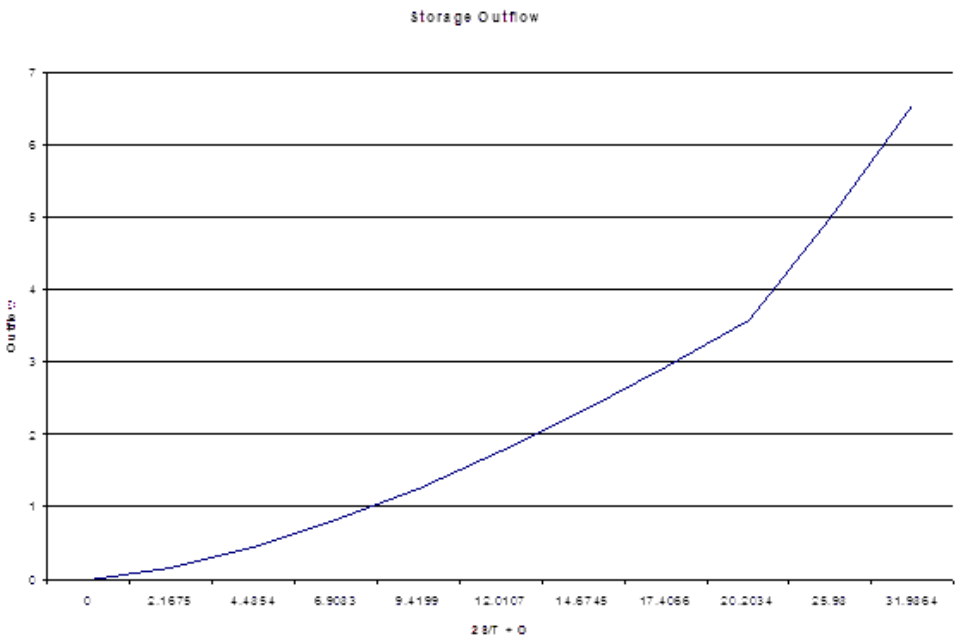
(Equation 7.30)

$$O \; vs \left(\frac{2S}{\Delta T} \right) + O$$

This relation can be plotted over the range of anticipated stages. Figure 7.9 illustrates a sample relationship.

The form of Equation 7.29 is especially useful because the terms on the left side of the equation are known. With the relation between the outflow and storage determined (Figure 7.9), the ordinates on the outflow hydrograph can be determined directly.

Figure 7.9 Storage-Outflow Relation



D. Procedure for Storage-Indication Method. The following steps are used to route an inflow flood runoff hydrograph through a storage system such as a reservoir or detention pond.

1. Acquire or develop a design flood runoff hydrograph.

2. Acquire or develop a stage-storage relation.

3. Acquire or develop a stage-outflow relationship.

4. Develop a storage-outflow relation curve (O vs. 2S/ Δ T + O).

5. At time step one (t = 1), assume an initial value for O_t as equal to I_t. Usually, at time step one, inflow equals zero, so outflow will be zero and $2S_1$□/T - O_1 = 0. Note that to start, t + 1 in the next step is 2.

6. Compute $2S_{t+1}/T + O_{t+1}$ using Equation 7.29.

7. From the storage-outflow relation, interpolate to find the value of outflow (O_{t+1}) at $(2S_{t+1})/(T) + O_{t+1}$ from Step 6.

8. Determine the value of $(2S_{t+1})/(T) - O_{t+1}$ using the relation $(2S_{t+1})/(T) - O_{t+1} = (2S_{t+1})/(T) + O_{t+1} - 2O_{t+1}$.

9. Assign the next time step to the value of t, e.g., for the first run through set t = 2.

10. Repeat Steps 6 through 9 until the outflow value (O_{t+1}) approaches zero.

11. Plot the inflow and outflow hydrographs. The peak outflow value should always coincide with a point on the receding limb of the inflow hydrograph.

12. Check conservation of mass to help identify success of the process by comparing the inflow volume to the sum of retained and outflow volumes using Equation 7.31.

(Equation 7.31)

$$\Delta T \times \sum I_t = S_r + \Delta T \times \sum O_t$$

where: S_r = volume of runoff completely retained, m^3 (ft^3)
ΣI_t = sum of inflow hydrograph ordinates, m^3 (ft^3)
ΣO_t = sum of outflow hydrograph ordinates, m^3 (ft^3)

There will be no retention volume if the outflow structure is at the flow line of the pond. A degree of imbalance can be expected due to the discretization process. If the difference is large yet the calculations are correct, reduce the time increment (T), determine the inflow hydrograph values for the new time steps, and repeat the routing process.

E. Example of Storage-Indication Method. Table 7.20 presents stage-storage and stage-outflow relations and the computed storage-outflow relation for a detention pond that was designed to reduce peak flows. The following is a sample calculation of the storage-outflow relation (2S/T+O) using a time increment of 15 minutes in Metric Units.

At stage = 0.1, S = 904.5 and O = 0.157. Then:

$$2S/\Delta T + O = (2)(904.5)/(15 \times 60) + 0.157 = 2.167 \text{ m}^3/\text{s}$$

Table 7.19 presents an inflow hydrograph and routing computations through the detention pond. The following presents sample calculations.

When t = 1, $I_1 = 0$, $I_2 = 0.5$, and $2S_1/\Delta T - O_1 = O$. Then:

$$2S_2/\Delta T + O_2 = I_1 + I_2 + 2S_1/\Delta T - O_1 = 0 + 0.5 + 0 = 0.5 \text{ m}^3/\text{s}$$

By interpolation (using Table 7.19),

$$O2 = 0 + (0.5 - 0)(0.1575 - 0) / (2.1675 - 0) = 0.363 \text{ m}^3/\text{s}$$

$$2S_2/\Delta T - O_2 = 2S_2/\Delta T + O_2 - 2O_2 = 0.5 - (2)(0.0363) = 0.427 \text{ m}^3/\text{s}$$

For brevity, the complete outflow ordinates are not shown in Table 7.20.

Similarly, in U.S. Customary Units, using Table 7.19:

Stage = 0.3 ft, S = 31938 ft^3 and O = 5.56 cfs

$$\frac{2S}{\Delta T} + O = \frac{2(31938)}{(15 \times 60)} + 5.56 = 76.53 \text{ cfs}$$

and sample calculations using Table 7.20 are:

when t = 1, $I_1 = 0$, $I_2 = 17.66$, and $2S_1/\Delta T_1 - O_1 = 0$, then:

$$\frac{2S_2}{\Delta T} + O_2 = I_1 + I_2 + \frac{2S_1}{\Delta T} - O_1 = 0 + 17.66 + 0 = 17.66 \; cfs$$

$$and \; O_2 = 0 + (17.66 - 0)\left(\frac{5.56}{76.53}\right) = 1.283 \; cfs$$

$$and \; \frac{2S_2}{\Delta T} - O_2 = 17.66 - 2(1.283) = 15.09 \; cfs$$

A plot of the resulting outflow hydrograph from the reservoir is superimposed on a plot of the inflow hydrograph (Figure 7.10). It is apparent in this example that the peak flow is attenuated from 141.24 cfs (4.00 m^3/s) to 121.75 cfs (3.45 m^3/s) by the effects of the available storage in the detention system.

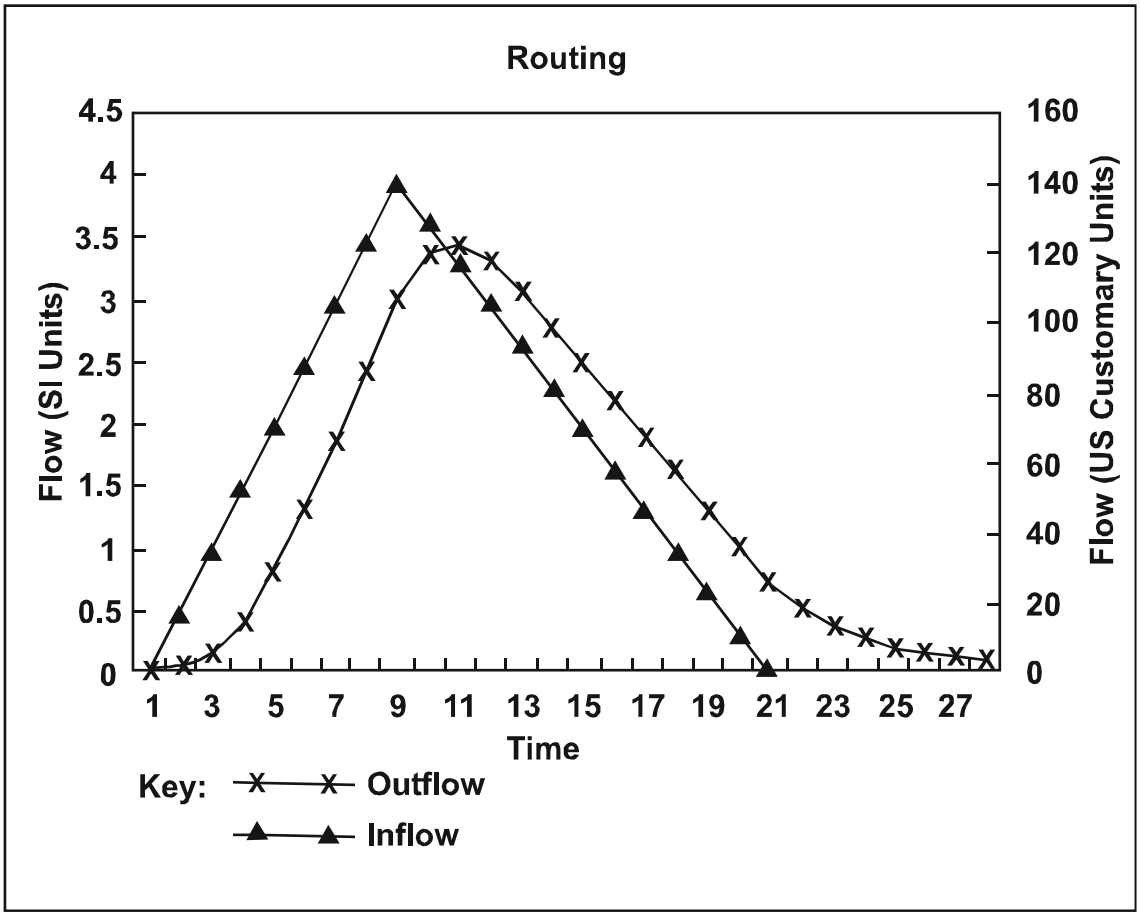
Table 7.19 Stage-Storage-Outflow and Computed Storage-Outflow Relation

Stage		Storage		Outflow		$2S/\Delta T+O$	
ft	m	ft^3	m^3	cfs	m^3/s	cfs	m^3/s
0.3	0.1	31938	905	5.56	0.16	76.53	2.17
0.7	0.2	64194	1818	15.73	0.45	158.38	4.49
1.0	0.3	96767	2741	28.89	0.82	243.93	6.91
1.3	0.4	129658	3672	44.49	1.26	332.62	9.42
1.6	0.5	162867	4613	62.17	1.76	424.10	12.01
2.0	0.6	196394	5562	81.72	2.31	518.16	14.67
2.3	0.7	230239	6521	102.99	2.92	614.63	17.41
2.6	0.8	264401	7488	125.82	3.56	713.38	20.20
3.3	1	333680	9450	175.84	4.98	917.35	25.98
3.9	1.2	404229	11448	231.15	6.55	1129.44	31.99

Table 7.20 Storage Routing Calculations

1	2	3		4		5		6		7		8	
Time step (t)	Time (min)	Inflow		$I_1 + I_2$		$2S/\Delta T-O$		$2S/\Delta T+O$		Outflow (O)		Stage	
		cfs	m^3/s	cfs	m^3/s	cfs	m^3/s	cfs	m^3/s	cfs	m^3/s	ft	m
1	0	0.00	0.00			0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	15	17.66	0.50	17.66	0.50	15.09	0.43	17.66	0.50	1.28	0.04	0.15	0.05
3	30	35.31	1.00	52.97	1.50	58.17	1.65	68.05	1.93	4.94	0.14	0.58	0.18
4	45	52.97	1.50	88.28	2.50	117.95	3.34	146.44	4.15	14.24	0.40	0.89	0.27
5	60	70.62	2.00	123.59	3.50	184.48	5.22	241.53	6.84	28.53	0.81	1.29	0.39
6	75	88.28	2.50	158.90	4.50	250.25	7.09	343.38	9.72	46.57	1.32	1.39	0.42
7	90	105.93	3.00	194.21	5.50	311.65	8.83	444.45	12.59	66.40	1.88	1.78	0.54
8	105	123.59	3.50	229.52	6.50	367.57	10.41	435.23	12.33	86.80	2.46	2.13	0.65
9	120	141.24	4.00	264.83	7.50	418.21	11.84	632.40	17.91	107.10	3.03	2.41	0.74
10	135	129.48	3.67	270.72	7.67	448.59	12.70	688.92	19.51	120.17	3.40	2.79	0.85
11	150	117.69	3.33	247.17	7.00	452.26	12.81	695.76	19.70	121.75	3.45	2.83	0.86
12	165	105.93	3.00	223.62	6.33	441.58	12.51	675.89	19.14	117.16	3.32	2.70	0.82
13	180	94.17	2.67	200.10	5.67	423.19	11.99	641.67	18.17	109.24	3.09	2.48	0.76
14	195	82.38	2.33	176.55	5.00	400.33	11.34	599.74	16.99	99.70	2.82	2.52	0.77
15	210	70.62	2.00	153.00	4.33	374.38	10.60	553.34	15.67	89.48	2.53	2.21	0.67
16	225	58.86	1.67	129.48	3.67	346.35	9.81	503.86	14.27	78.75	2.23	2.20	0.67
17	240	47.07	1.33	105.93	3.00	316.22	8.96	452.28	12.81	68.03	1.93	1.84	0.56
18	255	35.31	1.00	82.38	2.33	284.13	8.05	398.61	11.29	57.24	1.62	1.78	0.54
19	270	23.55	0.67	58.86	1.67	250.00	7.08	342.98	9.71	46.49	1.32	1.39	0.42
20	285	11.76	0.33	35.31	1.00	212.97	6.03	285.31	8.08	36.17	1.02	1.29	0.39
21	300	0.00	0.00	11.76	0.33	172.86	4.90	224.74	6.36	25.94	0.73	1.16	0.36
22	315	0.00	0.00	0.00	0.00	136.95	3.88	172.86	4.90	17.96	0.51	0.77	0.23
23	330	0.00	0.00	0.00	0.00	110.82	3.14	136.95	3.88	13.06	0.37	0.81	0.25
24	345	0.00	0.00	0.00	0.00	91.18	2.58	110.82	3.14	9.82	0.28	0.60	0.18
25	360	0.00	0.00	0.00	0.00	76.42	2.16	91.18	2.58	7.38	0.21	0.45	0.14
26	375	0.00	0.00	0.00	0.00	65.31	1.85	76.42	2.16	5.55	0.16	0.66	0.20
27	390	0.00	0.00	0.00	0.00	55.82	1.58	65.31	1.85	4.75	0.13	0.56	0.17
28	405	0.00	0.00	0.00	0.00	47.71	1.35	55.82	1.58	4.06	0.11	0.48	0.15

Figure 7.10 Plot of Inflow and Outflow Hydrographs



7.9 CHANNEL ROUTING OF FLOOD HYDROGRAPHS

A. Introduction to Channel Routing. Routing of flood hydrographs by means of channel routing procedures is useful in instances where known hydrographic data are located at a point other than the point of interest. This also is true in those instances where the channel profile or plan is changed in such a way as to alter the natural velocity or channel storage characteristics. Routing analysis estimates the change from the outflow hydrograph to the inflow hydrograph or through a reach. This section describes the Muskingum Method and the Muskingum-Cunge Method, which are lumped flow routing techniques that approximate storage effects in the form of a prism and wedge component (Chow, Maidment, and Mays, 1988).

Routing analysis is used to predict the magnitudes, volumes, and temporal patterns of the flow (often a flood wave) as it translates down a channel. The discussion in this section is limited to one-dimensional routing. Thus, at a given instant in time, variation in flow properties is computed from upstream to downstream, but not in the vertical or transverse dimensions of the channel.

Almost all flow in river channels is to some extent unsteady; that is, depth, velocity, and flow change with time. The extent of flow variation with time and distance may be such that steady flow computations may be are inappropriate. Some of the applications of unsteady flow analysis include:

- Runoff from precipitation.
- Transient flows released from reservoirs during operations for flood control, hydropower generation, recreation, etc.
- Irrigation flows affected by gates, pumps, diversions, etc.
- Ocean or lake waves.

- Tidal effects in rivers or estuaries.

Unsteady flow models based on the complete St. Venant equations, or some simplified form thereof, provide a means for analyzing the flow for engineering applications. The equations are generally written:

Continuity Equation:

(Equation 7.32)

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} - q = 0$$

Moment Equation:

(Equation 7.33)

$$\frac{\partial y}{\partial x} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{1}{g}\frac{\partial V}{\partial t} = S_o - S_f$$

where: Q = discharge, m^3/s (cfs)
x = distance, m
A = cross sectional area, m^2 (ft^2)
t = time, sec
q = lateral inflow or outflow, m^2/s (ft^2/s)
S_f = friction slope, m/m (ft/ft)
S_o = channel bed slope, m/m (ft/ft)
g = gravitational acceleration, m/s^2 (ft/s^2)

Routing methods are classified into two categories: hydrologic and hydraulic routing. Both methods attempt to account for volumetric changes in storage as water moves downstream. As such, both of these methods use some form of the continuity equation (Equation 7.32).

B. Hydrologic Routing. Hydrologic routing methods combine the continuity equation with some relationship between storage, outflow, and possibly inflow. These relationships can be assumed, empirical, or analytical in nature. An example of such a relationship might be a stage-storage rating, combined with a stage-discharge function such as Manning's equation:

(Equation 7.34)

$$Q = \frac{1}{n} A R^{2/3} S_f^{1/2}$$

where R = hydraulic radius, S_f = friction slope, and n = Manning's roughness coefficient, or a simple exponential relationship such as:

(Equation 7.35)

$$Q = aS^n$$

1. Hydraulic Routing. In hydraulic routing analysis, it is intended that the dynamics of the water or flood wave movement be more accurately described than in hydrologic routing. Therefore, hydraulic routing methods combine the continuity equation with a mathematical relationship describing the actual physics of the movement of the water. The momentum equation (Equation 7.34) is the common relationship employed.

Open Channel Flow (Henderson, 1966) expressed the momentum equation as:

(Equation 7.36)

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x} - \frac{1}{g}\frac{\partial v}{\partial t}$$

where S_f = the friction slope, S_o = the bed slope, y = depth, v = velocity, and x = distance longitudinally along the channel. Equations 7.33 and 7.36 are considered to be the full dynamic wave equations or the St. Venant equations. The solution of the full dynamic wave equations requires numerical analysis since no closed form solution of the equations exists.

The most common technique used in applying the St. Venant equations is to ignore those terms in the momentum equation which are thought to be negligible, thereby producing simplified hydraulic models. These simplified models are illustrated below:

Unsteady non-uniform flow (full momentum equation):

(Equation 7.37)

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x} - \frac{1}{g}\frac{\partial v}{\partial t}$$

Steady non-uniform (quasi-steady model – eliminate time differential):

(Equation 7.38)

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x}$$

Diffusion or non-inertial model (eliminate inertial terms):

(Equation 7.39)

$$S_f = S_o - \frac{\partial y}{\partial x}$$

Kinematic model:

(Equation 7.40)

$$S_f = S_o$$

The following sections are devoted to providing a working knowledge of two of the more popular methods: the Muskingum and the Muskingum-Cunge.

C. Muskingum Method. The Muskingum Method combines a prism component of storage, KO, and a wedge component, KX(I-O), to describe the total storage in the reach as the following equation:

(Equation 7.41)

$$S = K\left[XI + (1 - X)O\right]$$

where: S = total storage, m^3 (ft^3)
K = a proportionality constant representing the time of travel of a flood wave to traverse the reach(es). Oftentimes, this is set to the average travel time through the reach.
X = a weighting factor describing the backwater storage effects approximated as a wedge
I = inflow, m^3/s (cfs)
O = outflow, m^3/s (cfs)

The value of X depends on the amount of wedge storage; when X = 0 there is no backwater (reservoir type storage) and when X = 0.5 the storage is described as a full wedge. The weighting factor, X, ranges from 0.2 to 0.3 in natural streams. A value of 0.25 is typical. The method is very good when there is not a lot of channel storage and the routing distance is less than 2 km (1.24 miles).

Equation 7.42 represents the time rate of change of storage as the following:

(Equation 7.42)

$$\frac{S_{t+1} - S_t}{\Delta T} = \frac{K\{[XI_{t+1} + (1-x)O_{t+1}] - [XI_t + (1-X)O_t]\}}{\Delta T}$$

where: ΔT = time interval usually ranging from 0.3•K to K
t = time step number

Applying continuity to Equation 7.42 produces the Muskingum flow routing equation as follows:

(Equation 7.43)

$$O_{t+1} = C_1 \cdot I_{t+1} + C_2 \cdot I_t + C_3 \cdot O_t$$

where:

(Equations 7.44 – 7.46)

$$C_1 = \frac{\Delta T - 2K \quad X}{2K(1-X) + \Delta T}$$

$$C_2 = \frac{\Delta T + 2K \quad X}{2K(1-X) + \Delta T}$$

$$C_3 = \frac{2K(1-X) - \Delta T}{2K(1-X) + \Delta T}$$

By definition, the sum of C_1, C_2 and C_3 should be 1.

If measured inflow and outflow hydrographs are available, the designer may approximate K and X using Equation 7.47. One calculates X by plotting the numerator on the vertical axis and the denominator on the horizontal axis, and adjusting X until the loop collapses into a single line. The slope of the line equals K.

(Equation 7.47)

$$K = \frac{0.5\Delta T\left[\left(I_{t+1} + I_t\right) - \left(O_{t+1} + O_t\right)\right]}{X\left(I_{t+1} - I_t\right) + \left(1-X\right)\left(P_{t+1} - O_t\right)}$$

The designer may also approximate K and X using the Muskingum-Cunge Method described in Chow, 1988, or Fread, 1993.

D. Example of Muskingum Method. The example shown in Table 7.21 shows a triangular hydrograph routed through three 3280 feet (1000 meter) reaches of channel. The outflow hydrograph for each reach is used as the inflow for the next. The channel has a 'K' of 0.278 hours (1000 seconds) and an X of 0.2.

Table 7.21 Channel Routing Using the Muskingum Method

Time Step	Time (s)	Inflow cfs (m^3/s)	Outflow cfs (m^3/s)		
			Reach 1	Reach 2	Reach 3
1	0	0 (0)	0 (0)	0	0
2	1000	7062 (200)	1629.57 (46.15)	376.05 (10.65)	86.86 (2.46)
3	2000	14124 (400)	7438.05 (210.65)	2680.74 (75.92)	841.08 (23.82)
4	3000	10593 (300)	11766.35 (333.23)	7339.18 (207.85)	3331.15 (94.34)
5	4000	7062 (200)	10048.87 (284.59)	10348.30 (293.07)	7108.61 (201.32)
6	5000	3531 (100)	6936.30 (196.44)	9399.88 (266.21)	9381.87 (265.70)
7	6000	0 (0)	3502.05(99.18)	6712.43 (190.10)	8775.24 (248.52)
8	7000	0 (0)	808.25 (22.89)	3621.04 (102.55)	6475.15 (183.38)
9	8000	0 (0)	186.44(5.28)	1313.89 (37.21)	3747.45 (106.13)
10	9000	0 (0)	43.08 (1.22)	413.48 (11.71)	1667.69 (47.23)
11	10000	0 (0)	9.89 (0.28)	120.76 (3.42)	635.58 (18.00)
12	11000	0 (0)	2.12 (0.06)	33.90 (0.96)	219.63 (6.22)
13	12000	0 (0)	0.35 (0.01)	9.18 (0.26)	70.97 (2.01)
14	13000	0 (0)	0.00	2.47 (0.07)	21.89 (0.62)
15	14000	0 (0)	0.00	0.71 (0.02)	6.36 (0.18)
16	15000	0 (0)	0.00	0.00	1.77 (0.05)
17	16000	0 (0)	0.00	0.00	0.71 (0.02)
18	17000	0 (0)	0.00	0.00	0.00

Using Equations 7.44, 7.45, and 7.46 with a time increment of 1000 s:

$$C_1 = \frac{1000 - 2(1000)(0.2)}{2(1000)(1-0.2)+1000} = 0.23077$$

$$C_2 = \frac{1000 + 2(1000)(0.2)}{2(1000)(1-0.2)+1000} = 0.53846$$

$$C_3 = \frac{2(1000)(1-0.2) - 1000}{2(1000)(1-0.2)+1000} = 0.23077$$

Check:

$C_1 + C_2 + C_3 = 0.23077 + 0.23077 + 0.53846 = 1$

For time step two, (t = 2), first reach:

$O_2 = (0.23077)(7062) + (0.53846)(0) + (0.23077)(0) = 1629.70$ cfs

$O_2 = (0.23077)(200) + (0.53846)(0) + (0.23077)(0) = 46.15$ m^3/s

For time step three, (t = 3):

$O_3 = (0.23077)(14124) + (0.53846)(7062) + (0.23077)(1630) = 7438.16$ cfs

$O_3 = (0.23077)(400) + (0.53846)(200) + (0.23077)(46.15) = 210.65$ m^3/s

Repeat the process until the outflow hydrograph is complete. The outflow hydrograph from reach 1 becomes the inflow hydrograph for reach 2 and the process is repeated for reaches 2 and 3. Since the outflow hydrograph represents a displacement in distance as well as time, the peak outflow does not coincide with the receding limb of the inflow hydrograph.

E. Muskingum-Cunge Method.

1. Description of Method. "On the Subject of a Flood Propagation Computation Method (Muskingum Method)" (Cunge, 1969) states that the Muskingum method is essentially a finite difference form of the St. Venant equations after neglecting the inertia terms. This neglecting of the inertia terms implies the Muskingum method is of a kinematic nature. "On the Subject of a Flood Propagation Computation Method (Muskingum Method)" (Cunge, 1969) introduced alternative methods for calculating the routing coefficients in the Muskingum method, which effectively produces results based on diffusion analogy, rather than the kinematic nature of the basic Muskingum method. This method is known as the Muskingum-Cunge routing method.

The Muskingum-Cunge routing method produces good results for many applications without the computational problems associated with fully dynamic routing models. The method is non-linear in that routing coefficients, as a function of flow properties, are recalculated with each time step. The method requires input values of representative channel cross section, slope and roughness for the routing reach.

2. Assumptions. Use of the method requires the assumption that backwater effects are negligible and that channel geometry, slope and roughness data are adequate to characterize the hydraulics over the reach represented by a given cross section.

3. Limitations. The Muskingum-Cunge method combines the continuity equation and the diffusion form of the momentum equation. As such it accounts for flood wave attenuation, but not for reverse flow or backwater effects. The Muskingum-Cunge technique is not well suited for fast rising hydrographs such as those encountered in dam break situations.

4. Data Requirements. Muskingum-Cunge routing requires representative cross section geometry, slope and roughness for each reach simulated.

5. Development of Equations. The Muskingum-Cunge formulation is similar to the Muskingum formulation. The Muskingum-Cunge derivation begins with the continuity equation and includes the diffusion form of the momentum equation. These equations are combined and linearized, producing the following convective diffusion equation (Miller and Cunge, 1975).

(Equation 7.48)

$$\frac{\partial Q}{\partial t} + c\frac{\partial Q}{\partial x} = \mu\frac{d^2Q}{\partial x^2} + cq_x$$

where: Q = discharge, m^3/s (cfs)
t = time, sec
x = distance along channel, m (ft)
q_x = lateral inflow, m^2/s (ft^2/s)
c = wave celerity, m/s (ft/s)
μ = hydraulic diffusivity, m^2/s (ft^2/s)

The wave celerity is defined as follows:

(Equation 7.49)

$$c = \frac{dQ}{dA}$$

and the hydraulic diffusivity is defined:

(Equation 7.50)

$$\mu = \frac{Q}{2BS_o}$$

where S_0 is the bed slope and B is the top width of the flow area.

The solution of the Muskingum-Cunge procedure is accomplished by discretizing the equations on an x-t plane. The equation applied to the discretized plane, allowing for lateral inflow, is as follows:

(Equation 7.51)

$$Q_{j+1}^{n+1} = C_1 Q_j^n + C_2 Q_j^{n+1} + C_3 Q_{j+1}^n + C_4 Q_L$$

where the coefficients are defined to be:

(Equations 7.52 - 7.55)

$$C_1 = \frac{\frac{\Delta t}{K} + 2X}{\frac{\Delta t}{K} + 2(1 - X)}$$

$$C_2 = \frac{\frac{\Delta t}{K} - 2X}{\frac{\Delta t}{K} + 2(1 - X)}$$

$$C_3 = \frac{2(1 - X) - \frac{\Delta t}{K}}{\frac{\Delta t}{K} + 2(1 - X)}$$

$$C_4 = \frac{2\left(\frac{B}{K}\right) t}{\frac{\Delta t}{K} + 2(1 - X)}$$

6. Use and Estimation of Parameters. In the Muskingum-Cunge method, the travel time parameter, K, is calculated using the reach length, x, and the wave celerity, c, as:

(Equation 7.56)

$$K = \frac{\Delta x}{c}$$

and the parameter X is calculated as a function of the flow properties:

(Equation 7.57)

$$X = \tfrac{1}{2}\left(1 - \frac{Q}{BS_o c \Delta x}\right)$$

The parameters required to apply Muskingum-Cunge routing are channel cross section geometry (either as a regular trapezoid or rectangle, or as an eight-point cross section), channel slope, roughness, and the length of the channel element represented by these parameters. One must take particular care when choosing the computation interval, since the computation interval must not be longer than the time it takes for the wave to travel the reach distance. If the wave travels faster than the time step, the computation will not indicate the proper storage.

7.10 STATISTICAL ANALYSIS OF STREAM GAGE DATA

A. Introduction to Statistical Analysis of Stream Gage Data. The estimation of peak discharges of various recurrence intervals is one of the most common problems faced by engineers when designing for highway drainage structures. The problem can be divided into three categories:

1. If the site is at a gaging station, and the streamflow record is fairly complete (no missing data) and of sufficient length then flood frequency analysis using the log-Pearson Type III distribution will be used to determine the peak flows for the various return periods desired.

2. If there is no gage at the site, but there is a gage on the same stream that can be transposed (the gage's data can be transposed if $0.5 \leq \frac{A_g}{A_{Watershed}} \leq 1.5$), then the gaged data will be transposed to the site using an area relationship given in Section 7.3.H. or based on the method outlined in USGS SIR 08-5102.

3. If there is no gage on the stream and the watershed characteristics are within the bounds of the USGS SIR 08-5102 and/or USGS WRIR 00-4189 procedure, then USGS SIR 08-5102 and/or USGS WRIR 00-4189 will be applied to determine the peak flows for the various return periods desired based on the regression procedure discussed in Section 7.3.J.5. If USGS SIR 08-5102 and/or USGS WRIR 00-4189 is not appropriate for the specific site then a hydrograph model such as HEC-1 should be used to determine the peak discharges.

Bulletin 17B, *Guidelines for Determining Flood Flow Frequency* (U.S. Water Resources Council, 1982) is a guide that describes the data and procedures for computing flood flow frequency curves where stream gaging records of sufficient length are available. The Bulletin was intended for use for analyzing records of annual flood peak discharges, including both systematic records and historic data. There are numerous statistical distribution methods for establishing peak discharge versus frequency relations. The log Pearson Type III statistical distribution method has gained the most widespread acceptance and is suggested by Bulletin 17B. An outline of this method follows; however, the designer is not limited to using only this method, especially if the resulting discharge frequency relation does not seem to fit the data adequately (e.g. other methods have a smaller standard of error).

B. Sources of Stream Gage Records. Generally, for Department application, the designer will need to acquire a record of the annual peak flows for the appropriate gaging station. The following sources provide stream gage records:

- U.S. Geological Survey Water Resources Data - Pennsylvania, Surface Water. These are prepared annually and contain records for one water year per publication.
- International Boundary and Water Commission Water Bulletins.
- USGS database WATSTORE.
- INTERNET file transfer protocol (ftp) sites.
- Compact Disk Read Only Memory (various vendors).

The designer should ensure acquisition of the most recent published records.

C. Applicability and Limitations. For highway drainage purposes, a statistical analysis of stream gage data is applied only in those instances where there is adequate data from stream gaging stations. Bulletin 17B states that stream gage records must contain a minimum of 10 years of data to warrant a statistical analysis (see also Section 7.3.H). *Water-Quality Conditions During Low Flow in the Lower Youghiogheny River Basin, Pennsylvania, October 5-7, 1998* (USGS, 1998) contains a procedure for computing weighted peak flows and equivalent periods of record for stream gages. This procedure can be used to extend short gage records that meet the minimum requirement for number of years.

In some cases, a site needing a design peak discharge will be located on the same stream and near to the location of an active or discontinued stream gaging station with an adequate length of record. A list of all the active and discontinued gaging stations in the state can be obtained from the USGS in Harrisburg, PA.

Having determined that a suitable stream gage record exists, it is necessary to determine if any structures or organization may be affecting the peak discharges at the design site. The period of record for the gaging station's

annual-peak discharges should represent the same or similar basin conditions as that of the design state. A time series analysis may be required to determine if there is an impact caused by the upstream structure. Therefore, any gaged peak discharges not representing the basin conditions for the design site should be excluded from the analysis.

The most typical factors affecting peak discharges are urbanization and regulation by reservoirs. Densities of impervious cover less than 10% of the watershed area generally do not affect peak discharges. The existence in the watershed of a major reservoir or many small reservoirs or flood control structures can greatly affect the runoff characteristics. The length of record should be adjusted to include only those records that have been collected subsequent to the impoundment of water by reservoirs and subsequent to any major urbanization. If the resulting records then become too short as defined above, do not use the method outlined in Section 7.10.D.

D. Log-Pearson Type III Distribution. The log-Pearson Type III method for the statistical analysis of gaged flood data is applicable to just about any series of natural floods. Three statistical moments are involved in the analysis:

- The mean is approximately equal to the logarithm of the 2-year peak discharge.
- The standard deviation, which is the second moment of the logarithm of the discharges, can be compared to the slope of the plotted curve. (Although, with the consideration of the third moment, skew, there is no single slope to the curve.)
- The skew (the third moment of the logarithm of the peak discharges) represents the form of curvature to the plotted curve.

On log-probability paper, a negative skew of the flood-frequency curve is concave downward and for a positive skew, the curve is concave upward. If the skew is zero, the plotted relation forms a straight line, the distribution is defined as normal, and the standard deviation becomes the slope of that straight line. Because of these tendencies, the significance of the skew becomes especially important in the estimation of floods based upon extrapolated curves (i.e., for a large positive skew with limited years of record, the 100-year return period flood peak may be over predicted).

Because flooding is often erratic in Pennsylvania, a series of observed floods may include annual peak discharge rates which do not seem to belong to the population of the series. The values may be extremely large or extremely small with respect to the rest of the series of observations. Such values may be "outliers" which possibly should be excluded from the set of data to be analyzed. Additionally, adjustment can be made to incorporate historical data. Caution should be exercised when dealing with high outliers and if at all possible the record should be extended by incorporating historical data (see Bulletin 17B).

To make the analysis of the Log-Pearson Type III method easier, the computer program PEAKFQ is available from the USGS. Section 7.10.E. outlines the procedure for the Log-Pearson Type III method for understanding and reference only.

E. Log-Pearson Type III Analysis Procedure.

1. Acquire and assess annual peak discharge record. There are instances when the annual peak discharge data may be suspect. For projects containing stream gage data affected by outliers, coordination with the USGS is suggested to obtain the final design peak flows. The record should comprise only one discharge (maximum) per year. Note that the USGS water year is October through September.

2. Calculate the logarithm of the base 10 for each discharge value.

3. Calculate the statistics using Equations 7.58, 7.59, and 7.60.

(Equation 7.58)

$$\overline{Q}_L = \frac{\sum X}{N}$$

(Equation 7.59)

$$S_L = \left\{ \frac{\sum X^2 - \frac{(\sum x)^2}{N}}{N-1} \right\}^{\frac{1}{2}}$$

(Equation 7.60)

$$G_S = \frac{N^2\left(\sum X^3\right) - 3N\left(\sum X\right)\left(\sum X^2\right) + 2\left(\sum X\right)^3}{N(N-1)(N-2)S_L^3}$$

where: $\overline{Q}_L$ = mean of the logarithms of the annual peak discharges, m^3/s (cfs)
N = number of observations
X = logarithm of the annual peak discharge, m^3/s (cfs)
S_L = standard deviation of the logarithms of the annual peak discharge, m^3/s (cfs)
G_S = coefficient of skew of log values (station skew)

4. For each frequency (typically 2 through 100 year), calculate the logarithm of the discharge using Equation 7.61.

(Equation 7.61)

$$\log Q = \overline{Q}_L + KS_L$$

where: Q = flood magnitude, m^3/s (cfs)
K = a frequency factor for a particular return period and coefficient of skew

5. Plot discharge versus frequency on standard log probability paper.

6. Consider adjusting the calculations to accommodate a weighted skew and outliers as discussed in Sections 7.10.F. and 7.10.G., respectively.

F. Skew. There are three methods for determining the value of the skew coefficient for the log Pearson Type III curve fit.

1. The station skew is calculated directly from the gage data using Equation 7.60. This value may not represent the skew of the parent population accurately if the period of record is short or if there are extreme events in the period of record.

2. Figure 7.11 shows the value of generalized skew coefficients across Pennsylvania. The designer may use the generalized skew coefficient identified by the map based upon the project location.

3. The designer may compute a weighted skew. Refer to Bulletin 17B for the method. (Note: The mean square error for the generalized skew is 0.35 which replaces the value of 0.55 presented in Bulletin 17B.)

It is preferable to use a weighted skew as described in Bulletin 17B for all flood peak estimates. The weighted value and can also be obtained from the USGS in Harrisburg, PA.

G. Accommodating Outliers in the Data. The distribution of all the annual and historical peak discharges determines the shape of the frequency curve, and thus the design-peak discharges. For a Log-Pearson Type III analysis, the distribution of the high peak discharges affects the shape of the curve and the value of the low peak discharges.

In most peak-stream flow frequency analyses, the larger recurrence-interval peaks are needed more often than those for the lower recurrence intervals. Most design peaks, for example, are based on 10-year, 25-year, 50-year, or 100-year recurrence intervals rather than 2-year or 5-year intervals. Therefore, it would be more desirable for the shape of the frequency curve to be based on the distribution of the larger peaks.

This is accomplished by eliminating some of the lowest peak discharges from the analyses, using a low-outlier threshold. The peak discharges less than the low-outlier threshold are excluded from the analysis. The value for the low-outlier threshold, therefore, should exclude those peaks not indicative of the distribution for the higher peaks. This value can be chosen by reviewing the sequentially-ranked values for all of the peak discharges used in the analysis.

For example, the lowest sequentially ranked peak discharges for a station, in cubic meters per second (m^3/sec), are as follows: 0, 10, 25, 90, 450, 495, 630, 800, 1050. The largest difference between sequential values for these discharges is 360 m^3/s, which is the difference between 90 and 450 m^3/s. Therefore, the distribution of the peak discharges substantially changes below the value of 450 m^s/s, which could be used as the low value threshold.

In circumstances when removal of low outliers is appropriate, Equation 7.62 is an adaptation of the discussion in Bulletin 17B that was developed by Peter Smith, Texas DOT, to compute the low-outlier threshold.

(Equation 7.62)

$$LOT = 10^{\left(a\overline{Q}_L + bS_L + cG + d\right)}$$

where:
- LOT = estimated low-outlier threshold, m^3/s
- G = appropriate skew coefficient as discussed in Section 7.10.F.
- a = 1.09
- b = -0.584
- c = 0.140
- d = -0.799
- $\overline{Q}_L$ = mean of the logarithms of the annual peak discharges, m^3/s
- S_L = standard deviation of the logarithms of the annual peak discharge, m^3/s

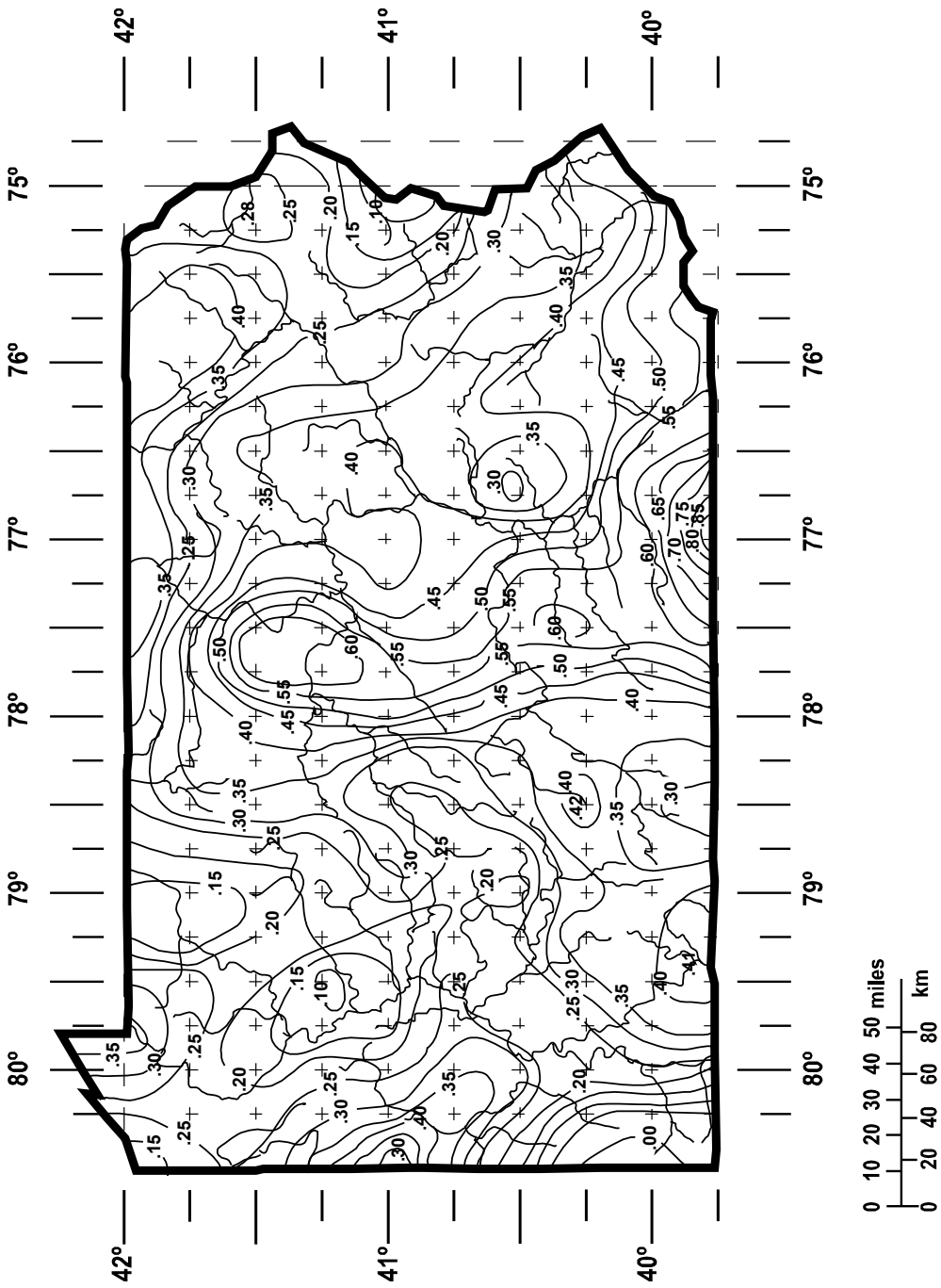
Higher-outlier thresholds represent extremely high peak discharges: those with a recurrence interval larger than indicated by the period of record for a station. For example, a 100-year peak discharge could be gaged during a 10-year period of record. The frequency curve thus would be unduly shaped by the 100-year period.

Efforts have been made by the USGS to identify high outliers, referred to as historical peaks. This has been done by identifying and interviewing long-term residents living near the gaging stations. In many cases, residents have identified a particular flood peak as being the highest since a previous higher peak. These peaks are identified as the highest since a specific date. In other cases, residents identify a specific peak as the highest since they have lived near the gaging station. Those peaks are identified as the highest since at least a specific date. The historical peaks may precede or be within the period of gaged record for the station.

For some stations, however, a historical-peak discharge may have been gaged without knowledge of its historical significance. When this is suspected for a station, the dates for historical peaks from nearby stations on basins of comparable size should be reviewed and compared to dates of floods for the suspect station. These dates and historical periods may be applicable to stations where this information is absent.

Having identified appropriate outliers, the designer should re-compute the statistics (Equations 7.60 and 7.61) using a data set which excludes values beyond the established outlier thresholds.

Figure 7.11
Pennsylvania
Generalized Skew
Coefficients of
Logarithms of Annual
Maximum Streamflow
(G)



Based on annual series records through water year 1977 from 139 Pennsylvania stream gauges and 148 gauges from adjoining states.

7.11 CHAPTER 7 NOMENCLATURE

Symbol	Definition	Units
A	Drainage area	ha or ac
A	Channel cross-sectional area in Manning's Equation	m^2 of ft^2
A_n	Nth sub-area size	ha or ac
A_g	Area of gaged basin	ha or ac
α	1.0 (metric), 3.28 (U.S. Customary)	m/s or ft/s
C	Weighted runoff coefficient	dimensionless
c	Wave celerity for use in Muskingum-Cunge Method	m/s or ft/s
C_f	Runoff coefficient adjustment factor due to return interval	dimensionless
C_n	Runoff coefficient for nth sub-area	dimensionless
C_0, C_1, C_2	Coefficients for Unit Peak Discharge Equation	dimensionless
C_1, C_2, C_3	Coefficients for Muskingum and Muskingum-Cunge Routing	dimensionless
d	Duration of unit excess rainfall	hr
F	Pond adjustment factor	dimensionless
G	Skew coefficient for Log Pearson Type III curve fit	dimensionless
G_S	Coefficient of skew of log values, station skew	dimensionless
g	Gravitational acceleration	m/s^2 or ft/s^2
I	Average rainfall intensity in the Rational Method Equation	mm/hr or in/hr
I	Inflow to reach in the Muskingum Method	m^3/s or cfs
I_a	Initial abstraction	mm or in
I_t	Inflow at time step number t for Hydrograph Routing	m^3/s or cfs
I_{t+1}	Inflow at time step number t + 1 for Hydrograph Routing	m^3/s or cfs
i	Rainfall intensity in the Overland Flow Segmental Method	mm/hr or in/hr
K	A frequency factor for Log Pearson Type III Analysis	dimensionless
K	Coefficient for use in Sheet Flow Kinematic Wave Equation	6.92 Metric/0.933 US
K	Coefficient for use in the Rational Method Equation	360 Metric/1 US
K	Coefficient for use in the Triangular Hydrograph Method	0.208 Metric/484 US
K	Coefficient for Overland Flow Equation	2949 Metric/1900 US
K	Proportionality constant representing the time of travel of a flood wave through the reach in the Muskingum method	hr
k	Intercept coefficient for Shallow Concentrated Flow Equation	dimensionless
L	Longest hydraulic length	m or ft
L_n	Length of nth reach along flow path	m or ft
LOT	Estimated low-outlier threshold for use in Bulletin 17B Statistical Frequency Analysis	m^3/s
μ	Hydraulic diffusivity for use in Muskingum-Cunge Method	m^2/s or ft^2/s
N	Number of observations	dimensionless
n	Manning's n-value	dimensionless
O	Outflow from reach in the Muskingum Method	m^3/s or cfs
O_t	Outflow at time step number t for Hydrograph Routing	m^3/s or cfs
O_{t+1}	Outflow at time step number t + 1 for Hydrograph Routing	m^3/s or cfs
P	Accumulated rainfall (potential maximum runoff) for NRCS Rainfall-Runoff Equation	mm or in
Q	Discharge	m^3/s or cfs
Q	Maximum rate of runoff in the Rational Method Equation	m^3/s or cfs
Q	Flood magnitude	m^3/s or cfs
$\overline{Q_L}$	Mean of the logarithms of the annual peak discharges in Log Pearson Type III Analysis	m^3/s or cfs
Q_g	WRC peak discharge at gage	m^3/s or cfs
q_x	Lateral inflow or outflow in Muskingum-Cunge Method	m^2/s or ft^2/s
q	Depth of storm runoff during time interval for Hydrograph Methods	mm or in
q_p	Peak rate of discharge	m^3/s or cfs
q_u	Unit peak discharge	$m^3/s/km^2/mm$ or $cfs/mi^2/in$

R	Runoff volume for NRCS Graphical Peak Discharge Method	mm or in
R	Hydraulic radius in Manning's Equation	m or ft
RCN or CN	Runoff Curve Number	dimensionless
S	Average slope of the watershed in percent	%
S	Potential max retention for NRCS Rainfall-Runoff Equation	mm or in
S	Total Storage in the Muskingum Method	m^3 or ft^3
S_f	Friction slope	m/m or ft/ft
S_L	Standard deviation of logarithms of annual peak discharge	m^3/s or cfs
S_o	Channel bed slope	m/m or ft/ft
S_p	Channel or flow slope in percent	%
S_r	Volume of runoff completely retained for Storage-Indication Method	m^3 or ft^3
S_t	Storage in the reservoir at time step number t for Hydrograph Routing	m^3 or ft^3
S_{t+1}	Storage in the reservoir at time step number t + 1 for Hydrograph Routing	m^3 or ft^3
s	Surface slope of the flow path	m/m or ft/ft
T	Total time along flow path in Time of Concentration Calculation	min
T_{lag}	SCS Lag time	hr
T_p	Time to peak runoff in Hydrograph Methods	hr
t	Time	general
T_c or t_c	Time of concentration	hr
t_n	Travel time over nth reach	min
t_t	Sheet flow travel time	min
V	Velocity	m/s or ft/s
v_n	Estimated flow velocity for nth reach	m/s or ft/s
x	Distance	general
X	Backwater storage effects weighting factor in the Muskingum method	dimensionless
X	Logarithm of the annual peak discharge in Log Pearson Type III Analysis	m^3/s or cfs
ΔT	Time increment	general
ΣI_t	Sum of inflow hydrograph ordinates for Storage-Indication Method	m^3 or ft^3
ΣO_t	Sum of outflow hydrograph ordinates for Storage-Indication Method	m^3 or ft^3

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CHAPTER 7, APPENDIX A

FIELD MANUAL FOR PENNSYLVANIA DESIGN RAINFALL INTENSITY CHARTS FROM NOAA ATLAS 14 VERSION 3 DATA

7A.0 INTRODUCTION

Previously used procedures to estimate design rainfall intensities, usually obtained from the *U.S. Weather Bureau Technical Paper No. 40* (Hershfield, 1961) or the *1986 Field Manual of PennDOT Storm-Intensity-Duration-Frequency Charts PDT-IDF* (Aron et al., 1986), have been updated in this appendix. The regional rainfall design curves in this Pennsylvania field manual were developed from frequency analyses based on hourly records from 278 daily and 139 hourly rainfall gages in Pennsylvania plus gages in surrounding states for a period of record from April 1, 1863 through December 31, 2000. The analysis leading to the design curves is fully described in this Appendix.

In performing the PDT-IDF analysis, it was found that there were regional differences in rainfall patterns between storm durations. For example, the lowest intensities and amounts for the five (5) minute storms are located in north central PA, whereas the lowest intensities and amounts for the twenty-four (24) hour storm are located in western PA. It was determined that one rainfall region map would not adequately represent the rainfall patterns. Therefore, the maps were developed based upon storm duration and frequency as shown in Table 7A.1.

7A.1 PROCEDURE FOR FINDING DESIGN INTENSITY VALUES

A. Objective. To obtain the design rainfall or return periods from 1 to 100-years and durations from 5 minutes to 24 hours and to obtain the 500-year, 24-hour precipitation.

Step 1 Determine the rainfall duration of the storm that will need to be analyzed. For the rational method, the required storm duration will be equal to the time-of-concentration.

Step 2 From Table 7A.1, determine what Rainfall Region Map should be utilized for the design storm duration of interest.

Table 7A.1 Appropriate Rainfall Region Map for each Storm Duration and Frequency

	Frequency							
Duration	**1 year**	**2 year**	**5 year**	**10 year**	**25 year**	**50 year**	**100 year**	**500 year**
5 min	C	C	C	C	B	B	B	-
10 min	C	C	C	C	C	C	C	-
15 min	A	A	A	A	C	C	C	-
30 min	A	A	A	A	A	C	C	-
60 min	A	A	A	A	A	C	C	-
2 hr	E	E	E	E	E	E	E	-
3 hr	E	E	E	E	E	E	E	-
6 hr	D	D	D	D	D	D	D	-
12 hr	F	F	F	F	F	F	F	-
24 hr	F	F	F	F	F	F	F	F

Step 3 Locate the area of interest on the Pennsylvania map for the Map determined in Step 2 (Figures 7A.1 through 7A.6) and note the region into which this area falls.

If a basin should be found to lie on the boundary between two regions, the intensities should be obtained from the two corresponding regional graphs and averaged. In the case that the basin is large enough to be divided into areas A_i and A_j of measurable size in the adjacent regions i and j, a weighted average intensity may be used.

$$I = \frac{I_i A_{i+} I_j A_j}{A_{i+} A_j}$$

Step 4 From the PDT-IDF curves for that region, determine the rainfall intensity.

The rainfall values for the five-minute through six (6) hour storms can be obtained directly from Tables 7A.2(a/b) through 7A.6(a/b) for each of the five regions or from interpolation from the PDT-IDF curves, Figures 7A.7(a/b) through 7A.16(a/b). For the twelve (12) and twenty-four (24) hour storms, the rainfall values can only be obtained directly from Tables 7A.2(a/b) through 7A.6(a/b) for each of the five regions.

Table 7A.2(a) Five (5) minute through twenty-four (24) hour storm totals for Region 1 (Metric).

Region 1								
Rainfall Total								
	1-Yr Storm	2-Yr Storm	5-Yr Storm	10-Yr Storm	25-Yr Storm	50-Yr Storm	100-Yr Storm	500-Yr Storm
Duration (Min)	cm	cm	cm	cm	cm	cm	cm	cm
5	0.70	0.83	1.00	1.13	1.29	1.39	1.47	
10	1.09	1.30	1.56	1.75	1.97	2.11	2.22	
15	1.34	1.59	1.91	2.15	2.44	2.61	2.76	
30	1.77	2.13	2.63	2.99	3.45	3.73	3.99	
60	2.16	2.62	3.30	3.81	4.48	4.92	5.33	
120	2.52	3.03	3.79	4.41	5.28	5.98	6.64	
180	2.77	3.32	4.15	4.82	5.80	6.55	7.34	
360	3.49	4.17	5.17	6.02	7.22	8.11	9.04	
720	4.30	5.14	6.33	7.38	8.93	10.09	11.32	
1440	5.18	6.19	7.59	8.74	10.40	11.80	13.30	17.11

Table 7A.2(b) Five (5) minute through twenty-four (24) hour storm totals for Region 1 (U.S. Customary).

Region 1								
Rainfall Total								
	1-Yr Storm	2-Yr Storm	5-Yr Storm	10-Yr Storm	25-Yr Storm	50-Yr Storm	100-Yr Storm	500-Yr Storm
Duration (Min)	in	in	in	in	in	in	in	in
5	0.28	0.33	0.39	0.45	0.51	0.55	0.58	
10	0.43	0.51	0.61	0.69	0.78	0.83	0.87	
15	0.53	0.63	0.75	0.85	0.96	1.03	1.09	
30	0.70	0.84	1.03	1.18	1.36	1.47	1.57	
60	0.85	1.03	1.30	1.50	1.76	1.94	2.10	
120	0.99	1.19	1.49	1.74	2.08	2.35	2.62	
180	1.09	1.31	1.63	1.90	2.28	2.58	2.89	
360	1.37	1.64	2.04	2.37	2.84	3.19	3.56	
720	1.69	2.02	2.49	2.91	3.52	3.97	4.46	
1440	2.04	2.44	2.99	3.44	4.09	4.65	5.24	6.74

Table 7A.3(a) Five (5) minute through twenty-four (24) hour storm totals for Region 2 (Metric).

Region 2								
Rainfall Total								
	1-Yr Storm	2-Yr Storm	5-Yr Storm	10-Yr Storm	25-Yr Storm	50-Yr Storm	100-Yr Storm	500-Yr Storm
Duration (Min)	cm	cm	cm	cm	cm	cm	cm	cm
5	0.76	0.91	1.08	1.22	1.40	1.52	1.63	
10	1.18	1.42	1.68	1.88	2.14	2.31	2.46	
15	1.45	1.73	2.07	2.32	2.65	2.86	3.06	
30	1.93	2.32	2.84	3.23	3.75	4.10	4.43	
60	2.36	2.87	3.60	4.15	4.88	5.40	5.93	
120	2.77	3.35	4.19	4.87	5.82	6.60	7.46	
180	3.05	3.67	4.59	5.34	6.41	7.29	8.27	
360	3.83	4.60	5.73	6.68	8.03	9.07	10.17	
720	4.73	5.67	7.01	8.20	9.97	11.36	12.84	
1440	5.68	6.80	8.37	9.70	11.67	13.40	15.32	20.70

Table 7A.3(b). Five (5) minute through twenty-four (24) hour storm totals for Region 2 (U.S. Customary).

Region 2								
Rainfall Total								
	1-Yr Storm	2-Yr Storm	5-Yr Storm	10-Yr Storm	25-Yr Storm	50-Yr Storm	100-Yr Storm	500-Yr Storm
Duration (Min)	in	in	in	in	in	in	in	in
5	0.30	0.36	0.43	0.48	0.55	0.60	0.64	
10	0.47	0.56	0.66	0.74	0.84	0.91	0.97	
15	0.57	0.68	0.81	0.91	1.04	1.13	1.20	
30	0.76	0.92	1.12	1.27	1.47	1.61	1.74	
60	0.93	1.13	1.42	1.63	1.92	2.13	2.33	
120	1.09	1.32	1.65	1.92	2.29	2.60	2.94	
180	1.20	1.45	1.81	2.10	2.52	2.87	3.25	
360	1.51	1.81	2.26	2.63	3.16	3.57	4.00	
720	1.86	2.23	2.76	3.23	3.92	4.47	5.06	
1440	2.24	2.68	3.30	3.82	4.60	5.27	6.03	8.15

Table 7A.4(a) Five (5) minute through twenty-four (24) hour storm totals for Region 3 (Metric).

Region 3								
Rainfall Total								
	1-Yr Storm	2-Yr Storm	5-Yr Storm	10-Yr Storm	25-Yr Storm	50-Yr Storm	100-Yr Storm	500-Yr Storm
Duration (Min)	cm	cm	cm	cm	cm	cm	cm	cm
5	0.82	0.98	1.16	1.30	1.50	1.65	1.79	
10	1.28	1.53	1.81	2.02	2.31	2.51	2.70	
15	1.57	1.88	2.22	2.49	2.85	3.11	3.36	
30	2.08	2.51	3.06	3.48	4.04	4.46	4.87	
60	2.56	3.11	3.89	4.48	5.27	5.88	6.53	
120	3.03	3.66	4.59	5.33	6.37	7.23	8.27	
180	3.33	4.02	5.03	5.85	7.02	8.03	9.19	
360	4.17	5.02	6.29	7.34	8.84	10.03	11.30	
720	5.16	6.19	7.69	9.02	11.00	12.62	14.37	
1440	6.19	7.42	9.16	10.66	12.94	14.99	17.35	24.30

Table 7A.4(b). Five (5) minute through twenty-four (24) hour storm totals for Region 3 (U.S. Customary).

Region 3								
Rainfall Total								
	1-Yr Storm	2-Yr Storm	5-Yr Storm	10-Yr Storm	25-Yr Storm	50-Yr Storm	100-Yr Storm	500-Yr Storm
Duration (Min)	in	in	in	in	in	in	in	in
5	0.32	0.39	0.46	0.51	0.59	0.65	0.71	
10	0.50	0.60	0.71	0.80	0.91	0.99	1.06	
15	0.62	0.74	0.88	0.98	1.12	1.22	1.32	
30	0.82	0.99	1.20	1.37	1.59	1.75	1.92	
60	1.01	1.23	1.53	1.77	2.08	2.32	2.57	
120	1.19	1.44	1.81	2.10	2.51	2.85	3.26	
180	1.31	1.58	1.98	2.30	2.77	3.16	3.62	
360	1.64	1.98	2.48	2.89	3.48	3.95	4.45	
720	2.03	2.44	3.03	3.55	4.33	4.97	5.66	
1440	2.44	2.92	3.61	4.20	5.10	5.90	6.83	9.57

Table 7A.5(a) Five (5) minute through twenty-four (24) hour storm totals for Region 4 (Metric).

Region 4								
Rainfall Total								
	1-Yr Storm	2-Yr Storm	5-Yr Storm	10-Yr Storm	25-Yr Storm	50-Yr Storm	100-Yr Storm	500-Yr Storm
Duration (Min)	cm	cm	cm	cm	cm	cm	cm	cm
5	0.89	1.06	1.24	1.39	1.61	1.78	1.95	
10	1.38	1.65	1.93	2.16	2.47	2.71	2.95	
15	1.69	2.02	2.38	2.66	3.06	3.36	3.66	
30	2.24	2.71	3.28	3.72	4.34	4.82	5.31	
60	2.76	3.36	4.18	4.82	5.67	6.36	7.12	
120	3.28	3.98	4.99	5.80	6.91	7.85	9.09	
180	3.61	4.36	5.47	6.37	7.64	8.77	10.11	
360	4.50	5.45	6.85	7.99	9.65	10.99	12.43	
720	5.59	6.72	8.37	9.84	12.03	13.88	15.89	
1440	6.69	8.03	9.94	11.61	14.21	16.59	19.38	27.89

Table 7A.5(b). Five (5) minute through twenty-four (24) hour storm totals for Region 4 (U.S. Customary).

Region 4								
Rainfall Total								
	1-Yr Storm	2-Yr Storm	5-Yr Storm	10-Yr Storm	25-Yr Storm	50-Yr Storm	100-Yr Storm	500-Yr Storm
Duration (Min)	in	in	in	in	in	in	in	in
5	0.35	0.42	0.49	0.55	0.63	0.70	0.77	
10	0.54	0.65	0.76	0.85	0.97	1.07	1.16	
15	0.67	0.79	0.94	1.05	1.21	1.32	1.44	
30	0.88	1.07	1.29	1.47	1.71	1.90	2.09	
60	1.09	1.32	1.65	1.90	2.23	2.51	2.80	
120	1.29	1.57	1.96	2.28	2.72	3.09	3.58	
180	1.42	1.72	2.16	2.51	3.01	3.45	3.98	
360	1.77	2.14	2.70	3.15	3.80	4.33	4.89	
720	2.20	2.65	3.29	3.87	4.74	5.46	6.26	
1440	2.64	3.16	3.91	4.57	5.60	6.53	7.63	10.98

Table 7A.6(a) Five (5) minute through twenty-four (24) hour storm totals for Region 5 (Metric).

Region 5								
Rainfall Total								
	1-Yr Storm	2-Yr Storm	5-Yr Storm	10-Yr Storm	25-Yr Storm	50-Yr Storm	100-Yr Storm	500-Yr Storm
Duration (Min)	cm	cm	cm	cm	cm	cm	cm	cm
5	0.95	1.13	1.32	1.48	1.72	1.91	2.11	
10	1.47	1.76	2.06	2.29	2.64	2.91	3.19	
15	1.81	2.16	2.53	2.82	3.27	3.61	3.96	
30	2.40	2.90	3.49	3.97	4.63	5.18	5.76	
60	2.96	3.61	4.47	5.15	6.06	6.84	7.72	
120	3.54	4.30	5.39	6.26	7.45	8.48	9.90	
180	3.90	4.71	5.92	6.89	8.25	9.51	11.03	
360	4.84	5.87	7.40	8.65	10.46	11.95	13.56	
720	6.02	7.25	9.04	10.66	13.07	15.14	17.42	
1440	7.20	8.64	10.73	12.57	15.49	18.19	21.40	31.49

Table 7A.6(b). Five (5) minute through twenty-four (24) hour storm totals for Region 5 (U.S. Customary).

Region 5								
Rainfall Total								
	1-Yr Storm	2-Yr Storm	5-Yr Storm	10-Yr Storm	25-Yr Storm	50-Yr Storm	100-Yr Storm	500-Yr Storm
Duration (Min)	in	in	in	in	in	in	in	in
5	0.37	0.45	0.52	0.58	0.68	0.75	0.83	
10	0.58	0.69	0.81	0.90	1.04	1.15	1.26	
15	0.71	0.85	1.00	1.11	1.29	1.42	1.56	
30	0.94	1.14	1.37	1.56	1.82	2.04	2.27	
60	1.17	1.42	1.76	2.03	2.39	2.69	3.04	
120	1.39	1.69	2.12	2.46	2.93	3.34	3.90	
180	1.53	1.86	2.33	2.71	3.25	3.75	4.34	
360	1.91	2.31	2.91	3.40	4.12	4.70	5.34	
720	2.37	2.86	3.56	4.20	5.15	5.96	6.86	
1440	2.83	3.40	4.22	4.95	6.10	7.16	8.43	12.40

1. Example. Determine the 10-year rainfall intensity for a drainage area in eastern Schuylkill County that has a time-of-concentration (T_c) of 12 minutes.

Step 1 The duration would be equal to the time-of-concentration or 12 minutes

Step 2 Since 12 minutes is less than half way between the midpoint between the 10- and 15-minute increment in Table 7A.1, the Map corresponding to the 10-minute duration would be utilized, or Map C. If the T_c value falls between values shown in Table 7A.1, one should round to the nearest value in the Table. For example, if a 14-minute T_c was obtained, the map shown for the 15-minute duration should be utilized, or Map A. A value of 12.5 would also be rounded to 15 minutes and Map A chosen. Note that rounding is only necessary for map determination. The original T_c value should be utilized to obtain the rainfall intensity.

Step 3 From Map C, the site would fall in Region 4.

Step 4 From the PDT-IDF curve for Region 4, the 10-year, 12-minute storm would be 12.2 cm per hour (4.8 in per hour).

NOTE: EQUATION 7.9 FOR THE RATIONAL METHOD IN METRIC UNITS USES mm/hr AS THE RAINFALL UNIT, NOT cm/hr. CHARTS IN APPENDIX 7A ARE IN UNITS OF cm/hr. THE RAINFALL INTENSITIES DERIVED FROM APPENDIX 7A WILL NEED TO BE CONVERTED FROM cm/hr TO mm/hr FOR USE IN THE RATIONAL METHOD, METRIC UNITS.

B. Composite Design Storms. In storm runoff modeling, design storms are often needed which are not only appropriate for the entire watershed, but for individual subareas as well. Design storms are typically classified by Average Recurrence Interval (ARI), which is established based upon the frequency of an event or how often an event is statistically likely to occur. The ARI is inversely equivalent to the probability that an event will occur in any given year. For instance, a 100-storm is inversely equivalent to 0.01 which means this event has a 1% chance of occurring in any given year. Similarly, the 25-year storm has is inversely equivalent to 0.04 which means this event has a 4% probability of occurring in any given year. A complete design storm, constructed as described below, will demonstrate the procedures used to obtain precipitation estimates for a given ARI which can then be used for design and analysis.

The composite design storm should be generated such that the maximum rain falling over any time span (centered around the storm peak), equals the design storm depth indicated for the corresponding durations.

As an example, a 10-year design storm for the same site will be constructed for a location in Map C, Region 4 for the 10-minute duration and Map A, Region 5 for the 20 through 80 minute durations. The storm is to be defined in 10-minute intervals, for a total duration of 80 minutes. The storm peak shall be placed at or right after the center of the storm and that the storm shape be approximately symmetrical.

Step 1 From Figures 7A.13(b) and 7A.14(b), obtain 10-year rainfall amounts for the 10 minute duration and from Figures 7A.15(b) and 7A.16(b), obtain 10-year rainfall amounts for the 20-… 80-minute durations (the appropriate map and region), and list them as in column (2) in Tables 7A.7(a) and 7A.7(b).

Step 2 Compute the incremental rainfall amounts between consecutive durations. Enter into column (3).

Step 3 Rearrange the rainfall increments from column (3) to column (4) in a quasi-symmetrical pattern, moving first and largest storm interval time to 40-50 minutes, the second to 30-40 minutes, the third to 50-60 minutes and so forth.

Step 4 Compute the rain intensities during the time increments, dividing column (4) by the time step (10 minutes) and multiplying by 60 minutes. Enter the intensities into column (5) and plot the hyetograph.

Table 7A.7(a) Composite 10-year storm in Region 5 (Metric).

(1) Storm duration (minutes) and Map	(2) Rainfall amount (cm)	(3) Rainfall increments (cm)	(4) Rearranged rainfall inc. (cm)	(5) Rearranged rainfall int. (cm/hour)
0	0.00			
		2.16	0.14	0.90
10 Map C*	2.16			
		1.02	0.35	2.10
20 Map A	3.18			
		0.79	0.42	2.52
30 Map A	3.97			
		0.42	1.02	6.12
40 Map A	4.39			
		0.41	2.16	12.96
50 Map A	4.80			
		0.35	0.79	4.74
60 Map A	5.15			
		0.29	0.41	2.46
70 Map A	5.44			
		0.14	0.29	1.74
80 Map A	5.58			

* For region 4

Table 7A.7(b). Composite 10-year storm in Region 5 (U.S. Customary).

(1) Storm duration (minutes) and Map	(2) Rainfall amount (in)	(3) Rainfall increments (in)	(4) Rearranged rainfall inc. (in)	(5) Rearranged rainfall int. (in/hour)
0	0.00			
		0.85	0.06	0.36
10 Map C*	0.85			
		0.40	0.14	0.84
20 Map A	1.25			
		0.31	0.17	1.02
30 Map A	1.56			
		0.17	0.40	2.40
40 Map A	1.73			
		0.16	0.85	5.10
50 Map A	1.89			
		0.14	0.31	1.86
60 Map A	2.03			
		0.11	0.16	0.96
70 Map A	2.14			
		0.06	0.11	0.66
80 Map A	2.20			

* For region 4

Figure 7A.1 Map A. 15-, 30- and 60-minute durations for storms occurring with an ARI of 1-, 2-, 5-, 10-years and 30- and 60-minute durations for storms occurring with an ARI of 25-years.

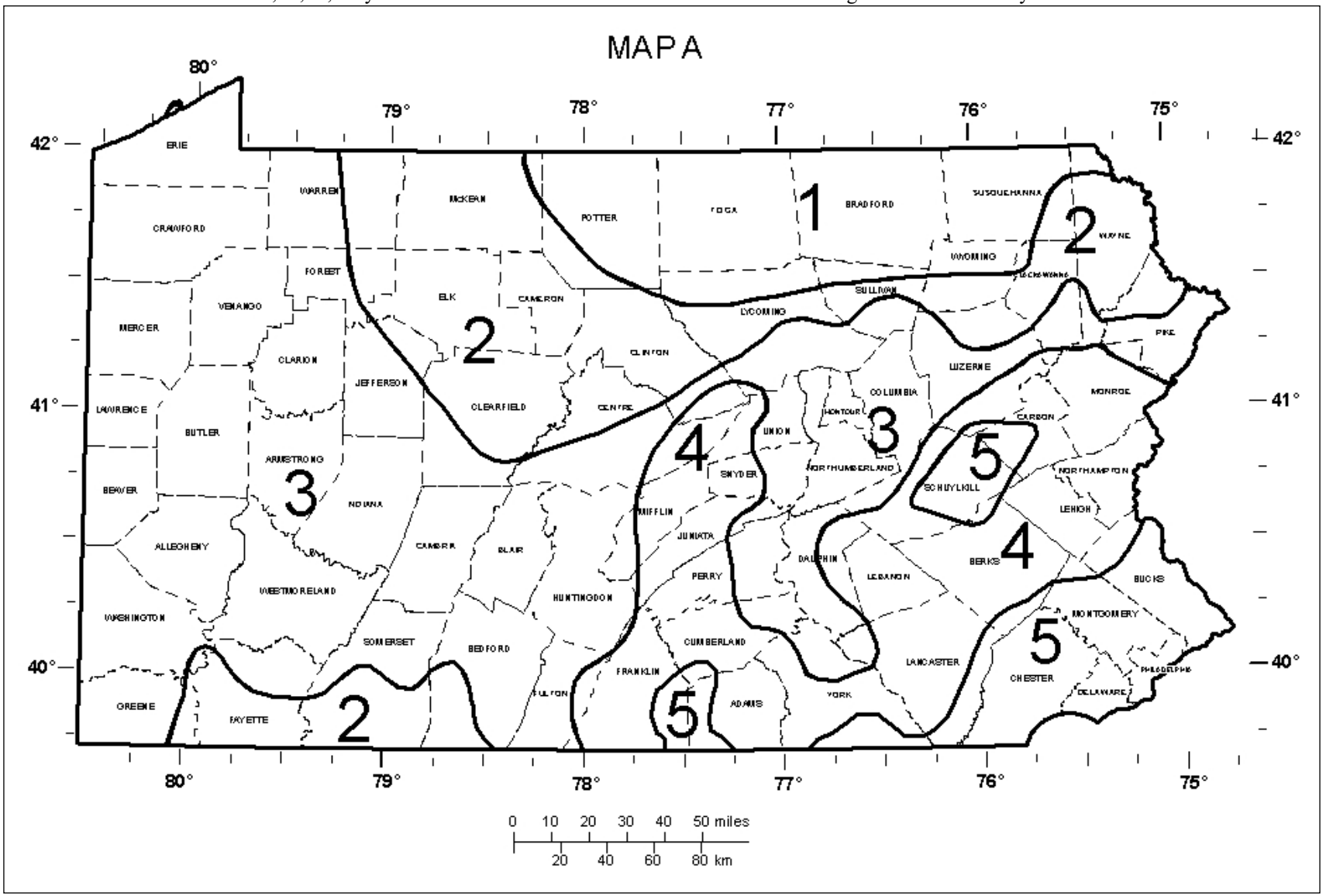


Figure 7A.2 Map B. 5-minute durations for storms occurring with an ARI of 25-, 50- and 100-years.

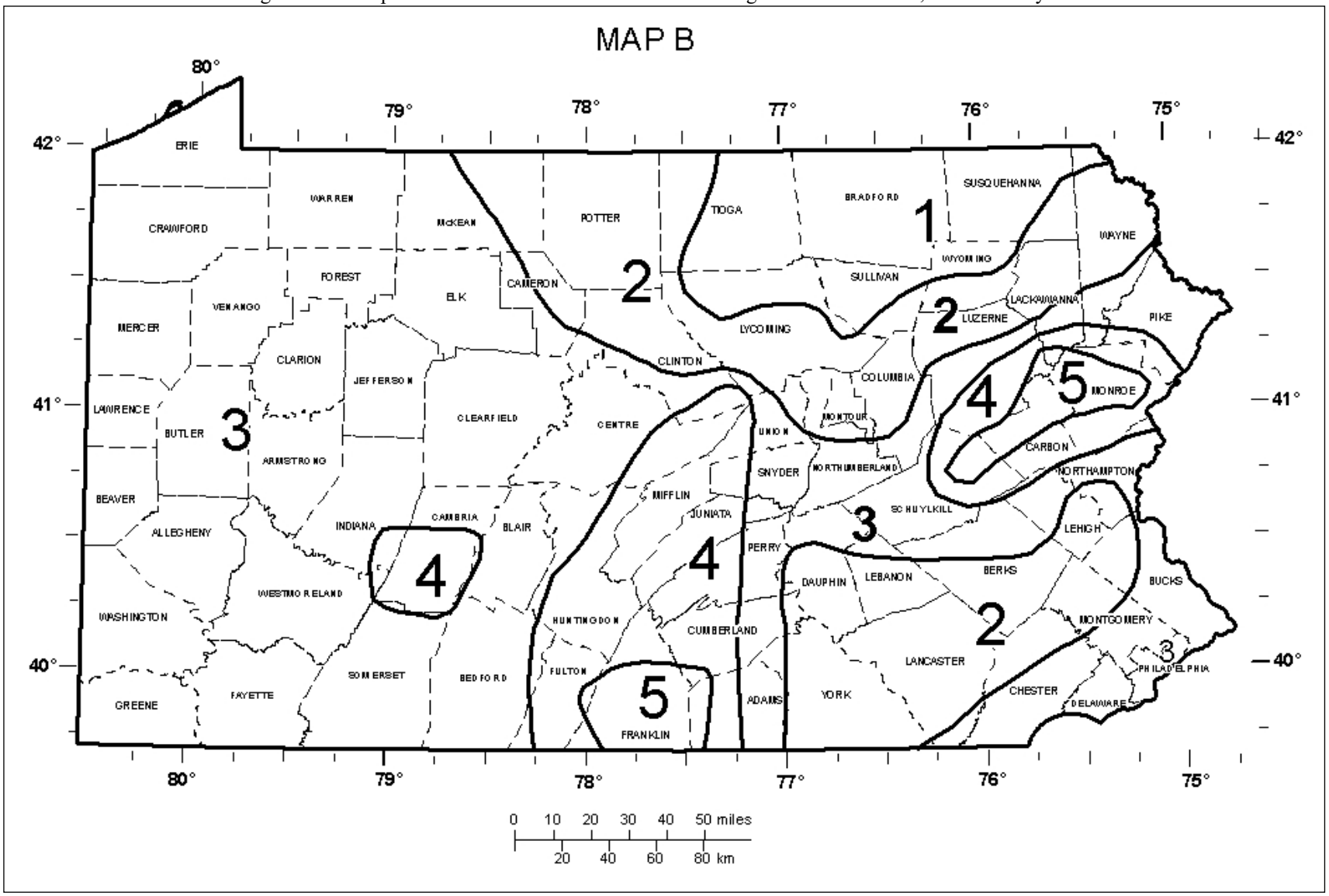


Figure 7A.3 Map C. 5- and 10-minute durations for storms occurring with an ARI of 1-, 2-, 5-, and 10-years, 10- and 15-minute durations for storms occurring with an ARI of 25-years and 10-, 15-, 30-, 60-minute durations for storms occurring with an ARI of 50- and 100-years.

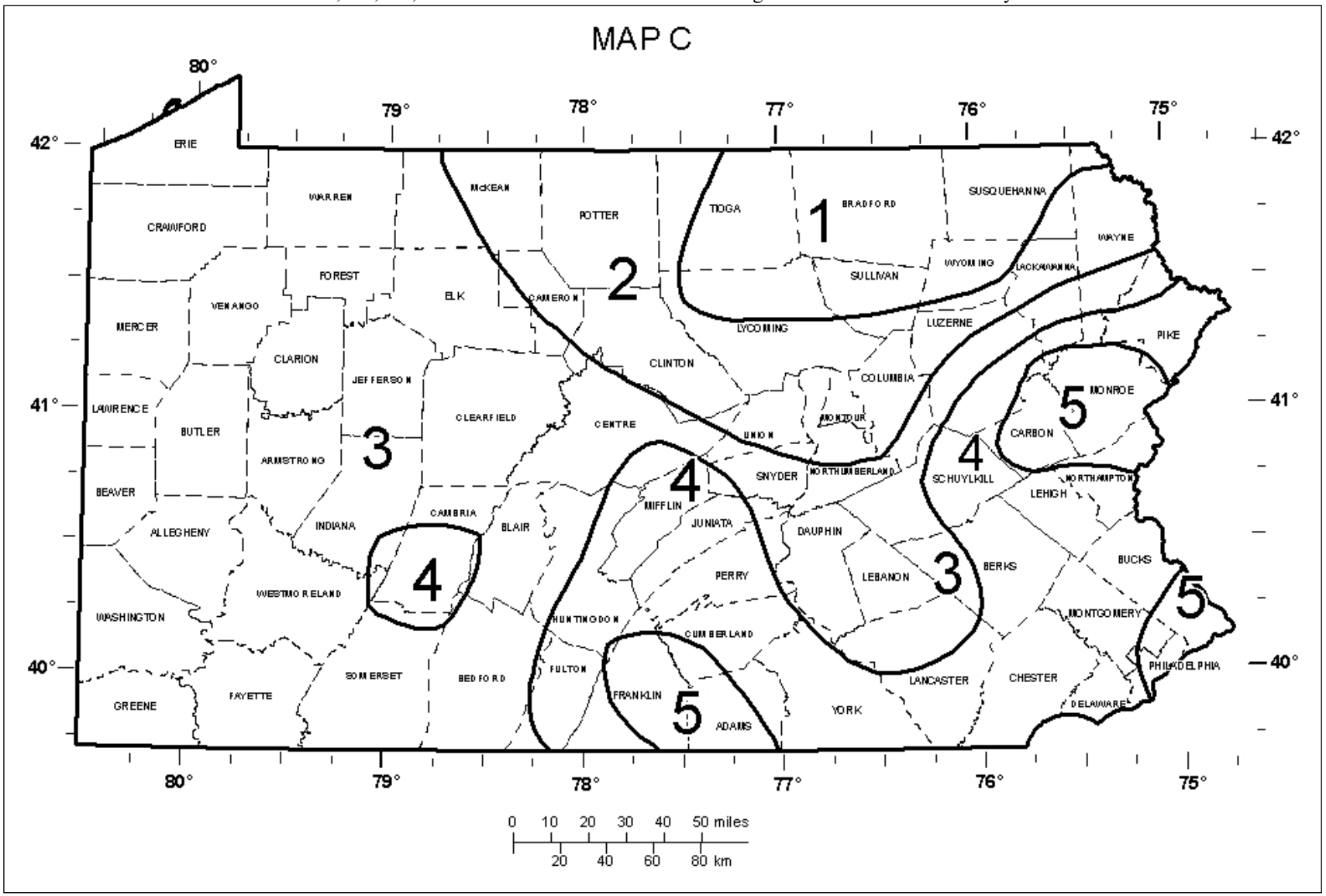


Figure 7A.4 Map D. 6-hour durations for storms occurring with an ARI of 1-, 2-, 5-, 10-, 25-, 50- and 100-years.

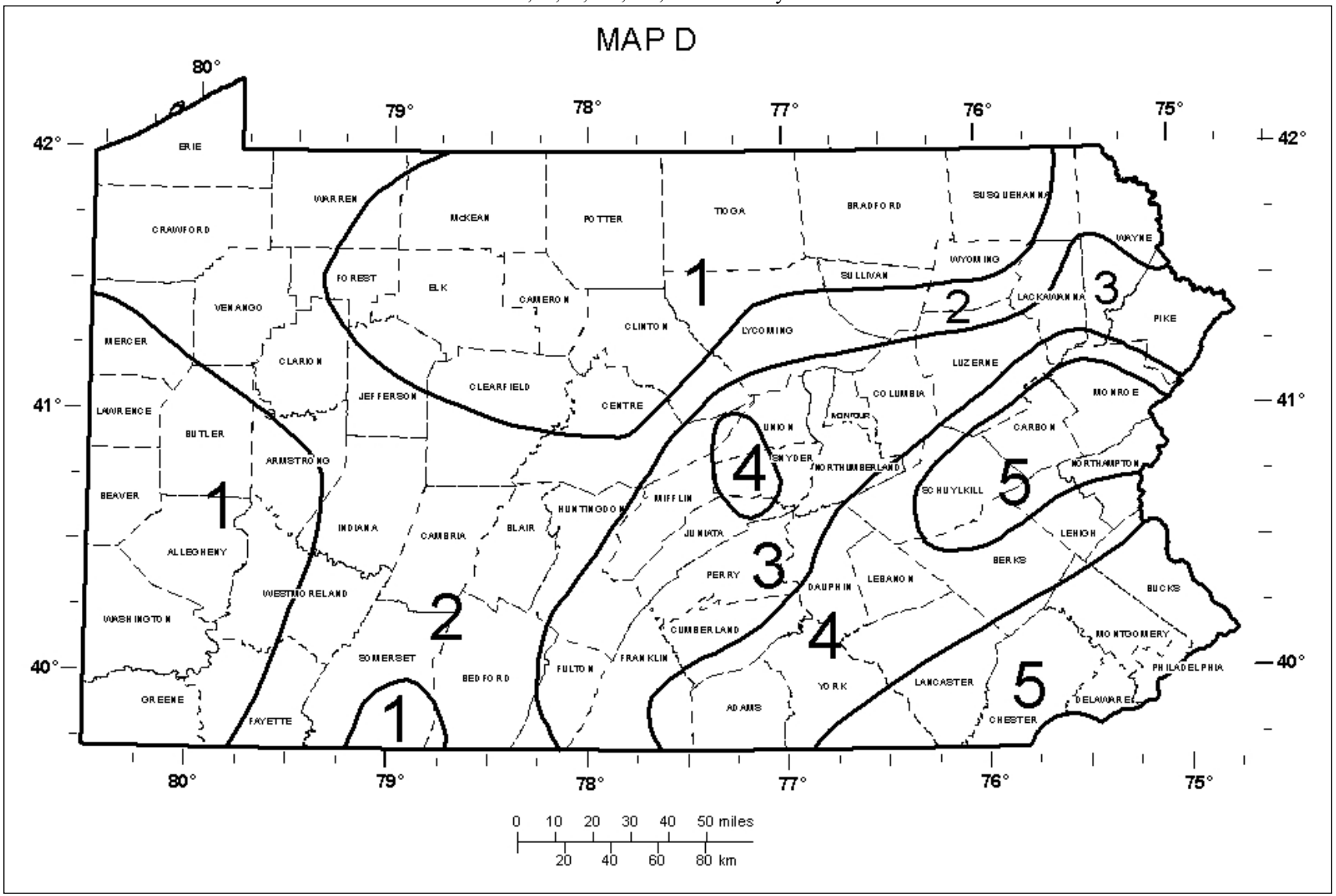


Figure 7A.5 Map E. 2- and 3-hour durations for storms occurring with an ARI of 1-, 2-, 5-, 10-, 25-, 50- and 100-years.

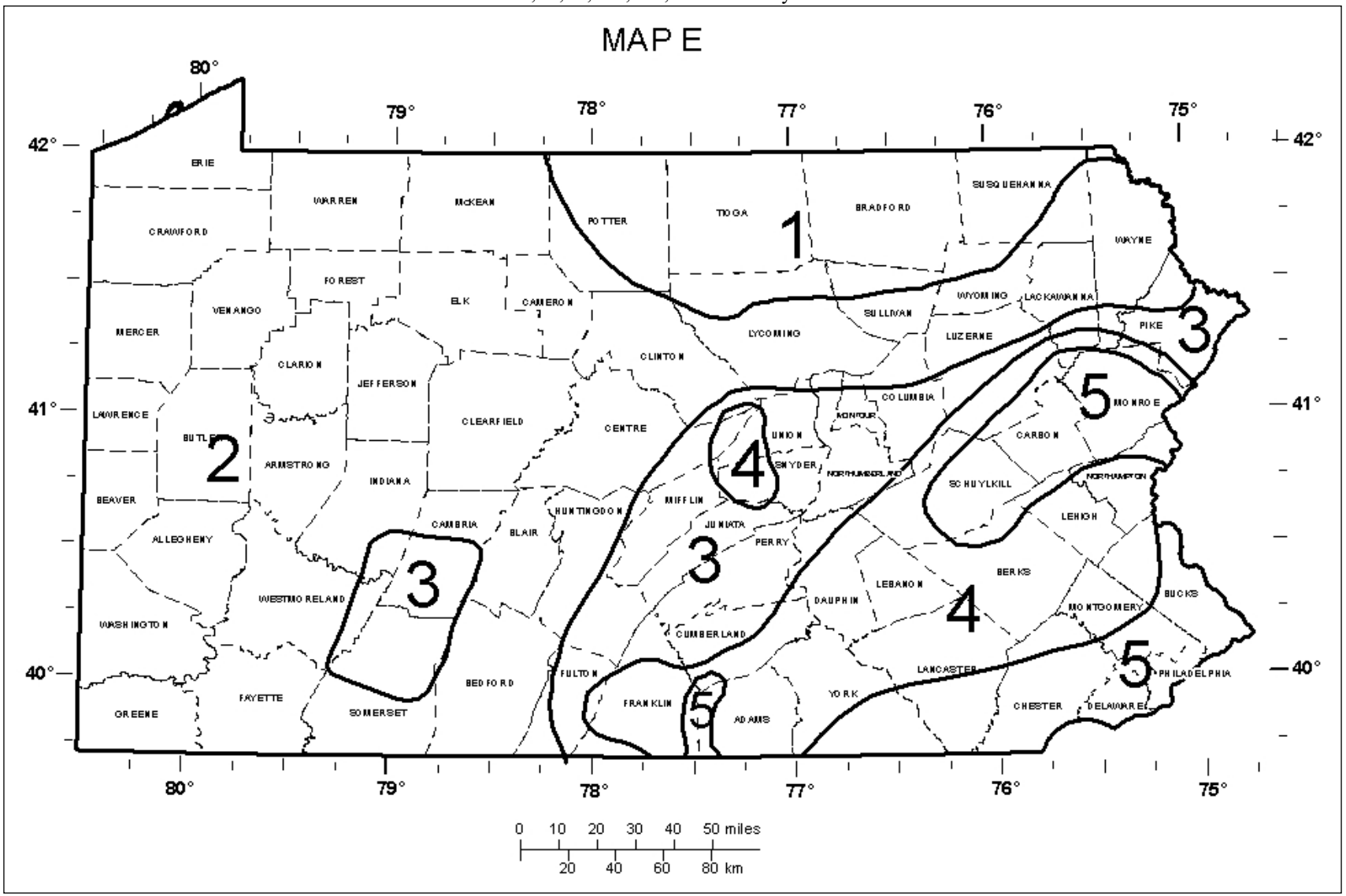


Figure 7A.6 Map F. 12- and 24-hour durations for storms occurring with an average recurrence interval (ARI) of 1-, 2-, 5-, 10-, 25-, 50-, and 100-years and the 24-hour duration for the 500-year frequency storm.

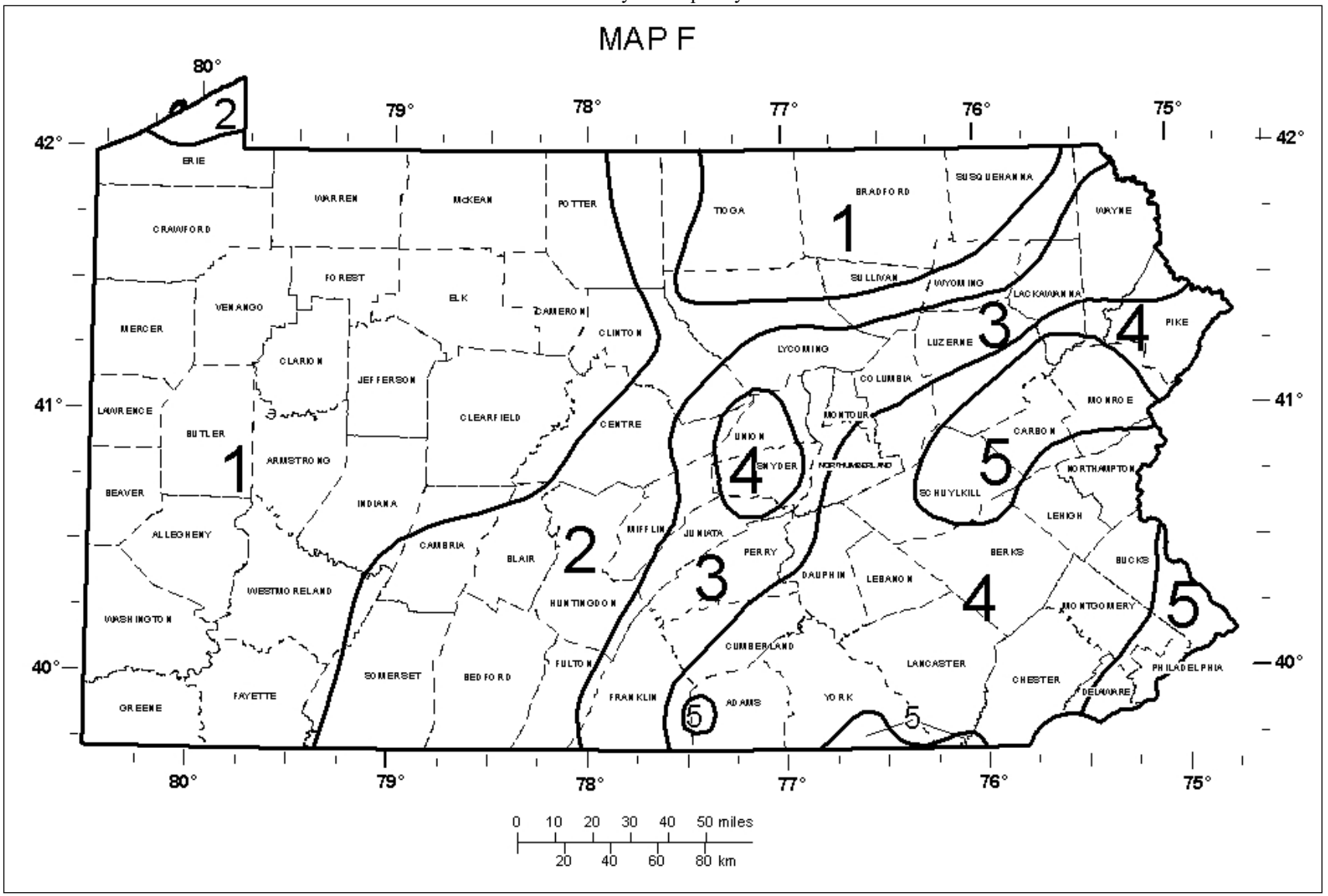


Figure 7A.7(a) Rainfall Intensity for 1- through 100-year Storms for Region 1 (Metric).

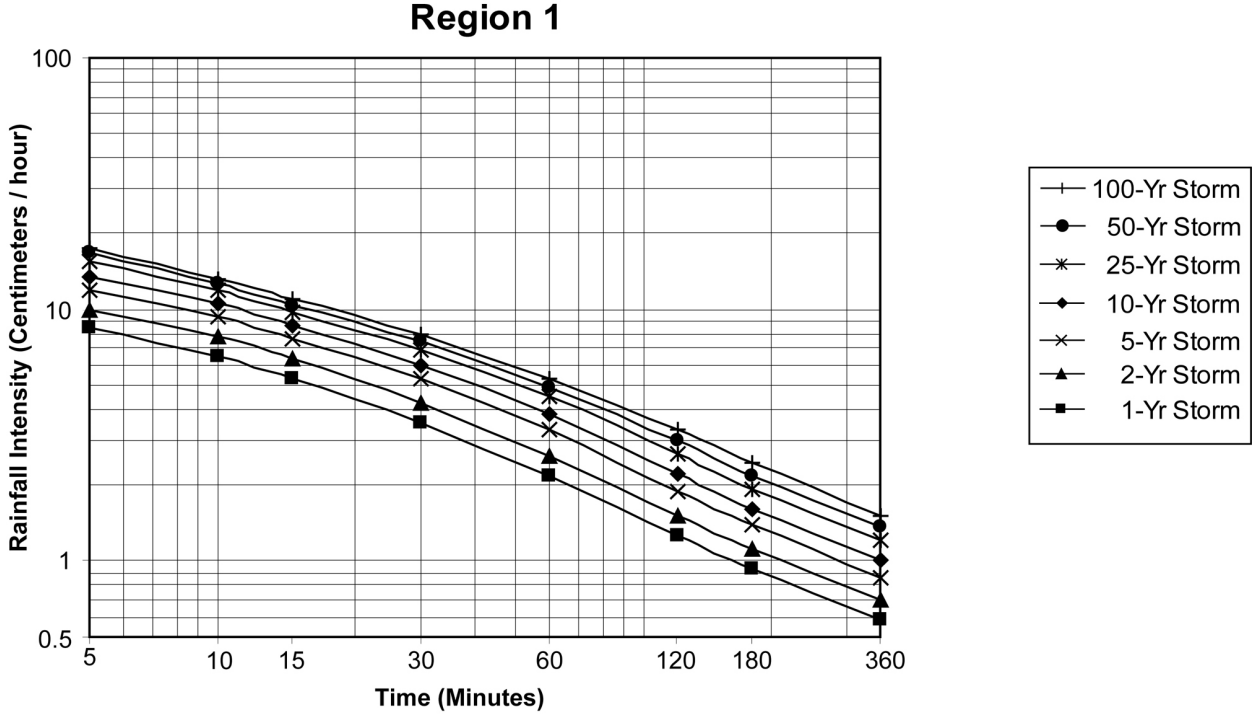


Figure 7A.7(b) Rainfall Amount for 1- through 100-year Storms for Region 1 (Metric).

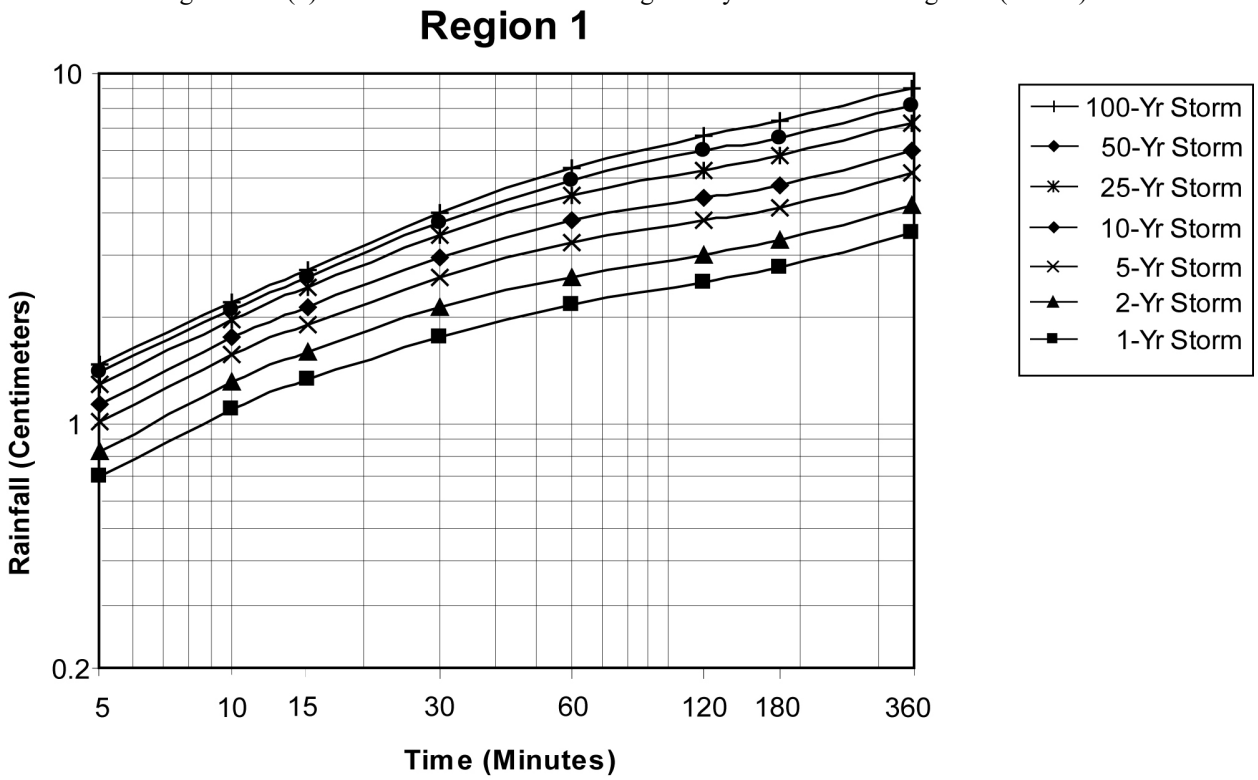


Figure 7A.8(a) Rainfall Intensity for 1- through 100-year Storms for Region 1 (U.S. Customary).

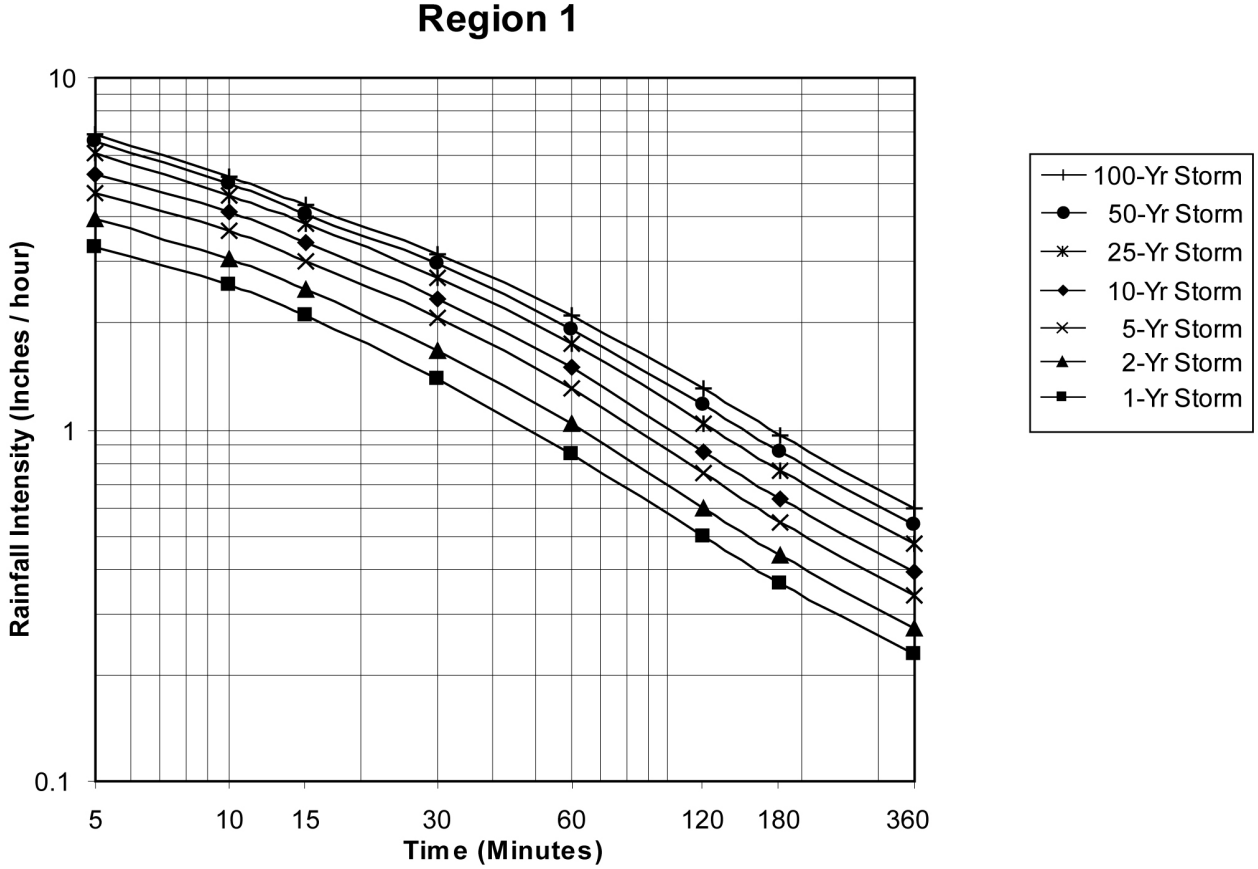


Figure 7A.8(b) Rainfall Amount for 1- through 100-year Storms for Region 1 (U.S. Customary).

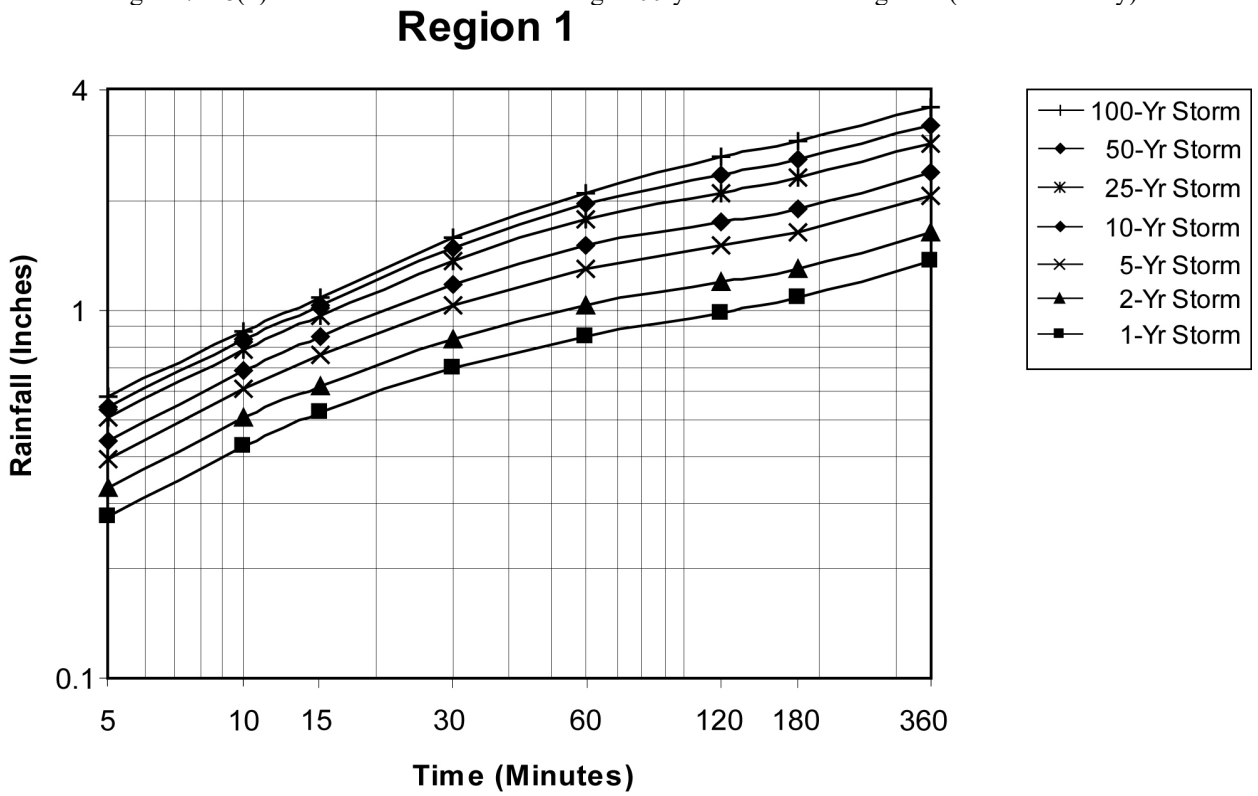


Figure 7A.9(a) Rainfall Intensity for 1- through 100-year Storms for Region 2 (Metric).

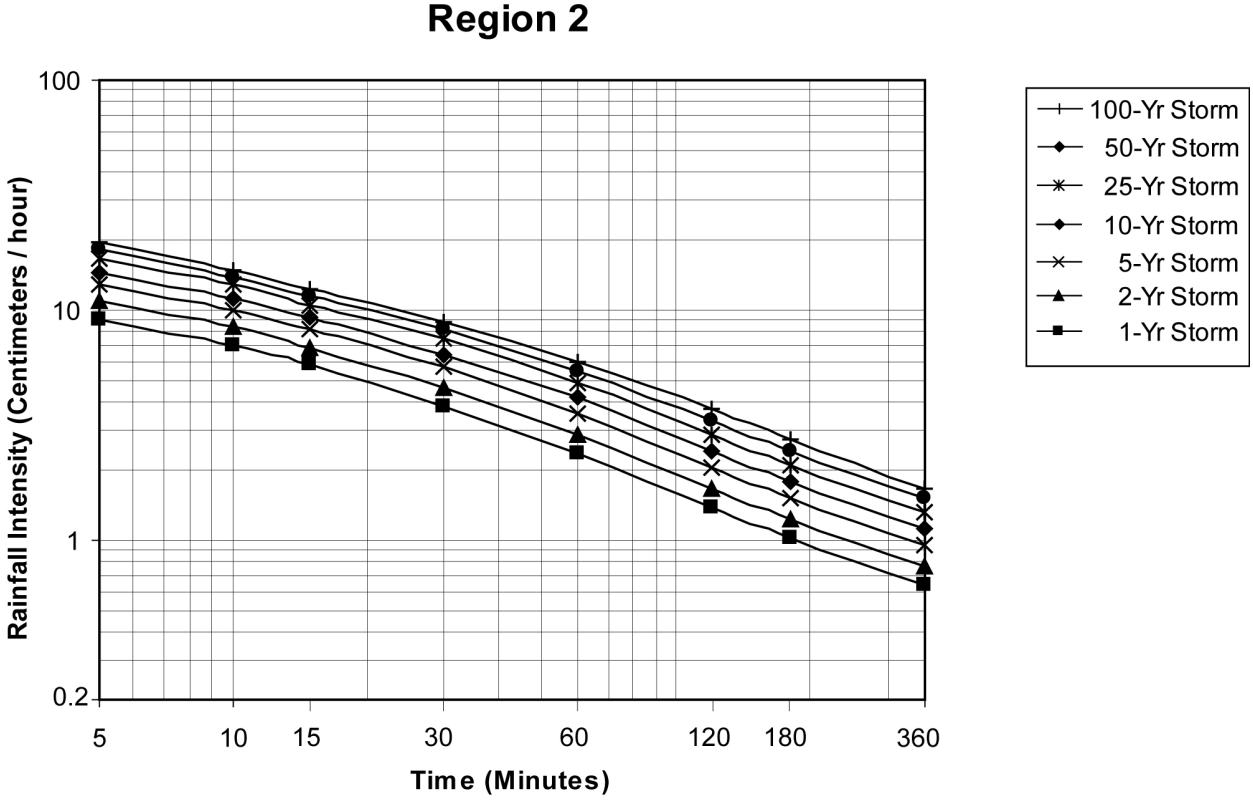


Figure 7A.9(b) Rainfall Amount for 1- through 100-year Storms for Region 2 (Metric).

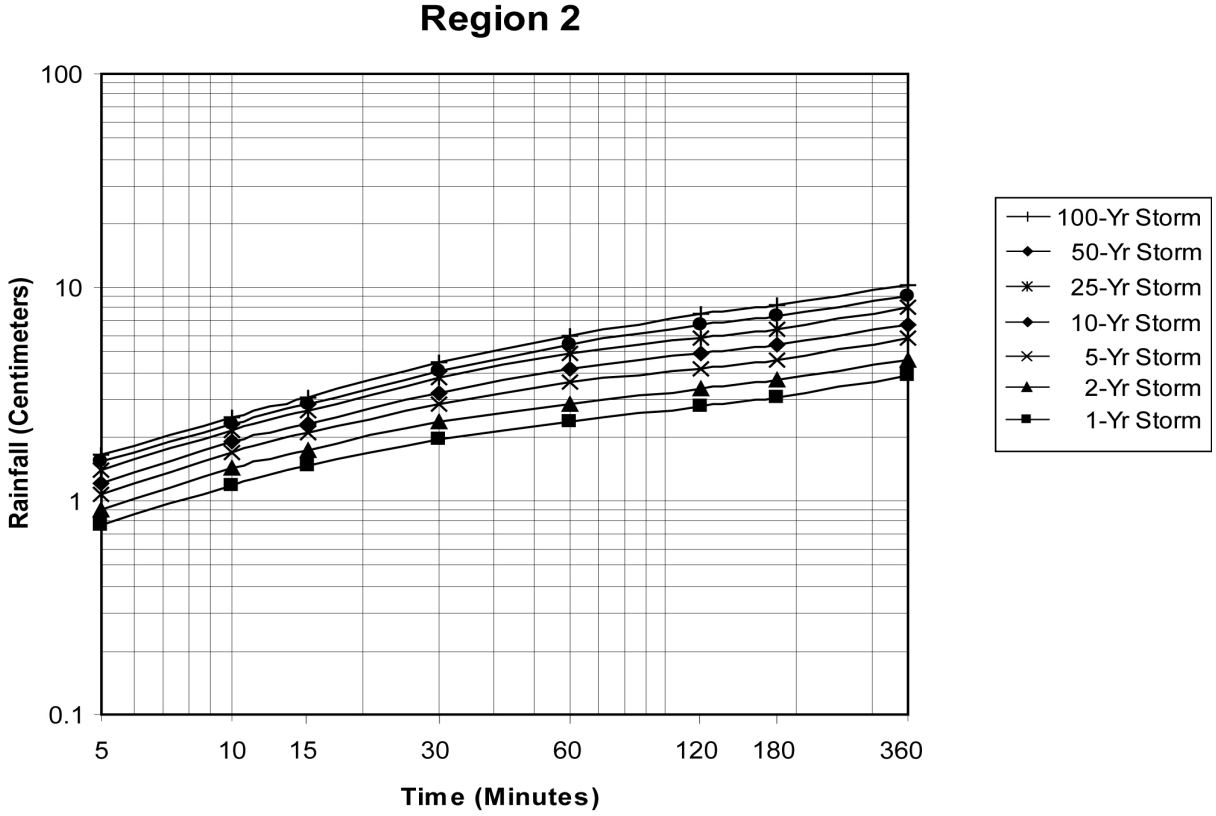


Figure 7A.10(a) Rainfall Intensity for 1- through 100-year Storms for Region 2 (U.S. Customary).

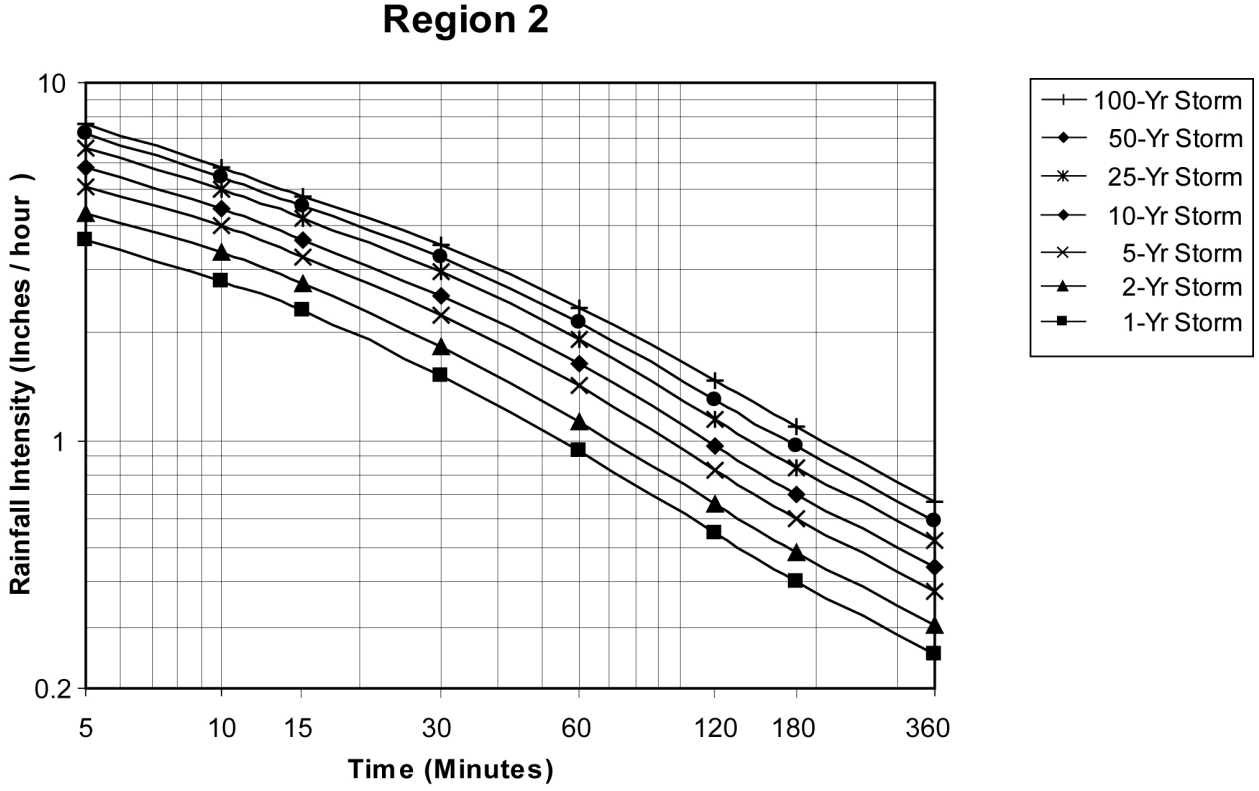


Figure 7A.10(b) Rainfall Amount for 1- through 100-year Storms for Region 2 (U.S. Customary).

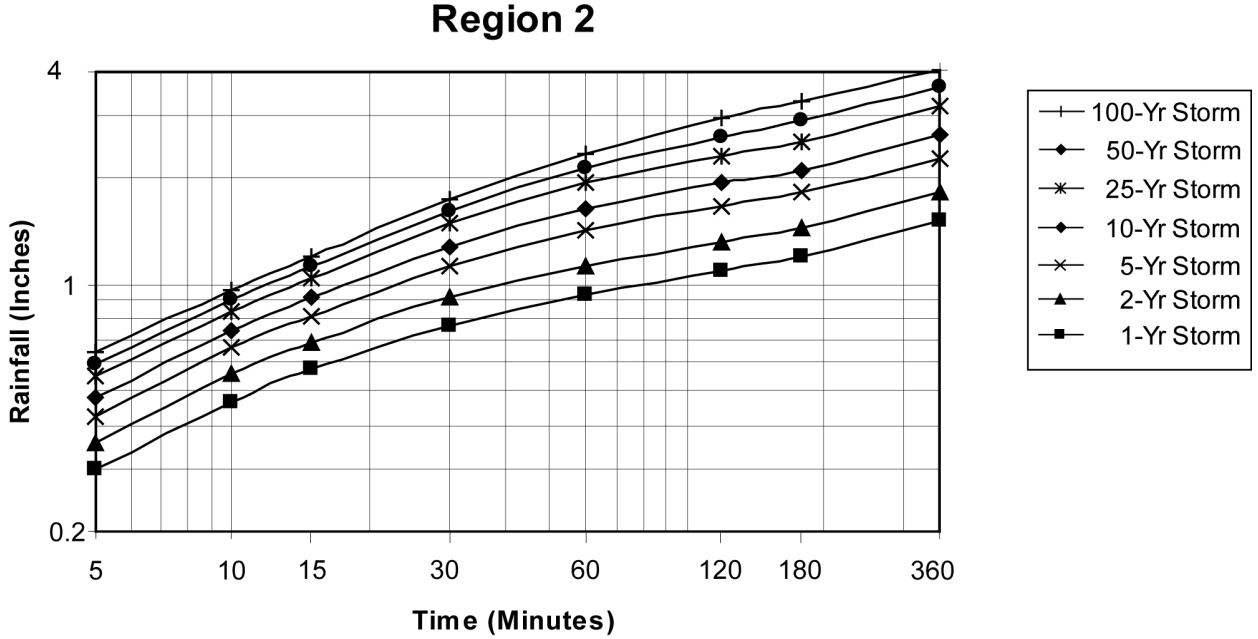


Figure 7A.11(a) Rainfall Intensity for 1- through 100-year Storms for Region 3 (Metric).

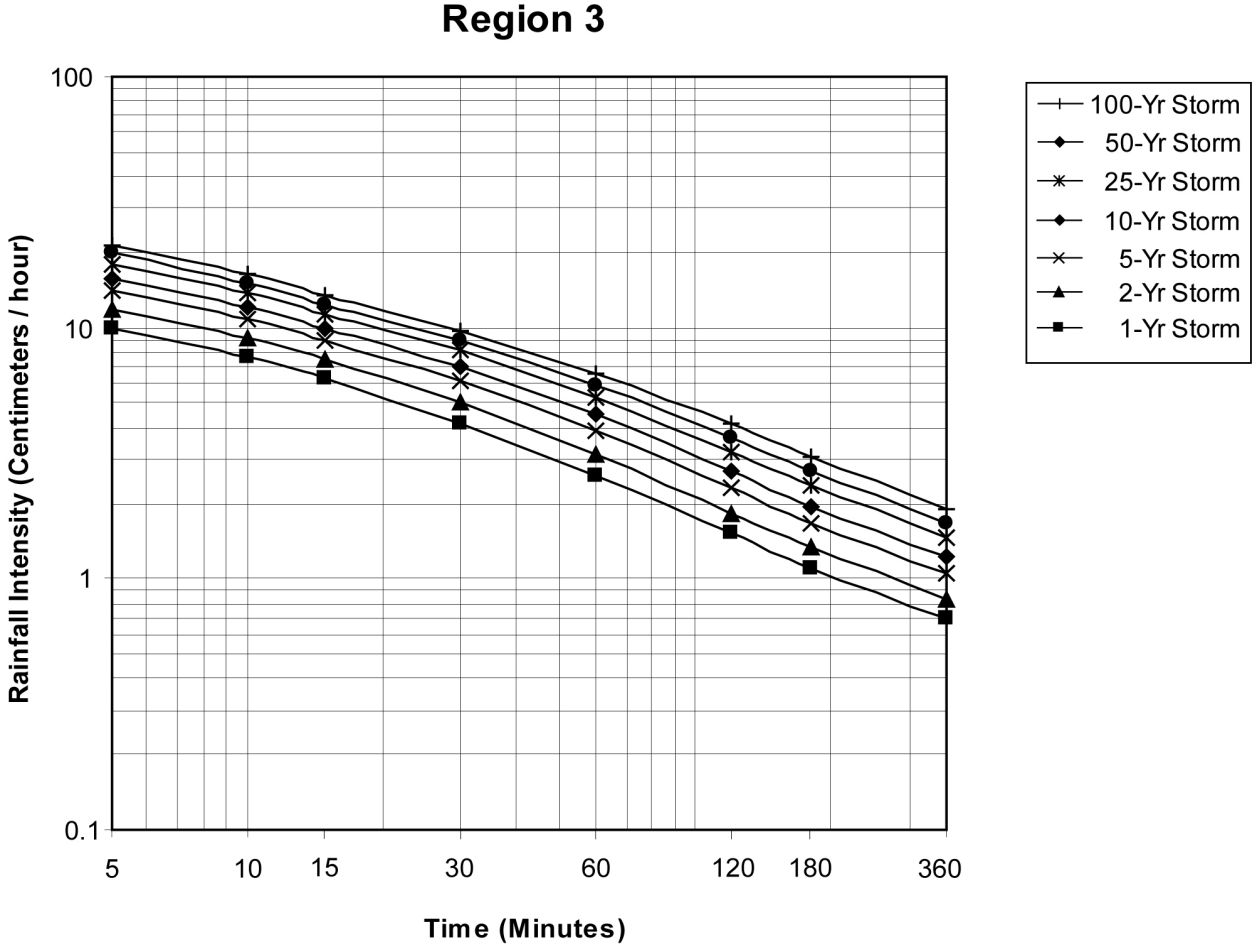


Figure 7A.11(b) Rainfall Amount for 1- through 100-year Storms for Region 3 (Metric).

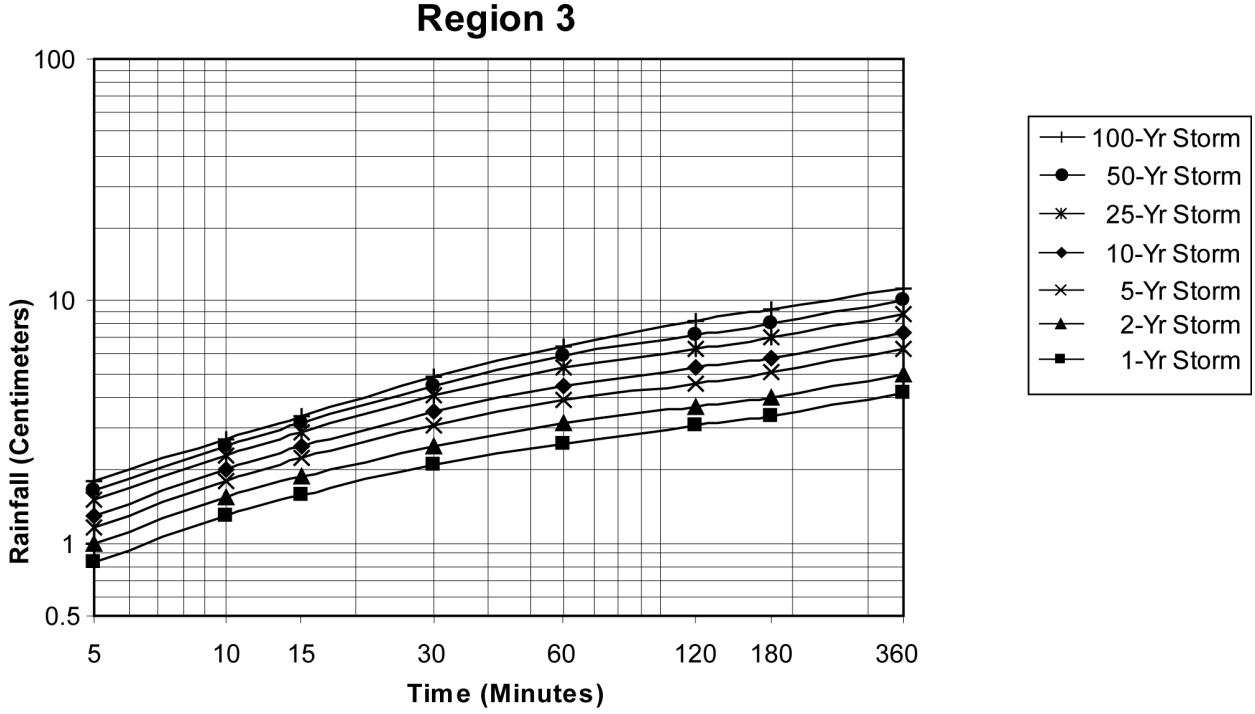


Figure 7A.12(a) Rainfall Intensity for 1- through 100-year Storms for Region 3 (U.S. Customary).

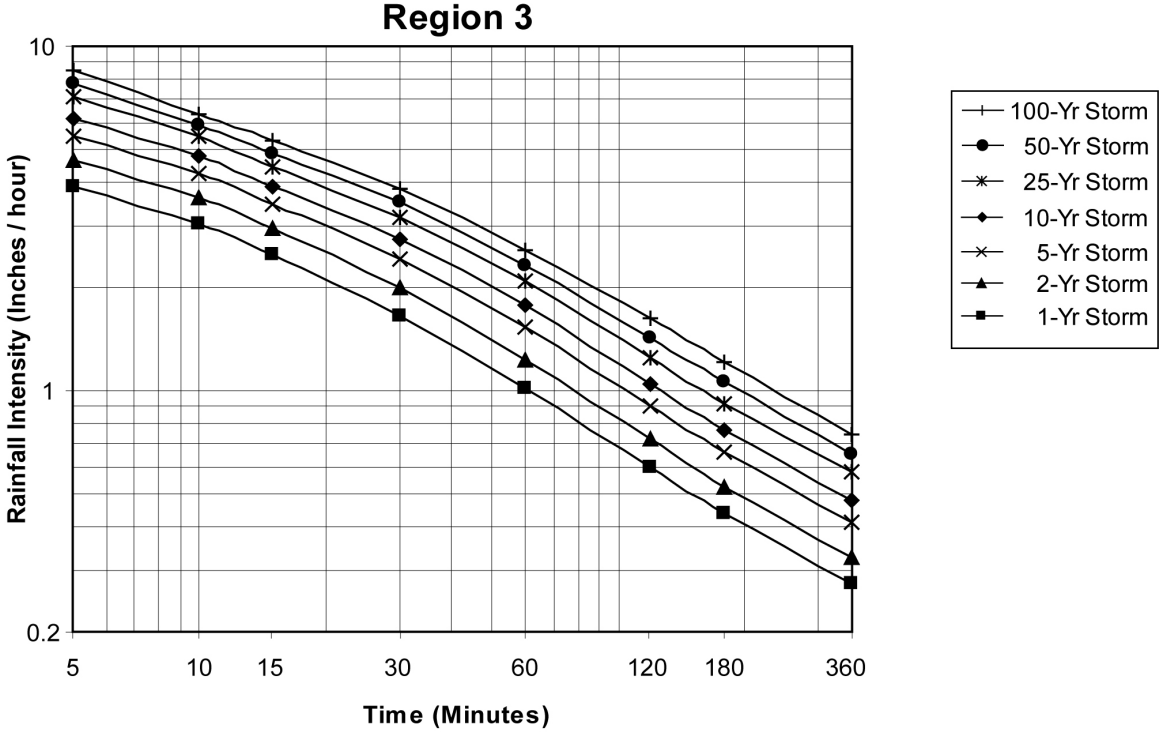


Figure 7A.12(b) Rainfall Amount for 1- through 100-year Storms for Region 3 (U.S. Customary).

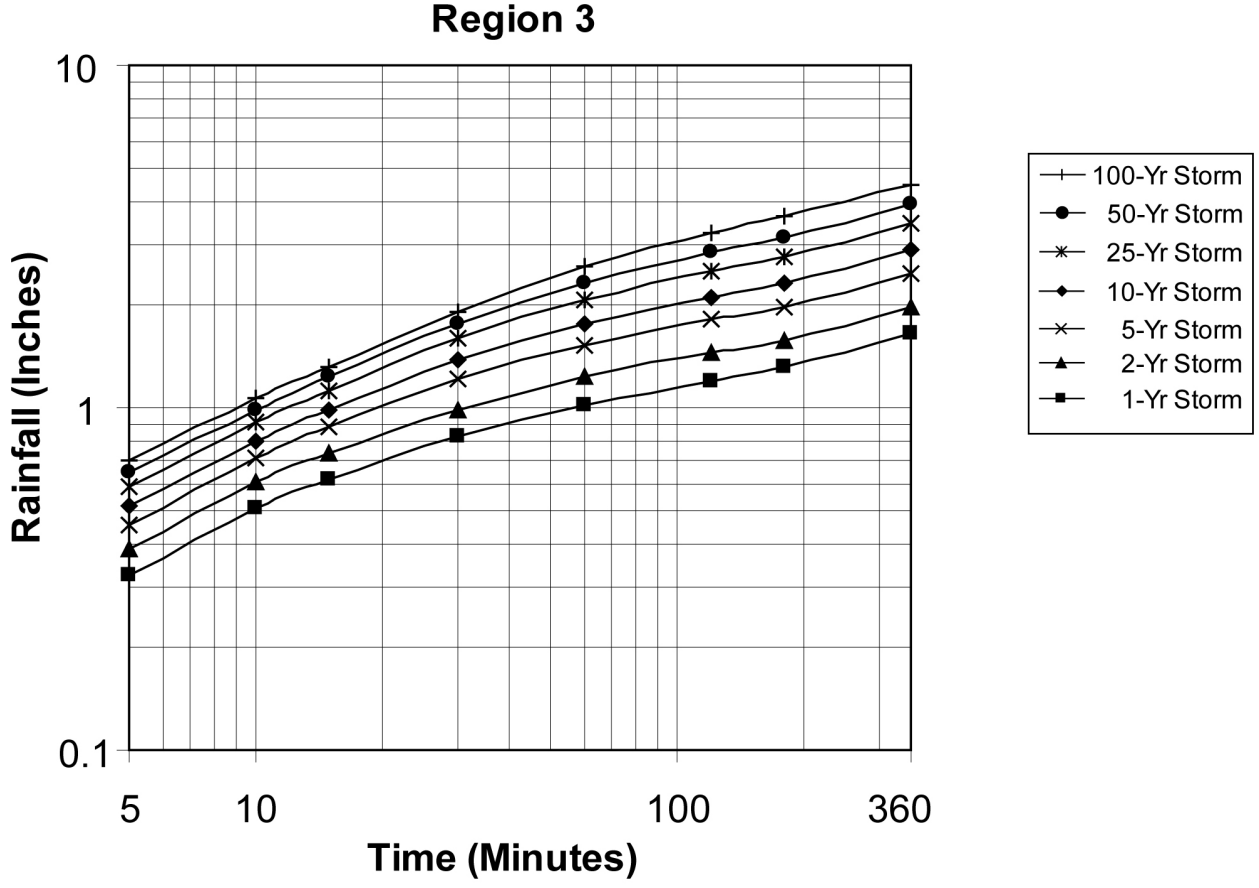


Figure 7A.13(a) Rainfall Intensity for 1- through 100-year Storms for Region 4 (Metric).

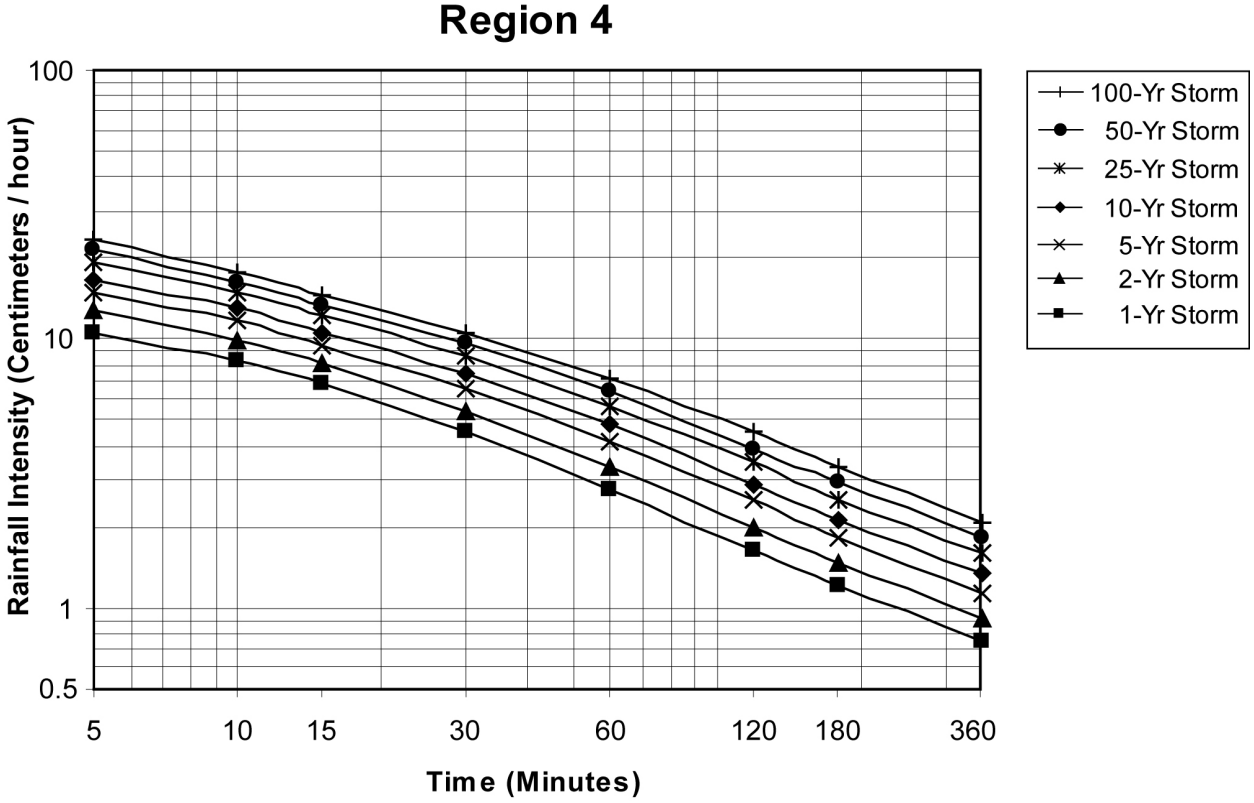


Figure 7A.13(b) Rainfall Amount for 1- through 100-year Storms for Region 4 (Metric).

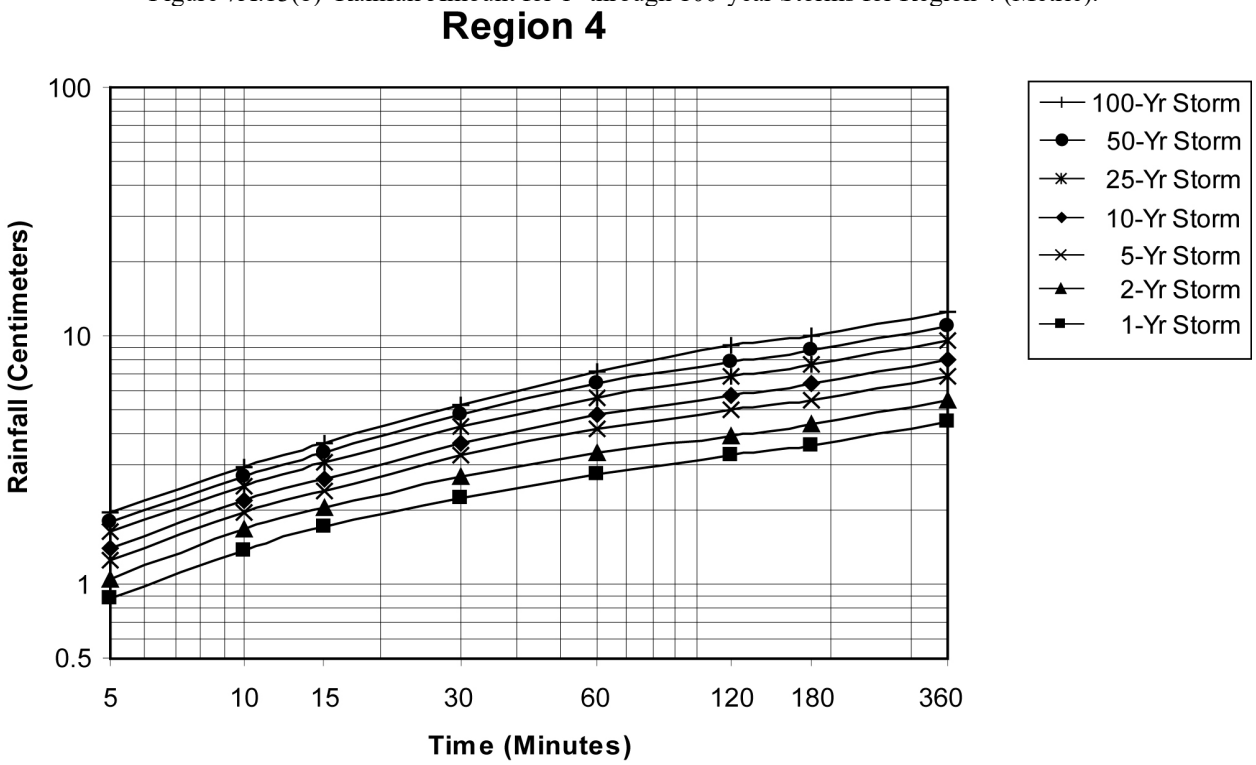


Figure 7A.14(a) Rainfall Intensity for 1- through 100-year Storms for Region 4 (U.S. Customary).

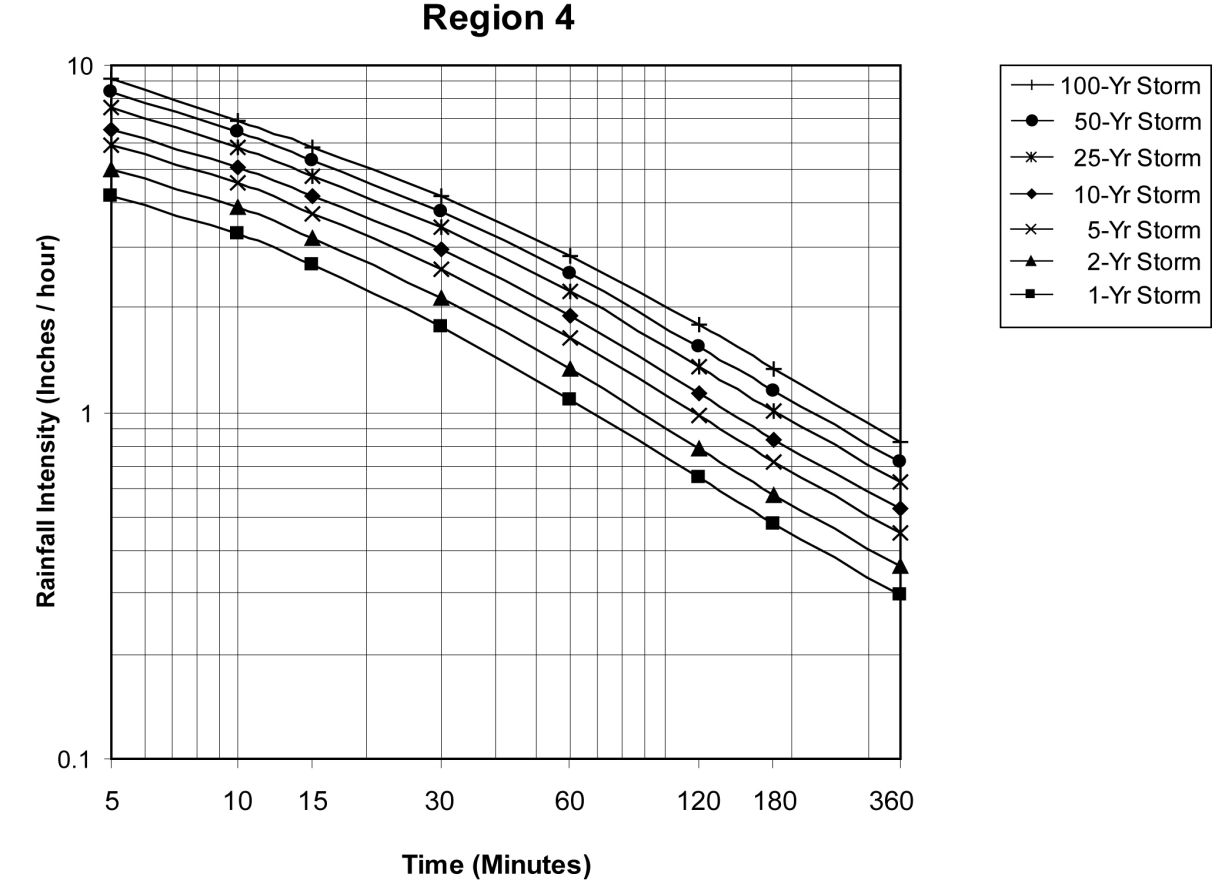


Figure 7A.14(b) Rainfall Amount for 1- through 100-year Storms for Region 4 (U.S. Customary).

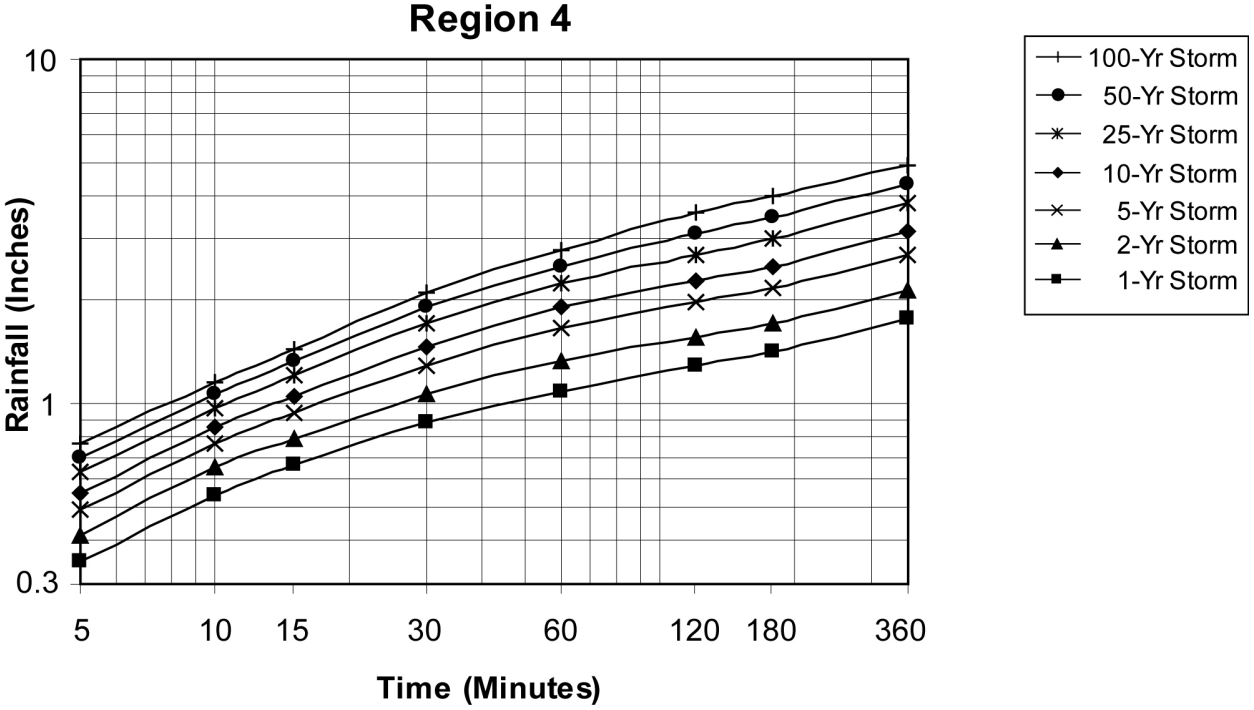


Figure 7A.15(a) Rainfall Intensity for 1- through 100-year Storms for Region 5 (Metric).

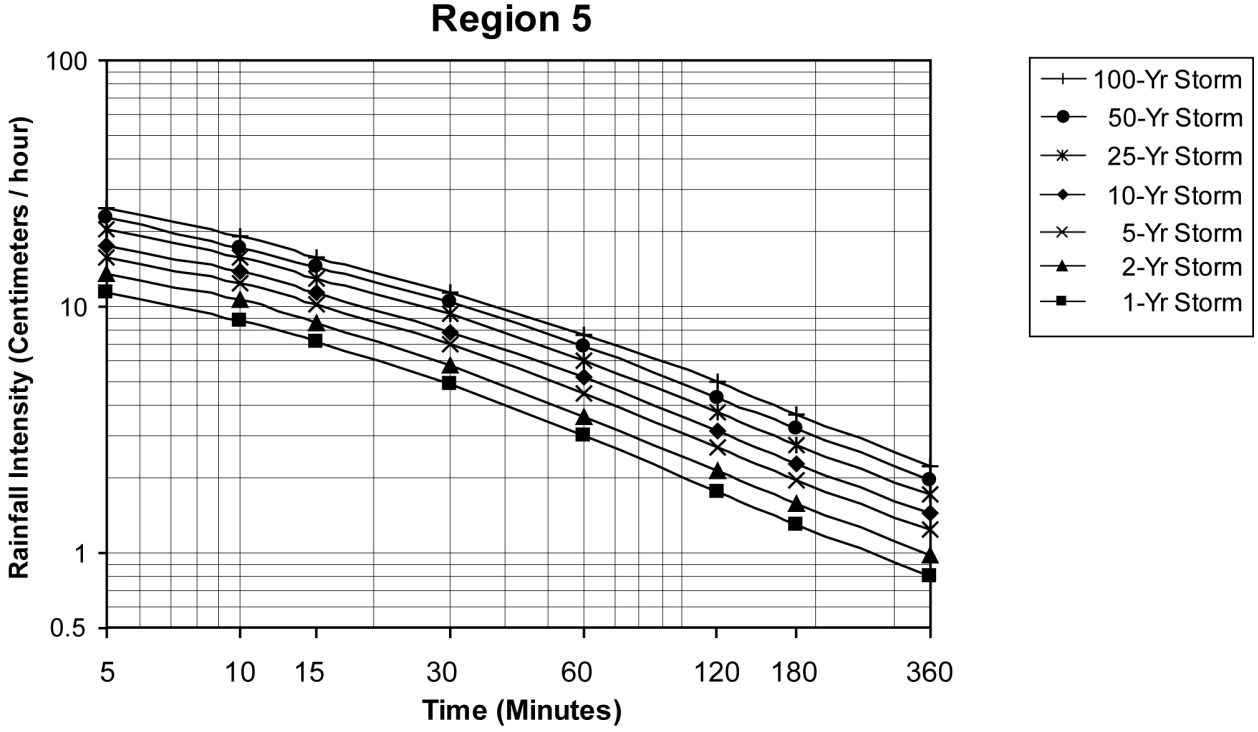


Figure 7A.15(b) Rainfall Amount for 1- through 100-year Storms for Region 5 (Metric).

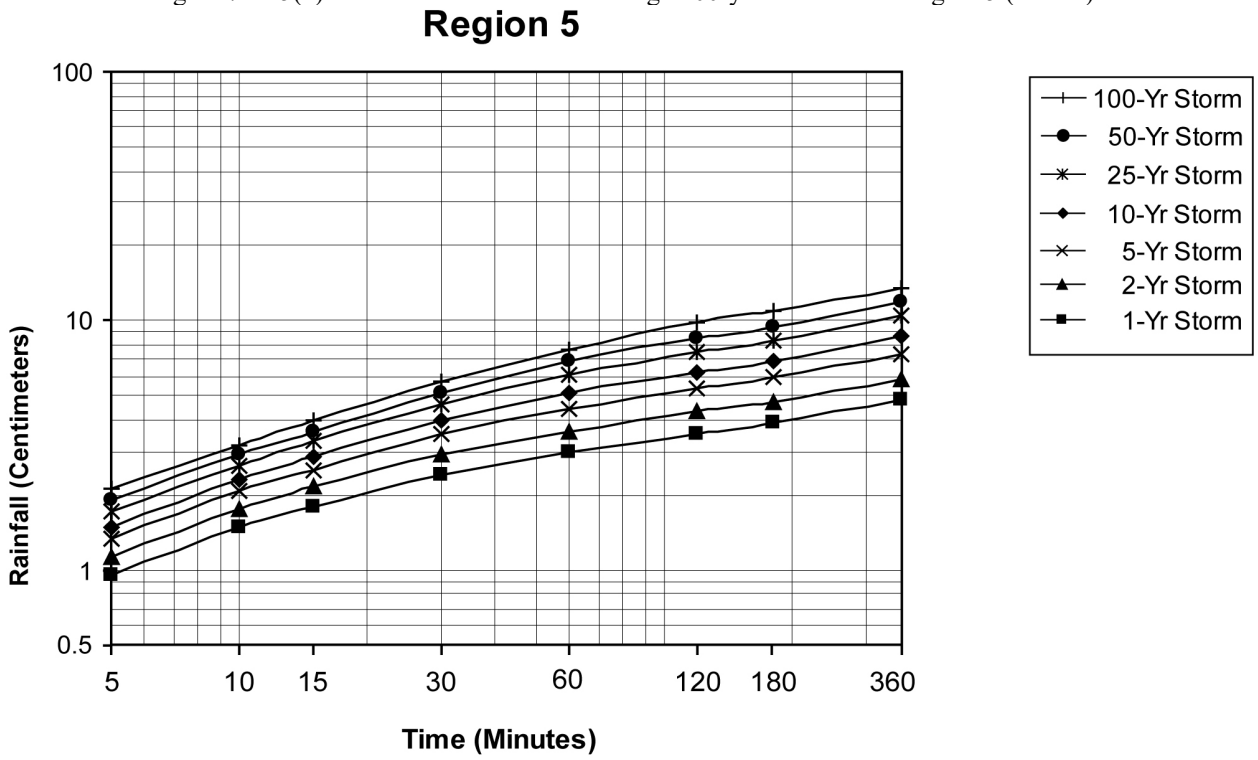


Figure 7A.16(a) Rainfall Intensity for 1- through 100-year Storms for Region 5 (U.S. Customary).

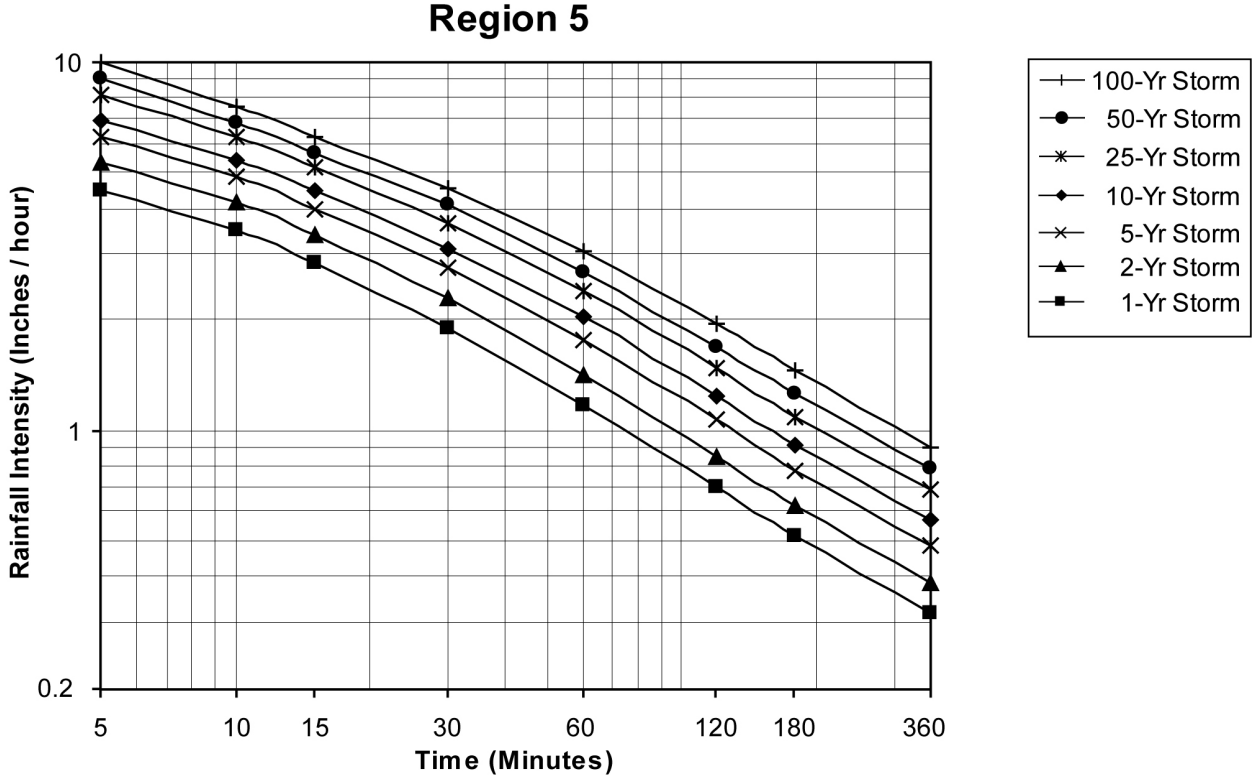
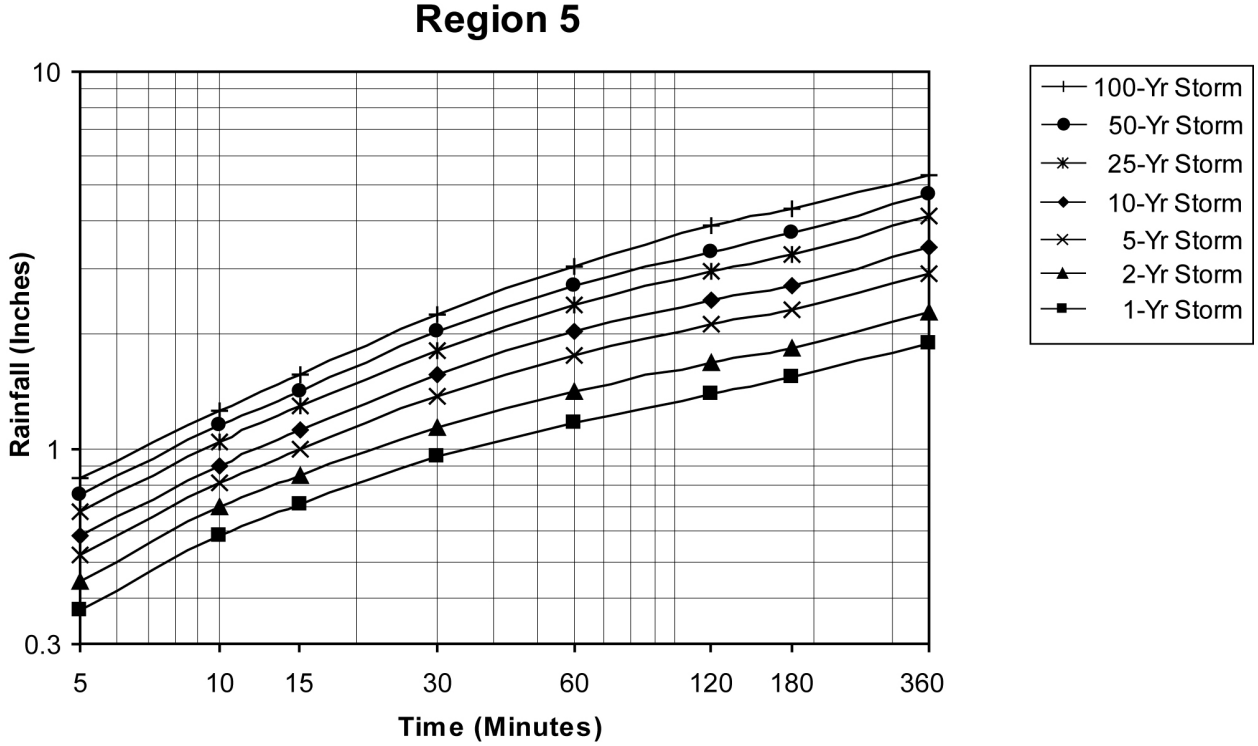


Figure 7A.16(b) Rainfall Amount for 1- through 100-year Storms for Region 5 (U.S. Customary).



7A.2 SAMPLE PROBLEMS UTILIZING THE PDT-IDF CURVES AND DATA

A. Sample 1. Rational Method – Rainfall Intensity – Computing Peak Flow (Q) for Storm Sewer and Inlet Sizing.

For culvert and pipe sizing where drainage areas are less than 80 hectares (200 acres) and full hydrographs are not required, often the rational method is the most suitable method to compute peak rates of runoff for design. The following example provides the process of obtaining the rainfall intensity for the determination of the rational peak flow value (Q) for the 10-year design storm.

Rational Method: 10-year Storm, the center of Armstrong County

Determine the Q to an inlet along a road.

Drainage Area (A): = 91.4 meters (300 feet) x 15.2 meters (50 feet) pavement width
= 1,389.3 square meters (15,000 sf)
= 0.14 hectares (0.34 ac)

T_c = 5 min
C = 0.95 from Table 7.6, page 7-27, Chapter 7, *Hydraulics*

Rainfall:

Step 1 Determine the appropriate rainfall region map using Table 7A.1. For the 10-year 5 minute storm, Map C would be the appropriate map to determine rainfall region.

Step 2 From Figure 7A.3, Map C, the center of Armstrong County would fall within Region 3

Step 3 From Figures 7A.11(a) or 7A.12(a), the 10-year, 5-minute rainfall intensity (I) would be 15.75 cm/hr (6.2 in/hr).

Step 4 Convert cm/hr to mm/hr by multiplying the metric value from Figure 7.A.11(a) by 10 mm/cm to yield 157mm/hr, for use in the rational equation.

NOTE: EQUATION 7.9 IN METRIC UNITS USES mm/hr AS THE RAINFALL UNIT, NOT cm/hr. CHARTS IN APPENDIX 7A ARE IN UNITS OF cm/hr.

Step 5 From Equation 7.9, Compute the flow (Q)

C = 0.95 (Table 7.6)
C_f = 1 (Table 7.7)
I = 157.5 mm/hr (6.2 in/hr)
A = 0.14 hectares (0.34 Ac)

METRIC

$$Q = CC_fIA/360$$
$$= (.95)(1)(157.5)(.14)/360$$
$$= 0.058\ m^3/s\ (cfs)$$

US CUSTOMARY

$$Q = CC_fIA\ (U.S.\ Customary)$$
$$= (.95)(1)(6.2)(.34)$$
$$= 2.0\ cfs$$

B. Sample 2. Rational Samples - Rainfall Intensity – Computing Peak Flow (Q) for Storm Culvert Sizing.

Using the rational method, determine the Q_{25} to size three roadside culverts from three separate contributing drainage areas in central Pike County.

Given:

Subarea	D.A. (ha)	D.A. (ac)	C (Table 7.6)	C_f (Table 7.7)	T_c (min)
1	0.49	1.2	.95	1.1	5.1
2	4.33	10.7	.87	1.1	22.5
3	2.31	5.7	.79	1.1	12.6

Step 1 From Table 7A.1, determine the map needed in order to obtain the required rainfall region.

Subarea	D.A. (ha)	D.A. (ac)	C	C_f	T_c (min)	Map
1	0.49	1.2	.95	1.1	5.1	B
2	4.33	10.7	.87	1.1	22.5	A
3	2.31	5.7	.79	1.1	12.6	C

Step 2 From each map (Figures 7A.2,1 and 3 respectively), determine what region should be utilized.

Subarea	D.A. (ha)	D.A. (ac)	C	C_f	T_c (min)	Map	Region
1	0.49	1.2	.95	1.1	5.1	B	3
2	4.33	10.7	.87	1.1	22.5	A	3
3	2.31	5.7	.79	1.1	12.6	C	4

Step 3 From the appropriate PDT-IDF curve, determine the rainfall intensity to be used for each subarea.

NOTE: EQUATION 7.9 IN METRIC UNITS USES mm/hr AS THE RAINFALL UNIT, NOT cm/hr. CHARTS IN APPENDIX 7A ARE IN UNITS OF cm/hr. THE RAINFALL INTENSITIES DERIVED FROM APPENDIX 7A WILL NEED TO BE CONVERTED FROM cm/hr TO mm/hr.

METRIC

Subarea	D.A. (ha)	C	C_f	T_c (min)	Map	Region	Figure	Rainfall Intensity From Figure (cm/hr)	Rainfall Intensity Conversion (mm/hr)
1	0.49	.95	1.1	5.1	B	3	7A.11(a)	17.78	177.80
2	4.33	.87	1.1	22.5	A	3	7A.11(a)	9.65	96.50
3	2.31	.79	1.1	12.6	C	4	7A.13(a)	13.71	137.10

US CUSTOMARY

Subarea	D.A. (ac)	C	C_f	T_c (min)	Map	Region	Figure	Rainfall Intensity (in/hr)
1	1.2	.95	1.1	5.1	B	3	7A.12(a)	7.0
2	10.7	.87	1.1	22.5	A	3	7A.12(a)	3.8
3	5.7	.79	1.1	12.6	C	4	7A.14(a)	5.4

Step 4 From Equation 7.9, Compute the flow (Q).

METRIC

Subarea	$Q = CC_fIA/360$	Q (m^3/s)
1	= (.95)(1.1)(177.80)(0.49)/360	0.25
2	= (.87)(1.1)(96.52)(4.33)/360	1.11
3	= (.79)(1.1)(137.16)(2.31)/360	0.76

US CUSTOMARY

Subarea	$Q = CC_fIA$	Q (cfs)
1	= (.95)(1.1)(7.0)(1.2)	8.8
2	= (.87)(1.1)(3.8)(10.7)	38.9
3	= (.79)(1.1)(5.4)(5.7)	26.7

C. Sample 3. 24-hour Storm Total and Composite Design Storms for PDT-IDF 'S' Curve and Hyetograph.

In storm runoff modeling, design storms are often needed for a 24-hour storm represented by a typical 'S' curve. A composite 24-hour design storm, constructed as described below, will serve this purpose.

The composite design storm should be generated such that the maximum rain falling over any time span (centered around the storm peak), equals the design storm depth indicated for the corresponding durations.

As an example, a 50-year design storm for the same site will be constructed for a location in southeast Bucks County. The storm is to be defined in 15-minute intervals, for a total duration of 24 hours (1440 minutes). It is advisable to place the storm peak at or right after the center of the storm and that the storm shape be approximately symmetrical. The process is best set up in an Excel spreadsheet. Since we are obtaining the 50-year Average Recurrence Interval (ARI) storm, the following maps will be utilized for each storm duration rainfall amount.

Step 1 From Table 7A.1, Determine which map applies to each duration for the 50-year storm as shown in Table 7A.8(a).

Step 2 Find the region the site is located within for each map determined in Step 1.
For the 5-minute duration, Figure 7A.2 (Map B) yields Region 3
For the 10-, 15-, 30-, and 60-minute duration, Figure 7A.3 (Map C) yields Region 5
For the 2 and 3-hour duration, Figure 7A.4 (Map E) yields Region 5
For the 6-hour duration, Figure 7A.5 (Map D) yields Region 5
For the 12 and 24-hour duration, Figure 7A.6 (Map F) yields Region 5

To better smooth the curve, it may be necessary to obtain values for storm durations not specified in Table 7A.1, for example the 90, 500, 800, 1000, 1200, 1300, 1350, and 1400 minute durations shown in Table 7A.8 (a) and (b). If a storm duration does not appear in Table 7A.1, the map that should be utilized should be the one for the closest storm duration in Table 7A.1. For instance, a storm duration of 500 minutes is closer to the 6 hour (360 minutes) duration than the 12 hour (720 minutes) duration. Therefore, map D should be utilized for the 50-year storm.

For storm durations that fall directly between two storm durations specified in Table 7A.1, the appropriate maps on either side of the selected time can be inspected to obtain the appropriate rainfall region. If the two maps place the site in two different regions the precipitation estimates for both regions should be obtained and the larger of the two precipitation values selected to complete the calculations. For instance, 90 minutes falls midway between the 60 and 120 minute durations. Therefore both maps C and E should be inspected.

It should be noted that the values for these intermediate storms not shown in Table 7A.1 may be adjusted slightly to provide for a smoother fitting curve. The values in table 7A.1 may not be modified.

Step 3 From Tables 7A.4(a) and 7A.4(b) or Figures 7A.11(b) and 7A12(b) (Region 3), obtain 50-year rainfall amount for the 5-minute duration. (1.65 cm (0.65 in))

From Tables 7A.6(a) and 7A.6(b) or Figures 7A.15(b) and 7A16.(b) (Region 5), obtain 50-year rainfall amounts for the 10-, 15-, 30-, 60-, 120-, 180-, 360-, 720-, and 1440-minute durations (from the appropriate map and region), and list them as in column (2) in Tables 7A.8(a/b) below. The Maps and Regions (columns 3 and 4) may be listed for reference.

Therefore, per Table 7A.8 (a/b), the 50-year, 24-hour rainfall total amount would be 18.19 cm (7.16 inches), which may be utilized for the storm total in TR-55, HEC-HMS or other computations.

Table 7A.8(a). Values obtained from 2007 PDT-IDF (Metric)

50-year -24-hour Storm			
1	2	3	4
Minutes	Amt (cm)	Map	Region
5	1.65	B	3
10	2.92	C	5
15	3.61	C	5
30	5.18	C	5
60	6.83	C	5
90	7.87	C, E	5
120	8.48	E	5
180	9.53	E	5
360	11.94	D	5
500	13.34	D	5
720	15.14	F	5
800	15.62	F	5
1000	16.38	F	5
1200	17.27	F	5
1300	17.65	F	5
1350	17.83	F	5
1400	17.98	F	5
1440	18.19	F	5

Table 7A.8(b). Values obtained from 2007 PDT-IDF (U.S. Customary)

50-year -24-hour Storm			
Minutes	Amt (in)	Map	Region
5	0.65	B	3
10	1.15	C	5
15	1.42	C	5
30	2.04	C	5
60	2.69	C	5
90	3.10	C, E	5
120	3.34	E	5
180	3.75	E	5
360	4.70	D	5
500	5.25	D	5
720	5.96	F	5
800	6.15	F	5
1000	6.45	F	5
1200	6.80	F	5
1300	6.95	F	5
1350	7.02	F	5
1400	7.08	F	5
1440	7.16	F	5

Figure 7A.17(a). Initial Plot of the Data (Metric)

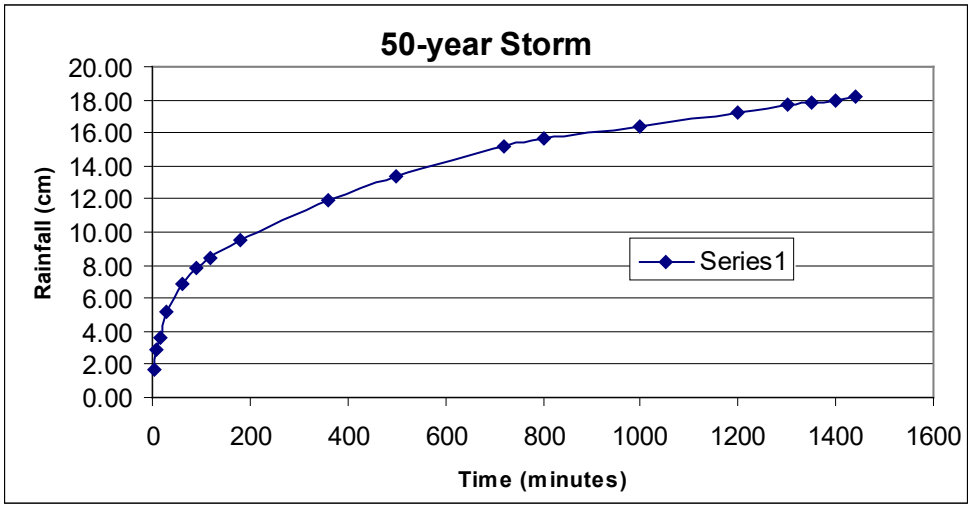
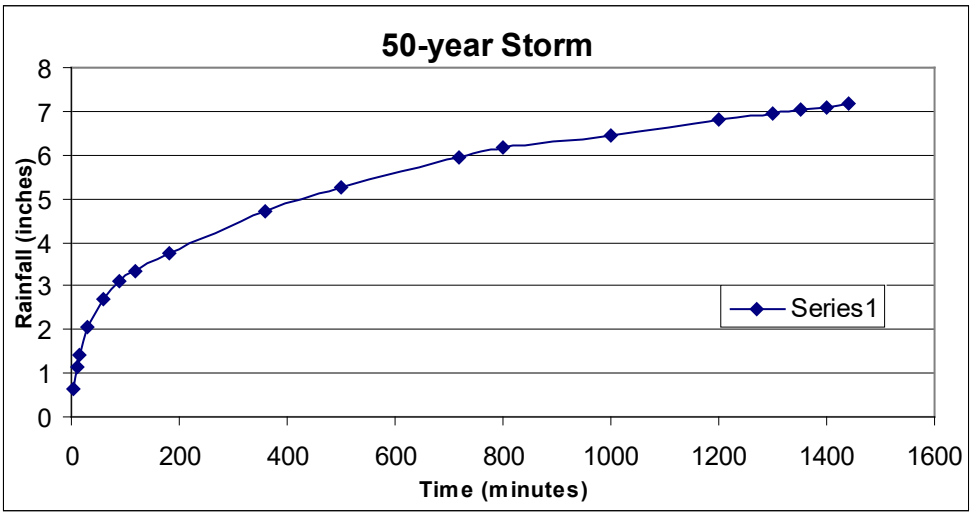


Figure 7A.17(b). Initial Plot of the Data (U.S. Customary)



Step 4 Determine the rainfall distribution, by plotting a curve of the rainfall amount vs. time as shown in Figure 7A.17 (a/b). It has been found that obtaining additional rainfall values for durations not specified in Table 7A.1 (a/b) provides for a smoother curve and a more accurate equation of the curve. Initial values should be obtained from the PDT-IDF curves Figures 7A.7 through 7A.16. Initial rainfall values can be plotted and fitted with a smooth curve to obtain rainfall values for the entire 24-hour event. It is important to not fit an equation to the entire curve, as it will not appropriately represent the center and end portions of the curve. Instead, the data should be divided into two portions, with separate curves and equations developed for the beginning and end portions of the storm, to more accurately estimate the rainfall throughout the event.

Step 5 Split the data into two Tables (halves), as shown in Tables 7A.9(a), 7A.9(b), 7A.10(a), and 7A.10(b). Then develop curves and equations to fit the data, and R^2 values for each half as shown in Figures 7A.18(a), 7A.18(b), 7A.19(a), and 7A.19(b).

Table 7A.9(a). Data from Time 0 to 500 Minutes (Metric)

Minutes	Amt (cm)
5	1.65
10	2.92
15	3.61
30	5.18
60	6.83
90	7.87
120	8.48
180	9.53
360	11.94
500	13.34

Table 7A.9(b). Data from Time 0 to 500 Minutes (U.S. Customary)

Minutes	Amount (in)
5	0.65
10	1.15
15	1.42
30	2.04
60	2.69
90	3.10
120	3.34
180	3.75
360	4.70
500	5.25

Figure 7A.18(a). Logarithmic Curve Developed from 0 to 500 Minutes (Metric)

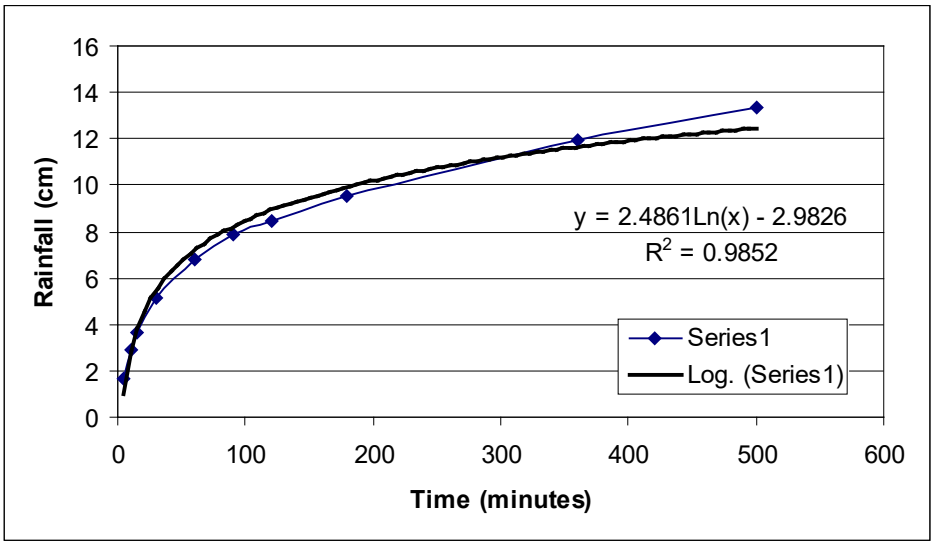


Figure 7A.18(b). Logarithmic Curve Developed from 0 to 500 Minutes (U.S. Customary)

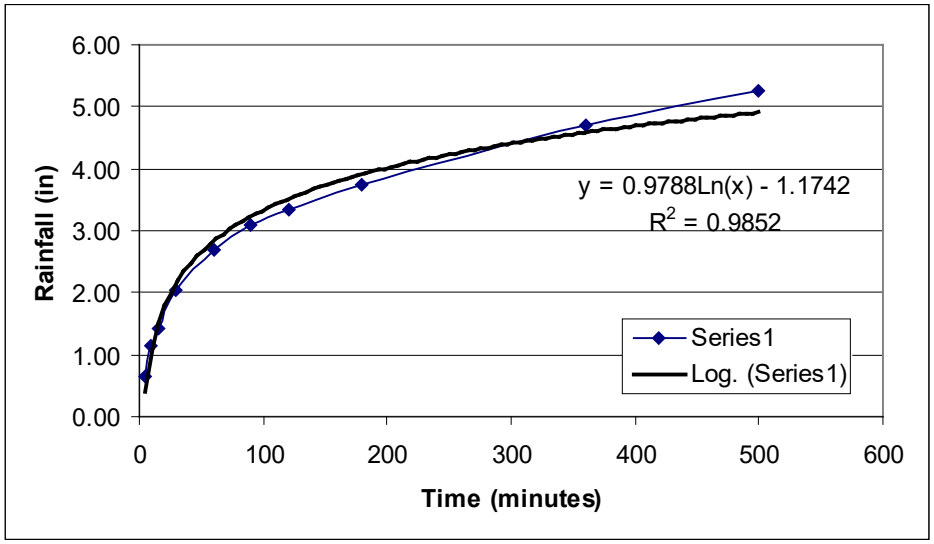


Table 7A.10(a). Data from Time 500 to 1440 Minutes (Metric)

Minutes	Amt (cm)
500	13.34
720	15.14
800	15.62
1000	16.38
1200	17.27
1300	17.65
1350	17.83
1400	17.98
1440	18.19

Table 7A.10(b). Data from Time 500 to 1440 Minutes (U.S. Customary)

Minutes	Amount (in)
500	5.25
720	5.96
800	6.15
1000	6.45
1200	6.8
1300	6.95
1350	7.02
1400	7.08
1440	7.16

Figure 7A.19(a). Logarithmic Curve Developed from 500 to 1440 Minutes (Metric)

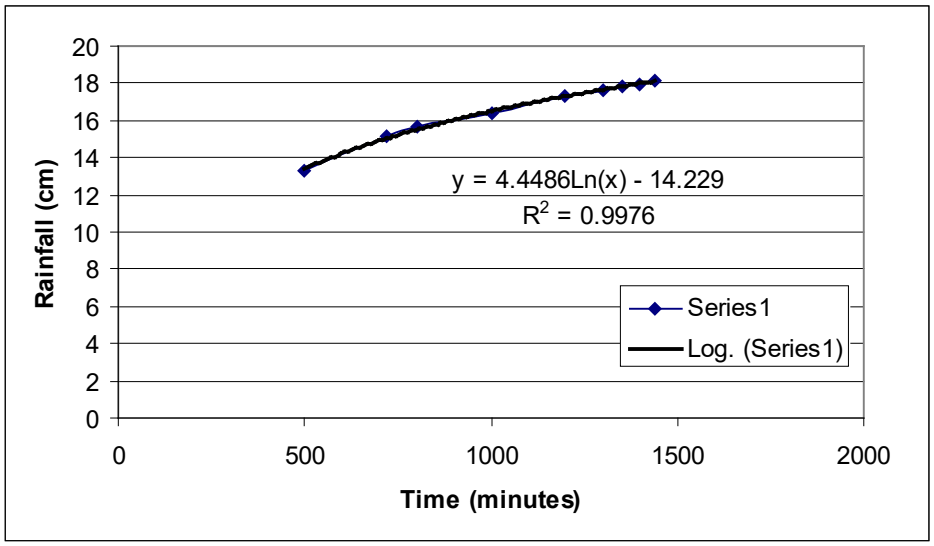
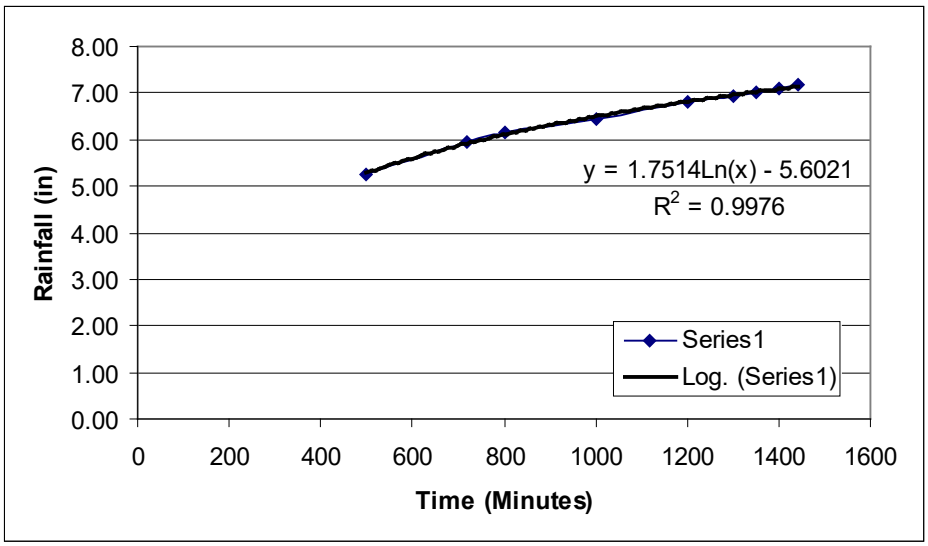


Figure 7A.19(b). Logarithmic Curve Developed from 500 to 1440 Minutes (U.S. Customary)



Step 6 Develop a table similar to Table 7.A.11 (a/b) using both curves. This table can be used to determine the time at which the two curves intercept and the development of a composite rainfall curve. The values in column (3) are from Tables 7A.8 (a/b) and based on the maps and the corresponding regional precipitation curves using the procedure discussed in Step 3. The values in Column 4 are calculated from the regression equation representing the curve for the time frame 0 to 500 minutes. Similarly the values in column (5) are calculated from the equation representing the curve for the time frame 500 to 1440 minutes. Negative values at the top of column (5) are not an issue since this portion of column (5) is not used in the composite curve. As can be seen in the Table 7.A.11(a/b), the numbers merge near time 315 minutes, which for this example is considered the intercept of the two curves.

Step 7 Create the composite rainfall curve in column (6) by using the rainfall data from column 4 for the first part of the event (green shading) and use the data from column (5) for the second part (orange shading), or later portion, of the storm.

Table 7A.11(a). Composite Storm Development (Metric)

Column									
1	2	3	4	5	6	7	8	9	10
Storm Duration	Storm Duration	Rainfall From PDM	Rainfall From Eqn. 1	Rainfall From Eqn. 2	Rainfall From Column 4 or 5	Incremental Rainfall	Rearranged Rainfall Distribution	Rearranged Rainfall Intensity	Fraction of 24-hr Rainfall
Hour	Minute	(cm)	(cm)	(cm)	(cm)	(cm)	(cm)	(cm/hr)	(%)
0	0	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	15	3.61	3.840	-2.182	3.840	3.840	0.047	0.188	0.003
	30	5.18	5.563	0.902	5.563	1.723	0.048	0.192	0.005
	45	6.01	6.571	2.705	6.571	1.008	0.049	0.197	0.008
1	60	6.83	7.286	3.985	7.286	0.715	0.050	0.201	0.011
	75	7.42	7.841	4.978	7.841	0.555	0.051	0.206	0.014
	90	7.87	8.294	5.789	8.294	0.453	0.053	0.211	0.016
	105	8.23	8.678	6.475	8.678	0.383	0.054	0.216	0.019
2	120	8.48	9.010	7.069	9.010	0.332	0.055	0.221	0.023
	135	8.84	9.302	7.593	9.302	0.293	0.057	0.227	0.026
	150	9.09	9.564	8.061	9.564	0.262	0.058	0.233	0.029
	165	9.32	9.801	8.485	9.801	0.237	0.060	0.239	0.032
3	180	9.53	10.018	8.872	10.018	0.216	0.061	0.245	0.036
	195	9.80	10.217	9.228	10.217	0.199	0.063	0.252	0.039
	210	10.06	10.401	9.558	10.401	0.184	0.065	0.260	0.043
	225	10.31	10.572	9.865	10.572	0.172	0.067	0.268	0.046
4	240	10.49	10.733	10.152	10.733	0.160	0.069	0.276	0.050
	255	10.64	10.884	10.422	10.884	0.151	0.071	0.285	0.054
	270	10.82	11.026	10.676	11.026	0.142	0.074	0.294	0.058
	285	10.97	11.160	10.917	11.160	0.134	0.076	0.304	0.062
5	300	11.18	11.288	11.145	11.288	0.128	0.079	0.315	0.067
	315	11.40	11.409	11.362	11.409	0.121	0.082	0.327	0.071
	330	11.54	11.525	11.569	11.569	0.160	0.085	0.339	0.076
	345	11.74	11.635	11.767	11.767	0.198	0.088	0.352	0.081
6	360	11.94	11.741	11.956	11.956	0.189	0.092	0.367	0.086
	375	12.12	11.842	12.138	12.138	0.182	0.096	0.383	0.091
	390	12.14	11.940	12.312	12.312	0.174	0.100	0.400	0.097
	405	12.29	12.034	12.480	12.480	0.168	0.105	0.419	0.102
7	420	12.37	12.124	12.642	12.642	0.162	0.110	0.439	0.108
	435	12.47	12.211	12.798	12.798	0.156	0.116	0.462	0.115
	450	12.60	12.296	12.949	12.949	0.151	0.121	0.485	0.121
	465	12.67	12.377	13.094	13.094	0.146	0.125	0.501	0.128
8	480	12.80	12.456	13.236	13.236	0.141	0.129	0.516	0.135
	495	12.88	12.533	13.373	13.373	0.137	0.134	0.538	0.143
	510	13.00	12.607	13.505	13.505	0.133	0.141	0.565	0.151
	525	13.40	12.679	13.634	13.634	0.129	0.146	0.583	0.159
9	540	13.54	12.749	13.760	13.760	0.125	0.151	0.603	0.167
	555	13.67	12.817	13.882	13.882	0.122	0.160	0.640	0.176
	570	13.80	12.883	14.000	14.000	0.119	0.162	0.647	0.185
	585	13.94	12.948	14.116	14.116	0.116	0.172	0.686	0.194
10	600	14.07	13.011	14.228	14.228	0.113	0.182	0.726	0.204
	615	14.20	13.072	14.338	14.338	0.110	0.189	0.757	0.215

Column									
1	2	3	4	5	6	7	8	9	10
Storm Duration	Storm Duration	Rainfall From PDM	Rainfall From Eqn. 1	Rainfall From Eqn. 2	Rainfall From Column 4 or 5	Incremental Rainfall	Rearranged Rainfall Distribution	Rearranged Rainfall Intensity	Fraction of 24-hr Rainfall
Hour	Minute	(cm)	(cm)	(cm)	(cm)	(cm)	(cm)	(cm/hr)	(%)
	630	14.34	13.132	14.445	14.445	0.107	0.199	0.796	0.226
	645	14.47	13.191	14.550	14.550	0.105	0.237	0.948	0.239
11	660	14.61	13.248	14.652	14.652	0.102	0.293	1.171	0.255
	675	14.74	13.304	14.752	14.752	0.100	0.383	1.533	0.276
	690	14.87	13.358	14.850	14.850	0.098	0.555	2.219	0.307
	705	15.01	13.412	14.946	14.946	0.096	1.008	4.032	0.362
12	720	15.14	13.464	15.039	15.039	0.094	3.840	15.360	0.574
	735	15.20	13.515	15.131	15.131	0.092	1.723	6.893	0.669
	750	15.27	13.566	15.221	15.221	0.090	0.715	2.861	0.709
	765	15.33	13.615	15.309	15.309	0.088	0.453	1.813	0.734
13	780	15.39	13.663	15.396	15.396	0.086	0.332	1.328	0.752
	795	15.46	13.710	15.480	15.480	0.085	0.262	1.048	0.767
	810	15.52	13.757	15.563	15.563	0.083	0.216	0.865	0.778
	825	15.58	13.803	15.645	15.645	0.082	0.198	0.791	0.789
14	840	15.65	13.847	15.725	15.725	0.080	0.184	0.737	0.800
	855	15.71	13.891	15.804	15.804	0.079	0.174	0.698	0.809
	870	15.77	13.935	15.881	15.881	0.077	0.168	0.672	0.818
	885	15.84	13.977	15.957	15.957	0.076	0.160	0.642	0.827
15	900	15.90	14.019	16.032	16.032	0.075	0.156	0.624	0.836
	915	15.96	14.060	16.106	16.106	0.074	0.151	0.603	0.844
	930	16.03	14.100	16.178	16.178	0.072	0.142	0.568	0.852
	945	16.09	14.140	16.249	16.249	0.071	0.137	0.548	0.860
16	960	16.15	14.179	16.319	16.319	0.070	0.133	0.531	0.867
	975	16.22	14.218	16.388	16.388	0.069	0.128	0.510	0.874
	990	16.28	14.256	16.456	16.456	0.068	0.122	0.488	0.881
	1005	16.34	14.293	16.523	16.523	0.067	0.119	0.475	0.887
17	1020	16.41	14.330	16.589	16.589	0.066	0.113	0.451	0.893
	1035	16.47	14.366	16.654	16.654	0.065	0.107	0.429	0.899
	1050	16.54	14.402	16.718	16.718	0.064	0.102	0.409	0.905
	1065	16.60	14.437	16.781	16.781	0.063	0.098	0.391	0.910
18	1080	16.66	14.472	16.843	16.843	0.062	0.094	0.375	0.916
	1095	16.73	14.506	16.905	16.905	0.061	0.090	0.359	0.921
	1110	16.79	14.540	16.965	16.965	0.061	0.086	0.346	0.925
	1125	16.85	14.574	17.025	17.025	0.060	0.083	0.333	0.930
19	1140	16.92	14.607	17.084	17.084	0.059	0.080	0.321	0.934
	1155	16.98	14.639	17.142	17.142	0.058	0.077	0.309	0.939
	1170	17.04	14.671	17.199	17.199	0.057	0.075	0.299	0.943
	1185	17.11	14.703	17.256	17.256	0.057	0.072	0.289	0.947
20	1200	17.27	14.734	17.312	17.312	0.056	0.070	0.280	0.951
	1215	17.23	14.765	17.367	17.367	0.055	0.068	0.272	0.954
	1230	17.30	14.795	17.422	17.422	0.055	0.066	0.264	0.958
	1245	17.36	14.826	17.476	17.476	0.054	0.064	0.256	0.961
21	1260	17.42	14.855	17.529	17.529	0.053	0.062	0.249	0.965
	1275	17.49	14.885	17.582	17.582	0.053	0.061	0.242	0.968

Column									
1	2	3	4	5	6	7	8	9	10
Storm Duration	Storm Duration	Rainfall From PDM	Rainfall From Eqn. 1	Rainfall From Eqn. 2	Rainfall From Column 4 or 5	Incremental Rainfall	Rearranged Rainfall Distribution	Rearranged Rainfall Intensity	Fraction of 24-hr Rainfall
Hour	Minute	(cm)	(cm)	(cm)	(cm)	(cm)	(cm)	(cm/hr)	(%)
	1290	17.55	14.914	17.634	17.634	0.052	0.059	0.236	0.971
	1305	17.61	14.943	17.685	17.685	0.051	0.057	0.230	0.975
22	1320	17.68	14.971	17.736	17.736	0.051	0.056	0.224	0.978
	1335	17.74	14.999	17.786	17.786	0.050	0.055	0.218	0.981
	1350	17.83	15.027	17.836	17.836	0.050	0.053	0.213	0.984
	1365	17.87	15.054	17.885	17.885	0.049	0.052	0.208	0.987
23	1380	17.93	15.082	17.934	17.934	0.049	0.051	0.203	0.989
	1395	18.00	15.108	17.982	17.982	0.048	0.050	0.199	0.992
	1410	18.06	15.135	18.029	18.029	0.048	0.049	0.194	0.995
	1425	18.12	15.161	18.076	18.076	0.047	0.048	0.190	0.997
24	1440	18.19	15.187	18.123	18.123	0.047	0.047	0.186	1.000

Table 7A.11(b). Composite Storm Development (U.S. Customary)

Column									
1	2	3	4	5	6	7	8	9	10
Storm Duration	Storm Duration	Rainfall From PDM	Rainfall From Eqn. 1	Rainfall From Eqn. 2	Rainfall From Column 4 or 5	Incremental Rainfall	Rearranged Rainfall Distribution	Rearranged Rainfall Intensity	Fraction of 24-hr Rainfall
Hour	Minute	(in)	(in.)	(in)	(in)	(in)	(in)	(in/hr)	(%)
0	0	0	0	0	0.000	0	0.000	0.000	0.000
	15	1.42	1.476	-0.859	1.476	1.476	0.019	0.074	0.003
	30	2.04	2.155	0.355	2.155	0.678	0.019	0.076	0.005
	45	2.37	2.552	1.065	2.552	0.397	0.019	0.077	0.008
1	60	2.69	2.833	1.569	2.833	0.282	0.020	0.079	0.011
	75	2.92	3.052	1.960	3.052	0.218	0.020	0.081	0.014
	90	3.1	3.230	2.279	3.230	0.178	0.021	0.083	0.016
	105	3.24	3.381	2.549	3.381	0.151	0.021	0.085	0.019
2	120	3.34	3.512	2.783	3.512	0.131	0.022	0.087	0.023
	135	3.48	3.627	2.989	3.627	0.115	0.022	0.089	0.026
	150	3.58	3.730	3.174	3.730	0.103	0.023	0.092	0.029
	165	3.67	3.823	3.340	3.823	0.093	0.024	0.094	0.032
3	180	3.75	3.909	3.493	3.909	0.085	0.024	0.097	0.036
	195	3.86	3.987	3.633	3.987	0.078	0.025	0.099	0.039
	210	3.96	4.060	3.763	4.060	0.073	0.026	0.102	0.043
	225	4.06	4.127	3.884	4.127	0.068	0.026	0.105	0.046
4	240	4.13	4.190	3.997	4.190	0.063	0.027	0.109	0.050
	255	4.19	4.250	4.103	4.250	0.059	0.028	0.112	0.054
	270	4.26	4.306	4.203	4.306	0.056	0.029	0.116	0.058
	285	4.32	4.358	4.298	4.358	0.053	0.030	0.120	0.062
5	300	4.4	4.409	4.388	4.409	0.050	0.031	0.124	0.067
	315	4.49	4.456	4.473	4.473	0.064	0.032	0.129	0.071
	330	4.54	4.502	4.554	4.554	0.081	0.033	0.133	0.076

Column									
1	2	3	4	5	6	7	8	9	10
Storm Duration	Storm Duration	Rainfall From PDM	Rainfall From Eqn. 1	Rainfall From Eqn. 2	Rainfall From Column 4 or 5	Incremental Rainfall	Rearranged Rainfall Distribution	Rearranged Rainfall Intensity	Fraction of 24-hr Rainfall
Hour	Minute	(in)	(in.)	(in)	(in)	(in)	(in)	(in/hr)	(%)
	345	4.62	4.545	4.632	4.632	0.078	0.035	0.139	0.081
6	360	4.7	4.587	4.707	4.707	0.075	0.036	0.144	0.086
	375	4.77	4.627	4.778	4.778	0.071	0.038	0.151	0.091
	390	4.78	4.665	4.847	4.847	0.069	0.039	0.157	0.097
	405	4.84	4.702	4.913	4.913	0.066	0.041	0.165	0.102
7	420	4.87	4.738	4.977	4.977	0.064	0.043	0.173	0.108
	435	4.91	4.772	5.038	5.038	0.061	0.045	0.182	0.115
	450	4.96	4.806	5.098	5.098	0.059	0.048	0.191	0.121
	465	4.99	4.838	5.155	5.155	0.057	0.049	0.197	0.128
8	480	5.04	4.869	5.211	5.211	0.056	0.051	0.203	0.135
	495	5.07	4.899	5.265	5.265	0.054	0.053	0.212	0.143
	510	5.12	4.928	5.317	5.317	0.052	0.056	0.222	0.151
	525	5.28	4.956	5.368	5.368	0.051	0.057	0.230	0.159
9	540	5.33	4.984	5.417	5.417	0.049	0.059	0.238	0.167
	555	5.38	5.011	5.465	5.465	0.048	0.063	0.253	0.176
	570	5.44	5.037	5.512	5.512	0.047	0.066	0.264	0.185
	585	5.49	5.062	5.557	5.557	0.045	0.069	0.275	0.195
10	600	5.54	5.087	5.601	5.601	0.044	0.073	0.290	0.205
	615	5.59	5.111	5.645	5.645	0.043	0.078	0.311	0.216
	630	5.65	5.135	5.687	5.687	0.042	0.085	0.341	0.228
	645	5.7	5.158	5.728	5.728	0.041	0.098	0.392	0.242
11	660	5.75	5.180	5.768	5.768	0.040	0.115	0.461	0.258
	675	5.8	5.202	5.808	5.808	0.039	0.151	0.604	0.279
	690	5.86	5.224	5.846	5.846	0.038	0.218	0.874	0.309
	705	5.91	5.245	5.884	5.884	0.038	0.397	1.587	0.365
12	720	5.96	5.266	5.921	5.921	0.037	1.476	5.906	0.572
	735	5.99	5.286	5.957	5.957	0.036	0.678	2.714	0.667
	750	6.01	5.306	5.992	5.992	0.035	0.282	1.126	0.707
	765	6.04	5.325	6.027	6.027	0.035	0.178	0.714	0.732
13	780	6.06	5.344	6.061	6.061	0.034	0.131	0.523	0.750
	795	6.09	5.363	6.094	6.094	0.033	0.103	0.413	0.764
	810	6.11	5.381	6.127	6.127	0.033	0.093	0.373	0.777
	825	6.14	5.399	6.159	6.159	0.032	0.078	0.313	0.788
14	840	6.16	5.416	6.191	6.191	0.032	0.075	0.298	0.799
	855	6.19	5.434	6.222	6.222	0.031	0.071	0.286	0.809
	870	6.21	5.451	6.252	6.252	0.030	0.068	0.270	0.818
	885	6.24	5.468	6.282	6.282	0.030	0.064	0.255	0.827
15	900	6.26	5.484	6.312	6.312	0.029	0.061	0.246	0.836
	915	6.29	5.500	6.341	6.341	0.029	0.059	0.237	0.844
	930	6.31	5.516	6.369	6.369	0.028	0.056	0.224	0.852
	945	6.34	5.532	6.397	6.397	0.028	0.054	0.216	0.860
16	960	6.36	5.547	6.425	6.425	0.028	0.052	0.209	0.867
	975	6.39	5.562	6.452	6.452	0.027	0.050	0.201	0.874
	990	6.41	5.577	6.479	6.479	0.027	0.048	0.192	0.881

Column									
1	2	3	4	5	6	7	8	9	10
Storm Duration	Storm Duration	Rainfall From PDM	Rainfall From Eqn. 1	Rainfall From Eqn. 2	Rainfall From Column 4 or 5	Incremental Rainfall	Rearranged Rainfall Distribution	Rearranged Rainfall Intensity	Fraction of 24-hr Rainfall
Hour	Minute	(in)	(in.)	(in)	(in)	(in)	(in)	(in/hr)	(%)
	1005	6.44	5.592	6.505	6.505	0.026	0.047	0.187	0.887
17	1020	6.46	5.606	6.531	6.531	0.026	0.044	0.177	0.893
	1035	6.49	5.621	6.556	6.556	0.026	0.042	0.169	0.899
	1050	6.51	5.635	6.582	6.582	0.025	0.040	0.161	0.905
	1065	6.54	5.649	6.606	6.606	0.025	0.038	0.154	0.910
18	1080	6.56	5.662	6.631	6.631	0.024	0.037	0.147	0.916
	1095	6.59	5.676	6.655	6.655	0.024	0.035	0.142	0.920
	1110	6.61	5.689	6.679	6.679	0.024	0.034	0.136	0.925
	1125	6.64	5.702	6.702	6.702	0.024	0.033	0.131	0.930
19	1140	6.66	5.715	6.726	6.726	0.023	0.032	0.126	0.934
	1155	6.69	5.728	6.749	6.749	0.023	0.030	0.122	0.939
	1170	6.71	5.741	6.771	6.771	0.023	0.029	0.118	0.943
	1185	6.74	5.753	6.793	6.793	0.022	0.028	0.114	0.947
20	1200	6.8	5.766	6.815	6.815	0.022	0.028	0.110	0.951
	1215	6.79	5.778	6.837	6.837	0.022	0.027	0.107	0.954
	1230	6.81	5.790	6.859	6.859	0.021	0.026	0.104	0.958
	1245	6.84	5.802	6.880	6.880	0.021	0.025	0.101	0.961
21	1260	6.86	5.813	6.901	6.901	0.021	0.024	0.098	0.965
	1275	6.89	5.825	6.922	6.922	0.021	0.024	0.095	0.968
	1290	6.91	5.836	6.942	6.942	0.020	0.023	0.093	0.971
	1305	6.94	5.848	6.962	6.962	0.020	0.023	0.090	0.975
22	1320	6.96	5.859	6.982	6.982	0.020	0.022	0.088	0.978
	1335	6.99	5.870	7.002	7.002	0.020	0.021	0.086	0.981
	1350	7.02	5.881	7.022	7.022	0.020	0.021	0.084	0.984
	1365	7.04	5.892	7.041	7.041	0.019	0.020	0.082	0.987
23	1380	7.06	5.902	7.060	7.060	0.019	0.020	0.080	0.989
	1395	7.09	5.913	7.079	7.079	0.019	0.020	0.078	0.992
	1410	7.11	5.923	7.098	7.098	0.019	0.019	0.077	0.995
	1425	7.14	5.934	7.116	7.116	0.019	0.019	0.075	0.997
24	1440	7.16	5.944	7.135	7.135	0.018	0.018	0.073	1.000

Step 8 Compute the incremental rainfall amounts in column (7) by subtracting consecutive values entered in column (6).

Step 9 Rearrange the rainfall increments from column (7) to column (8) in a quasi-symmetrical pattern, moving first and largest storm interval time to 720 minutes. Place the second largest incremental rainfall amount at 735 minutes and the third largest incremental rainfall amount immediately preceding the peek, at 705 minutes. Continuing placing the incremental rainfall amounts on either side of the peak until all of the incremental rainfall values fill column 8. This data represents the rainfall distribution for a particular event and could then be input into a hydrologic model such as HEC-HMS for additional hydrologic analysis. Plot the hyetograph from Column 8 as shown in Figures 7A.20(a) and 7A.20(b).

Figure 7A.20(a). Hyetograph for the 50-year Storm, Region 5 (Metric)

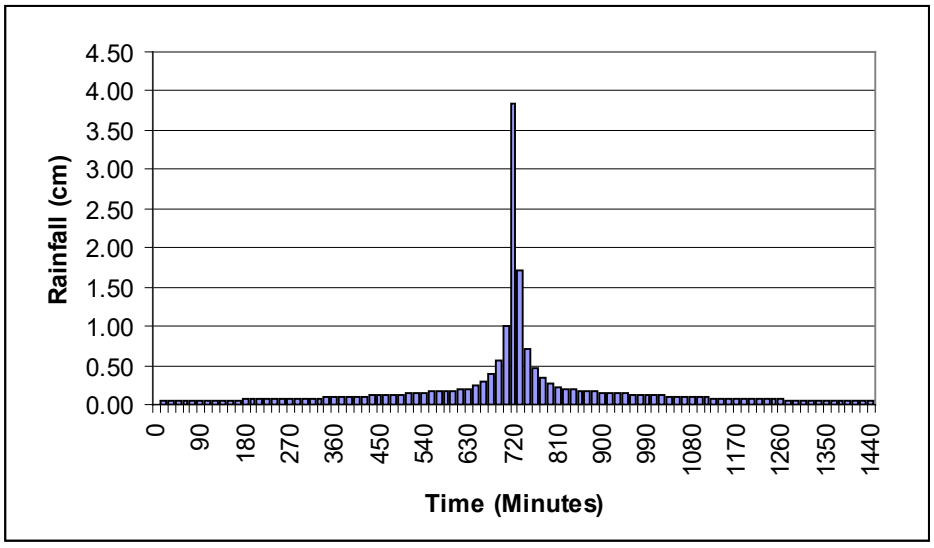
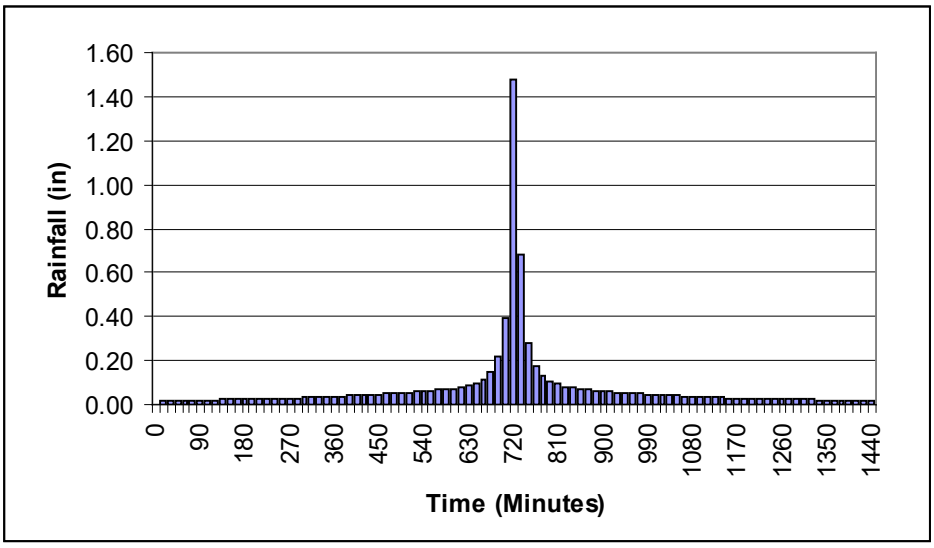


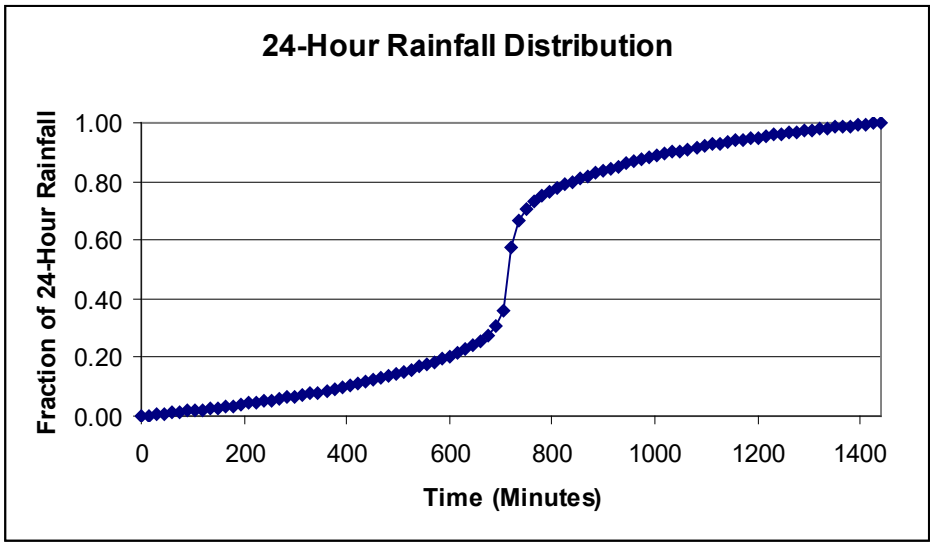
Figure 7A.20(b). Hyetograph for the 50-year Storm, Region 5 (U.S. Customary)



Step 10 (Optional) If incremental intensities are required, divide rearranged rainfall distribution in column (8) by the time step (15 minutes) and multiply by 60 minutes as shown in Tables 7A.7(a/b) to obtain in incremental intensity in units of depth of rainfall/hour.

Step 11 (Optional) To plot the 'S' Curve of the rainfall pattern, the rearranged incremental rainfall values in column (8) can be divided by the total 24-hour storm precipitation of 18.12 cm (7.16 in) to obtain the fraction of the 24-hour precipitation that has fallen for any given time increment. A running total of each time increment is placed in column (10) to obtain the total fraction of rainfall that has fallen with respect to time. The 'S' Curve is plotted using columns (2) and (10) as shown in Figure 7A.21. Since the 'S' curve is a plot of ratios of a unit of rainfall, the metric and U.S. customary units will plot identical.

Figure 7A.21. S-Curve for the 50-year Storm, Region 5 (Metric and U.S. Customary)



Note: It should be noted that the values obtained from the equations will vary slightly from the values obtained from the PDT-IDF charts and graphs. The variation is typically not significant enough to have a significant impact on the hydrograph or peak flow results. However, if the engineer sees it justifiable, the columns could be multiplied by the ratio of the difference in values to match the exact number. Also, one may notice in the hyetograph and tables, that there is a slight shift in rainfall difference for each increment near the intercept. This will not significantly affect the peaks. If warranted, the user may readjust the values at each time increment to smooth the curve.

7A.3 REFERENCES

Aron, G., D.J. Wall, E.L. White, C.N. Dunn, D.M. Kotz (1986). Field Manual of Pennsylvania Department of Transportation Storm Intensity-Duration-Frequency Charts (PDT-IDF). Department of Civil Engineering and Institute for Research on Land and Water Resources, The Pennsylvania State University. May 1986.

Hershfield, D. M. (1961). *Rainfall Frequency Atlas of the United States for Durations from 30 minutes to 24 hours and Return Periods from 1 to 100 years*. Weather Bureau Technical Paper No. 40, U.S. Weather Bureau, Washington D.C.

CHAPTER 7, APPENDIX B

FREQUENTLY ASKED QUESTIONS FOR PENNSYLVANIA DESIGN RAINFALL INTENSITY CHARTS FROM NOAA ATLAS 14 VERSION 3 DATA

7B.0 INTRODUCTION

The questions and answers have been grouped into the following sections:

- Background Questions
- Technical Questions
- Using Publication 584, *PennDOT Drainage Manual*, Chapter 7, Appendix A, Section 7A.1

7B.1 FREQUENTLY ASKED QUESTIONS

A. Background Questions.

1. What are the Pennsylvania Design Rainfall Intensity Charts contained in Publication 584, *PennDOT Drainage Manual*, Chapter 7, Appendix A, *Field Manual For Pennsylvania Design Rainfall Intensity Charts From NOAA Atlas 14 Version 3 Data*?

- Chapter 7, Appendix A contains the new Pennsylvania Department of Transportation - Intensity Duration Frequency Curves (PDT-IDF) based upon the new NOAA Atlas 14 Version 3 precipitation data and replaces the FIELD MANUAL, Pennsylvania Department of Transportation STORM INTENSITY-DURATION FREQUENCY CHARTS (PDT-IDF) (Aron, et. al., 1986a, 1986b).

2. Why are new IDF charts needed?

- They reflect new and updated precipitation data. Since the 1986 PDT-IDF Field Manual was generated, 17 years of additional rainfall records have been collected by the National Oceanic and Atmospheric Administration (NOAA). The availability of more comprehensive and updated precipitation data available through NOAA Atlas 14 warranted an update to the PDT-IDF curves. The 1986 PDT-IDF curves were based on data from 1948 to 1983, and included 153 data analysis sites. The 2007 PDT-IDF curves are based on data from 1863 to 2000, and included 278 daily rainfall recording gages and 139 hourly rainfall recording gages. The additional years of data plus addition stations utilized provide for more current and accurate data.
- They solve discrepancies between the 1986 values and NOAA Atlas 14, Version 3 values. In comparing the rainfall data from the on-line NOAA Atlas 14, Version 3 data with the 1986 PDT-IDF curves, it was noticed that there were discrepancies in rainfall values obtained. This warranted a further investigation as to why and where the discrepancies occurred.

3. Why are there differences between the rainfall values published in 1986 and 2007.

- The 1986 study only utilized 35 years of data whereas the 2007 utilized 137 years of data.
- This additional data normalized the impacts from major storms in the 1986 analysis.
- The gage network utilized in the 2007 analysis was denser, therefore providing more accurate and localized data.
- More up-to-date statistical methods and distributions were utilized for the NOAA Atlas 14, Version 3 data.
- NOAA Atlas 14, Version 3 used different spatial interpolation methods.
- The 2007 PDT-IDF analysis to produce the maps was performed using GIS whereas the 1986 PDT-IDF version relied on overlaying data on light tables.

4. What are the major differences between the old and new curves?

- The regional boundaries have shifted. The more inclusive dataset used to develop the new curves resulted in a change in rainfall depths and intensities throughout the State. Due to this, the rainfall regional boundaries also changed.
- The number of regional maps have increased. The 1986 PDT-IDF curves resulted in one map of five delineated rainfall regions; the region boundaries remained unchanged regardless of storm duration and frequency. The 2007 study still utilizes five regions, but six maps were developed to properly show the shifting region boundaries for different storm durations and frequencies. Recent statistical evidence based on additional data and a higher concentration of rainfall recording gages has now shown that the region boundaries vary depending on the duration and frequency of storms. For example, the lowest intensities and amounts for the five (5-) minute storms are located in Northcentral Pennsylvania, whereas the lowest intensities and amounts for the twenty-four (24-) hour storm are located in Western Pennsylvania. It was determined that one rainfall region map would not adequately represent the rainfall patterns. Therefore, the maps and curves were developed based upon storm duration and frequency
- Regional selection is based on duration and frequency. For the 1986 PDT-IDF curves, one simply had to find the site location on the single rainfall region map, find out what rainfall district the site of interest was located in, and determine the rainfall from the appropriate PDT-IDF curve. For the 2007 PDT-IDF curves, one first has to find the appropriate rainfall region map from Table 7A.1, then follow the same procedure for the appropriate map and region.
- The maximum duration shown is 360 minutes. The 1986 PDT-IDF curves were plotted up to 24-hour durations. The 2007 PDT-IDF curves in Chapter 7, Appendix A (Figures 7A.7 through 7A.16) are plotted only up to 360 minutes. This allowed a larger scale for the plotting since the x-axis was reduced, allowing for more accurate interpretation from the curves. The 12- and 24-hour duration values were simply put in Tables 7A.2 through 7A.6.

5. Why did the rainfall values go down in Western and Southwestern Pennsylvania?

- The additional rainfall data statistically lowered the values. Since there were only 35 years of data used to determine the 1986 PDT-IDF curves, there were two major hurricanes in the Western and Southwestern portions of the State during the 35-year period that skewed the rainfall distribution upwards in this area in the 1986 PDT-IDF curves. Therefore, rainfall intensities and amounts decreased between the 1986 PDT-IDF and 2007 PDT-IDF curves, whereas the additional years of data utilized in the 2007 study "dampened" the affects of the hurricane data. This also resulted in a shift of the region boundaries. For instance, in Southwestern Pennsylvania, for the 24 hour storm duration, portions of Region 4 changed to Region 1, and portions of Region 3 changed to Region 1.

6. Is this new data reliable and defensible?

- Yes. The new data and resulting 2007 PDT-IDF curves went through a rigorous QA/QC procedure, was peer reviewed by Dr. Arthur Miller, formerly of Penn State University and Tye Parzybok, Consultant to NOAA in development of NOAA Atlas 14, Version 3. As a result, the 2007 PDT-IDF Curves and Tables were found to be consistent and accurate.

7. Why can't the rainfall values just be obtained from the NOAA Atlas 14 web site as opposed to obtaining them from the new PDT-IDF curves?

- Users needed a consistent, verifiable source for rainfall data. NOAA Atlas 14 data is constantly being updated. The Department needs to have consistency in obtaining rainfall values for design and for associated calculation reviews rather than having values constantly changing for a particular area. In addition, the Department did not want to have to rely on the availability of a web site if emergency applications were needed. The new PDT-IDF curves are developed from the NOAA Atlas 14, Version 3 data. PennDOT designers and their consultants are familiar with the PDT-IDF procedure. PennDOT did not want designers and reviewers gathering information from the NOAA Atlas 14 website at two different points in time and obtaining different values.

8. Will Publication 13M, Design Manual. Part 2, *Highway Design*, Chapter 10 have its PDT-IDF curves replaced with the new curves?

- The old (1986) PDT-IDF curves will be taken out of Publication 13M, Design Manual. Part 2, *Highway Design*, Chapter 10. The new (2007) PDT-IDF curves are now available in Publication 584, *PennDOT Drainage Manual*, Chapter 7, Appendix A, *Field Manual For Pennsylvania Design Rainfall Intensity Charts From NOAA Atlas 14 Version 3 Data* and will no longer be available in Publication 13M, Design Manual. Part 2, *Highway Design*, Chapter 10. Rather, Publication 13M, Design Manual. Part 2, *Highway Design*, Chapter 10 will reference Publication 584, *PennDOT Drainage Manual*, Chapter 7, Appendix A, *Field Manual For Pennsylvania Design Rainfall Intensity Charts From NOAA Atlas 14 Version 3 Data* for new rainfall data.

9. Will the new (2007) PDT-IDF curves be available in an interactive setting (computer program) that will allow users to import a drainage area boundary and obtain the corresponding rainfall data?

- Not at this time.

10. Can the 1986 PDT-IDF curves continue to be used for PennDOT projects?

- With the issuance of Publication 584, the 1986 PDT-IDF curves will no longer be acceptable for projects on all projects that have waterway and/or NPDES permit applications submitted to PA DEP and/or the County Conservation District after February 27, 2009. Exceptions to this policy must be requested by December 1, 2008. Requests should be submitted from the District Executive to the Director of the Bureau of Design to the attention of the Chief Bridge Engineer. The new PDT-IDF Curves are to be implemented on new and existing designs when no additional cost or delay will occur.

B. Technical Questions.

1. Why are the rainfall intensity and amount curves (Figures 7A.7 through 7A.16) truncated at 360 minutes?

- For accuracy in reading the curves. When the curves were developed, each curve was originally plotted with the 12-hour and 24-hour duration data included. This resulted in a scale of the curves that was skewed. This made it difficult to interpolate rainfall values from the curves. For most projects in which the Rational Method is used to determine flow, the time of concentration (T_c) will be used as the storm duration. The T_c for a small watershed (<200 acres) will almost always be less than 360 minutes. Therefore, it was decided to truncate the curves at 360 minutes to increase their accuracy of use and put the 12- and 24-hour rainfall values in Tables 7A.2 through 7A.6. The curves can be used for where the storm duration falls between the increments in those tables up to 360 minutes.

2. Can the rainfall data from the 2007 PDT-IDF curves be utilized for hydrologic methods or models such as HEC-1, HEC-HMS, TR-55, TR-20 and EFH-2?

- Yes. The rainfall data should be used for all methods and models that require rainfall data for watershed modeling.

3. Why is there only one duration value (24-hour) for the 500-year storm in Tables 7A.2 through 7A.6 and in Map F? Could Map F be used for other 500-year storm durations?

- The 500-year storm should only be required with a 24-hour duration. The 500-year storm event is used as the superflood to check potential scour. Therefore, the standard 24-hour storm is the only duration that will be used to determine the effects of a 500-year storm for most projects.

C. Using Publication 584, *PennDOT Drainage Manual*, Chapter 7, Appendix A, Section 7A.1.

1. How are rainfall intensities and amounts obtained for storms with durations greater than 360 minutes?

- Use Tables 7A.2 through 7A.6 for 12-hour and 24-hour duration storms. Although the curves are truncated at 360 minutes (6 hours) on the duration axis, the data for 12- and 24-hour duration storms are still included in Tables 7A.2 through 7A.6. These tables present the total rainfall for durations of 5-, 10-, 15-, 30-, 60-, 120-, 180-, 360-, 720-, and 1440-minutes for the 1-, 2-, 5-, 10-, 25-, 50-, and 100-year storm frequencies. The rainfall total for a 100-year, 24-hour storm in Region 3 may be read from Tables 7A.4(a) or 7A.4(b) as 17.35 cm (6.83 inches). The intensity can be calculated as the total depth divided by the duration:

  17.83 cm / 24 hours = 0.74 cm/hr **(Metric)**
  6.83 inches / 24 hours = 0.28 in/hr **(US Customary)**

2. If the duration falls directly between two duration values in Table 7A.1, how is it determined which map should be used?

- Select the regional map by rounding to the closest duration shown in the table. If a duration value falls between two storm durations, the storm duration in Table 7A.1 closest to the required storm duration would be utilized to obtain the Rainfall Region Map. For instance, for a 10-year storm with a watershed T_c of 12.5 minutes, (which would equate to a storm duration of 12.5 minutes), the rounded up 15 minute storm duration would be chosen from Table 7A.1 to obtain Map A. If a storm duration of 12.4 minutes was obtained, the rounded down value of 10 minutes would produce the use of Map C for the 10-year storm.

  For most drainage designs where the drainage area to the project site is less than 200 acres, the Rational Method is applied to determine flow rates. The time of concentration (T_c) is used as the storm duration, and typically results in storm durations in between those on the curves. If a 25-year storm is being analyzed in Johnstown and the duration is 23.5 minutes, Region 3 from Map A becomes the appropriate region because 23.5 minutes is more than midway between 15- and 30-minute durations.

3. If I rounded the storm duration up or down to obtain the Rainfall Region Map from Table 7A.1, which duration would I use when it comes time to obtain the rainfall total or intensity from the curves (Figures 7A.7 through 7A.16) or from Tables 7A.2 through 7A.6?

- Use the actual storm duration calculated. To obtain the rainfall total or intensity from the curves or tables 7A.2 through 7A.6, one would use the actual storm duration required. To obtain the rainfall total values for a duration that falls between two categories in Tables 7A.2 through 7A.6, one would take the weighted average based upon the difference in rainfall values of the two durations on either side of the design duration. For instance, if the 25-year, 500-minute storm total was required for Region 1, the following calculations would be required:

Metric	US Customary
o 25-year, 360-minute storm total = 7.22 cm	o 25-year, 360-minute storm total = 2.84 inches
o 25-year, 720-minute storm total = 8.93 cm	o 25-year, 720-minute storm total = 3.52 inches
o (500 min-360 min)/(720 min-360 min) = 0.39 or 39 %	o (500 min-360 min)/(720 min-360 min) = 0.39 or 39 %
o (8.93 cm - 7.22 cm) = 1.71 cm x 0.39 = 0.67 cm	o (3.52 in - 2.84 in) = 0.68 in x 0.39 = 0.27 inches
o 7.22 cm + 0.67 cm = 7.89 cm for a 500-minute duration	o 2.84 in + 0.27 in = 3.11 inches for a 500-minute duration

4. For the rational method, or where rainfall intensity is required, and the storm duration is between duration values listed in Tables 7A.2 through 7A.6, how is the rainfall total determined? Will linear interpolation between values produce accurate results?

- Use the curves to obtain the rainfall intensity. Whereas the actual duration of a storm (time of concentration of a watershed) is not used to determine the region (12-minutes is between 10-minutes and 15-minutes and is closer to 10-minutes, therefore, a 10-minute duration is used to determine the appropriate region), the actual duration of a storm should be used to determine the rainfall intensity. Because the curves were plotted on logarithmic scale, linear interpolation will not produce accurate results. The physical duration should be read along the x-axis of the appropriate region plot, and a vertical line should be drawn to intersect the design storm of interest. The corresponding rainfall intensity value may then be read along the y-axis of the plot.

  Example: The duration is equal to the time-of-concentration of 12 minutes, and the design storm is the 10-year storm.

  The original T_c (or duration) value should be utilized to obtain the rainfall intensity.

  From Table 7A.1 Map C would be the appropriate map. From Map C, assume the site would fall in Region 4.

  From the PDT-IDF curve for Region 4, the 10-year, 12-minute storm would be 12.2 cm/hr (4.8 in/hr).

5. What is the process to be used for projects located in two different regions?

- For small sites, use the average value for the two regions. For large watersheds, a weighted average should be developed. If a basin should be found to lie on the boundary between two regions, the rainfall totals or intensities should be obtained from the two corresponding regional graphs and averaged. In the case that the basin is large enough to be divided into areas A_i and A_j of measurable size in the adjacent regions i and j, a weighted average intensity may be used.

$$I = \frac{I_i A_{i+} I_j A_j}{A_{i+} A_j}$$

6. What if a site is in one region for one frequency or duration, and another region for another frequency or duration?

- Just follow the process established for each storm frequency and duration. Should a project be located in an area such that the region changes between different storm durations or frequencies, each storm should be analyzed separately with the appropriate rainfall region determined for the desired storm duration and frequency. For example, the City of Johnstown in Cambria County (40° 20' N, 78° 55' W) is in Region 4 for a 25-year storm with a 15-minute duration (Map C). However, the City is in Region 3 for a 25-year storm with a 30-minute duration (Map A).

7. Can the curves be applied to projects in which the drainage area lies in a portion of another state?

- Yes. If more than 50% of the drainage area is located in Pennsylvania, the Pennsylvania rainfall curves may be used for the entire project. If more than 50% of the drainage area is in another state, rainfall data should be obtained for both states. Should the rainfall data appear to be consistent for both states a weighted average may be used. If the data is not consistent between states, the Pennsylvania rainfall data should be used.

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CHAPTER 7, APPENDIX C

IMPLEMENTATION GUIDE FOR PENNSYLVANIA DESIGN RAINFALL INTENSITY CHARTS FROM NOAA ATLAS 14 VERSION 3 DATA

7C.0 INTRODUCTION

Several examples on how to use the 2007 PDT-IDF curves were developed to guide designers with the use of these curves. These examples are presented in this appendix.

7C.1 OBJECTIVES

The Pennsylvania Design Rainfall Intensity Charts have been updated using NOAA Atlas 14, Version 3 data, including data from 278 daily and 139 hourly rainfall gages for a period of record from April 1, 1863 through December 31, 2000. The new rainfall charts and 2007 PDT-IDF curves are presented in Chapter 7, Appendix A, *Field Manual For Pennsylvania Design Rainfall Intensity Charts From NOAA Atlas 14 Version 3 Data*. The curves replace the previously used 1986 Field Manual of PennDOT Storm-Intensity-Duration-Frequency Charts PDT-IDF to estimate design rainfall intensities.

The purpose of this appendix is to determine the precipitation intensities and depths for various storm frequencies. The storm frequency of interest will be given along with the project location. Refer to Chapter 7, Appendix A, *Field Manual For Pennsylvania Design Rainfall Intensity Charts From NOAA Atlas 14 Version 3 Data* for PennDOT's rainfall regions and precipitation tables and curves.

The procedure outlined in Chapter 7, Appendix A, *Field Manual For Pennsylvania Design Rainfall Intensity Charts From NOAA Atlas 14 Version 3 Data* for finding design intensity values will be used during this workshop and is restated below.

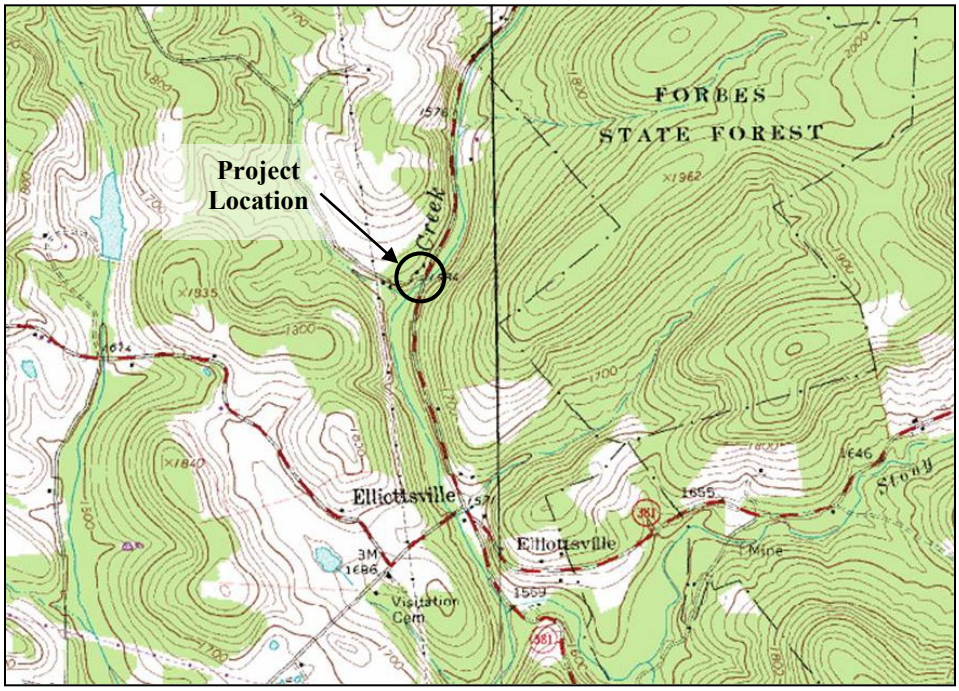
Step 1 – Determine the rainfall duration of the storm that needs to be analyzed (typically 24 hours). For the Rational method, the storm duration is equal to the time-of-concentration.
Step 2 – Use Table 7A.1 to determine which Rainfall Region Map should be used.
Step 3 – Locate the basin area on the Map determined in Step 2 and note the rainfall region.
Step 4 – Using either Tables 7A.2-7A.6 or Figures 7A.7-7A.16, determine the rainfall intensity or total rainfall.

7C.2 PRECIPITATION FOR 24-HOUR STORM DURATION

The next few examples illustrate obtaining precipitation depths for 24-hour storms. For project sites that meet the criteria for the HEC-1, HEC-HMS, TR-55, and EFH-2 hydrologic models, a 24-hour precipitation depth is required as an input.

A. Bridge Replacement over Big Sandy Creek. Perform the H&H analysis for a bridge replacement on Wharton Furnace Road over Big Sandy Creek in Fayette County, Pennsylvania. The project is located at 39°46'38"N, 79°37'40"W and the total drainage area to the bridge is 15.6 square miles. In order to calculate the hydrology using the HEC-1 or HEC-HMS models, you will need a precipitation depth in inches. Find the 24-hour precipitation depth for the 25-year event.

Figure 7C.1. Project location map for bridge replacement over Big Sandy Creek.



Step 1 – Rainfall duration and storm frequency to be analyzed: 24-hour, 25-year
Step 2 – Determine Rainfall Region Map (Table 7A.1) for storm frequency/duration: Map F
Step 3 – Region where the drainage area is located on the Rainfall Region Map: Region 1

Is the drainage area on the boundary or split between two regions (circle one)?

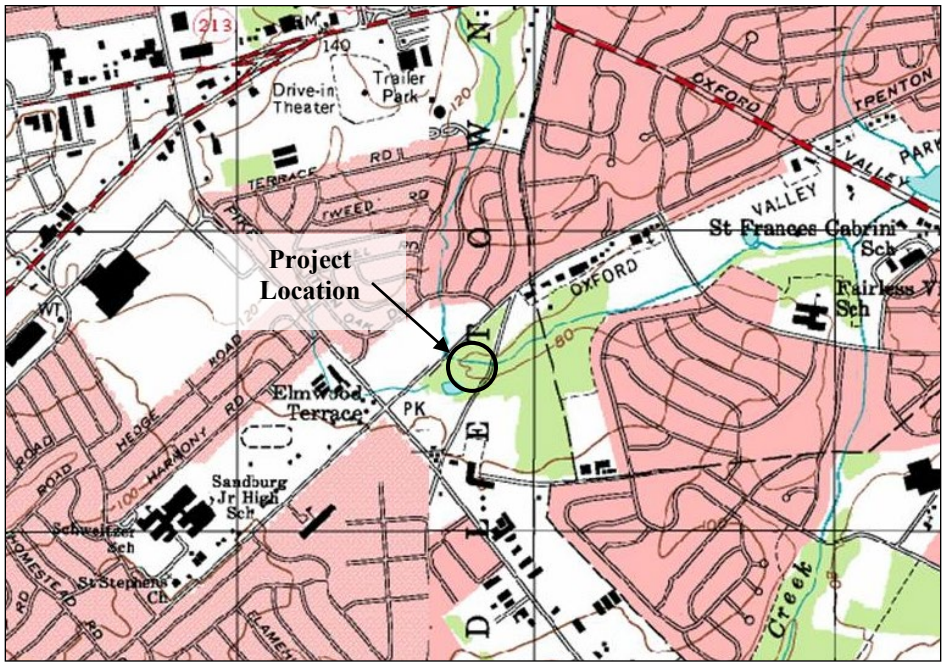
Yes (No)

If the drainage area is between two regions, the rainfall intensities from the two corresponding regional graphs should be averaged. If the basin is large enough to measure the drainage area in the adjacent rainfall regions, a weighted average intensity may be used.

Step 4 – Determine the rainfall depth: 4.09 inches [Table 7A.2(b)]

B. Culvert Replacement on Queen Anne Creek Tributary. Perform the H&H analysis for a culvert replacement on Trenton Road over a tributary to Queen Anne Creek in Bucks County, Pennsylvania. The project is located at 40°10'10"N, 74°52'27"W and the drainage area to the crossing is 1.27 square miles. In order to calculate the hydrology using the TR-55 method, you will need a precipitation depth in inches. Find the 24-hour precipitation depth for the 100-year event.

Figure 7C.2. Project location map for culvert replacement over Queen Anne Creek Tributary.



Step 1 – Rainfall duration and storm frequency to be analyzed: 24-hour, 100-year
Step 2 – Determine Rainfall Region Map (Table 7A.1) for storm frequency/duration: Map F
Step 3 – Region where the drainage area is located on the Rainfall Region Map: Region 5

Is the drainage area on the boundary or split between two regions (circle one)?

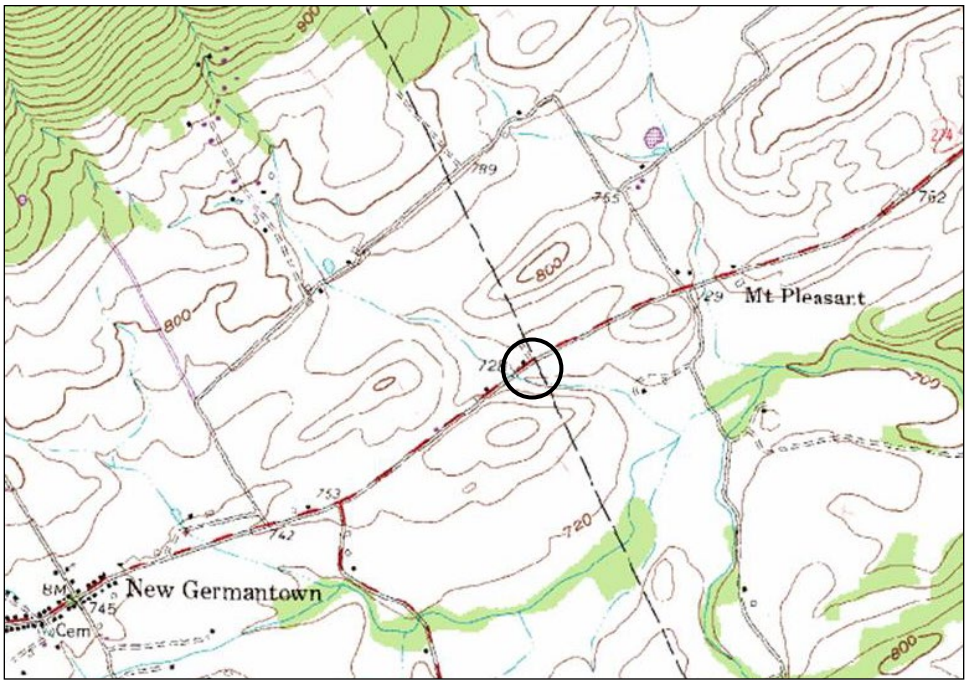
Yes (No)

If the drainage area is between two regions, the rainfall intensities from the two corresponding regional graphs should be averaged. If the basin is large enough to measure the drainage area in the adjacent rainfall regions, a weighted average intensity may be used.

Step 4 – Determine the rainfall depth: 8.43 inches [Table 7A.6(b)]

C. Culvert Replacement over Sherman Creek Tributary. Perform the H&H analysis for a small culvert replacement on SR 0274 over a tributary to Sherman Creek in Perry County, Pennsylvania. The project is located at 40°18'57"N, 77°33'10"W and the drainage area to the crossing is 0.97 square miles. In order to calculate the hydrology using the HEC-1, HEC-HMS, and EFH-2 models, you will need a precipitation depth in inches. Find the 24-hour precipitation depth for the 25-year event.

Figure 7C.3. Project location map for culvert replacement over Sherman Creek Tributary.



Step 1 – Rainfall duration and storm frequency to be analyzed: 24-hour, 25-year
Step 2 – Determine Rainfall Region Map (Table 7A.1) for storm frequency/duration: Map F
Step 3 – Region where the drainage area is located on the Rainfall Region Map: Region 3

Is the drainage area on the boundary or split between two regions (circle one)?

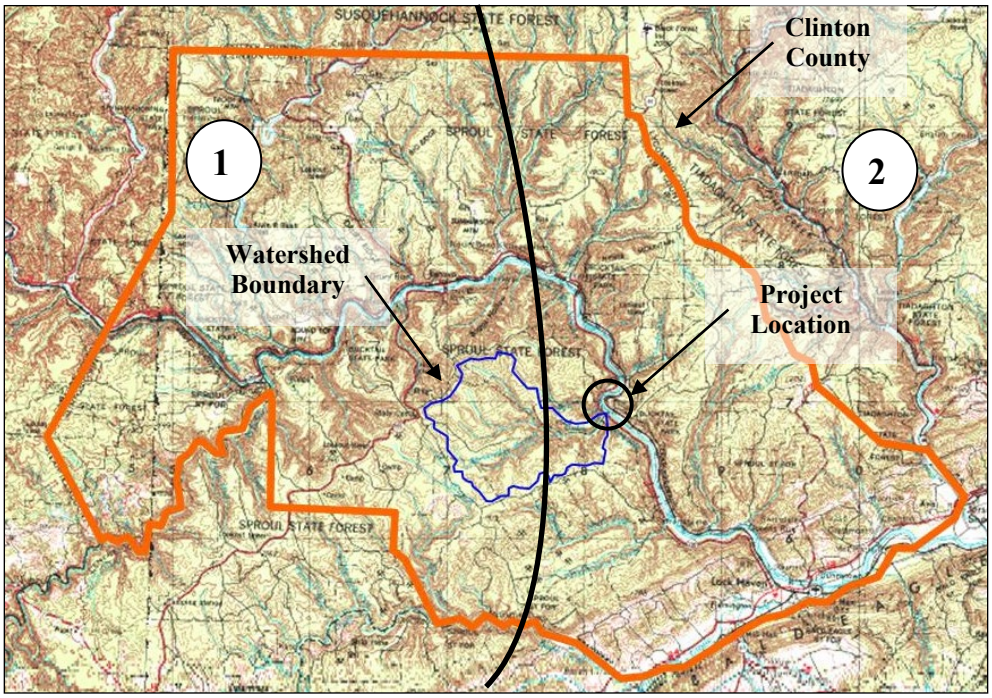
Yes (No)

If the drainage area is between two regions, the rainfall intensities from the two corresponding regional graphs should be averaged. If the basin is large enough to measure the drainage area in the adjacent rainfall regions, a weighted average intensity may be used.

Step 4 – Determine the rainfall depth: 5.10 inches [Table 7A.4(b)]

D. Bridge Replacement over Baker Run. Perform the H&H analysis for the SR 0120 bridge replacement over Baker Run in Clinton County, Pennsylvania. The project is located at the eastern edge of the watershed at 41°14'45"N, 77°36'28"W. The drainage area to the crossing is 35 square miles and the western edge of the watershed is at 41°14'12"N, 77°45'02"W. In order to calculate the 100-year flow using the HEC-1 and HEC-HMS models, you will need a precipitation depth in inches. Find the 24-hour precipitation depth.

Figure 7C.4. Project location map for bridge replacement over Baker Run.



Step 1 – Rainfall duration and storm frequency to be analyzed: 24-hour, 100-year
Step 2 – Determine Rainfall Region Map (Table 7A.1) for storm frequency/duration: Map F
Step 3 – Region where the drainage area is located on the Rainfall Region Map: Region 1-2

Is the drainage area on the boundary or split between two regions (circle one)?

(Yes) No

If the drainage area is between two regions, the rainfall intensities from the two corresponding regional graphs should be averaged. If the basin is large enough to measure the drainage area in the adjacent rainfall regions, a weighted average intensity may be used. Estimate 60% in Region 1 and 40% in Region 2

Step 4 – Determine the rainfall depth: 5.56 inches (5.24 inches for Region 1 [Table 7A.2(b)] x 0.60 + 6.03 inches for Region 2 [Table 7A.3(b)] x 0.40)

7C.3 PRECIPITATION FOR RATIONAL METHOD

The next few examples illustrate obtaining rainfall intensities (inches/hour) for project sites that meet the criteria for the Rational method. Remember that the storm duration must equal the time of concentration. If the drainage area is between two regions, the rainfall intensities from the two corresponding regional graphs should be averaged.

A. Drainage Design, Blair County. Design a drainage pipe for a project near Altoona in Blair County, Pennsylvania. The project is located at 40°28'33"N, 78°23'39"W. The time of concentration for the drainage area to the pipe is 5 minutes and you are sizing the pipe for the 10-year event.

Step 1 – Rainfall duration and storm frequency to be analyzed: 5-minute, 10-year
Step 2 – Determine Rainfall Region Map (Table 7A.1) for storm frequency/duration: Map C
Step 3 – Region where the drainage area is located on the Rainfall Region Map: Region 3

Is the drainage area on the boundary or split between two regions (circle one)?

Yes (No)

Step 4 – Determine the rainfall intensity: 6.1 inches/hour [Figure 7A.12(a)]

B. Pipe Replacement, Tioga County. Replace a drainage cross-pipe for a project near Tioga in Tioga County, Pennsylvania. The project is located at 41°54'56"N, 77°08'53"W. The time of concentration for the drainage area to the pipe is 15 minutes and you are designing the pipe for the 10-year event.

Step 1 – Rainfall duration and storm frequency to be analyzed: 15-minute, 10-year
Step 2 – Determine Rainfall Region Map (Table 7A.1) for storm frequency/duration: Map A
Step 3 – Region where the drainage area is located on the Rainfall Region Map: Region 1

Is the drainage area on the boundary or split between two regions (circle one)?

Yes (No)

Step 4 – Determine the rainfall intensity: 3.4 inches/hour [Figure 7A.8(a)]

C. Culvert Design, Lackawanna County. Perform the drainage design for a project near Old Forge in Lackawanna County, Pennsylvania. The project is located at 41°23'00"N, 75°45'00"W. The time of concentration for the drainage area to the pipe of interest, which is a small culvert, is 20 minutes and you need to calculate the 100-year flow to assess the risk of flooding upstream from the crossing.

Step 1 – Rainfall duration and storm frequency to be analyzed: 20-minute, 100-year
* Round 20-minute T_c to 15-minute duration to use Table 7A.1
Step 2 – Determine Rainfall Region Map (Table 7A.1) for storm frequency/duration: Map C
Step 3 – Region where the drainage area is located on the Rainfall Region Map: Region 2-3

Is the drainage area on the boundary or split between two regions (circle one)?

(Yes) No

Step 4 – Determine the rainfall intensity: 4.5 inches/hour (4.3 inches/hour for Region 2 [Figure 7A.10(a)] + 4.7 inches/hour for Region 3 [Figure 7A.12(a)] / 2)

D. Drainage Design, York County. Perform the drainage design for the SR 74 project in York City, York County, Pennsylvania. The project is located at 39°57'00"N, 76°43'00"W. The time of concentration for the drainage area to the pipe of interest is 7.5 minutes and you need to calculate the 100-year flow.

Step 1 – Rainfall duration and storm frequency to be analyzed: 7.5-minute*, 100-year
* Round 7.5-minute T_c to 10-minute duration to use Table 7A.1
Step 2 – Determine Rainfall Region Map (Table 7A.1) for storm frequency/duration: Map C
Step 3 – Region where the drainage area is located on the Rainfall Region Map: Region 4

If they rounded down to 5 minutes, they would have used Map B and located the site in Region 2 – this is not correct.

Is the drainage area on the boundary or split between two regions (circle one)?

Yes (No)

Step 4 – Determine the rainfall intensity: 7.8 inches/hour [Figure 7A.14(a)]

CHAPTER 8

OPEN CHANNELS

8.0 INTRODUCTION TO OPEN CHANNELS

A. Open Channels. An open channel is a defined area consisting of a free water surface subject to atmospheric pressure. Open channels may be natural or manmade. Natural streams usually consist of a normal or low flow channel and adjacent floodplains. In this chapter, the term "open channel" will include the total conveyance facility (the floodplain and stream channel).

Open channel hydraulics is of particular importance to highway design because of the interrelationship of channels to all highway hydraulic structures. In the hydraulic analysis and design of bridges and culverts, open channel hydraulic principles are utilized to evaluate the effects of proposed structures on water surface profiles, flow and velocity distributions, lateral and vertical stability of the channel, stream regime, flood risk, and the potential reaction of the stream to changes in variables such as structure type, shape, location and scour control measures. Any channels that qualify as waters of the Unites States will require a Chapter 105 permit if they are encroached.

The hydraulic design process for open channels consists of establishing criteria, developing and evaluating alternatives, and selecting the alternative which best satisfies the established criteria.

The purposes of this chapter are to specify required design criteria, discuss design strategies, and outline channel design and analysis procedures. The principles of open channel flow hydraulics are applicable to all drainage facilities, including culverts. The two types of open channels that are examined in this Chapter are natural stream channels and artificial roadside channels or ditches.

Stream channels are usually natural channels with their sizes and shapes determined by natural forces. The cross section of a stream channel ordinarily consists of a main channel that conveys low flows and a floodplain that transports flood flows. Stream channels are usually shaped over time by sediment loads and water discharges.

Artificial channels are man-made channels with regular geometric cross sections. Artificial channels, such as roadside channels, and drainage ditches, can be lined with erosion-resistant material if necessary.

A roadside channel is normally trapezoidal or "V"-shaped in cross section and lined with grass or a special protective lining (flexible or rigid). Roadside channels often parallel the highway embankment within the limits of the highway right-of-way.

The primary function of a roadside channel is to collect surface runoff from the highway and areas which drain to the right-of-way, and to convey the accumulated runoff to acceptable outlet points. Another 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.

B. Design Process. The hydraulic design process for open channels consists of establishing criteria, developing and evaluating alternatives, and selecting the alternative which best satisfies the criteria. Capital investment and future costs may be considered, including maintenance and flood damages to properties, traffic service requirements, and the stream and floodplain environment. Risks should be evaluated as warranted by the flood hazard at the site, economics, and current engineering practices.

8.1 CHANNEL DESIGN CRITERIA

A. Stream Channel Criteria. The hydraulic effects of floodplain encroachments should be evaluated for a range of discharges for any major highway facility. These discharges may include the discharges associated with the design flood, the 100-year flood, the regulatory floods, superflood(s), and designated stormwater management discharges. Relocation or realignment of a stream channel should be avoided wherever practical. If relocation is necessary, the cross sectional shape, plan-view, roughness, sediment transport, and slope should conform to the original conditions insofar as practical. Some means of energy dissipation may be necessary when channel velocities are excessive, or when the original conditions cannot be duplicated. On the other hand, instream structures, such as rock vanes, may be necessary to increase channel velocities of low flows passing through a bridge in order to reduce the amount of sediment that builds up under a bridge and, in turn, reduce future maintenance.

Stream bank stabilization should be provided, when appropriate, to counteract any stream disturbance such as an encroachment. Stream encroachments can be defined as a structure or activity which changes, expands or diminishes the course, current or cross section of a watercourse, floodway, or body of water. Both upstream and downstream banks, as well as the local site, should be stabilized.

Features such as dikes and levees associated with natural channel modifications should have a sufficient top width for access for maintenance equipment. Turnaround points should be provided throughout and at the end of any such feature.

B. Roadside Channel Criteria. The alignment, cross section, and grade of roadside channels are usually constrained 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.

Roadside channel side slopes usually are defined by the roadway cross section. The side slopes of an unlined channel must not exceed the angle of repose of the soil. Roadside channels may be designed with rigid or flexible linings. Flexible linings in channels are better suited to a changing channel shape than rigid linings, which is one of many reasons why flexible linings are generally preferred. However, rigid linings may be better suited for highly-erosive flows.

8.2 OPEN CHANNEL HYDRAULIC PRINCIPLES

A. Continuity Equation. The continuity equation is derived from the conservation of mass in fluid mechanics. For the special case of steady flow of an incompressible fluid, it assumes the following form:

(Equation 8.1)

$$Q = A_1 v_1 = A_2 v_2$$

where: Q = discharge, m^3/s (cfs)
A = flow cross sectional area, m^2 (ft^2)
v = mean cross sectional velocity, m/s (ft/s) (perpendicular to the cross section)

The subscripts 1 and 2 refer to successive cross sections along the flow path. The continuity equation also assumes that the velocities v_1 and v_2 are perpendicular to the areas A_1 and A_2, respectively.

B. Channel Capacity. Most open channel computational procedures use the Manning Equation for uniform flow as a basis for analysis.

(Equation 8.2)

$$v = \frac{x}{n} R^{2/3} S^{1/2}$$

where: x = 1.0 (metric), 1.486 (U.S. Customary)
n = Manning roughness coefficient (a coefficient for quantifying the roughness characteristics of the channel)

R = hydraulic radius, m (ft), R = A / P
P = wetted perimeter of flow (the length of the cross section in direct contact with the water), m (ft)
S = slope of the energy grade line, m/m (ft/ft) (Note: For uniform, steady flow, S is also equal to the channel slope, m/m (ft/ft)

Manning's Equation can be combined with the continuity equation to determine the channel uniform flow capacity, as shown in Equation 8.3.

(Equation 8.3)

$$Q = \frac{x}{n} A R^{2/3} S^{1/2}$$

For convenience, Manning's Equation in this chapter will assume the form of Equation 8.3. Since Manning's Equation does not allow a direct solution for depth of water (given discharge, longitudinal slope, roughness characteristics, and channel dimensions), an indirect solution to compute the depth of flow is necessary. An alternative method is to develop a stage-discharge relationship for flow in the stream.

Procedures for developing the stage-discharge relationship include certain basic parameters:

- Geometric descriptions of typical cross sections.
- Identification and quantification of stream roughness characteristics.
- A longitudinal water surface slope.

Because of their importance and significance to the final result, careful consideration by the designer is necessary for an appropriate selection and estimation of these parameters.

C. Conveyance. In channel analysis, it is often convenient to group the channel cross sectional properties in a single term called the channel conveyance (K). Conveyance represents the carrying capacity of a stream cross section based entirely upon its geometry and roughness characteristics as shown in Equation 8.4.

(Equation 8.4)

$$K = \frac{x}{n} A R^{2/3}$$

Manning's Equation can then be written as:

(Equation 8.5)

$$Q = K S^{1/2}$$

Conveyance is useful when computing the distribution of overbank flood flows in the cross section and the flow distribution through the opening in a proposed stream crossing (see Section 8.2.R.).

D. Total Energy. A form of the Bernoulli Equation is used to show the total energy in a cross section, which is the sum of a flow's kinetic and potential energies as shown in Equation 8.6.

(Equation 8.6)

$$H = \frac{P}{\gamma_w} + z + \frac{v^2}{2g}$$

where: H = total energy head, m (ft)
P = pressure, N/m^2 (lb/ft^2)
γ_w = unit weight of water, 9810 N/m^3 (62.4 lb/ft^3)
z = elevation head, m (ft)
$v^2/2g$ = average velocity head, h_v, m (ft)
g = gravitational acceleration, 9.81 m/s^2 (32.2 ft/s^2)

In open channel computations, it often is useful to define the total energy head as the sum of the specific energy head and the elevation of the channel bottom with respect to some datum.

(Equation 8.7a)

$$H = z + E$$

$$E = y + \frac{v^2}{2g}$$

where: y = depth of flow, m (ft)

It should be noted that the pressure head, P/γ_w, has been replaced by the depth of flow. This assumes that the pressure term in Equation 8.6 can be approximated by hydrostatic pressure force.

For some applications, it may be more practical to compute the total energy head as a sum of the water surface elevation (relative to mean sea level) and velocity head.

(Equation 8.7b)

$$H = WS + \frac{v^2}{2g}$$

where: WS = water-surface elevation or stage, m (ft) = z + y

E. 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, the specific energy becomes the sum of the depth of flow and velocity head.

(Equation 8.8)

$$E = y + \alpha \frac{v^2}{2g}$$

where: α = kinetic energy coefficient, as described below.

F. Kinetic Energy Coefficient. (α) Channel roughness, non-uniformities in channel geometry, bends, and upstream obstructions are some of the numerous factors that cause variations in velocity from point to point in a cross section. Because the velocity distribution in a river varies from a maximum in the main channel to a minimum along the banks, the average velocity head does not give a true measure of the kinetic energy of the flow. A weighted average value of the kinetic energy is obtained by multiplying the average velocity head by the kinetic energy coefficient (α). The kinetic energy coefficient is taken to have a value of 1.0 for turbulent flow in prismatic channels (channels of constant cross section, roughness, and slope), but may be significantly different than 1.0 in natural channels. The kinetic energy coefficient can be computed with the following equation (Equation 2.11 from the HEC-RAS Hydraulic Reference Manual):

(Equation 8.9)

$$\alpha = \frac{\sum \left(q_i v_i^2 \right)}{Q v^2} = \frac{A_t^2 \sum \frac{K_i^3}{A_i^2}}{K_t^3}$$

where: v_i = average velocity in subsection, m/s (ft/s) (see Section 8.2.R.)
q_i = discharge in subsection i, m^3/s (cfs) (see Section 8.2.R.)
K_i = conveyance in subsection i, m^3/s (cfs) (see Section 8.2.C.)
A_i = flow area of same subsection i, m^2 (ft^2)
K_t = total conveyance for cross section, m^3/s (cfs)
A_t = total flow area of cross section, m^2 (ft^2)

The subsections should isolate any places where ineffective or upstream flow is suspected. Then, by omitting the subsections or assigning very large roughness coefficients to them, a more realistic kinetic energy coefficient will be computed. In some cases, kinetic energy coefficients in excess of 3 may be calculated. If adjacent cross sections have comparable values, or if the changes are not sudden between cross sections, such values can be accepted. If the change

is sudden, however, some attempt should be made to attain uniformity, such as using more cross sections to achieve gradual change.

G. Energy Balance and the Energy Grade Line. The parameters in the Energy Equation are illustrated in Figure 8.1. The Energy Balance Equation relates the total energy of an upstream section (1) along a channel with the total energy of a downstream section (2).

(Equation 8.10)

$$z_1 + y_1 + \alpha_1 \frac{v_1^2}{2g} = z_2 + y_2 + \alpha_2 \frac{v_2^2}{2g} + h_f + other\ losses$$

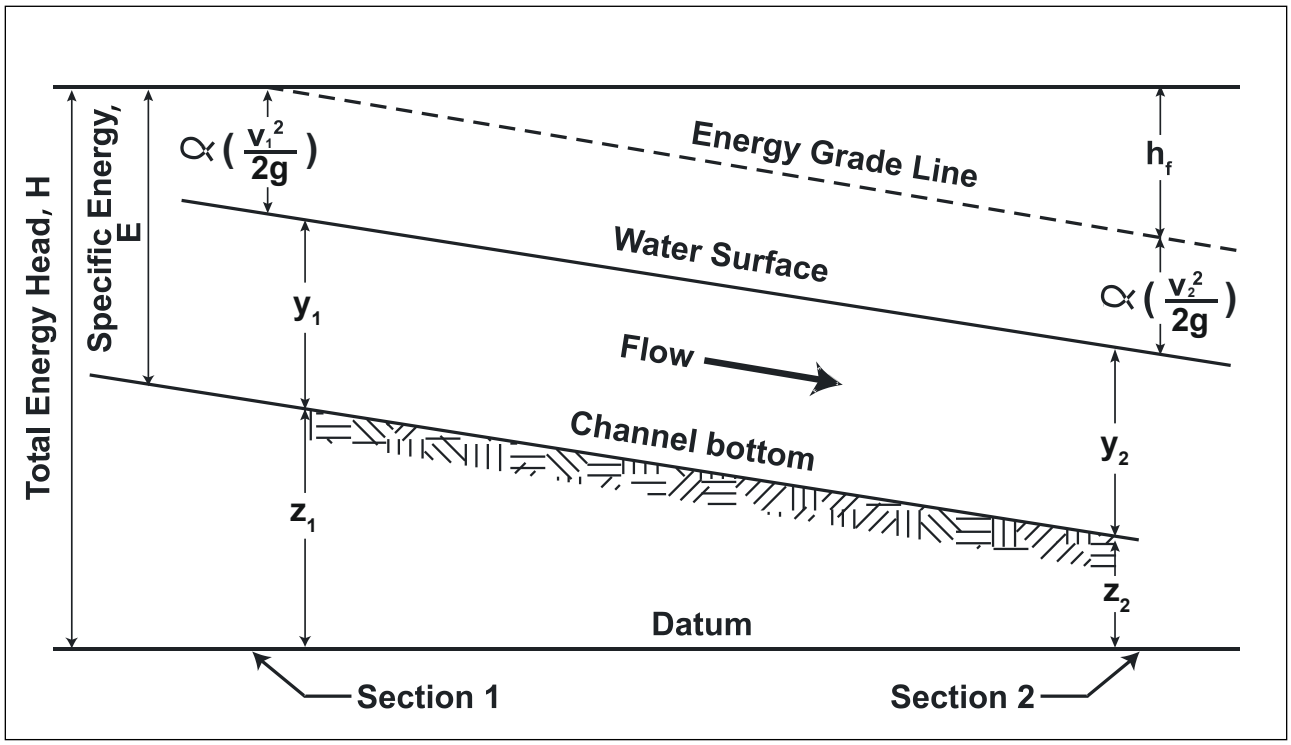
where: h_f = friction head loss from upstream to downstream, m (ft)

The energy grade line (EGL) is the line that joins the elevations of the total energy head associated with a water surface profile (see Figure 8.1).

H. Uniform Depth. Uniform depth (y_n) of flow (sometimes referred to as normal depth of flow) occurs when there is uniform flow in a channel or conduit. Uniform depth occurs when the discharge, slope, cross sectional geometry, and roughness characteristics are constant through a reach of stream.

I. Critical Depth. By plotting specific energy against depth of flow for constant discharge, a specific energy diagram is obtained (see Figure 8.2). When specific energy is minimum, the corresponding depth is critical depth (y_c). Critical depth of flow is a function of discharge and channel geometry, and is independent of slope. For a given discharge and cross section, only one critical depth exists.

Figure 8.1 EGL for Water Surface Profile



Critical depth (y_c) can be calculated with the following equation:

(Equation 8.11)

$$y_c = \sqrt[3]{\frac{q^2}{g}}$$

where: q = discharge per unit of width, $\frac{m^3}{m \cdot s}\left(\frac{ft^3}{ft \cdot s}\right)$

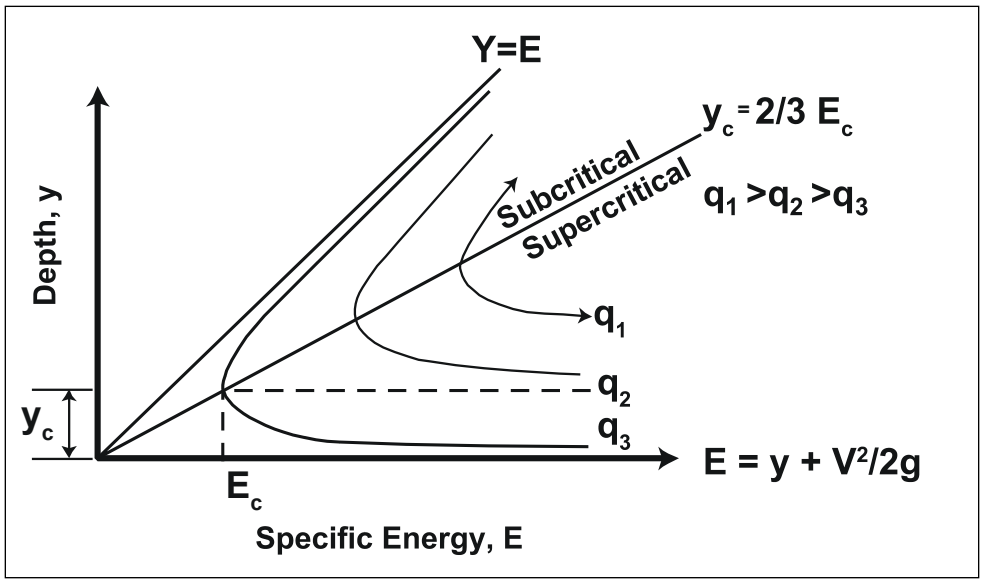
The critical depth for a given discharge and arbitrary cross section can be determined iteratively with the following equation:

(Equation 8.12)

$$\frac{Q^2}{g} = \frac{A^3}{T_c}$$

where: T_c = water surface width for critical flow, m (ft)

Figure 8.2 Typical Specific Energy Diagram



J. Froude Number. The Froude Number (F_r) is an important dimensionless parameter in open channel flow. It represents the ratio of inertial forces to gravitational forces and is calculated using Equation 8.13.

(Equation 8.13)

$$F_r = \sqrt{\frac{Q^2 T}{gA^3}}$$

where: T = channel top width at the water surface, m (ft)

This expression for the Froude number applies to any single-section channel of non-rectangular shape. For a rectangular channel, the Froude Number can be written as:

(Equation 8.14)

$$F_r = \frac{v}{\sqrt{gy}}$$

The Froude Number at critical depth is always 1.0, representing a balance between inertial and gravitational forces.

K. Flow Types. A brief description and discussion of the principal types of open channel flow are necessary since the methods of analysis, as well as the necessary assumptions, depend on the type of flow under study.

Open channel flow is usually classified as:

- Uniform or non-uniform.
- Steady or unsteady.
- Subcritical, critical, or supercritical.

Non-uniform, unsteady, subcritical flow is the most common type of flow in an open channel. Due to the complexity and difficulty involved in the analysis of non-uniform, unsteady flow, most hydraulic computations are made with certain simplifying assumptions which allow the application of steady, uniform, or gradually varied flow principles and one-dimensional methods of analysis as explained below.

L. Steady, Uniform Flow. Steady flow implies that the discharge at a point does not change with time, and uniform flow requires that there is no change in the magnitude or direction of velocity with distance along a streamline such that the geometry of flow does not change with distance along a channel. Steady, uniform flow is an idealized concept of open channel flow which seldom occurs in natural channels and is difficult to obtain in a hydraulic laboratory.

A further assumption of rigid, uniform boundary conditions is necessary to satisfy the conditions of constant flow depth along the channel. Alluvial, sand bed channels do not exhibit rigid boundary characteristics.

M. Gradually Varied Flow. For many practical highway applications the flow is steady and changes in width, depth, or direction (along a streamline) are sufficiently small so that the flow can be considered to be uniform (see Section 8.2.L.)

Changes in channel characteristics often occur over a relatively long distance so that the flow is varied gradually. Consideration of such flow conditions usually is reasonable for calculation of water surface profiles, especially for the hydraulic design of bridges.

N. Subcritical/Supercritical Flow. Many Pennsylvania streams flow in what is regarded as a subcritical flow regime (sometimes referred to as a mild slope regime). Subcritical flow occurs when uniform depth of flow is greater than critical depth of flow (see Figure 8.2). A Froude number less than 1.0 indicates subcritical flow. This type of flow is tranquil and slow, and implies flow control from the downstream direction; therefore, the calculations are carried out from downstream to upstream.

Supercritical flow is often characterized as rapid or shooting, with flow depths less than critical depth. A Froude number greater than 1.0 indicates supercritical flow. Control occurs from upstream and calculations are carried out from upstream to downstream.

The distinction between subcritical and supercritical flow is important in the analysis of open channel flow. The location of control sections and the method of analysis will depend on which type of flow occurs within the channel reach under study.

O. Cross Sections. The cross section should be representative of the stream reach. The choice of the cross section should not be left entirely to a field survey party. The location and orientation of the cross section used in the channel analysis should be considered carefully. Proper selection of cross sections is imperative to produce accurate and meaningful water surface profiles.

The distance that cross sections extend up or down stream will depend on the extent of changes in water surface elevation and whether flow is supercritical, subcritical, or mixed. The cross sections used for the hydraulic computations should have the following characteristics:

- Normal (perpendicular) to the general direction of stream flow.
- Representative of average conditions within the stream reach (i.e., geometry, slope, vegetative cover, and roughness).
- In wide floodplains, it may be necessary to bend the cross sections to ensure that the section is perpendicular to the streamlines or the flow direction. Adjacent cross sections cannot overlap each other.

Cross sections must extend outward to the highest expected water-surface elevation considered in the model (e.g., the survey needs to ensure that the highest elevation is above the expected water surface). Cross sections may be obtained by a direct field survey, photogrammetric methods, and/or extraction from contour maps where available.

Cross sections are representative of the geometric and roughness characteristics of the stream reach in question. Figure 8.3 provides examples of plotted cross sections. All cross sections used as part of the hydraulic analysis of a waterway should extend beyond the limits of the floodplain as depicted on cross section 3 of Figure 8.3.

The following suggestions are provided for location of cross sections:

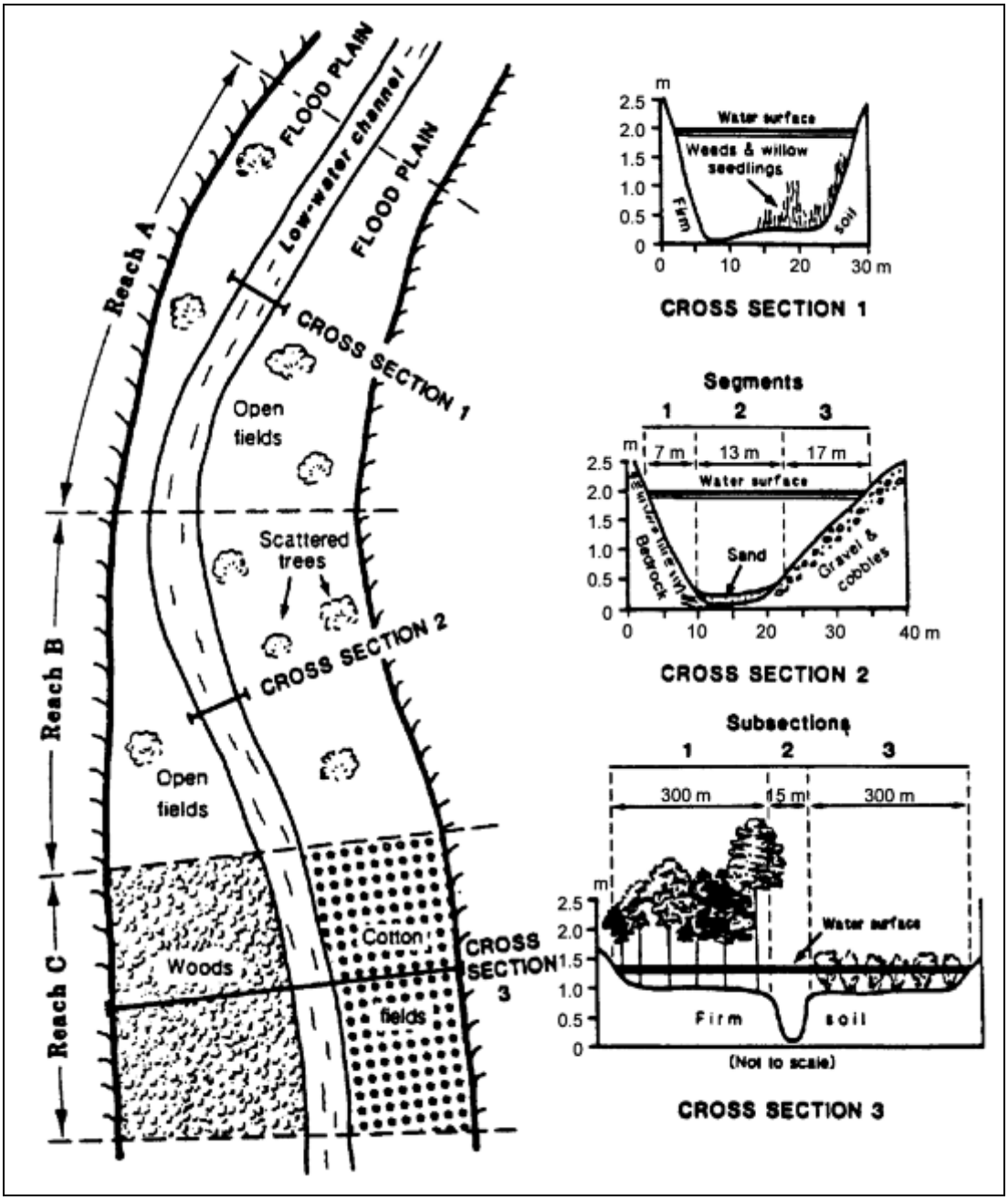
- Sections along the right-of-way line can be misleading hydraulically because they may represent only very local, cleared conditions which are not reflective of the stream reach. For similar reasons, avoid cross sections along utility easements and other narrow cleared areas.
- Try to avoid local depressions or crests that are not reflective of a whole stream reach.
- Generally, spacing between sections should be approximately 1.5 to 2 times the estimated floodplain width. A notable exception to this is at structures where more definition is required.
- Cross sections should be perpendicular to the flow.
- For structures, at a minimum the modeled reach should extend 150 m (500 feet) upstream and downstream of the structure and have bounding structure sections:
    - Immediately upstream of the proposed and/or existing crossings
    - Immediately downstream of the proposed and/or existing crossings
- More cross sections are required for extensive channel changes.
- Refer to the HEC-RAS manual for further guidance on cross section locations.

P. Roughness Coefficients. Roughness characteristics and elements in a channel offer resistance to flow through friction and are quantified in Manning Equation by means of "roughness coefficients." Roughness in channels may be attributed to any of the following:

- Boulders.
- Vegetation.
- Channel irregularities.
- Meanders.

Typically, roughness coefficients range from 0.009 to 0.175. The designer must quantify this parameter on the basis of experience, reference to design guide tables, and systematic estimating procedures.

Figure 8.3 Typical Cross Sections



Suggested values for the Manning roughness coefficient may be found in design charts such as Table 8.1. Any convenient, published design guide can be referenced for these values. Usually, reference to more than one guide can be productive in that more opinions are available. A productive and systematic approach for this task may be found in the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains* (FHWA, 1984).

In design it is important to understand that Manning's n-value determination may be inexact and subjective; however, once the Manning's n-values have been chosen *carefully*, they should not be adjusted just to provide another answer. If there is uncertainty about particular Manning's n-value choices, consult a more experienced designer.

When calibrating a water surface profile model, the parameters that typically are adjusted are the Manning's n-value, and the expansion and contraction coefficients.

Usually, it is not appropriate to assign a single roughness coefficient to a cross section for the reasons discussed in the following sections. Typical rivers and streams in Pennsylvania have very different roughness characteristics in the main channel versus the out-of-bank floodplains.

Q. Wetted Perimeter Weighted Manning's n-Value. In some instances, such as a trapezoidal section under a bridge, the Manning's n-value may vary drastically within a section, but the section should not be subdivided. If the Manning's n-value varies as such, then a weighted Manning's n-value (n_w) should be used. The equation for this procedure is defined by:

(Equation 8.15)

$$n_w = \left[\frac{\sum_{i=1}^{N} \left(P_i n_i^{1.5} \right)}{\sum P} \right]^{2/3}$$

where: P_i = subsection wetted perimeter, m (ft)
n_i = subsection Manning's n-value
$_i$ = subsection

Table 8.1 Roughness Coefficients n-values for Manning's Equation (Channels, Flood Plains and Excavated Channels)

Natural Streams			
	Minimum	**Normal**	**Maximum**
Main Channels:			
• Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
• Same as above, but more stones and weeds	0.030	0.035	0.040
• Clean, winding, some pools and shoals	0.033	0.040	0.045
• Same as above, but some weeds and stones	0.035	0.045	0.050
• Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
• Clean, winding, some pools and shoals, some weeds and many stones	0.045	0.050	0.060
• Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
• Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages:			
• Bottom: gravels, cobbles and few boulders	0.030	0.040	0.050
• Bottom: cobbles with large boulders	0.040	0.050	0.070
Floodplains			
Pasture, no brush:			
• Short grass	0.025	0.030	0.035
• High grass	0.030	0.035	0.050
Cultivated areas:			
• No crop	0.020	0.030	0.040
• Mature row crops	0.025	0.035	0.045
• Mature field crops	0.030	0.040	0.050
Brush:			
• Scattered brush, heavy weeds	0.035	0.050	0.070
• Light brush and trees, in winter	0.035	0.050	0.060
• Light brush and trees, in summer	0.040	0.060	0.080
• Medium to dense brush, in winter	0.045	0.070	0.110
• Medium to dense brush, in summer	0.070	0.100	0.160
Trees:			
• Dense willows, summer, straight	0.110	0.150	0.200
• Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
• Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
• Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
• Same as above, but with flood stage reaching branches	0.100	0.120	0.160
Excavated or Dredged Channels			
Earth, straight and uniform			
• Clean, recently completed	0.016	0.018	0.020
• Clean, after weathering	0.018	0.022	0.025
• Gravel, uniform section, clean	0.022	0.025	0.030
• With short grasses, few weeds	0.022	0.027	0.033

*For a complete list of Manning's n values, refer to Table 3.1 of the HEC-RAS Hydraulic Reference Manual.

R. Cross Section Subdivision for Conveyance Calculations. The determination of total conveyance and the velocity coefficient for a cross section requires that flow be subdivided into units for which the velocity is uniformly distributed. The approach used is to subdivide flow in the overbank areas using the input cross section Manning's n-

value break points (locations where Manning's n-values change) as the basis for subdivision (Figure 8.4). Conveyance is calculated within each subdivision using the Manning equation (Equation 8.16).

(Equation 8.16)

$$Q = KS_f^{1/2}$$

where K is defined as the conveyance in the subdivision and computed by Equation 8.17:

(Equation 8.17)

$$K = \frac{x}{n} AR^{2/3}$$

The incremental conveyances in the overbanks are summed to obtain a conveyance for the left overbank and the right overbank. The main channel conveyance normally is computed as a single conveyance element. The total conveyance for the cross section is obtained by summing the three subdivision conveyances (left, channel, and right).

An alternative method is to calculate conveyance between every coordinate point in the overbanks (Figure 8.5). The conveyance is then summed to get the total left overbank and right overbank values. This method is used in the U.S. Army Corps HEC-2 program and in the WSPRO Computer program. The wetted perimeter is the cross section boundary in contact with water.

Figure 8.4 Conveyance Subdivision

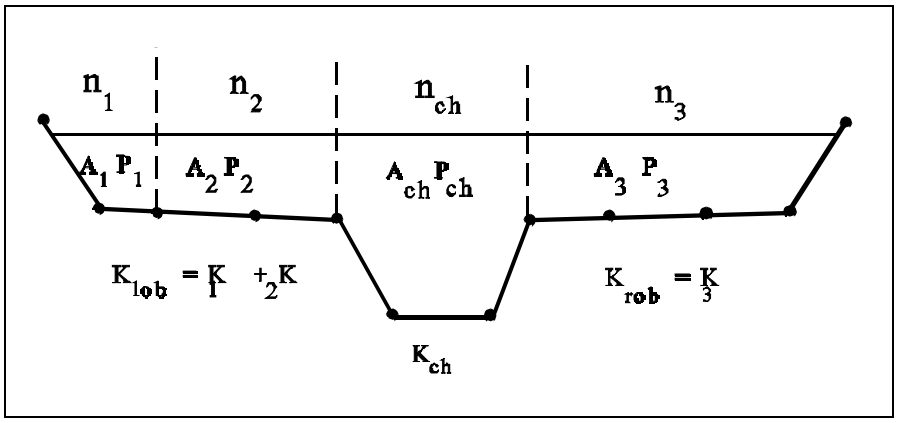
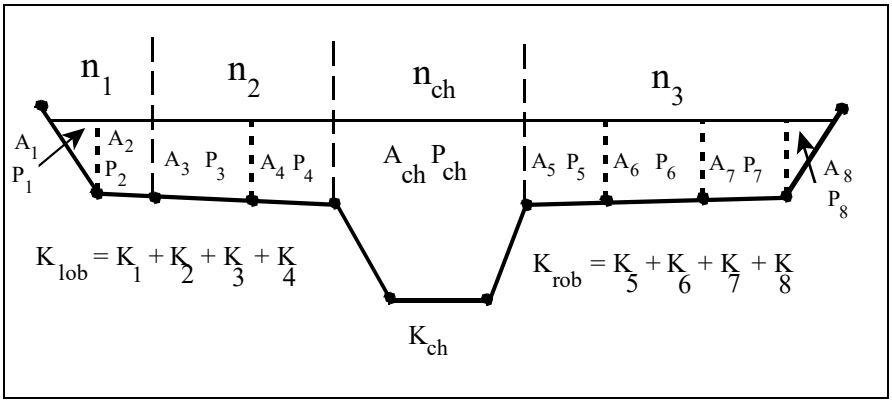


Figure 8.5 Alternative Conveyance Subdivision Method
(HEC-2 and WSPRO style)



The two methods for computing conveyance will produce different answers whenever portions of the overbanks have ground sections with significant vertical slopes. In general, the break of the subdivision by Manning n will provide a lower total conveyance for the same water surface elevation.

In order to test the significance of the two methods of computing conveyance, comparisons were performed using 97 data sets from the HEC profile accuracy study (HEC-23). Water surface profiles were computed for the 1% chance event using the two methods for computing conveyance. The results of the study showed that the Manning n break points default approach will generally produce a higher computed water surface elevation. Out of the 2048 cross section locations, 47.5% had computed water surface elevations within 30.48 mm (0.1 ft), 71% within 60.96 mm (0.2 ft), 94.4% within 122 mm (0.4 ft), 99.4% within 304.8 mm (1 ft).

The results from these comparisons do not show which method is more accurate; they only show differences. In general, subdividing the section by Manning's n is more commensurate with the Manning equation and the concept of separate flow elements. Further research, with observed water surface profiles, will be needed to make any conclusions about the accuracy of the two methods.

8.3 CHANNEL ANALYSIS

A. Overview of Channel Analysis Methods. The depth and velocity at which a given discharge will flow in a channel of known geometry, roughness, and slope can be determined through hydraulic analysis. The depth and velocity of flow are necessary for the design and analysis of channel linings and highway drainage structures.

Two methods commonly are used in the hydraulic analysis of open channels. The Slope-Conveyance Method is a simple application of Manning Equation which may be used to determine tailwater rating curves for culverts and storm drains. This procedure can be used to analyze situations in which uniform or nearly uniform flow conditions exist. The Standard Step-Backwater Method is used to compute the complete water surface profile in a stream reach to evaluate the unrestricted water surface elevations. It is used to analyze gradually varied flow conditions in streams and is especially appropriate for the hydraulic design of bridges and culverts.

Generally, the Slope-Conveyance Method requires more judgment and assumptions than the Standard Step-Backwater Method. In many situations, however, use of the Slope-Conveyance Method is justified (e.g., standard roadway ditches, culverts, storm drain outfalls, etc.).

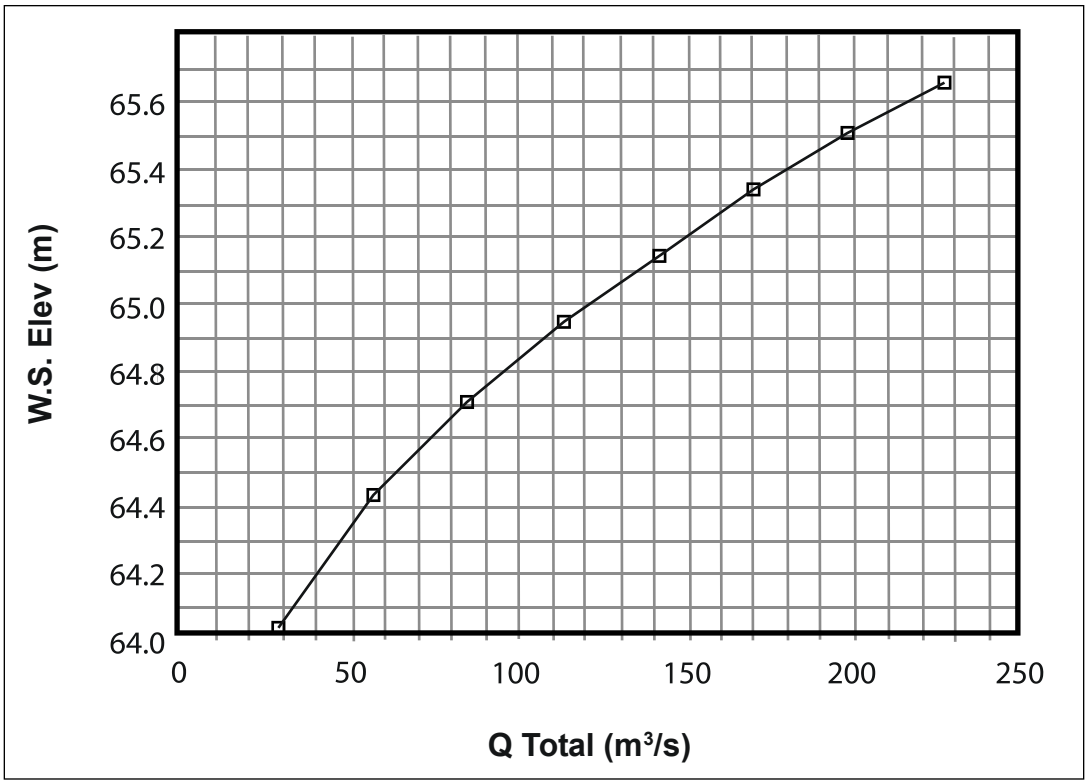
B. Stage-Discharge Relationship. A stage-discharge curve is a graph of water surface elevation versus flow rate in a channel. The stage-discharge relationship is one of the most important factors considered in analysis and design. The total discharge for the stream, normal flow channel, and floodplain may be computed for various depths. The data, plotted in graphic form (sometimes termed a "rating curve"), gives the designer a visual display of the relationship between water surface elevations and discharges. A stage-discharge curve is shown in Figure 8.6.

For channel design, an accurate stage-discharge relationship is necessary to evaluate the interrelationships of flow characteristics and to establish alternatives for the following:

- Width.
- Depth of flow.
- Freeboard.
- Conveyance capacity and type.
- Required degree of stabilization.

The stage-discharge relationship also enables the designer to evaluate a range of conditions as opposed to a preselected design flow rate.

Figure 8.6 Typical Stage-Discharge Curve

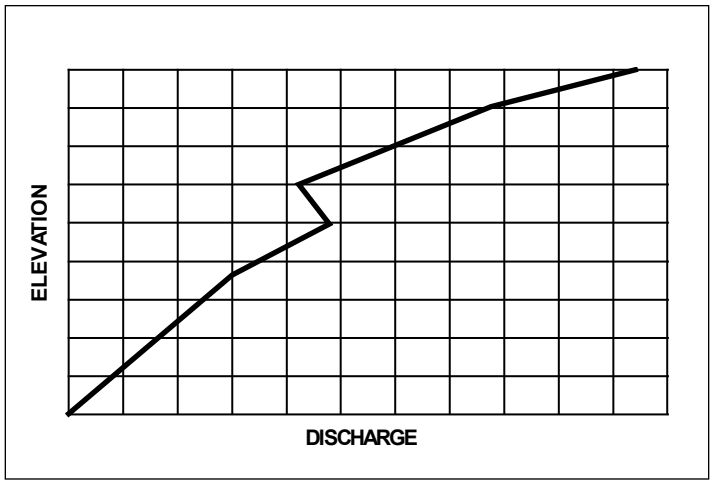


The plot of elevation-discharge should be examined carefully for evidence of the "switchback" characteristic described below. The plot also should be examined to determine whether it is realistic. For example, a stream serving a small watershed should reflect reasonable discharge rates for apparent high water elevations.

If the cross section is improperly subdivided, the mathematics of Manning Equation may cause a switchback. A switchback results when the calculated discharge decreases with an associated increase in elevation or depth (see Figure 8.7). Because of the small increase in depth, a small increase in cross sectional area and large increase in wetted perimeter cause a net decrease in the hydraulic radius. The discharge computed using the smaller hydraulic radius and the slightly larger cross sectional area is lower than the previous discharge for which the water depth is lower. More subdivisions within such cross sections should be used in order to avoid the switchback.

A switchback can occur in any type of conveyance computation. Computer logic can be seriously confused if a switchback occurs in any cross section being used in a program. For this reason, the cross section always should be subdivided with respect to both vegetation and geometric changes. Note that the actual Manning's n-value may be the same in adjacent subsections. However, too many subdivisions can result in problems, too. If the conveyance computations are carried out corresponding to the procedures used in HEC-2 or WSPRO, the computation for water surface elevations become a function of the number of subdivisions as pointed out in Section 8.2.R. Many of the computer models such as HEC-RAS, allow up to 500 (x,y) data points to describe the geometry of the cross section. Depending upon the computer model, using this many points can lead to differences in computed water surface elevations.

Figure 8.7 Switchback in Stage-Discharge Curve



8.4 SLOPE CONVEYANCE METHOD

A. Introduction to the Slope-Conveyance Method. The Slope-Conveyance Method, sometimes referred to as the Slope-Area Method, is a one-dimensional stream modeling procedure used extensively for stage-discharge development. This method is based on Manning Equation for uniform flow in a channel. The Slope-Conveyance Method has the advantage that it is a relatively simple, usually inexpensive, and expedient procedure. The results are highly sensitive to both the longitudinal slope and roughness coefficients which are subjectively assigned. For normal culvert design, the Slope-Conveyance Method often is sufficient for determining tailwater (TW) depth at the culvert outlet.

The procedure involves an iterative development of calculated discharges associated with assumed water surface elevations in a typical section. The series of assumed water surface elevations and associated discharges comprises the stage-discharge relationship.

When stream gage information exists, a measured relationship may be available. Such a measured stage-discharge relationship usually is termed a rating curve.

B. Application and Limitations of the Slope-Conveyance Method. The Slope-Conveyance Method normally is applied to relatively small stream crossings or those in which no unusual flow characteristics are anticipated. The reliability of the results depends on the accuracy of the supporting data, the appropriateness of the parameter assignments (Manning's n-values and longitudinal slopes), and the designer's selection of the typical cross section. If the crossing is a more important one, or if there are unusual flow characteristics, some other procedure should be used.

The computer program HY-8 performs the Slope-Conveyance Method, and may be used by the designer as an alternative to performing the calculations by hand.

C. Data Requirements. A channel cross section and associated roughness and slope data that can be considered as typical of the stream reach under consideration are required for this analysis. While not absolutely necessary, it is suggested that this cross section be located downstream from the proposed drainage facility site. The closer to the proposed site a typical cross section is taken, the less error there will be in the final water surface elevation.

A "typical" cross section should be located for the analysis. If such a cross section cannot be found, then a "controlling" cross section (also downstream) should be used. The depth of flow in a "controlling" cross section is controlled by a constriction of the channel, a damming effect across the channel, or possibly an area with extreme roughness coefficients. The cross section should be normal to the direction of stream flow under flood conditions. Once the cross section has been obtained, Manning roughness coefficients (Manning's n-values) should be applied. The cross section should be subdivided with vertical boundaries at significant changes in cross section shape or at

changes in vegetation cover and roughness components. See Section 8.2.R. for suggestions on subdividing cross sections.

The slope is the third important parameter necessary to perform a Slope-Conveyance analysis. Manning Equation is based on the slope of the water surface, which often corresponds to the average slope of the channel bed. However, some reaches of stream may have a water surface slope quite different from the bed slope during flood flow.

The least expensive and most expedient method of slope-determination is to survey and analyze the bed profile for some distance in a stream reach.

D. Slope-Conveyance Procedure. The calculation of the stage-discharge relationship should proceed as described in this section. Tables 8.3 to 8.7 represent an example of the progression of these calculations based on the cross section shown in Figure 8.8. The result of this procedure is a stage-discharge curve, as shown in Figure 8.9. The design discharge or any other discharge then can be used to estimate (usually done by interpolation) an associated water surface elevation.

1. Select a trial starting depth and apply it to a plot of the cross section.

2. Compute the area and wetted perimeter for each submerged subsection.

3. Compute the subsection discharges with Manning Equation using the subsection values for roughness, area, wetted perimeter, and slope. Manning Equation is repeated below for convenience (See Equation 8.3). The sum of all of the incremental discharges represents the total discharge for each assumed water surface elevation.

(Equation 8.18)

$$Q = \frac{x}{n} A R^{2/3} S^{1/2}$$

NOTE: If necessary, the average velocity for the section can be computed by substituting the total section area and discharge into the continuity equation (See Equation 8.1).

(Equation 8.19)

$$v = \frac{Q}{A}$$

Figure 8.8 Sample Cross Section

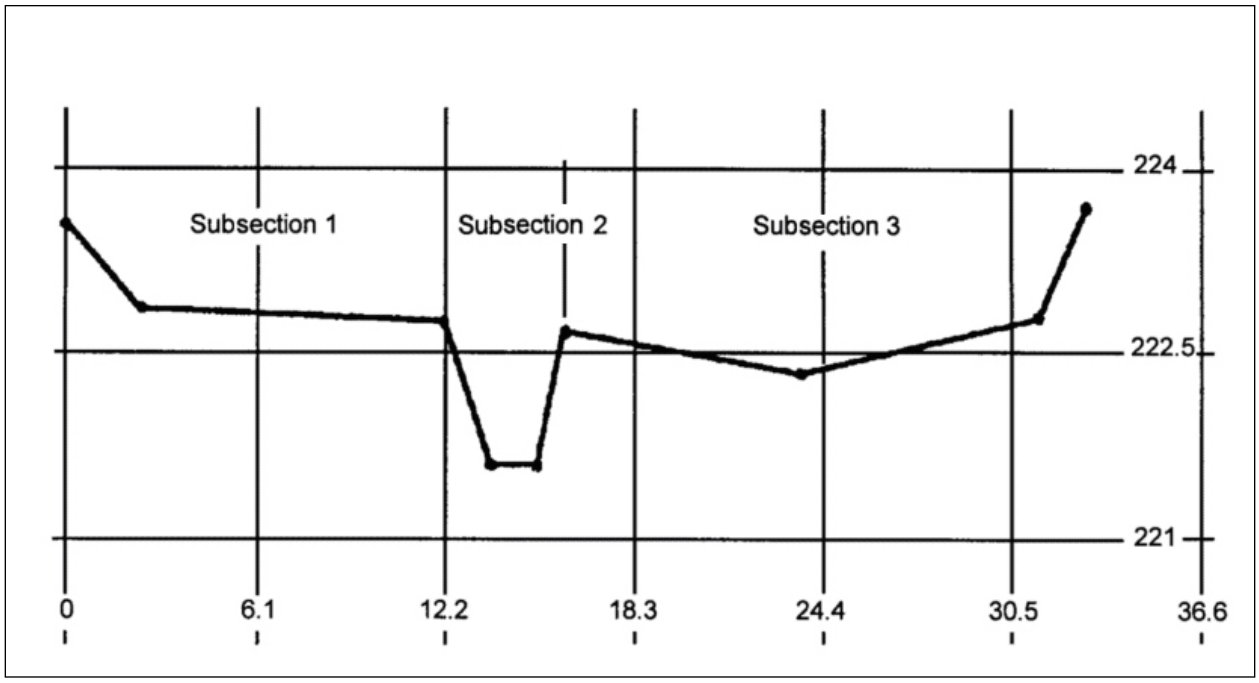
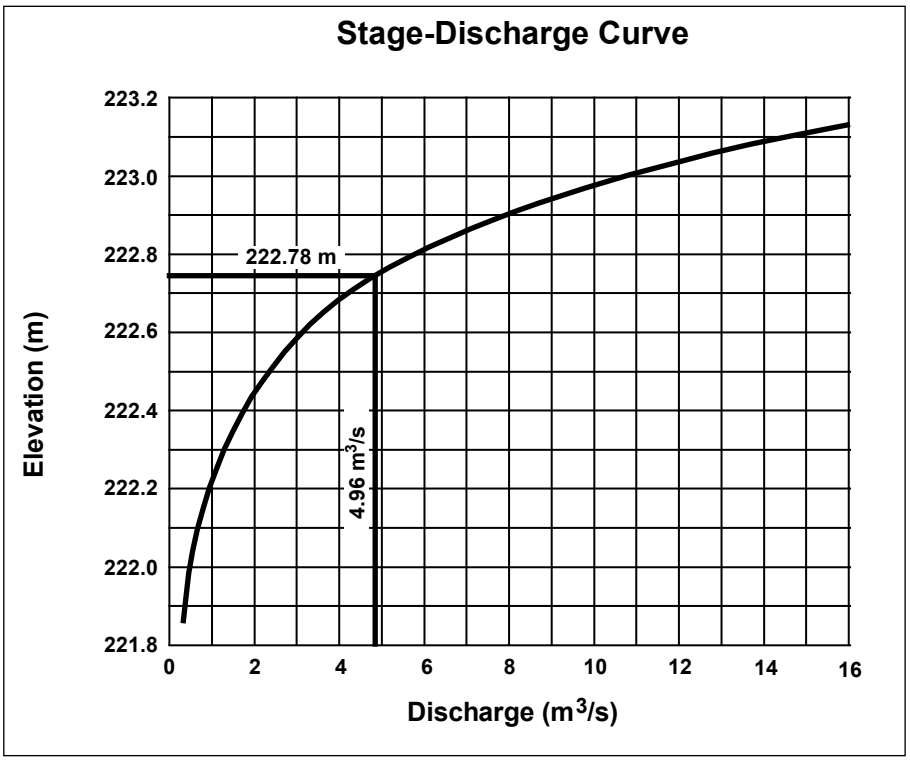


Figure 8.9 Example Stage-Discharge Curve



If there is discharge (water flowing) in the overbanks, the energy correction factor, α may be applied to the cross section and appropriate energy grade line computed. However, often this effect is ignored.

4. Tabulate or plot the water surface elevation and resulting discharge.

5. Repeat the above steps with a new channel depth, or add a depth increment to the trial depth. The choice of elevation increment is somewhat subjective; however, if the increments are less than about 0.075 m (0.025 ft), considerable calculation is required. On the other hand, if the increments are greater than 0.5 m (1.65 ft), the resulting stage-discharge relationship may not be detailed enough for use in design.

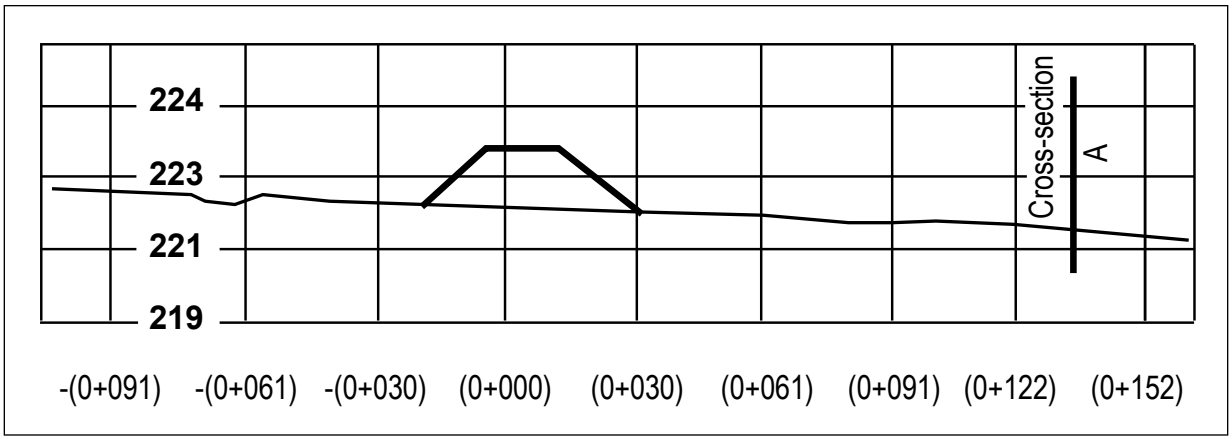
Example 1:

Given: The 25-year discharge, Q_{25} = 4.96 m^3/s (175 cfs). Cross section information is given in the following table of surveyed data points for a typical cross section.

Table 8.2 Cross Section Data (Section "A" - Sta. 1+36)

Distance (m)	Elevation (m)	Manning's "n"
0.0 (0.0 ft)	223.51 (733.3 ft)	0.06
2.44 (8.0 ft)	222.81 (731.0 ft)	0.06
12.19 (40 ft)	222.69 (730.6 ft)	0.035
13.72 (45 ft)	221.53 (726.8 ft)	0.035
15.24 (50 ft)	221.53 (726.8 ft)	0.035
16.15 (53 ft)	222.63 (730.4 ft)	0.05
23.77 (78 ft)	222.26 (729.2 ft)	0.05
31.39 (103 ft)	222.75 (730.8 ft)	0.05
32.92 (108 ft)	223.66 (733.8 ft)	0.05

Figure 8.10 Stream Profile



Referring to Figure 8.8, Subsection 1 consists of an overbank area with light brush and trees. Subsection 2 is in the main channel of this stream and comprises a clean, straight stream with a few weeds and rocks. Subsection 3 is in the right floodplain and includes some scattered brush with considerable weediness.

Find: Use the slope-conveyance method to develop a stage-discharge curve for the channel cross section at station 0+136 which is located downstream from a highway culvert. Determine the tailwater elevation at the outlet of the culvert (assume a channel station of 0+030 for this location) for the 25-year flood.

STREAM CROSS SECTION "A"

Solution: The slope of the stream can be determined by examining the reach from stream Station – (0+091) to the typical section at Station 0+136. The flow line differential for this reach is 0.61 m (2 ft) (in 227 m (745 ft) of stream reach). Therefore, the slope (S) is 0.0027 m/m (ft/ft).

Figure 8.9 can be used to assist in the development of a stage-discharge curve for this typical section. Assuming water surface elevations beginning at 221.83 m (727.80 ft), calculate pairs of water surface elevation/discharge for plotting on a stage-discharge curve. Illustrative calculations with arbitrary increments of water surface elevation of 0.3 m (1 ft) were used. A plotted stage-discharge curve is shown in Figure 8.9. The water elevation for Q_{25} = 4.96 m^3/s (175 cfs) is 222.78 m (730.70 ft).

Since the calculation section for the stream is downstream of the culvert site, it will be necessary to project the water surface elevation as determined from the typical section at station 0+136 to represent the tailwater elevation at stream Station 0+030. Therefore, the projected tailwater (TW) levels are calculated as follows:

TW_{25} = 222.78 + (136-30)(0.0027)
= 222.78 + 0.29
= 223.07 m = 731.86 ft

Table 8.3 Cross Section Data at Elevation 221.83 m (727.80 ft)

Elevation = 221.83 m (727.8 ft) Slope = 0.0027							
Sub-section ID	I	II	III	IV	V	VI	Totals/Average
Area (m^2) (ft^2)		0.56 (6.0)					0.56 (6.0)
Wetted Perimeter (m) (ft)		2.42 (7.92)					2.42 (7.92)
Hydraulic Radius (m) (ft)		0.23 (0.76)					
$R^{2/3}$		0.38 (.83)					
N	0.060	0.035	0.050				
ΔQ (m^3/s) (cfs)		0.31 (11.00)					0.31 (11.00)
Sub-section Vel. (m/s) (ft/s)		0.56 (1.83)					0.56 (1.83)

Table 8.4 Cross Section Data at Elevation 222.14 m (728.80 ft)

Elevation = 222.20 m (728.8 ft) Slope = 0.0027							
Sub-section ID	I	II	III	IV	V	VI	Totals/Average
Area (m^2) (ft^2)		1.33 (14.30)					1.33 (14.30)
Wetted Perimeter (m) (ft)		3.33 (10.91)					
Hydraulic Radius (m) (ft)		0.40 (1.31)					
$R^{2/3}$		0.54 (1.20)					
N	0.060	0.035	0.050				
ΔQ (m^3/s) (cfs)		1.07 (37.80)					1.07 (37.80)
Sub-section Vel. (m/s) (ft/s)		0.81 (2.64)					0.81 (2.64)

Table 8.5 Cross Section Data at Elevation 222.14 m (729.81 ft)

Elevation = 222.14 m (729.81 ft) Slope = 0.0027							
Sub-section ID	I	II	III	IV	V	VI	Totals/Average
Area (m^2) (ft^2)		2.31 (24.82)					2.31 (24.82)
Wetted Perimeter (m) (ft)		4.24 (13.90)					
Hydraulic Radius (m) (ft)		0.54 (1.79)					
$R^{2/3}$		0.67 (1.47)					
N	0.060	0.035	0.050				
ΔQ (m^3/s) (cfs)		2.28 (80.60)					2.28 (80.60)
Sub-section Vel. (m/s) (ft/s)		0.99 (3.25)					0.99 (3.25)

Table 8.6 Cross Section Data at Elevation 222.74 m (730.61 ft)

Elevation = 222.74 m (730.61 ft) Slope = 0.0027							
Sub-section ID	I	II	III	IV	V	VI	Totals/Average
Area (m^2) (ft^2)	0.00 (0.00)	3.22 (34.69)	3.35 (36.01)				6.57 (70.70)
Wetted Perimeter (m) (ft)	0.36 (1.19)	4.87 (15.97)	14.38 (47.18)				
Hydraulic Radius (m) (ft)	0.00 (0.00)	0.66 (2.17)	0.23 (0.76)				
$R^{2/3}$	0.00 (0.00)	0.76 (1.68)	0.38 (0.84)				
N	0.060	0.035	0.050				
ΔQ (m^3/s) (cfs)	0.00 (0.00)	3.64 (128.52)	1.32 (46.48)				4.96 (175.00)
Sub-section Vel. (m/s) (ft/s)	0.01 (0.05)	1.13 (3.70)	0.39 (1.29)				0.75 (2.47)

Table 8.7 Cross Section Data at Elevation 222.74 m (730.78 ft)

Elevation = 222.74 (730.78 ft)	Slope = 0.0027						
Sub-section ID	I	II	III	IV	V	VI	Totals/Average
Area (m^2) (ft^2)	0.13 (1.36)	3.43 (36.90)	4.11 (44.22)				7.67 (82.48)
Wetted Perimeter (m) (ft)	4.50 (14.75)	3.96 (15.97)	15.17 (49.84)				
Hydraulic Radius (m) (ft)	0.03 (0.06)	0.87 (2.31)	0.27 (0.89)				
$R^{2/3}$	0.09 (0.20)	0.91 (1.75)	0.42 (0.92)				
N	0.060	0.035	0.050				
ΔQ (m^3/s) (cfs)	0.01 (0.36)	4.03 (142.28)	1.79 (63.06)				5.82 (205.70)
Sub-section Vel. (m/s) (ft/s)	0.08 (0.26)	1.18 (3.86)	0.43 (1.43)				0.76 (2.49)

8.5 STANDARD STEP-BACKWATER METHOD

A. Introduction to the Step-Backwater Method. The Standard Step-Backwater Method uses Bernoulli's Equation to "step" the stream water surface along a profile, typically in an upstream direction (subcritical flow). The Standard Step-Backwater Method is a one-dimensional stream modeling procedure.

By definition, the Standard Step-Backwater Method accommodates gradually varied, steady flow situations. It should be noted that this method typically is more data and time intensive, but is more reliable than the Slope-Conveyance Method.

The Standard Step-Backwater Method is utilized by the following computer programs:

- U.S. Army Corps of Engineers HEC-RAS and HEC-2.
- U.S. Geological Survey WSPRO (the Federal Highway Administration also sponsors WSPRO under the name of HY-7).

A stage-discharge relationship can be derived from the water surface profiles for each of several discharge rates.

The calculation process for the Standard Step-Backwater Method often is tedious. With widespread access to computers, the usual channel analysis by Standard Step-Backwater can be accomplished better by computer programs such as those listed above; however, the designer should ensure that the particular application complies with the limitations of the program used.

B. Application and Limitations of the Standard Step-Backwater Method. The Standard Step-Backwater Method should be used for analysis in the following instances:

- Results from the Slope-Conveyance Method may not be accurate enough or too time consuming.
- The drainage facility's level of importance deserves a more sophisticated channel analysis.
- The channel is highly irregular with numerous or significant variations of geometry, roughness characteristics, and stream confluences.
- A controlling structure affects backwater.

The Standard Step-Backwater procedure is applicable to most open channel flow scenarios, including streams having an irregular channel with the cross section consisting of a main channel and separate overbank areas with individual Manning's n-values.

The Standard Step-Backwater Method can be used either for supercritical ("steep slope regime") or for subcritical ("mild slope regime") flow.

C. Data Requirements for the Standard Step-Backwater Method. At least two cross sections are required to complete this procedure; often, many more will be necessary. The number and frequency of cross sections required is a direct function of the irregularity of the stream reach. Generally speaking, the more irregular the reach, the greater the number of cross sections that will be required. The cross sections should be representative of the reach between them. A system of measurement or stationing between cross sections also is required. Roughness characteristics (Manning's n-values) and associated subsection boundaries must be evaluated for all of the cross sections.

The selection of cross sections used in this method is critical. As the irregularities of a stream vary along a natural stream reach, the influence of the varying cross sectional geometry must be accommodated. This means that transitional cross sections must be incorporated into sections making up the stream reach.

While there is considerable flexibility in the procedure concerning the computed water surface profile, it is important to know any controlling water surface elevations. For example, critical depth may occur at a cross section that is constricted. This section would be considered a control and water surface would be known at this section (i.e., critical depth).

D. Standard Step-Backwater Procedure. The Standard Step-Backwater Method uses the Energy Balance Equation (repeated here for convenience), which allows the water surface elevation at the upstream section (2) to be computed given a known water surface elevation at the downstream section (1). Figure 8.1 illustrates the quantities given in the Energy Balance Equation.

(Equation 8.20)

$$z_2 + y_2 + \alpha_2 \frac{v_2^2}{2g} = z_1 + y_1 + \alpha_1 \frac{v_1^2}{2g} + h_f + other\ losses$$

The following procedure assumes that cross sections, stationing, discharges, and Manning's n-values already have been established.

1. Generally, for most situations, the assumption of subcritical flow will be appropriate to start the process. Subsequent calculations will check this assumption.

Determine a starting water surface elevation. For subcritical flow, begin at the most downstream cross section. Use one of the following methods to establish a starting water surface elevation:

- Measured elevation.
- Use the Slope-Conveyance Method to determine the stage for an appropriate discharge.
- Use an existing (verified) rating curve.

2. Referring to Figure 8.11, consider the starting water surface to be Section 1 and calculate the following:

y_1 = TW - flowline elevation
z_1 = flowline elevation

$$V_1 = \frac{Q_1}{A_1}$$

$$h_{v1} = \frac{V_1^2}{2g}$$

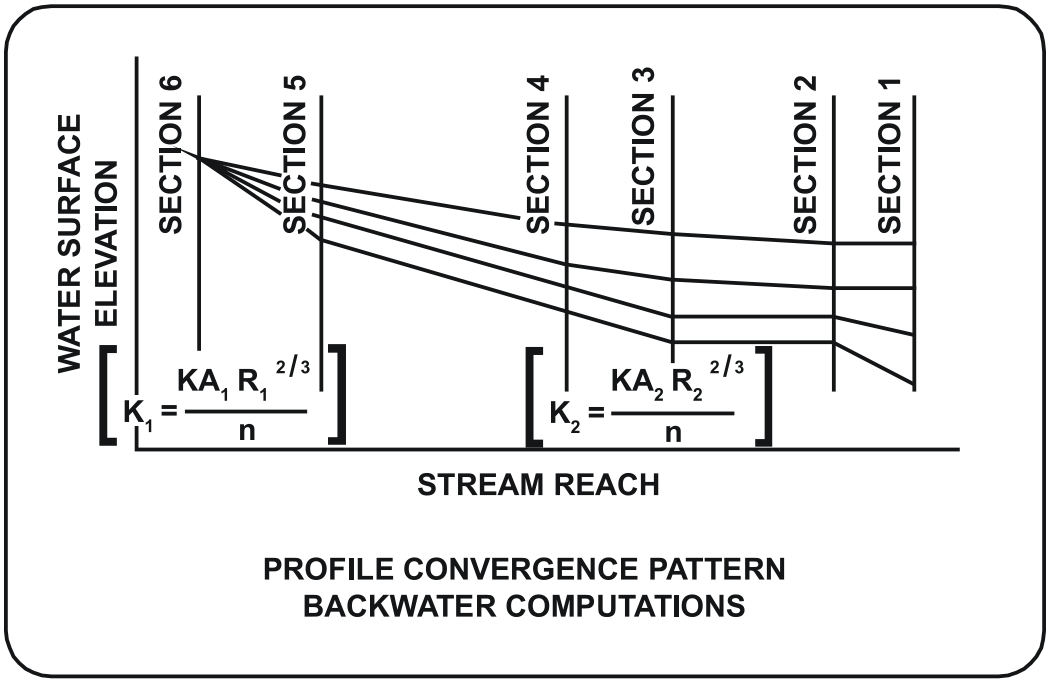
3. Assume a depth y_2 at Section 2, which, for the first reach, will be the section immediately upstream from the starting section. Using y_2, calculate the following:

z_2 = flowline elevation

$$V_2 = \frac{Q_2}{A_2}$$

$$h_{v2} = \frac{V_2^2}{2g}$$

Figure 8.11 Water Surface Profile Convergence



4. Calculate the friction head losses (h_f) between the two sections. From Equation 8.5 with $S_f = h_f/L$

(Equation 8.21)

$$h_f = \left(\frac{Q}{K_{ave}}\right)^2 L$$

$$\left(K_1 = \frac{xA_1R_1^{2/3}}{n}\right) \qquad \left(K_2 = \frac{xA_2R_2^{2/3}}{n}\right)$$

and

(Equation 8.22)

$$K_{ave} = \frac{K_1 + K_2}{2}$$

5. Calculate the adjustment coefficients (α1 and α2) using Equation 8.9.

6. Where appropriate, calculate expansion/contraction losses.

- expansion losses, h_e

(Equation 8.23)

$$h_e = K_e \frac{\Delta(V)^2}{2g} = K_e\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)$$

where: $K_e = 0.3$ for a gentle expansion
$K_e = 0.5$ for a sudden expansion

- contraction losses, h_c

(Equation 8.24)

$$h_c = K_c \frac{\Delta(V)^2}{2g} = K_c\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)$$

where: $K_c = 0.1$ for a gentle contraction
$K_c = 0.3$ for a sudden contraction

Other losses, such as eddy losses, are estimated as 10% of the friction head loss where the quantity $h_{v1} - h_{v2}$ is positive and 30% thereof when it is negative. Bend losses often are disregarded as an unnecessary refinement.

7. Check the energy equation for balance.

(Equation 8.25)

$$Energy = z_2 + y_2 + \alpha_2 \frac{V_2^2}{2g}$$

(Equation 8.26)

$$Computed\ Energy = z_1 + y_1 + \alpha\ \frac{V_1^2}{2g} + h_f + h_e + h_c$$

- If Energy - Computed Energy are within a reasonable tolerance (0.01 ft (0.003 m)), then the assumed depth at Section 1 is okay. This will be the calculated water surface depth at Section 1. Proceed to Step 8.

- If Energy - Computed Energy are not within a reasonable tolerance (0.01 ft (0.003 m)), go back to Step 3 using a different assumed depth.

8. Determine the critical depth at the cross section (see Section 8.2.I., Critical Depth) and find the uniform depth by iteration. If critical depth is greater than uniform depth, then the profile at that cross section is supercritical. For supercritical flow, the process is similar but the calculations must begin at the upstream section and proceed downstream.

9. Assign the calculated depth from Step 7 as the downstream elevation (Section 1) and the next section upstream as Section 2, and repeat Steps 2 through 7.

This process is repeated until all of the sections along the reach have been addressed.

E. Profile Convergence. When the Standard Step-Backwater Method is used and the starting water surface elevation is unknown or undefined, a computer can be useful in calculating several backwater profiles based on several arbitrary starting elevations. If these profiles are plotted as shown in Figure 8.11, they will tend to converge to a common curve at some point upstream because each successive calculation brings the water level nearer the uniform depth profile.

The purpose of plotting the curves and finding the convergence point is to determine where the proposed structure site is in reference to the convergence point. If the site is in the vicinity or upstream of the convergence point, the calculations have started far enough downstream to define a proper tailwater from an unknown starting elevation. Otherwise, the calculations may have to begin at a point further downstream.

F. Example of the Standard Step-Backwater Method. The Standard Step-Backwater procedure is illustrated in the following example.

Four cross sections along a reach are shown in Figures 8.12 through 8.15. Each cross section is separated by 152.4 m (500 ft), and is subdivided according to geometry and roughness. The calculations shown in Table 8.8 represent one set of water-surface calculations. An explanation of Table 8.8 follows the calculations.

Figure 8.12 Cross Section at Station 9.79 (Farthest Upstream)

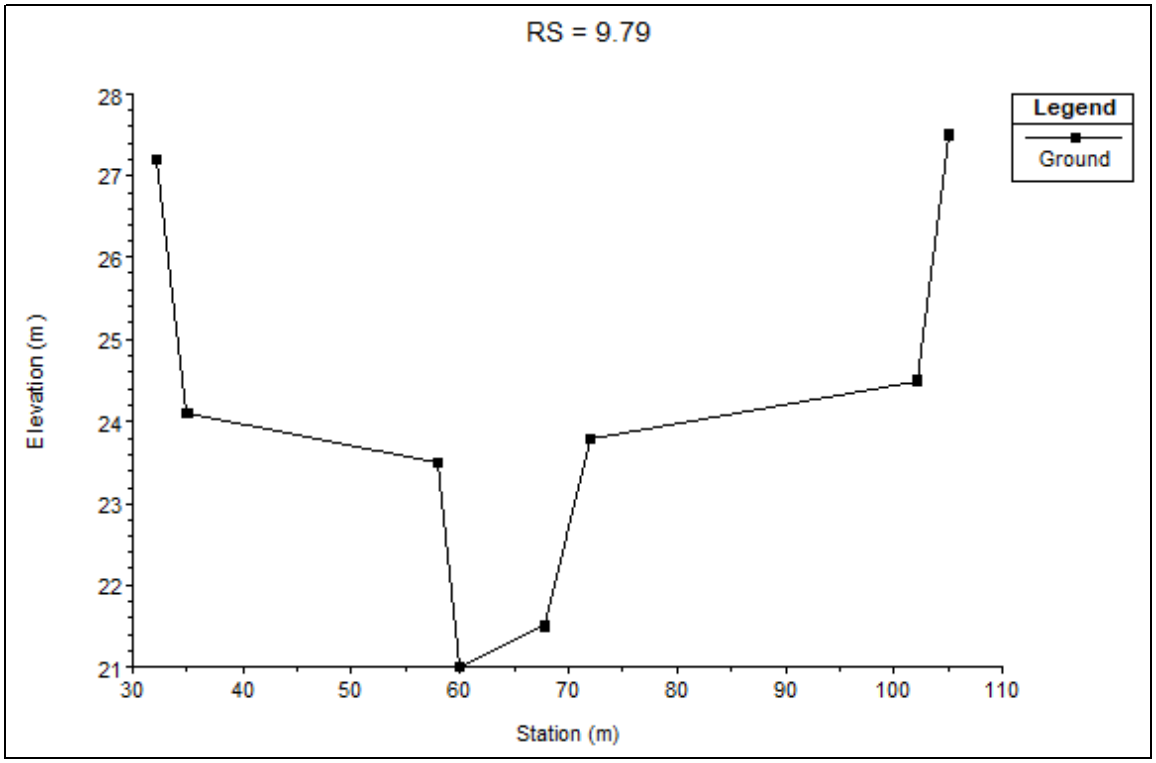


Figure 8.13 Cross Section at Station 9.7

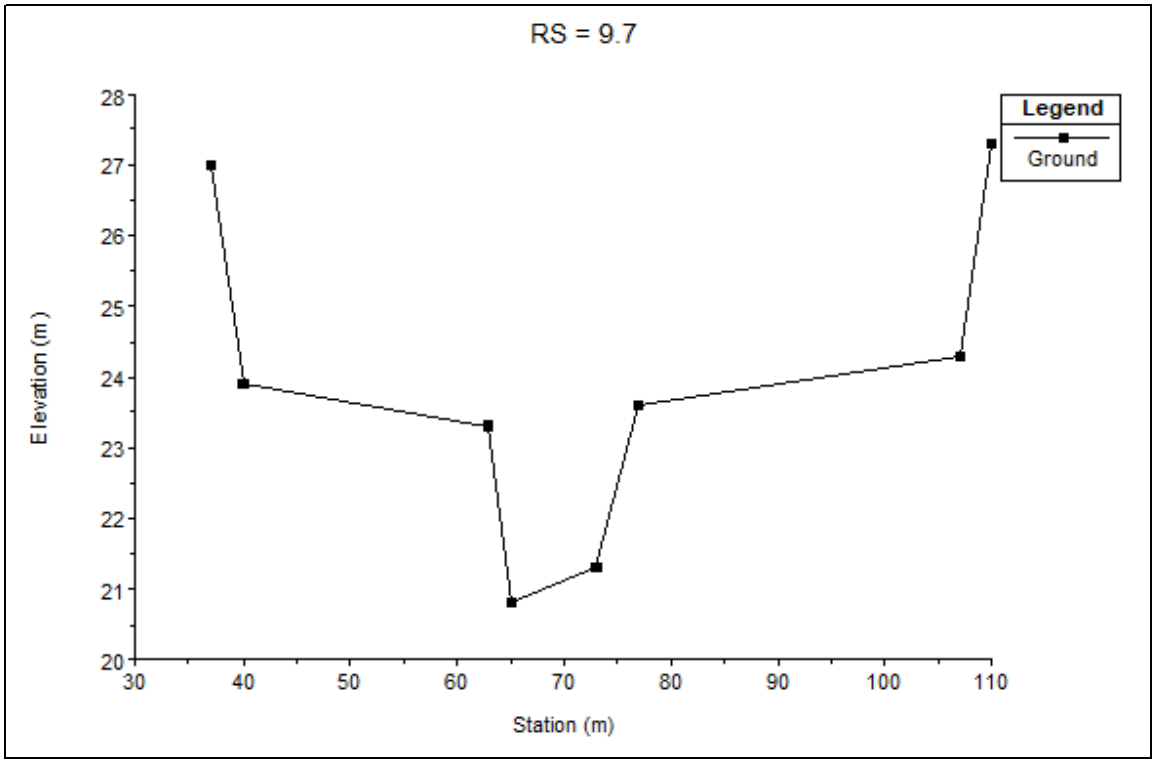


Figure 8.14 Cross Section at Station 9.6

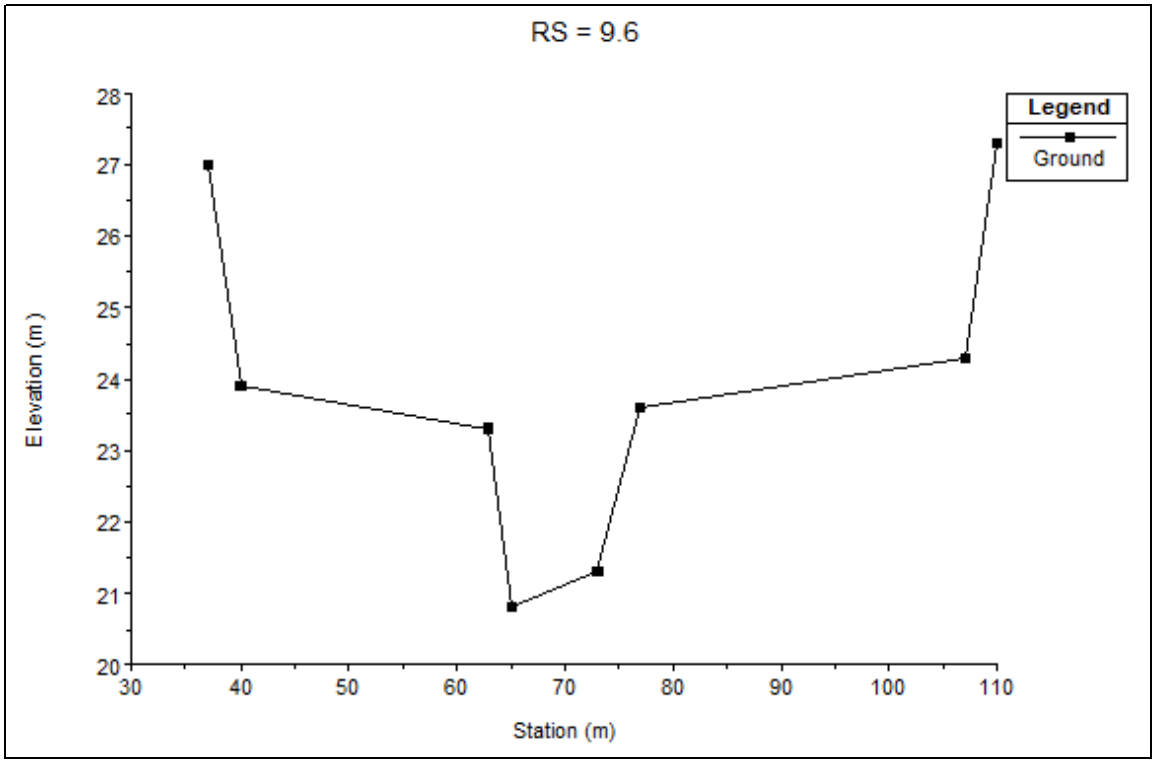


Figure 8.15 Cross Section at Station 9.5 (Farthest Downstream)

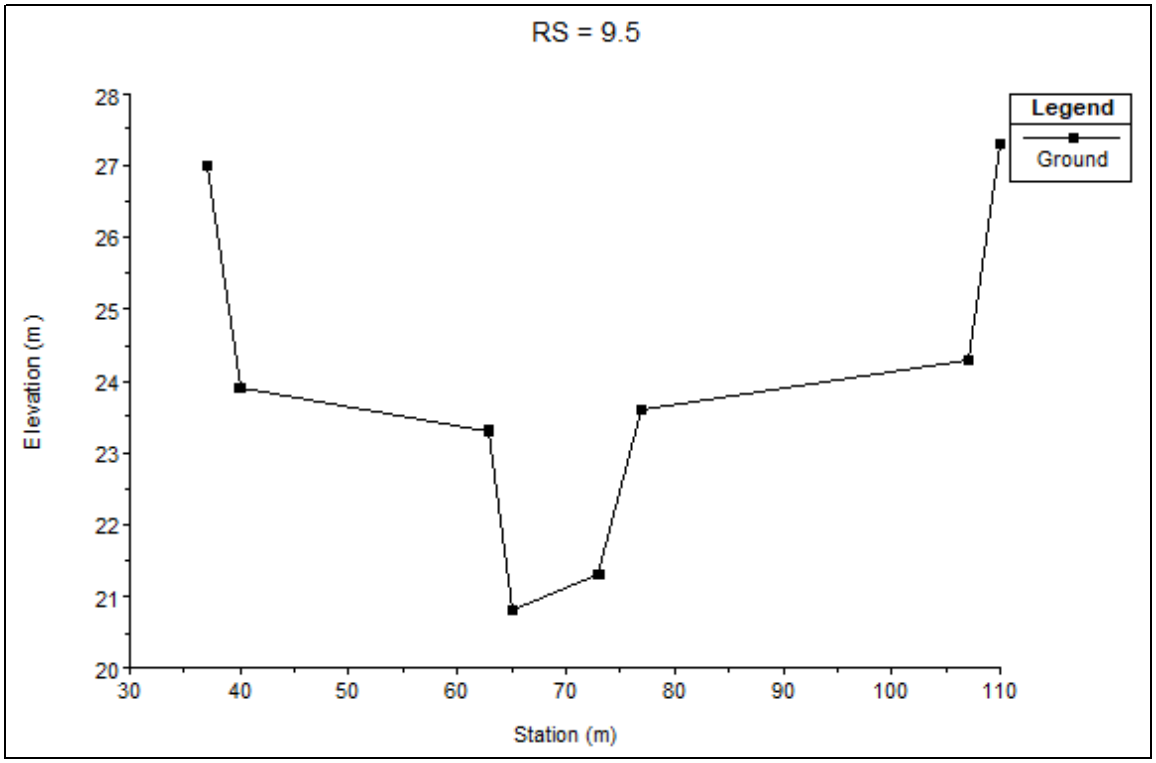


Table 8.8 Water-Surface Calculations (Q = 57 m^3/s (2000 cfs))

1 Cross-Section ID	2 Cross-Section Station	3 Assumed WS (m) (ft)	4 Computed WS (m) (ft)	5 Area (m^2) (ft^2)	6 Wetted Perimeter (m), (ft)	7 Hydraulic Radius (m), (ft)	8 n	9 K (m^3/s) (ft^3/s)	10 K_{avg} (m^3/s) (ft^3/s)	11 S_f	12 Avg S_f	13 L (m) (ft)	14 h_f (m) (ft)	15 $[K_1(K_2/A_1)^2]$ (m^4/s^2) (ft^4/s^2)	16 α	17 V (m/s) (ft/s)	18 $V^2/2g$ (m) (ft)	19 $(V^2/2g)$ (m),(ft)	20 h_e (m) (ft)	21 WS (m) (ft)
9.5	0		24.17 (79.2)																	
				17.1 (190.4)	25.85 (84.8)	0.68 (2.23)	0.060	229.0 (8080)						38354 (4439213)						
				47.0 (505.6)	18.83 (61.7)	2.50 (8.2)	0.035	2472.7 (87255)						6838227 (791478074)						
				13.6 (146.3)	34.89 (114.4)	0.39 (1.3)	0.050	145.7 (5141)						16641 (1926082)						
				78.3 (842.4)				2847.3 (100474)	2847.5 (100481)	0.00040	0.00000	0 (0)	0 (0)	6893221 (797843253)	1.83	0.73 (2.4)	0.049 (.16)	0.000 (0.000)	0.000 (0.000)	0.000 (0.000)
9.6	152.4 (500)	24.23 (79.47)	24.23 (79.47)																	
				17.7 (190.4)	25.85 (84.8)	0.68 (2.23)	0.060	229.2 (8088)						38419 (4446736)						
				47.0 (505.6)	18.83 (61.7)	2.50 (8.2)	0.035	2472.7 (87255)						6838227 (791478074)						
				13.6 (146.3)	34.89 (114.4)	0.39 (1.3)	0.050	145.7 (5141)						16641 (1926082)						
				78.4 (843.5)				2847.5 (100481)	2847.4 (100477)	0.00040	0.00040	152.4 (500)	0.06 (.19)	6893286 (797850777)	1.83	0.73 (2.4)	0.049 (.16)	0.000 (0.000)	0.000 (0.000)	0.061 (.20)
9.7	304.8 (1000)	24.45 (80.19)	24.30 (79.70)																	
				21.1 (227)	26.04 (85.4)	0.81 (2.65)	0.060	304.9 (10759)						63864 (7391821)						
				49.3 (530)	18.82 (61.7)	2.62 (8.59)	0.035	2677.6 (94486)						7895180 (913813165)						
				18.2 (196)	35.08 (115.1)	0.52 (1.70)	0.050	235.2 (8300)						39255 (4543498)						
				88.6 (953)				3217.8 (113548)	3032.6 (107013)	0.00031	0.00035	152.4 (500)	0.05 (.16)	7998300 (925748601)	1.88	0.64 (2.1)	0.040 (.13)	0.010 (.03)	0.001 (.003)	0.064 (.21)
	304.8 (1000)	24.29 (79.67)	24.29 (79.67)																	
				16.9 (182)	25.81 (84.6)	0.66 (2.16)	0.060	212.8 (7509)						33664 (3896378)						
				46.5 (500)	18.83 (61.8)	2.47 (8.10)	0.035	2425.5 (85590)						6605322 (764520910)						
				12.6 (136)	34.85 (114.3)	0.36 (1.18)	0.050	127.4 (4496)						13082 (1514152)						
				76.0 (818)				2765.7 (97595)	2806.6 (99038)	0.00042	0.00041	152.4 (500)	0.06 (.19)	6652069 (769931556)	1.81	0.75 (2.46)	0.052 (.17)	-0.003 (-.01)	-.001 (-.003)	0.059 (.19)
9.79	457.2 (1500)	24.35 (79.86)	24.33 (79.80)																	
				12.1 (130)	21.43 (70.3)	0.57 (1.87)	0.060	138.1 (4873)						17948 (2077358)						
				37.4 (402)	16.11 (52.8)	2.32 (7.61)	0.035	1871.8 (66051)						4693783 (543273326)						
				7.8 (84)	27.28 (89.5)	0.29 (.95)	0.050	68.1 (2403)						5160 (597234)						
				57.3 (188)				2078.1 (73331)	2421.9 (85463)	0.00075	0.00055	152.4 (500)	0.08 (.26)	4716892 (545948035)	1.73	0.99 (3.24)	0.087 (.28)	-0.035 (-.11)	-.010 (-.03)	0.039 (.12)

Column 1 - Column 1 contains the cross section identification name.

Column 2 - This column contains the stream cross section station number.

Column 3 - The assumed water surface elevation must agree with the resulting computed water surface elevation within ± 0.05 m (0.2 ft) (or some other allowable tolerance) for trial calculations to be successful.

Column 4 - Column 4 is the stage-discharge (rating) curve value for the first section; thereafter, it is the value calculated by adding WS (Column 21) to the computed water surface elevation for the previous cross section.

Column 5 - A is the cross sectional area. If the section is complex and has been subdivided into several parts (e.g., left overbank, channel, and right overbank), then use one line of the form for each subsection and add to get the total area of cross section (A_t)

Column 6 - This column contains the wetted perimeter. If the section is subdivided, then one line will be used for each subsection's wetted perimeter.

Column 7 - R is the hydraulic radius. Use the same procedure as for Column 5 if the section is complex, but do not add subsection values.

Column 8 - n is Manning coefficient of channel roughness.

Column 9 - K is the conveyance and is determined using Equation 8.4. This column contains the total conveyance for the cross section. If the cross section is complex, add subsection K values to get the total conveyance (K_t).

Column 10 - K_{ave}, the average conveyance for the reach, is computed with the following equation:

(Equation 8.27)

$$K_{ave} = \frac{1}{2}\left(K_{ds} + K_{us}\right)$$

Column 11 - This column contains the friction slope at the current section and is computed using Equation 8.28.

(Equation 8.28)

$$S_f = \left(\frac{Q}{K}\right)^2$$

Column 12 - The average friction slope is determined using Equation 8.29.

(Equation 8.29)

$$S_{f\,ave} = \left(\frac{Q}{K_{ave}}\right)^2$$

Column 13 - L is the distance between cross sections.

Column 14 - The energy loss due to friction (h_f) through the reach is calculated using Equation 8.30.

(Equation 8.30)

$$h_f = S_{ave}L$$

Column 15 - This column contains part of the expression relating distributed flow velocities to an average value (see Column 16). If the section is complex, one of these values should be calculated for each subsection, and all subsection values should be added to get a total. If one subsection is used, Column 15 is not needed and the kinetic energy coefficient (Column 16) equals 1.0.

Column 16 - The kinetic energy coefficient (α) is calculated with Equation 8.9.

Column 17 - The average velocity (v) for the cross section is calculated with the continuity equation (Equation 8.1).

Column 18 - This column contains the average velocity head, corrected for flow distribution.

Column 19 - This column contains the difference between the downstream and upstream velocity heads. A positive value indicates velocity is increasing; therefore, a contraction coefficient should be used to account for "other losses." A negative value indicates the expansion coefficient should be used in calculating "other losses."

Column 20 - The "other losses" are calculated by multiplying either the expansion coefficient (K_e) or contraction coefficient (K_c) by the absolute value of Column 18. That is, for expansion, the change in velocity head will be negative, but the head loss must be positive.

Column 21 - ΔWS is the change in water surface elevation from the previous cross section. It is the algebraic sum of Columns 14, 19, and 20.

8.6 TWO-DIMENSIONAL BACKWATER MODELS

A. Introduction to Two-Dimensional Modeling. The water surface elevation determination methods previously presented in this chapter are based on one-dimensional analysis. These one-dimensional backwater analysis methods can only approximate flow conditions assuming that the flow has a predominant velocity in one direction ignoring lateral and vertical velocity components. The water surface profiles are very approximate since only the one-dimensional aspects are considered. A two-dimensional backwater analysis considers the velocity vectors in two dimensions and provides more accurate estimates of flow vectors and water surface elevations.

Various types of two-dimensional backwater analysis computer models have been developed. Such models may be used productively by the designer who has adequate background and training in fluid mechanics, mathematics, and computer applications.

B. Finite Element Surface Water Modeling System (FESWMS). The Finite Element Surface Water Modeling System (FESWMS) is designed for use on a microcomputer. An extensive amount of detailed data and input is required; however, some progress has been made toward simplifying and automating the data input by use of computer graphics. For most highway drainage design problems, two-dimensional analysis probably is not justified; however, if a channel analysis situation arises in which a high accuracy is required, contact the main office for more information regarding FESWMS.

C. Application and Limitations of Two-Dimensional Models. Productive and cost-effective application of a two-dimensional backwater model generally is limited to the following situations:

- The crossing is highly sensitive in terms of policy, economy, and physical conditions.
- There is identifiable two-dimensional flow which significantly affects the roadway or associated drainage facilities.
- Other less sophisticated models would fail to yield acceptable results.

Three situations where the use of a 2-dimensional hydraulic model should be considered includes:

- Multiple structures are located across a shared floodplain;
- The structure is near a confluence;
- And/or the structure is located on a stream bed.

An additional element must be considered in the use of FESWMS. The output requires considerable interpretation and reduction before it can be useful. This requires special skill and training on the part of the user.

D. Data Requirements for FESWMS Application. The data requirements for FESWMS are roughly equivalent to one-dimensional models. Numerous ground points with both horizontal and vertical definition may be required for a comprehensive analysis by FESWMS. Elements of the channel system must be identified and quantified with regard to roughness characteristics. Geometric parameters of the roadway and associated drainage facilities must be well

defined and entered as part of the input. The use of digital terrain models greatly simplifies the data requirements for any two-dimensional models.

E. Surface-water Modeling System. Surface-water Modeling System (SMS) is a graphical modeling environment that supports creation of input data for FESWMS and RMA-2. SMS allows the user to use GIS objects to create a conceptual model. After general parameters are assigned to the GIS objects, SMS automatically generates the mesh and assigns boundary conditions, thus reducing the time required to construct the model. The software is supported by FHWA and can be used as a front-end software package to FESWMS. SMS also gives the designer a very clear representation of the data by creating tables and graphs, along with .AVI files. Due to the extensive output generated, SMS requires a fast processor.

8.7 STREAM CHANNEL DESIGN

The analysis of a stream 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 for the design storm(s). An assessment of the existing channel usually is necessary to determine the potential for problems that might result from a proposed action. The level of detail of channel studies will be commensurate with the risk associated with the proposed action and with the environmental sensitivity of the stream and adjoining floodplain.

Although the following step-by-step outline may not be appropriate for all projects, it does provide a logical sequence of actions that usually will apply.

Step 1 Assemble Site Data and Project File

1. Data collection:

- Topographic, site and location maps.
- Roadway profile.
- Photographs.
- Field view notes, minutes and summaries.
- Design data for nearby upstream and downstream structures.
- Gauge records.
- Historic flood data and information from local sources including local county maintenance offices.

2. Studies by other agencies:

- FEMA Flood Insurance Studies.
- Floodplain studies.
- Watershed studies.
- Stormwater Management Plans.
- Army Corps of Engineer project studies.

3. Environmental considerations:

- Floodplain encroachment.
- Floodway designation.
- Wildlife habitat.
- Commitments in NEPA documents.

4. Design criteria:

- PennDOT design policy.
- Regulatory design criteria.

Step 2 Determine the Project Scope

1. Determine level of assessment:

- Stability of existing channel.
- Potential for damage.
- Sensitivity of the stream.

2. Determine type of hydraulic analysis:

- Qualitative assessment.
- Slope Conveyance Method.
- Step-backwater analysis.

3. Determine additional survey information:

- Extent of streambed profiles.
- Locations of cross sections.
- Elevations of flood-prone properties.
- Details of existing structures.
- Physical and hydraulic properties of bed and bank materials.

Step 3 Evaluate Hydrologic Variables

1. Compute discharges for selected frequencies.

2. Consult Chapter 7, *Hydrology*.

Step 4 Perform Hydraulic Analysis

1. Qualitative Assessment.

2. Slope Conveyance Method (see Section 8.4):

- Select representative cross section.
- Select appropriate n values.
- Compute stage-discharge relationship.

3. Step-backwater analysis:

- Select appropriate cross sections.
- Determine n values, boundary conditions, and other data.
- Progress through the cross sections to calculate the final total energy in the stream and predicted water surface elevations for the stream reach.

4. Compare solution to known high water marks (if available).

Step 5 Perform Stability Analysis

1. Geomorphic factors.

2. Hydraulic factors.

3. Stream response to change

Step 6 Design Countermeasures

1. Criteria for selection:

- Erosion mechanism.
- Stream characteristics.
- Construction and maintenance requirements.
- Safety.
- Vandalism considerations.
- Cost.

2. Types of countermeasures:

- Meander migration countermeasures.
- Bank stabilization (Chapter 8.10, Bank Protection).
- Bend control countermeasures.
- Channel braiding countermeasures.
- Degradation countermeasures.
- Aggradation countermeasures.

3. For additional information:

- Design Manual Part 4.
- HEC-23, *Bridge Scour and Stream Instability Countermeasures* (FHWA, 2001a).
- *Stream Corridor Restoration, Principles, Practices, and Processes* (NRCS, 1998).
- HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, 2000).
- HEC-11, *Design of Riprap Revetment* (FHWA, 1989a).
- HEC-15, *Design of Roadside Channels with Flexible Linings* (FHWA, 1988).
- HEC-20, *Stream Stability at Highway Structures* (FHWA, 2001d).
- *River Engineering for Highway Encroachments* (FHWA, 2001c).
- U.S. Army Corps of Engineers Technical Publications.

Step 7 Documentation

- Prepare report and file with background information.
- See Chapter 4, *Documentation and Document Retention*.

8.8 ROADSIDE CHANNEL DESIGN

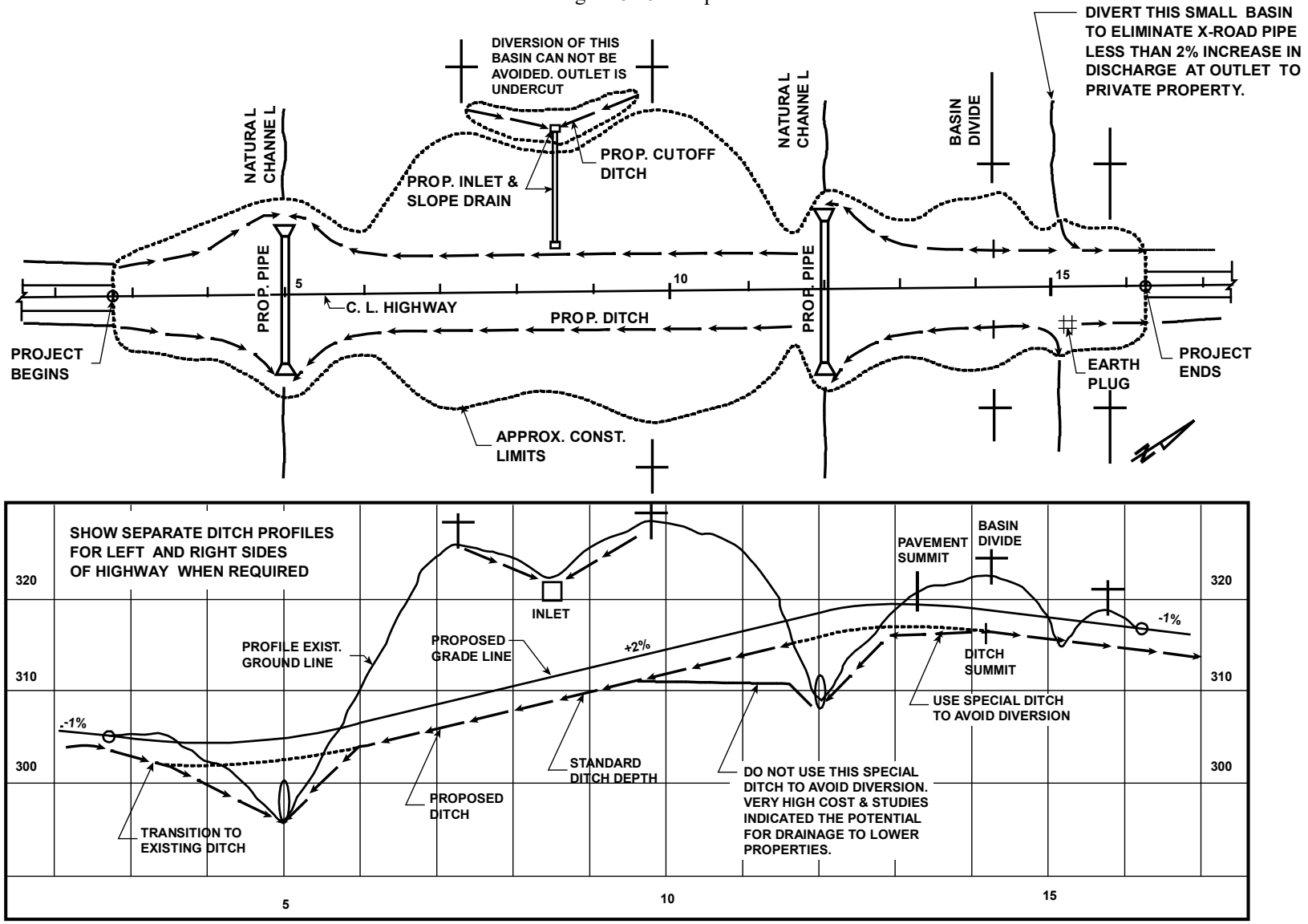
A roadside channel is an open channel that roughly parallels the highway and is typically within the limits of the right-of-way. Roadside channels also may be referred to as roadside ditches, swales, or diversion ditches. These channels normally are trapezoidal or V-shaped in cross section and lined with grass or a special protective lining. The considerations and procedures discussed in this section and in Section 8.11 generally are applicable to median ditches and other small excavated channels.

Roadside channels should accommodate the design discharge in a safe, functional, and economical manner. Their alignment, cross section, and grade usually are constrained by the geometric and safety standards applicable to the highway project.

Roadside channels may be designed with rigid or flexible linings. Flexible linings in channels conform better to a changing channel shape, whereas rigid linings cannot. Flexible linings also are less expensive generally, permit infiltration and exfiltration, have a natural appearance, and usually provide better habitat opportunities for local flora and fauna; however, an erosive force of high magnitude may be resisted better by a rigid lining. To address post-construction stormwater controls, flexible linings would provide a simple stormwater control if the erosive forces can be managed.

An example of a roadside channel plan/profile is shown in Figure 8.16.

Figure 8.16 Sample Roadside Channel



A. Channel Design Considerations. Local soil conditions, flow depths, and velocities within the channel are usually the primary considerations in channel design; however, terrain and safety considerations have considerable influence.

B. Channel Linings. Channel lining may be desirable or necessary to accomplish the following:

- Minimize maintenance.
- Resist the erosive forces of flowing water.
- Increase the velocity and/or conveyance to improve hydraulic efficiency.
- Limit the channel size for right-of-way or safety considerations.
- Prevent the formation of sinkhole in karst areas.

Highway drainage channel linings vary in cost, durability, hydraulic roughness, and appearance. Wherever possible, highway drainage channel design should make use of native, natural materials such as grass, crushed rock and earth; however, it often is necessary to use other types of materials for hydraulic, economic, safety, aesthetic and environmental reasons. When designing a channel consider the stormwater benefits that certain materials have and use those materials when reasonable to satisfy the NPDES permitting requirements for post-construction controls. For example, grass or other plantings would provide more stormwater benefits than a rock lined channel.

NOTE: Refer to the HEC-15, *Design of Roadside Channels with Flexible Linings* (FHWA, 1988) for comprehensive descriptions, advantages, and disadvantages of rigid and flexible channel linings.

C. Rigid Linings. The most common types of rigid linings include the following:

- Cast-in-place concrete.
- Soil cement.
- Grout bags.
- Grouted riprap.

Rigid channel linings have certain disadvantages when compared to natural or earth-lined channels. The initial construction cost of rigid linings usually is greater than the cost of flexible linings. Maintenance costs also may be high because rigid linings are susceptible to damage by undercutting, hydrostatic uplift, and erosion along the longitudinal interface between the lining and the underlying material.

Rigid linings have the further disadvantage from a stormwater runoff perspective by inhibiting natural infiltration in locations where infiltration is desirable or permissible. Smooth linings usually cause high flow velocities with scour occurring at the terminus of these sections, unless controlled with riprap or other energy dissipating devices.

Rigid linings may be undesirable in areas where water quality considerations are of major concern. Contaminants may be transported to the receiving waters whereas a vegetative or flexible type of lining may filter the contaminants from the runoff.

Aesthetics should be considered when determining the location and type of a paved ditch or channel. Concrete lining can be used on very flat slopes to increase the velocity and efficiently remove water from ponded areas or to reduce the size of channel required to handle the design discharge. However, there may be situations where the increased velocity resulting from concrete lining is undesirable and rock lining or an energy dissipator should be used.

D. Permanent Flexible Linings. Permanent flexible linings include the following:

- Rock riprap.
- Wire-enclosed riprap (gabions).
- Vegetative lining.
- Geotextile fabrics.
- Articulated blocks and mats.

Flexible linings generally are less costly to construct, have self-healing qualities which reduce maintenance costs, permit infiltration and exfiltration, present a more natural appearance, and may provide safer roadsides.

From a stormwater management (SWM) perspective, flexible linings permit natural infiltration and generally reduce velocities compared to smooth, rigid lining. Vegetated linings aid in filtering contaminants from runoff. Check dams installed in channels with flexible lining increase runoff retention capacity. Additional information on the use of check dams in vegetated swales can be found in Chapter 14, *Post-Construction Stormwater Management*.

Various species of grass may be used as permanent channel lining if flow depths, velocities, and soil types are within acceptable tolerances for vegetative lining. The turf may be established by sodding or seeding. Sod usually is more expensive than seeding, but it has the advantage of providing immediate protection. Some type of temporary protective covering often is required for seed and top soil until vegetation becomes established.

The use of sod should be carefully evaluated as a ditch or channel liner since sodding is expensive, labor intensive, and must be installed only during certain periods of the year when soil conditions are conducive to grass root establishment. Sod as specified in Publication 408, *Specifications,* is predominately comprised of Kentucky Bluegrass grass species and is not desirable for areas which cannot or will not be readily maintained by mowing to keep the sod properly established. Sod can be used for intermittent flows or velocities up to 2.7 m/s (9 ft/s) where soils are erosion resistant, where immediate protection is required, or where aesthetically a more pleasing appearance is desired, such as when construction adjoins established lawns or other fine turf areas. Sod lining should not be used in the vicinity of pipe outlets since pipe outlet velocities generally exceed allowable sod or other grass stand velocities.

Satisfactory performance from any of the erosion protection materials is dependent on proper installation techniques and, in particular, the installation of terminal ends, material joints and overlaps and edges at the crest, toe and sides.

E. Temporary Flexible Linings. The following are classified as temporary flexible linings:

- Geotextile fabrics.
- Straw with net.
- Curled wood mat.
- Jute, paper, or synthetic net.
- Synthetic mat.

Temporary channel lining and protecting covering may consist of jute matting or excelsior (wood wool) mats. Straw or wood chip mulches tacked with asphalt usually are not well suited for channel invert lining but may be used for side slopes. Geotextile materials, known as soil stabilization mats, may be used for protective linings in ditches and on side slopes. These materials are not biodegradable and may serve as permanent soil reinforcement while enhancing the establishment of vegetation.

F. Channel Lining Design Procedure. A suggested design procedure for roadside channels is comprised of the following seven basic steps. Each project is unique, but this procedure is applicable for most PennDOT applications. Generally, to optimize the roadside channel system design, several trial runs may be necessary before a final design is achieved. In addition, if vegetative lining is chosen, an analysis of both temporary and permanent conditions is often necessary.

1. Establish a roadside plan.

- Collect available site data.
- Obtain or prepare existing and proposed plan/profile layouts including highway, culverts, bridges, etc.
- Determine and plot on the plan the locations of natural basic divides and roadside channel outlets.
- Lay out the proposed roadside channels to minimize diversion flow length, velocity or time.

2. Obtain or establish cross section geometry.

- Identify features which may restrict cross section design including right-of-way limits, trees or environmentally sensitive areas, utilities, and existing drainage facilities.
- Provide channel depth adequate to drain the subbase and minimize freeze-thaw effects.
- Choose channel side slopes based on geometric design criteria including safety, economics, soil, aesthetics, and access.

- Establish the bottom width of trapezoidal channel.

3. Determine initial channel grades.

- Plot initial grades on plan-profile layout (slopes in roadside ditch cuts are usually controlled by highway grades).
- Establish a minimum grade to minimize ponding and sediment accumulation.
- Consider the influence of type of lining on grade.
- Where possible, avoid features which may influence or restrict grade, such as utility locations.

4. Check flow capacities and adjust as necessary.

- Compute the design discharge at the downstream end of a channel segment (see Chapter 7, *Hydrology*). Temporary channels are to be designed to convey the peak discharge from the 2-year frequency storm event (5-year in Special Protection Watersheds). All permanent channels are to be designed to convey the 10-year storm peak discharge.
- Set preliminary values of channel size, roughness, and slope.
- Determine the maximum allowable depth of channel allowing for freeboard.
- Check the flow capacity using Manning Equation and single cross section analysis.
- If the capacity is inadequate, possible adjustments are as follows:
  - Increase bottom width.
  - Make channel side slopes flatter.
  - Make channel (longitudinal) slope steeper.
  - Provide smoother (lower Manning's n-value) channel lining.
  - Install drop inlets and a parallel storm sewer beneath the channel to supplement capacity.
- Provide smooth transitions at changes in channel cross sections.
- Provide extra channel storage where needed to replace floodplain storage, to encourage infiltration, or to reduce peak discharge. Guidance for this can be found in Chapter 7, *Hydrology* and Chapter 14, *Post Construction Stormwater Management*.

5. Determine whether a protective lining is needed.

- Set preliminary values for channels discharging to sediment traps or basins.
- If the anticipated velocity (Table 8.12) or shear stress (Table 8.11) exceeds the maximum permissible for the type of soil present, a protective lining is needed (Step 6).

6. Determine channel lining or protection needed.

- Start with Table 8.9 to help select appropriate lining(s) based on various design considerations.
- Two methods may be used to determine if channel lining is adequate: Maximum Allowable Velocity or Permissible Shear Stress.
  - The velocity method may be used for slopes less than 10%.
  - The shear stress method may be used for all slopes.
- Permissible Shear Stress
  - Calculate normal flow depth, y_o, in m (ft) at design discharge.
  - Compute maximum design shear stress on a channel bottom and channel side as:

$$\tau_d = \gamma_w \, y_o \, S \qquad \textit{(Equation 8.31)}$$

where: γ_w = unit weight of water, N/m^3 (lbf/ft^3)
τ_d = maximum design shear stress at on a channel bottom and channel side, N/m^2 (lb/ft^2)
S = slope of channel bottom, m/m (ft/ft)

Select a lining and determine the permissible shear stress (τ_p in N/m^2 (lb/ft^2)) from Table 8.11 using the retardance class from Table 8.13.

- Maximum Allowable Velocity

- Calculate the design velocity, v_d, in m/s (ft/s) using Manning Equation.
- Select a lining and determine the maximum allowable velocity (v_a in m/s (ft/s)) from Table 8.12.

- If $\tau_d < \tau_p$ or $v_d < v_a$, 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).
  - Decrease channel slope.
  - Decrease slope in combination with drop structures.
  - Increase channel width or flatten side slopes.

7. Analyze outlet points and downstream effects.

- Identify any adverse impacts to downstream properties which may result from one of the following at the channel outlet:
  - Increase or decrease in discharge.
  - Increase in velocity of flow.
  - Confinement of sheet flow.
  - Change in outlet water quality.
  - Diversion of flow from another watershed (try to avoid this wherever possible).
- Mitigate any adverse impacts identified above. Possibilities include the following:
  - Enlarge outlet channel or install control structures to provide detention of increased runoff in channel.
  - Install velocity control structures.
  - Increase capacity or improve lining of downstream channel.
  - Install sedimentation/infiltration basins.
  - Install sophisticated weirs, level spreaders, 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.

Determine whether the permissible velocity design method (for channels with bed slopes less than 10%) or the shear stress lining design method will be used. If the permissible velocity method is used, make sure that the design velocity does not exceed the permissible velocity listed in Table 8.12 (with attached notes) as applicable. If the shear stress method is used, make sure the anticipated shear stress does not exceed that shown in Table 8.11 or manufacturer's recommendations (based upon independent testing for at least 20 minutes). For additional information, refer to HEC-15, *Design of Roadside Channels with Flexible Linings* (FHWA, 2005a) for design of steep gradient channels and ditches with flexible linings or HEC-11, *Design of Riprap Revetment* (FHWA, 1989) for design of channels with rigid linings.

Use of the channel design worksheet in the *PA DEP E&S Manual* is recommended for temporary and permanent channel lining calculations.

Table 8.9 Channel Lining Application and Considerations

Lining Type	Considerations					
	Functional Longevity	**Immediate Stabilization**	**Seasonal**	**Construction Effort**	**Initial Cost**	**SWM Benefits**
Grass	Perm	No	Yes	Minimal	Low	Yes
Sod	Perm	Yes	Yes	Intensive	High	Yes
RECP [1]	Temp/Perm	Yes	No	Moderate	Moderate	Yes
Rock (Riprap)	Perm	Yes	No	Moderate	Moderate	No
Concrete	Perm	Yes	Yes	Intensive	High	No

[1] Rolled Erosion Control Products

The Manning's n-value should be adjusted according to the type of channel lining and flow depth. Refer to Table 8.10 for appropriate n-values.

Table 8.10 Adjusted Manning's n-Values for Various Linings

Lining Category	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 Element	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
Temporary	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Permanent	TRM	0.036	0.025	0.021
	Riprap	See Figure 8.18		
	Gabion	0.030	0.028	0.026
	Reno Mattress	0.030	0.028	0.026
Vegetated	Classes A-E	See Equation 8.32		

Table adapted from HEC-15

Table 8.11 Allowable Shear Stresses for Various Linings

Lining Category	Lining Type	Allowable Unit Shear Stress	
		Pa	lb/ft^2
Unlined – Easily Eroded Soils [1]	Silts, Fine-Medium Sands	1.4	0.03
	Coarse Sands	1.9	0.04
	Very Coarse Sands	2.4	0.05
	Fine Gravel	4.8	0.10
Unlined – Erosion Resistant Soils [2]	Clay Loam	12.0	0.25
	Silty Clay Loam	8.6	0.18
	Sandy Clay Loam	4.8	0.10
	Loam	3.4	0.07
	Silt Loam	5.7	0.12
	Sandy Loam	1.0	0.02
	Gravely, Stony, Channery Loam	2.4	0.05
	Stony or Channery Silt Loam	3.4	0.07
Non-Reinforced Vegetation	Class A	177.2	3.70
	Class B	100.6	2.10
	Class C	47.9	1.00
	Class D	28.7	0.60
	Class E	16.8	0.35
Temporary RECPs	Mulch Control Netting	See Table 8.15	
	Open Weave Textile		
	Erosion Control Blanket Without a Net		
	Single Net Erosion Control Blanket		
	Double Net Erosion Control Blanket		
Permanent RECPs [3]	Turf Reinforcement Mat – Type 5.A	288	6.0
	Turf Reinforcement Mat – Type 5.B	384	8.0
	Turf Reinforcement Mat – Type 5.C	480	10.0
	Turf Reinforcement Mat – Type 5.D	575	12.0
	Turf Reinforcement Mat – Type 5.E	575	12.0
	Turf Reinforcement Mat – Type 5.F [4]	575	12.0
Riprap Lining	R-3	48	1.0
	R-4	96	2.0
	R-5	144	3.0
	R-6	192	4.0
	R-7	240	5.0
	R-8	384	8.0
	Gabion – 305 mm (12 in)	225	4.7
	Gabion – 457 mm (18 in)	249	5.2
	Gabion – 914 mm (36 in)	397	8.3
	Reno Mattress – 152 mm (6 in)	206	4.3
	Reno Mattress – 229 mm (9 in)	220	4.6

[1] Soils having an erodibility K factor greater than 0.37.
[2] Soils having an erodibility K factor less than or equal to 0.37.
[3] Values for vegetated condition; unvegetated condition is 96 Pa (2.0 lb/ft^2).
[4] High Performance.

Table 8.12 Permissible Velocities for Various Linings

Lining Category	Lining Type / Soil Material	Permissible Velocity	
		m/s	ft/sec
Unlined [13]	Fine sand, noncolloidal	0.4	1.5
	Sandy loam, noncolloidal	0.5	1.7
	Silt loam, noncolloidal	0.6	2.0
	Alluvial silts, noncolloidal	0.6	2.0
	Ordinary firm loam	0.7	2.5
	Stiff clay, very colloidal	1.1	3.7
	Alluvial silts, colloidal	1.1	3.7
	Fine gravel	0.7	2.5
	Graded, loam to cobbles, noncolloidal	1.1	3.7
	Graded, silt to cobbles, colloidal	1.2	4.0
	Coarse gravel, noncolloidal	1.2	4.0
	Cobbles and shingles	1.5	5.0
	Shales and hardpans	1.8	6.0
Vegetated – Easily Eroded Soils [1, 5-12] Non-Reinforced	[3] Seed Mix. 0-5% Slope	1.2	4.0
	[3] Seed Mix. 5-10% Slope	0.9	3.0
	[4] Sod 0-5% Slope	1.5	5.0
	[4] Sod 5-10% Slope	1.2	4.0
	[4] Sod > 10% Slope	0.9	3.0
Vegetated – Erosion Resistant Soils [2, 5-12] Non-Reinforced	[3] Seed Mix. 0-5% Slope	1.5	5.0
	[3] Seed Mix. 5-10% Slope	1.2	4.0
	[4] Sod 0-5% Slope	2.1	7.0
	[4] Sod 5-10% Slope	1.8	6.0
	[4] Sod > 10% Slope	1.2	4.0
Riprap Lining	R-3	2.0	6.5
	R-4	2.7	9.0
	R-5	3.4	11.5
	R-6	3.9	13.0
	R-7	4.3	14.5
	Gabion - 305 mm (12 in)	4.6	15.0
	Gabion - 457 mm (18 in)	5.5	18.0
	Gabion - 914 mm (36 in)	6.7	22.0
	Reno Mattress - 152 mm (6 in)	1.8	6.0
	Reno Mattress - 229 mm (9 in)	3.6	12.0

[1] Soils having an erodibility K factor greater than 0.37.
[2] Soils having an erodibility K factor less than or equal to 0.37.
[3] Grass Mixture Formulas, as specified in Publication 408, *Specifications*, Section 804.2.
[4] Cultivated SOD, as specified in Publication 408, *Specifications*, Section 809.2 (suggested for intermittent flow only).
[5] Use a maximum 0.9 m/s (3.0 ft/s) if only sparse cover can be established or maintained.
[6] Use 0.9 - 1.2 m/s (3.0 - 4.0 ft/s) under normal conditions if the vegetation is to be established by seeding.
[7] Use 1.2 - 1.5 m/s (4.0 - 5.0 ft/s) if a dense, vigorous sod is obtained quickly or if water can be diverted out the waterway while vegetation is being established.
[8] Use 1.5 - 1.8 m/s (5.0 - 6.0 ft/s) on well-established, good quality sod.
[9] Use 1.8 m/s (6.0 ft/s) to 2.1 m/s (7.0 ft/s) may be used only on established, excellent quality sod.
[10] If erosion resistant materials supplement the vegetative lining, increase by 0.6 m/s (2.0 ft/s).
[11] A rock lined low flow channel should be incorporated when base flow exists.
[12] Use sod only where there is sufficient soil cover to allow proper stapling of the sod.
[13] Based on clear water discharges. Reference: FHWA, HDS No. 3, Design Charts for Open Channel Flow.

G. Vegetated Lining. Vegetation effectively stabilizes most channels and is the preferred method of channel stabilization. However, the seeded areas must be protected until the seeds can germinate, grow, and establish a good vegetative surface cover. Establishing a grass-lined ditch or channel depends on several factors. See Table 8.11 and Table 8.12 for the allowable design shear stresses and velocities, respectively, for grass and sod. Evaluate and specify the appropriate channel protection needed based on slope and velocity constraints. For a complete listing of seeding formulas refer to Publication 408, *Specifications*. In instances where steep slopes or high discharge rates exceed design guidelines for vegetated channels, linings constructed from erosion-resistant materials should be considered. Provide a suitable temporary liner wherever vegetative linings are proposed. Separate calculations may be provided for each proposed vegetated lining: one for when the vegetation is fully established, and one for when the grass is still growing.

Table 8.13 Retardance Class for Lining Materials

Retardance Class	Cover	Condition
A	Weeping Lovegrass	Excellent stand, tall (average 760 mm) (2.5 ft)
	Yellow Bluestem Ischaemum	Excellent stand, tall (average 915 mm) (3.0 ft)
B	Bermuda grass	Good stand, tall (average 305 mm) (1.0 ft)
	Native grass mixture: little bluestem, bluestem, blue gamma, other short and long stem midwest grasses	Good stand, unmowed
	Weeping lovegrass	Good stand, tall (average 610 mm) (2.0 ft)
	Lespedeza sericea	Good stand, not woody, tall (average 480 mm) (1.57 ft)
	Alfalfa	Good stand, uncut (average 280 mm) (.91 ft)
	Weeping lovegrass	Good stand, unmowed (average 330 mm) (1.08 ft)
	Blue grama	Good stand, uncut (average 330 mm) (1.08 ft)
C	Crabgrass	Fair stand, uncut (255 to 1220 mm) (.83 to 4.0 ft)
	Bermuda grass	Good stand, mowed (average 150 mm) (.50 ft)
	Common lespedeza	Good stand, uncut (average 280 mm) (.91 ft)
	Grass-legume mixture: summer (orchard grass redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (150-200 mm) (.50 - .65 ft)
	Centipede grass	Very dense cover (average 150 mm) (.50 ft)
	Kentucky bluegrass	Good stand, headed (150 - 305 mm) (.50 - 1.0 ft)
D	Bermuda grass	Good stand, cut to 65 mm (.21 ft)
	Common lespedeza	Excellent stand, uncut (average 115 mm) (.38 ft)
	Buffalo grass	Good stand, uncut (75 - 150 mm) (.25 - .50 ft)
	Grass-legume mixture: fall, spring (orchard grass redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (100 - 125 mm) (.33 - .41 ft)
	Lespedeza serices	After cutting to 50 mm (.16 ft) (very good before cutting)
E	Bermuda grass	Good stand, cut to 40 mm (.13 ft)
	Bermuda grass	Burned stubble

The Manning's roughness coefficient for grass-lined channels varies based on grass properties and the shear force exerted by flow on the grass. This is because the applied shear from the flow causes the grass stems to bend. This reduces the stem height relative to the depth of flow which reduces the roughness. Manning's roughness coefficient for grass-lined channels should be calculated using Equation 8.32.

$$n = \alpha C_n \tau_e^{-0.4} \qquad \textit{(Equation 8.32)}$$

where,

- n = Manning's roughness coefficient
- α = unit conversion constant, 1.0 (SI), 0.213 (CU)
- C_n = grass roughness coefficient, Table 8.14
- τ_o = mean boundary shear stress, N/m^2 (lb/ft^2)
  = $\gamma R S_o$, where γ = specific weight of water, R= hydraulic radius, S_o = channel slope

The grass roughness coefficient, C_n, accounts for the combined effect of the grass stem height and density-stiffness of the subject grass. Values of C_n for SCS standard retardance classes are provided in Table 8.14.

Table 8.14 Grass Roughness Coefficient, C_n, for SCS Retardance Classes

Retardance Class	A	B	C	D	E
Stem Height, mm (in)	910 (36.0)	610 (24.0)	200 (8.0)	100 (4.0)	40 (1.6)
C_n	0.605	0.418	0.220	0.147	0.093

H. Rolled Erosion Control Products (RECPs). In the late 1960's, faced with the limitation of conventional mulching techniques, manufacturers initiated the development of what has become a diverse group of products known as Rolled Erosion Control Products, or RECPs. This category consists of prefabricated products such as mulch control nets, open-weave geotextiles, erosion control blankets, and turf reinforcement mats. Manufactured from wood excelsior, straw, jute, coir, polyolefins, PVC and nylon, this growing family of materials enables designers to incorporate the superiority of longfibered mulches with the tensile strength of dimensionally stable nets, meshes and geotextiles.

The Erosion Control Technology Council (ECTC) developed standardized terminology for these products, which FHWA and many other DOTs have adopted and incorporated into their construction specifications.

First, it is important to understand the difference between temporary and permanent RECPs.

1. Temporary RECPs. For applications where natural vegetation alone will provide sufficient permanent erosion protection, furnish a temporary RECP with the necessary longevity and performance properties to effectively control erosion and assist in the establishment of vegetation under the anticipated immediate site conditions.

2. Permanent RECPs. For applications where natural vegetation alone will not sustain expected flow conditions and/or provide sufficient long-term erosion protection, furnish a permanent RECP with the necessary performance properties to effectively control erosion and reinforce vegetation under the expected long-term site conditions.

PennDOT's designations for RECPs are described in Publication 13M, Design Manual, Part 2*, Highway Design*, Chapter 13 and in Publication 408, *Specifications*. The designations for RECPs are as follows:

1. Mulch Control Netting. Mulch control netting is a planar woven natural fiber or extruded geosynthetic mesh used as a temporary degradable RECP to anchor loose fiber mulches.

2. Open Weave Textile (OWT). Open-weave textile is a degradable RECP that is composed of natural or polymer yarns woven into a matrix. OWT can be used together with straw mulch to retain soil moisture and to increase the density and thickness of the lining. OWT is more flexible, thinner and less dense compared to erosion control blankets. Placement of OWT is usually done immediately after seeding operations.

3. Erosion Control Blanket (ECB). Erosion control blanket is a degradable RECP that is composed of an even distribution of natural or polymer fibers that are mechanically, structurally, or chemically bound together to form a continuous mat. One or both sides of the mat may be reinforced with a rapidly degrading, synthetic or natural fiber net. ECB is stiffer, thicker and denser than an open-weave textile. When ECBs are used and ultimately degrade, the long-term erosion protection is provided by the established vegetation.

4. Turf Reinforcement Mat (TRM). Turf reinforcement mat is a non-degradable RECP composed of UV stabilized synthetic fibers, filaments, netting and/or wire mesh processed into a three-dimensional matrix. TRMs provide sufficient thickness, strength and void space to permit soil filling and establishment of grass roots within the matrix. TRM is stiffer, thicker, and denser than an erosion control blanket. Two methods of seeding can be used with TRM. One choice is to seed before placement of the TRM, which allows the plant stems to grow through the mat. The second choice is to first place the TRM then cover the mat with soil and then seed. This method allows the plant roots to grow within the mat.

Table 8.15 Temporary RECPs

ULTRA SHORT-TERM - Typical 3 months functional longevity		
Type	**Product Description**	**Permissible Shear Stress [1]**
1.A	Netting/Open Weave Textile	48 Pa (1.0 lb/ft^2)
1.B	Erosion Control Blanket Without a Net	48 Pa (1.0 lb/ft^2)
1.C	Single Net Erosion Control Blanket	72 Pa (1.5 lb/ft^2)
1.D	Double Net Erosion Control Blanket	84 Pa (1.75 lb/ft^2)
SHORT-TERM - Typical 12 months functional longevity		
Type	**Product Description**	**Permissible Shear Stress [1]**
2.A	Netting/Open Weave Textile	48 Pa (1.0 lb/ft^2)
2.B	Erosion Control Blanket Without a Net	48 Pa (1.0 lb/ft^2)
2.C	Single Net Erosion Control Blanket	72 Pa (1.5 lb/ft^2)
2.D	Double Net Erosion Control Blanket	84 Pa (1.75 lb/ft^2)
EXTENDED-TERM - Typical 24 months functional longevity		
Type	**Product Description**	**Permissible Shear Stress [1]**
3.A	Open Weave Textile	96 Pa (2.0 lb/ft^2)
3.B	Erosion Control Blanket	96 Pa (2.0 lb/ft^2)
LONG-TERM - Typical 36 months functional longevity		
Type	**Product Description**	**Permissible Shear Stress [1]**
4.A	Open Weave Textile	108 Pa (2.25 lb/ft^2)
4.B	Erosion Control Blanket	108 Pa (2.25 lb/ft^2)

[1] Minimum shear stress RECP (unvegetated) can sustain without physical damage or excess erosion [> 12.7 mm (0.5 in) soil loss] during a 30-minute flow event in large-scale testing.

Adapted from ECTC, Standard Specifications for Rolled Erosion Control Products, 2017

Table 8.16 Permanent RECPs

Type	Product Description	Unvegetated Permissible Shear Stress [1]	Vegetated Permissible Shear Stress [1]
5.A	Turf Reinforcement Mat	96 Pa (2.0 lb/ft^2)	288 Pa (6.0 lb/ft^2)
5.B	Turf Reinforcement Mat	96 Pa (2.0 lb/ft^2)	384 Pa (8.0 lb/ft^2)
5.C	Turf Reinforcement Mat	96 Pa (2.0 lb/ft^2)	480 Pa (10.0 lb/ft^2)
5.D	Turf Reinforcement Mat	96 Pa (2.0 lb/ft^2)	575 Pa (12.0 lb/ft^2)
5.E	Turf Reinforcement Mat	96 Pa (2.0 lb/ft^2)	575 Pa (12.0 lb/ft^2)
5.F	High Performance Turf Reinforcement Mat	96 Pa (2.0 lb/ft^2)	575 Pa (12.0 lb/ft^2)

[1] Minimum shear stress RECP (unvegetated) can sustain without physical damage or excess erosion [> 12.7 mm (0.5 in) soil loss] during a 30-minute flow event in large-scale testing.

Adapted from ECTC, Standard Specifications for Rolled Erosion Control Products, 2017

I. Rock (Riprap) and Concrete. Aesthetics and stormwater issues should be considered when determining the location and type of a ditch or channel. Concrete lining can be used on very flat slopes to increase the velocity and efficiently remove water from ponded areas or to reduce the size of channel required to handle the design discharge. There may be situations where the increased velocity resulting from concrete lining is undesirable and rock lining or an energy dissipator should be used. Rock channel lining is usually reserved for steep slopes where high velocities and/or shear stresses are anticipated, or adjacent to hydraulic structures. It is worth noting that from a stormwater management aspect, vegetated channels are preferred over concrete and rock channels because of their water quantity and quality benefits.

Determine the rock size of rock lining for roadside channels from the procedures outlined below, or from those in HEC-15, *Design of Roadside Channels with Flexible Linings* (FHWA, 2005a). The rock size, gradation, and thickness should be determined in accordance with Publication 408, *Specifications*, Section 850. The design $D_{15\text{-}50}$ size (15-50% passing) consistent with Publication 408, *Specifications*, should be used as the D_{50} size for design purposes.

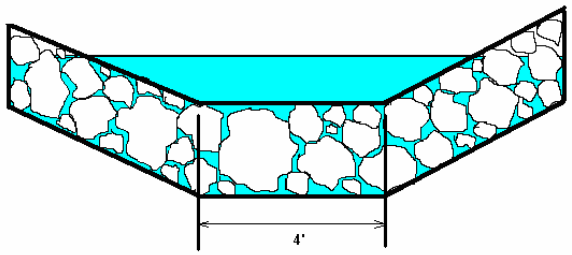
Table 8.12 shows the maximum permissible velocities for different size rock. Rock-lined channels may be sized on the basis of the maximum permissible velocity under the following conditions:

- The channel alignment is straight or gradually turning.
- Channel slope <10%.
- A geotextile lining underlayment is provided.

Due to the rapid increase in shear stress with the increasing bed slope, a 40% void space may be assumed in the riprap on channel bottoms (not side slopes) for gradients > 10%. Since this void space is filled before flow occurs on top of the riprap liner, the cross-sectional area of that void space may be subtracted from that of the in-channel flow prior to determining the anticipated flow depth to be used in the shear stress computation. If this results in a flow depth less than zero, use maximum velocity to size the riprap. If the resulting shear stress is still greater than the maximum allowable, and the 12:1 flow width to depth ratio is reached, Reno mattresses or gabions should be considered.

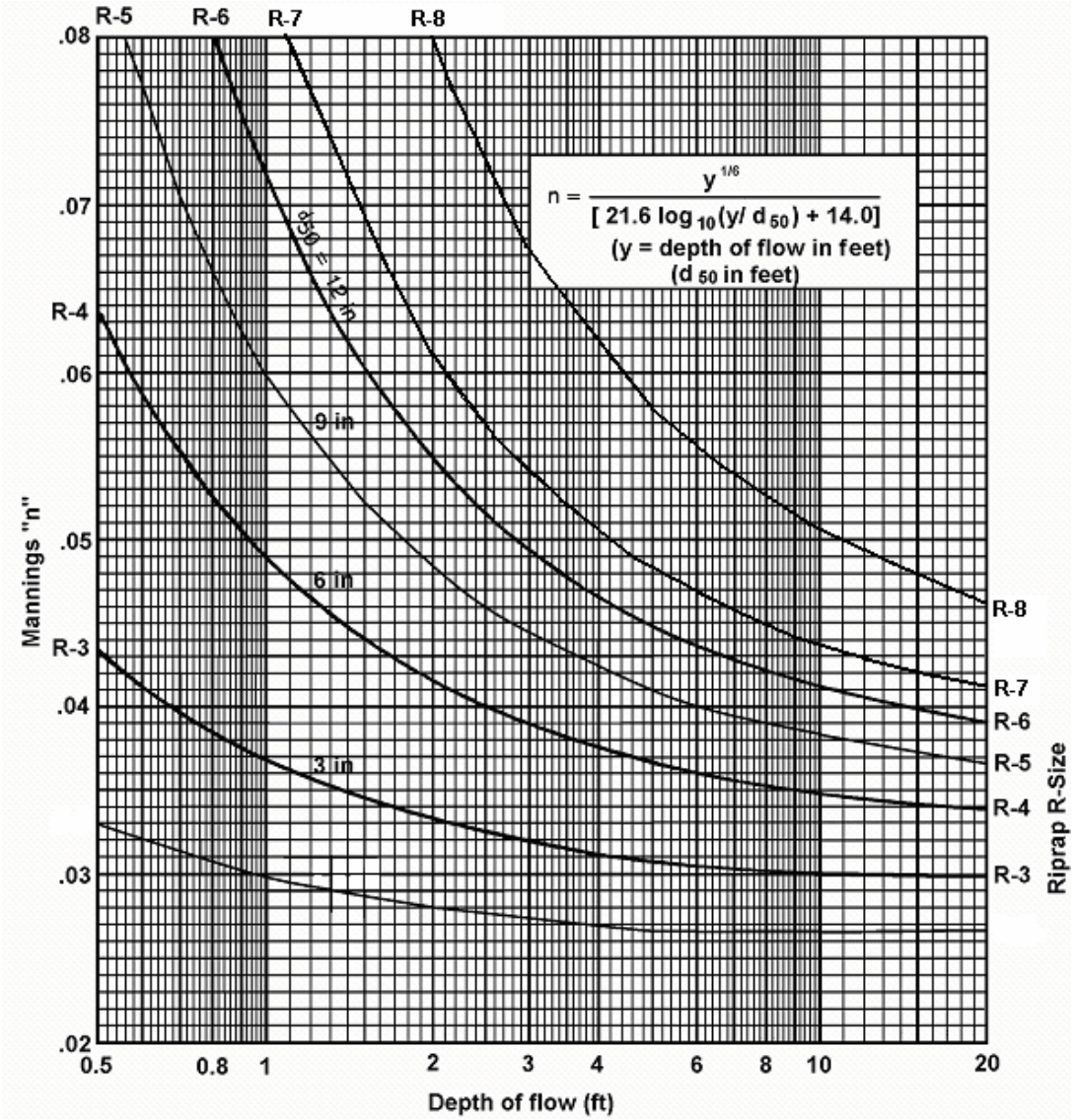
The maximum allowable shear stresses for riprap channel linings are given in Table 8.11. Manning's n-value used should be taken from Figure 8.18. Since the roughness coefficient varies significantly with the size of the rock and the depth of flow, use of standard n-values may result in undersizing of the channel or of the riprap protection.

Figure 8.17 Void Space in Riprap Channel Bottom



Example: A 1.2 m (4 ft) wide trapezoidal channel is being designed with R-4 riprap. The maximum stone size for R-4 riprap is 305 mm (12 in). Therefore, the minimum cross-sectional area of the riprap is 0.37 m^2 (4 ft^2) on the bottom of the channel. Assuming 40% void space, 0.15 m^2 (1.6 ft^2) may be subtracted from the cross-sectional area of the water flowing on top of the stone.

Figure 8.18 Manning's n-Values for Riprap Channels



From the PA DEP Erosion and Sediment Pollution Control Program Manual, 2012

8.9 STREAM STABILITY ISSUES

A. Stream Geomorphology. Fluvial or stream geomorphology is the study of the development and configuration of the earth's surface as formed by streams. River mechanics is an inclusive term primarily dealing with the action of rivers on the earth's surface, including their response to natural and artificial modifications. Planning and location engineers should be conscious of fluvial geomorphology and request the services of designers to quantify natural changes and changes that may occur as a result of stream encroachments, crossings, or channel modifications.

Fluvial geomorphology and river mechanics are not new subjects; however, methods of quantifying the interrelation of variables are relatively recent developments. For many years, engineers intuitively have considered many of these factors. The theories and knowledge available today make it possible to predict various reactions to changes and, more importantly, to establish thresholds for tolerance to change.

1. Non-alluvial Channels. A non-alluvial stream is a channel which has a boundary of bedrock. There are non-alluvial channels which can be degrading their beds. An example of such a stream is Bentley Creek in Tioga County. Many mountain streams are classified as non-alluvial and a hydraulic analysis may be performed utilizing rigid boundary theory.

2. Alluvial Channels. Alluvial streams are streams whose beds and banks are composed of materials that are transported and deposited by water, such as clay, silt, gravel, or various combinations of these materials. Streams have inherent dynamic qualities by which changes continually occur in the stream's position and shape. Changes may be slow or rapid, but all streams are subjected to forces that cause changes to occur. In these streams, banks erode, sediments are deposited, and islands and side channels form and disappear in time. The banks and adjacent floodplains usually contain a large proportion of sand, even though the surface strata may consist of silt and clay; thus, the banks erode and cave with relative ease.

Most alluvial channels exhibit a natural instability which results in continuous shifting of the stream through the following phenomena:

- Erosion and deposition at bends.
- Formation and destruction of islands.
- Development of oxbow lakes.
- Formation of braided channel sections.

The degree of channel instability varies with the following:

- Hydrologic events.
- Bank and bed instability.
- Type and extent of vegetation on the banks.
- Floodplain use.
- Upstream changes in land use.

The engineer should identify these characteristics and understand the relationship of the actions and reactions of forces tending to effect change. This knowledge enables the engineer to estimate the rates of change and evaluate potential upstream and downstream effects of natural change and proposed local channel modifications.

The potential response of the stream to natural and proposed changes may be quantified with the basic principles of river mechanics. The engineer should understand and use these principles to minimize the potential effect of these dynamic systems on highways and the adverse effects of highways on stream systems.

B. Levels of Assessment. A qualitative assessment of the river response to proposed highway facilities is possible with 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 is suggested as a tool to quantify the expected scour and/or sedimentation of potential problem locations. *Highways in the River Environment* (FHWA, 1990) and *Users Manual for BRISTARS* (Molinas, 2000) should be consulted to evaluate the problem.

The natural stream channel will assume a geomorphological form that will be compatible with the sediment load and discharge history that 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.

The analysis and design of a stream channel usually will 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 follows.

1. 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.

2. 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 explained adequately. Data requirements include Level 1 data in addition to the information needed to establish the hydrology and hydraulics of the stream.

3. 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 analyses where losses and liability costs are high. This level of analysis may require professionals experienced with mathematical modeling techniques for sediment routing 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.

C. 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	Width variability	Degree of braiding
Valley setting	Bar development	Degree of anabranching - degree to which >flow is divided at normal and lower stages by large islands or, more rarely, by large bars; the width of individual islands or bars is greater than approximately three times the average bankfull water surface width (AASHTO, 2005)
Natural levees	Flow variability	
Sinuosity	Floodplains	
	Apparent incision	
	Channel boundaries	

Hydraulic Factors

Magnitude, frequency and duration of floods
Bed configuration
Resistance to flow
Water surface profiles

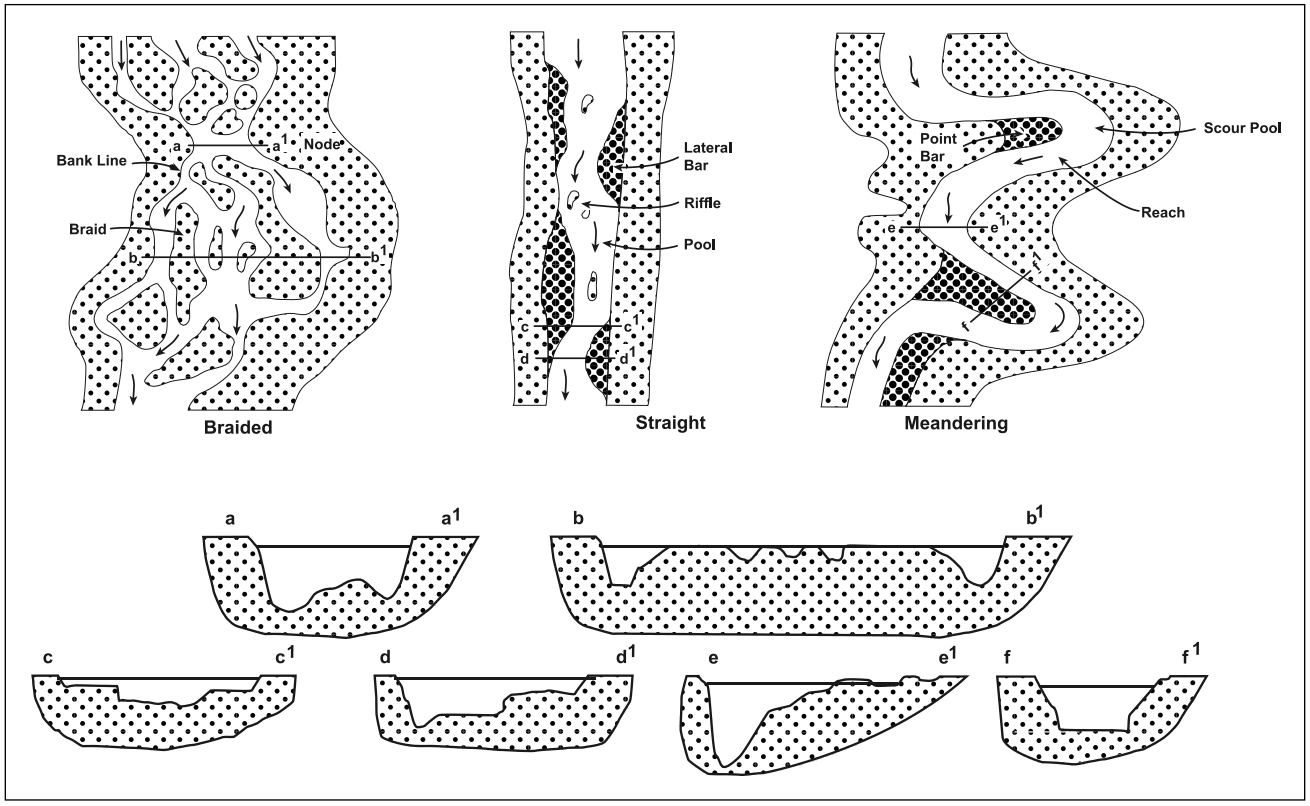
Rapid and unexpected changes may occur in streams in response to manmade activities in the watershed, such as alteration of vegetative cover. Changes in perviousness can alter the hydrology of a stream, sediment yield and channel geometry. Channelization, stream channel straightening, stream levees and dikes, bridges and culverts,

reservoirs and changes in land use can have major effects on stream flow, sediment transport, channel geometry, and location. Knowing that human activities can influence stream stability helps the designer to anticipate some of the problems that can occur.

Natural disturbances such as floods, drought, earthquakes, landslides, volcanoes and forest fires also can 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 also will occur to the stream channel.

D. Stream Classification. Figure 8.19 illustrates straight, meandering, and braided streams, the three main natural channel patterns.

Figure 8.19 Natural Stream Patterns



1. Straight Streams. A stream is classified as straight when the ratio of the length of the thalweg (path of deepest flow; see Figure 8.20(a)) to the length of the valley is less than 1.5. This ratio is known as the sinuosity of the stream. Degrees of sinuosity are illustrated in Figure 8.20(b).

Figure 8.20(a) Thalweg Location in Plan-View and Cross Section

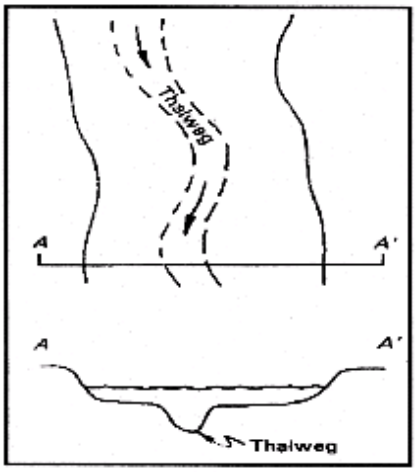
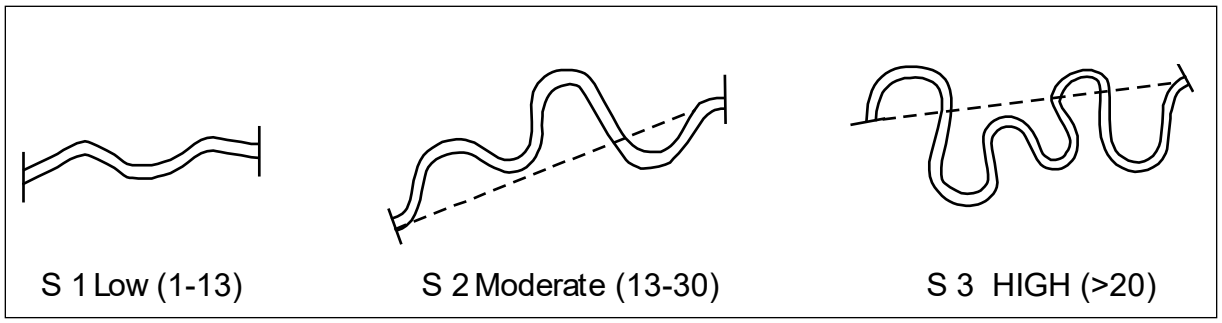


Figure 8.20(b) Various Degrees of Sinuosity

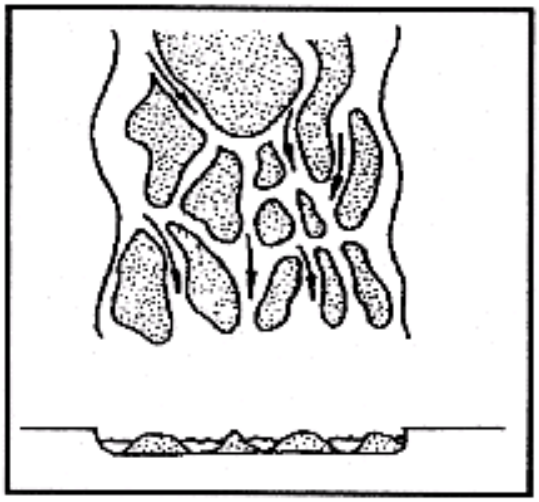


Straight channels are sinuous to the extent that the thalweg usually oscillates transversely within the low flow channel, and the current is deflected from one side to the other. The current oscillation usually results in the formation of pools on the outside of bends while lateral bars, resulting from deposition, form on the inside of the bends (Figure 8.19).

Straight reaches of alluvial channels may be only a temporary condition. Aerial photography and topographic maps may reveal former locations of the channel and potential directions of further movement.

2. Braided Streams. A braided stream is one that consists of multiple and interlaced channels (Figure 8.21). Braiding is caused by bank caving and by large quantities of bed load which the stream is unable to transport. The bed load, or contact load, in a stream is that portion of the total sediment discharge which moves along the bed by rolling or sliding and may, at times, be suspended by the flow. Deposition occurs when the supply of sediment exceeds the stream's transport capacity.

Figure 8.21 Plan-View and Cross Section of a Braided Stream

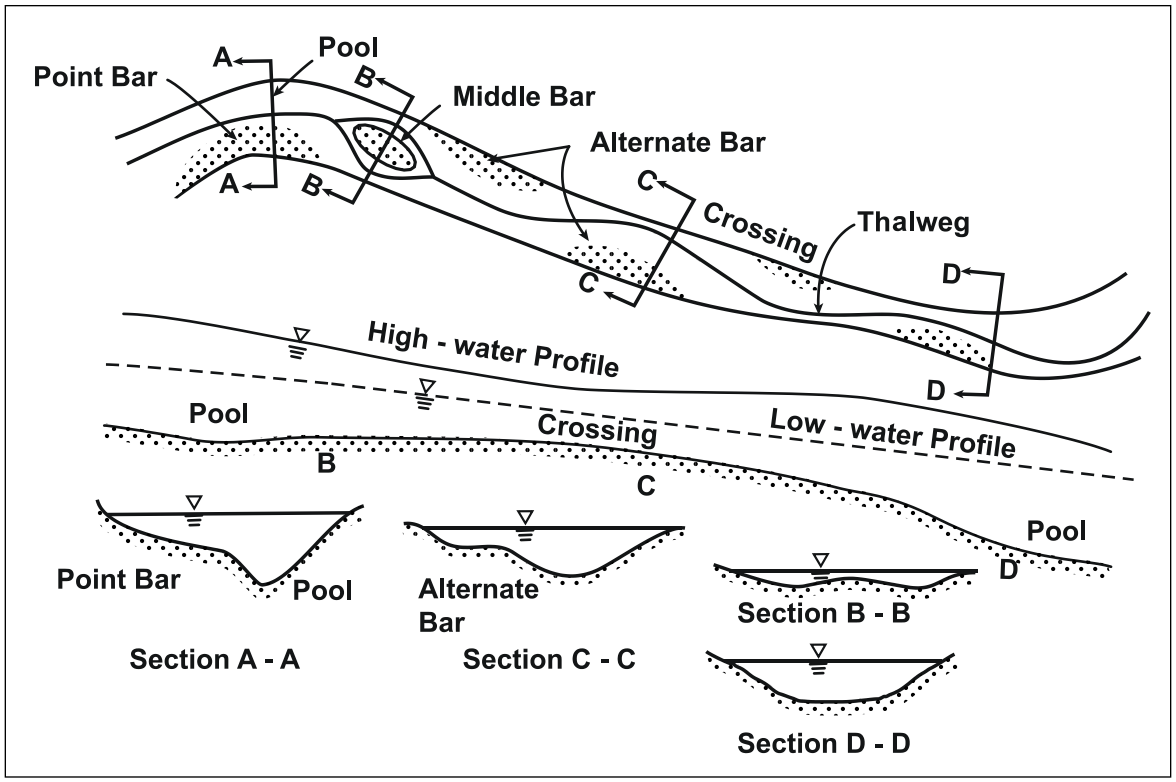


As the stream bed aggrades from deposition, the downstream channel reach develops a steeper slope, resulting in increased velocities. Multiple channels develop on the milder upstream slope as additional sediment is deposited within the main channel. These interlaced channels cause the overall channel system to widen, resulting in additional bank erosion. The eroded material may be deposited within the channel to form bars which may become stabilized islands. At flood stage, the flow may inundate most of the bars and islands, resulting in the complete destruction of some and changing the location of others. A braided stream is generally unpredictable and difficult to stabilize because it changes alignment rapidly, is subject to degradation and aggradation, and is very wide and shallow even at flood stage.

3. Meandering Streams. A meandering stream consists of alternating S-shaped bends (see Figure 8.19). In alluvial streams, the channel is subject to both lateral and longitudinal movement through the formation and destruction of bends.

Bends are formed by the process of erosion and sloughing of the banks on the outside of bends and by the corresponding deposition of bed load on the inside of bends to form point bars. The point bar constricts the bend and causes erosion in the bend to continue, accounting for the lateral and longitudinal migration of the meandering stream (Figure 8.22).

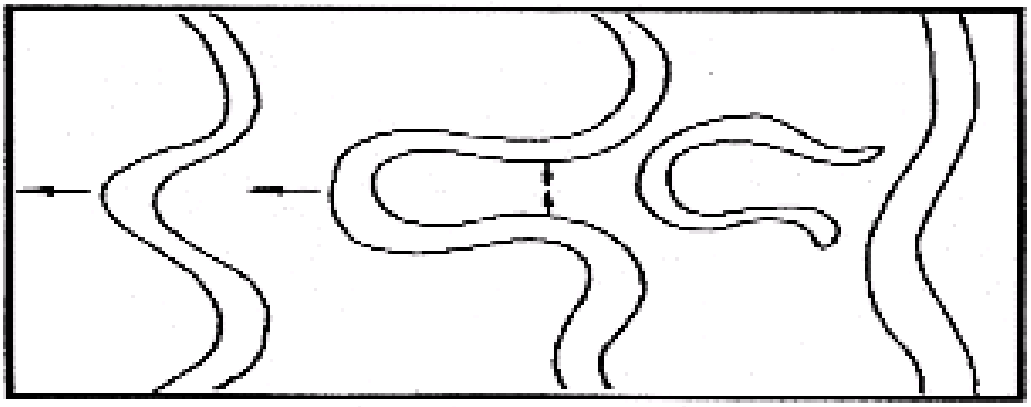
Figure 8.22 Plan-View and Cross Section of a Meandering Stream



As a meandering stream moves along the path of least resistance, the bends will move at unequal rates because of differences in the erodibility of the banks and floodplain. Bulbs form, which are ultimately cut off, resulting in oxbow lakes (see Figure 8.23). After a cutoff is formed, the stream gradient is steeper and the stream tends to adjust itself upstream and downstream, and a new bend may develop. Comparison of aerial photographs taken over a period of years is suggested for estimating the rate and direction of the meander movement. Local history also may help to quantify the rate of movement.

Modification of an alluvial channel from its natural meandering tendency into a straight alignment usually requires confinement within armored banks because they may be very unstable. Straightening meandering channels can result in steeper gradients, degradation, and bank caving upstream as the stream attempts to reestablish an equilibrium. The eroded material will be deposited downstream, resulting in reduced stream slopes, reduced sediment transport capacity, and possible braiding. However, a confined stream may become braided or degrade its bed due to the steeper gradient in the straight alignment. The braiding or degradation often extends beyond the limits of local armor protection. When an unprotected straight channel is constructed, the current will tend to oscillate transversely and initiate the formation of bends. Eventually, even protected straight channel reaches may be destroyed as a result of the natural migration of meanders upstream of the modified channel.

Figure 8.23 Migration Leading to Formation of Oxbow-Lake



4. Graded and Poised Streams. A graded stream has sufficient slope and energy to transport the material delivered to the stream and generally is stable. This definition applies to the average condition of the stream over a period of years. At any one time, there will be isolated locations where aggradation or degradation is occurring. The term "poised" applies to a stream that, over a period of time, is neither degrading nor aggrading.

Graded and poised streams are dynamically balanced, and any change altering that condition may lead to action by the stream to reestablish the balance. For example, if the channel gradient is increased, as occurs with a cutoff, the sediment transport capacity of the flow is increased and additional scouring results, thereby reducing the slope. The transport capacity of the downstream reach has not been altered; therefore, the additional bed load carried downstream will be deposited as a result of upstream scour.

As the aggradation progresses, the stream slope below the deposition is increased, and the transport capacity is adjusted to the extent required to carry the additional material through the entire reach. This process will continue until a new balance is achieved and the effect could extend to considerable distance before and after the cutoff.

E. Stream Modifications. The effects of stream modification on a natural stream vary greatly. Section 8.10 deals with analyzing the effects that proposed channel modifications may have on streams and the effects that the stream response may have on highway facilities. Except for certain minor maintenance activities, stream modifications require a Chapter 105 waterway permit.

1. Environmental Considerations. The potential environmental impacts and the possible need for stream impact mitigation measures should be a primary consideration for the designer. Unless it can be determined through comparisons that the highway improvement will permanently or significantly affect a stream, mitigation practices generally will not be warranted, but may be mandated by the regulatory agency.

Unfortunately, channel modifications often are viewed in a negative sense. The environmental team should recognize that channel modifications may be necessary and can provide environmental enhancement. Also, channel modifications that are compatible with the existing aquatic environment sometimes can be constructed at little or no extra cost.

Less aquatic habitat is available when a channel is shortened to accommodate highway construction. This not only decreases the aquatic biomass, but also reduces the amount of surface water available for recreation and sport fishing. The significance of this effect can be estimated by comparing the amount of surface water area, riparian and upland wetland area, and stream length that will be lost with the existing amount in the geographic area. If there will be a loss, particularly of wetlands, resource and regulatory agencies may raise objections in light of the national "no net loss" policy currently prevailing. In some instances, such habitat loss may be acceptable when combined with mitigation measures, but such measures should prevent habitat damage beyond the channel change limits.

Enhancement of the channel may be accomplished during stream reconstruction at little additional cost, and perhaps at less cost where reconstruction is essential to the needs of the highway project. It may even be possible to reconstruct the surface water resource in a manner that eliminates an existing environmental problem. This might include:

- Incorporating sinuosity into a straight stream reach.
- Relocating the channel to avoid contamination from minerals or other pollution sources.
- Adjusting the flow depth and width to better utilize low flows.
- Providing an irregular shaped channel section to encourage development of an overhanging bank.
- Improving the riparian vegetation.

The natural channel profile can be preserved in several ways. The most common practices are the following:

- Using a drop type grade control structure (check dam).
- Maintaining the existing channel slope.
- Increasing the channel length by constructing an artificial meander.

Culverts can provide another alternative similar to using drop structures. The culvert flowline slope can be increased to accommodate the elevation difference caused by foreshortening a channel. The increased erosion associated with steep culverts is localized at the outlet where erosion protection can be provided.

The cross sectional shape of the modified channel is very important. If it is relatively stable, the existing channel cross section should be simulated if it has low flow depths and velocities, and adequate minimum flow requirements. The cross sectional shape may be determined by hydraulically analyzing simple and easy-to-construct shapes that approximate the preferred natural channel geometry. The analysis generally compares the stage-discharge, stage-velocity, and stage-sediment relationships of the natural channel with the modified channel.

Stream relocations may impair water quality temporarily. The problem is primarily sediment-related, except for those rare instances where adverse minerals or chemicals are exposed, diverted, or intercepted. With a channel relocation, the new channel should be constructed in dry conditions wherever possible. Following completion, the downstream end should be opened first to allow a portion of the new channel to fill as much as possible. Next, the upstream end should be opened slowly to minimize erosion and damage to constructed habitat.

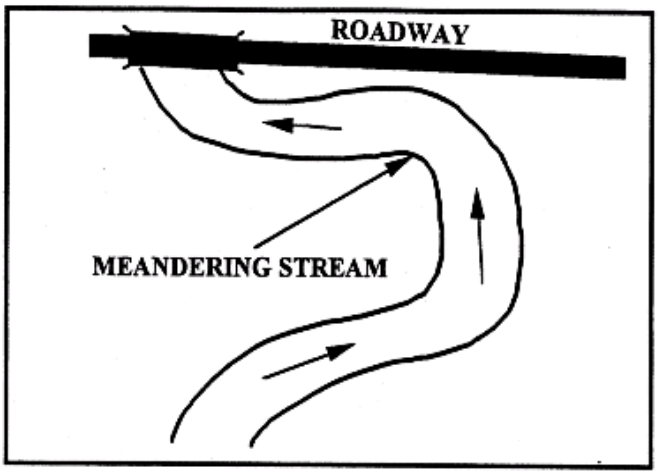
Where the channel relocation interferes with the existing channel, it may be desirable or required to construct rock and gravel dikes, or to use other filtering devices or commercially available dikes, to isolate the construction site, thereby limiting the amount of sediment entering the water.

F. Stream Meander Considerations. Many streams have a strong propensity to meander. The sinuosity of the main channel is a general characteristic of a stream and can vary with the discharge and the type of soil through which the stream passes. Meanders are formed by the erosive force of the stream water as it undercuts the main channel bank. The bank support is lost and material caves into the water to be re-deposited downstream. As the erosion on the outer bank of the meander migrates in a downstream direction, material from upstream deposits on the inside of the bend. This progression of stream meandering can have serious effects on highway crossings. Approach roadway embankment and bridge abutments often are threatened by this migration.

With reference to the example given in Figure 8.24, lengthening the bridge may not always be cost effective as a countermeasure to the damage potential from the meander. In this example, both the bridge and the approach roadway are threatened by the natural meandering course of the river. River training or some type of erosion control may be more effective and economical. Several measures and devices have been used successfully in Pennsylvania. For example, several spur dikes were designed to stabilize the Loyalsock Creek near Montoursville. The creek had meandered approximately 400 feet eastward, threatening to undermine the embankment of the Route 220 By-Pass. The spur dikes were designed to force the stream back to its original course. The system has been in place for more than a decade and is functioning as intended.

In order to protect the roadway from the threat of meanders, yet remain synchronous with nature, it is important to devise countermeasures which are environmentally sound, act naturally, are economically viable, and physically effective.

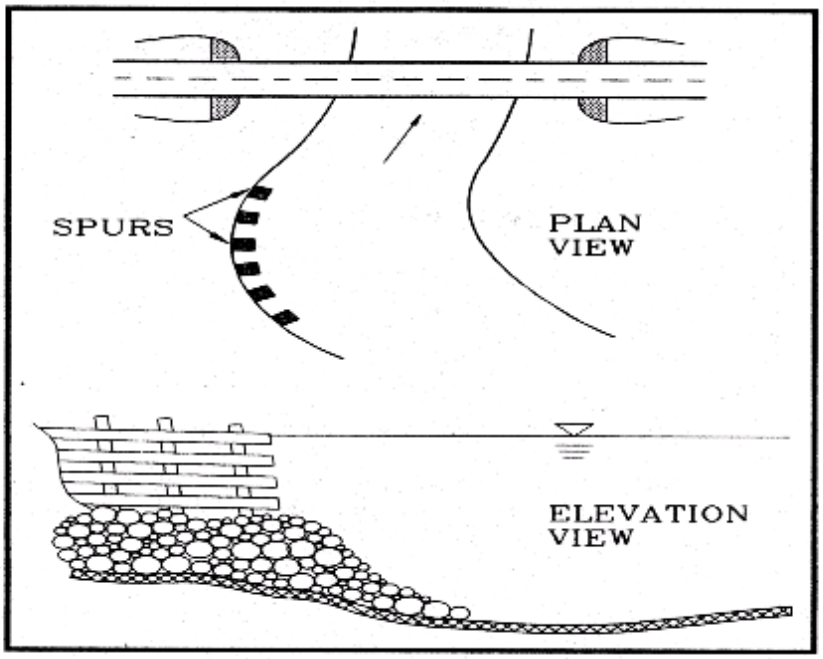
Figure 8.24 Meandering Stream Threatening Bridge and Approach Roadway



1. Meander Migration Countermeasures. Generally, it is good practice to locate bridge crossings on a relatively straight reach of stream between bends. When this is not practical, countermeasures such as spur or jetty type control structures may be needed (see Figure 8.25). These sometimes are referred to as linear structures, permeable or impermeable, projecting into the channel from the bank for some of the following purposes:

- Altering flow direction.
- Protecting the channel bank.
- Inducing deposition.
- Reducing flow velocity along the bank.

Figure 8.25 Permeable Fence Spurs as Meander Migration Countermeasures

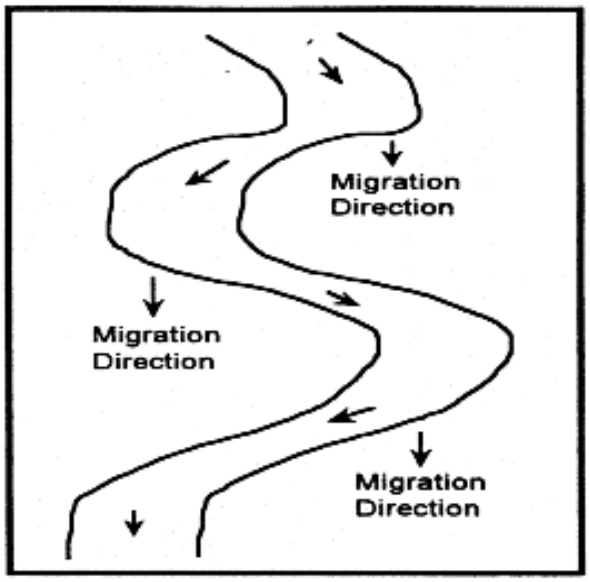


Control structures may or may not cause the typical cross section of flow in a meandering stream to become more symmetrical. For many locations, countermeasures may not be required for several years because of the time required for the bend to move to a location where it begins to threaten the highway facility. In other streams, however, bends may migrate at such a rate that countermeasures will be required after a few years or a few flood events. In such cases, the countermeasure may be installed during initial construction.

In some instances, stabilizing channel banks at a highway stream crossing can cause a change in the channel cross section, and may alter the stream sinuosity upstream of the stabilized banks. See Section 8.10 for more information on channel stabilization and bank protection.

Figure 8.26 illustrates meander migration in a natural stream. If sinuosity increases due to artificial stream stabilization, then meander amplitude may increase. Meander radii in other parts of the reach may become smaller and deposition may occur because of reduced slopes. The channel width-depth ratio may increase as a result of bank erosion and deposition. Ultimately, cutoffs can occur.

Figure 8.26 Meander Migration in a Natural Stream



For further guidance refer to:

- HEC-11, *Design of Riprap Revetment* (FHWA, 1989a).
- HEC-15, *Design of Roadside Channels with Flexible Linings* (FHWA, 1988).
- HEC-20, *Stream Stability at Highway Structures, Third Edition* (FHWA, 2001d).
- HEC-23, *Bridge Scour and Stream Instability Countermeasures* (FHWA, 2001a).
- HDS-6, *River Engineering for Highway Encroachments* (FHWA, 2001c).
- *Model Drainage Manual*, Chapter 17, *Bank Protection* (AASHTO, 2005).

8.10 THE EFFECTS OF CHANNEL ALTERATIONS

The effects of channel modification and the importance of the effects on a natural stream vary greatly. This section deals with analyzing the effects proposed channel modifications may have on streams and the effects the stream response may have on highway facilities. It is important to note that except for certain minor maintenance activities, channel alterations require some form of a Chapter 105 waterway permit.

It may be necessary to modify a stream in order to make it more compatible with the highway facility and the physical constraints imposed by local terrain or land use. The modifications may involve changes in alignment or conveyance. Changes may be necessary to accommodate the highway requirements, but they must be evaluated to assess short term and long term effects on the stream system.

An accumulation of background data on the existing stream should be available from previously completed planning and location studies, and a preliminary highway design should be available in sufficient detail to indicate the extent of required channel modifications.

Certain types of streams may have a very wide threshold of tolerance to change in alignment, grade and section. In contrast, small changes can cause significant impacts on sensitive recreational streams or other water resource values. An analysis of the tolerance to change may reveal that detrimental effects will not result from necessary changes. If detrimental effects are recognized, plans should be developed to mitigate the effect to within tolerable limits. Just as deliberate steps can be taken to avoid and mitigate adverse effects, positive actions can be taken to enhance certain aspects of an existing stream system, often to the economic benefit of the highway. For example, active upstream headcutting can be controlled with standard culverts or specially designed culverts so that many hectares of land along the stream banks will not be lost and the highway facility will be protected from the headcutting. Also, through

coordination and cooperation with fish and wildlife agency personnel, stabilization measures necessary to protect the highway may be adapted or modified to improve aquatic habitat (Figure 8.27).

Volume X of the *Highway Drainage Guidelines* (AASHTO, 2003) addresses the subject of highway impacts on surface water environments. The effects of specific structural facilities, such as bridges and culverts, also are covered in Volume IV and Volume VII.

A. Channel Realignment. Channel realignment may disrupt the natural system regime balance. The effect of these changes may be negligible or significant. The time effect relationship depends on the magnitude, duration and frequency of floods, fluvial geomorphology, and the nature or extent of the modification.

Channel realignment may involve changes in aquatic habitat by the removal of stream bottom materials which provide a diverse habitat for fish and substrate for aquatic insects. Reduction in stream side vegetation will sometimes affect water temperature and shelter as well as bank stability.

Figure 8.27 Stabilization Measures Adapted to Improve Aquatic Habitat



When dealing with channel modifications, the preferred procedure is:

- Establish the nature of the present regime (slope, section, meander pattern and stage-discharge relationship).
- Determine thresholds for changes in the various regime parameters.
- Duplicate the existing regime, where possible, but keep within the established thresholds for change, where duplication is not practical or possible.

Regime parameters which should be considered when stream modification is necessary are discussed in the following sections.

B. Slope Modification. Stream realignment occasionally may result in decreased channel slope but, more often, the modification will increase the channel slope. A localized increase in channel slope may introduce channel responses which are reflected for considerable distances upstream and downstream of the project.

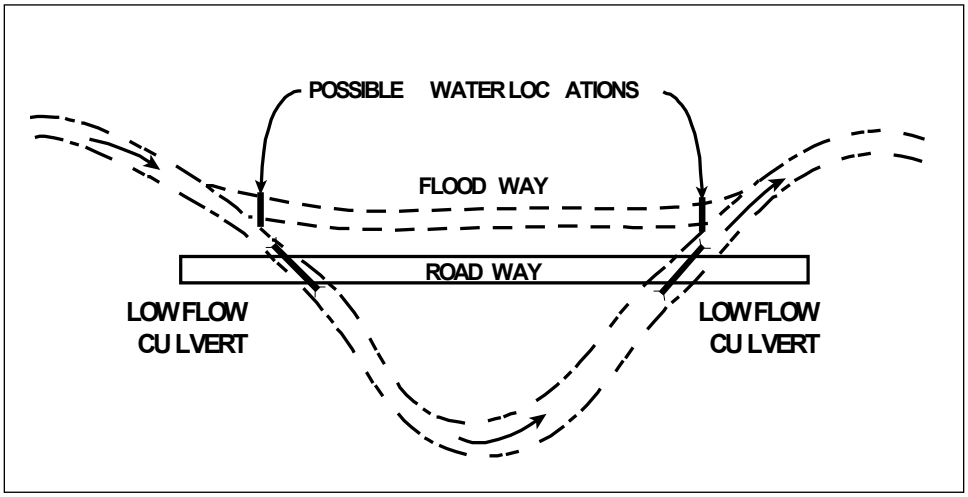
The stream response may be in the form of a regime change from a meandering to a braided channel, or sediment transport through the steepened reach may be increased sufficiently to cause degradation upstream of the realignment and aggradation downstream. Banks may become unstable and require structural stabilization measures to prevent erosion. Tributary channels entering the steepened main channel may be subject to headcutting with deposition occurring at or downstream of the confluence.

Grade control structures in the form of a series of weirs or chutes may be utilized to minimize increases in gradient provided there is some assurance that the normal meandering tendency of the channel will not bypass these structures in time. If topography permits, meanders may be employed to reduce stream gradient to existing or threshold levels. These meanders may require stabilization in order to assure continued effectiveness and stability. The type and extent of stabilization required are dependent upon bank materials and the velocity-discharge relationship. Again, as with grade control structures, a river system with migrating meanders can attack the protected meander reach from upstream of the protected limits.

C. Section Modification. Channel cross section characteristics are a major consideration where stream realignment is necessary. It is desirable to duplicate features such as shape, side slopes, bed material and roughness characteristics and to maintain roughly the same proportional flow in the channel and floodplains.

Complete preservation of the existing low flow conditions often is possible where realignment is desirable in order to eliminate multiple bridge or culvert structures. Figure 8.28 illustrates providing for flood flows in a relocated channel while maintaining the natural stream alignment for normal flow.

Figure 8.28 Flood Flow Channel Modifications

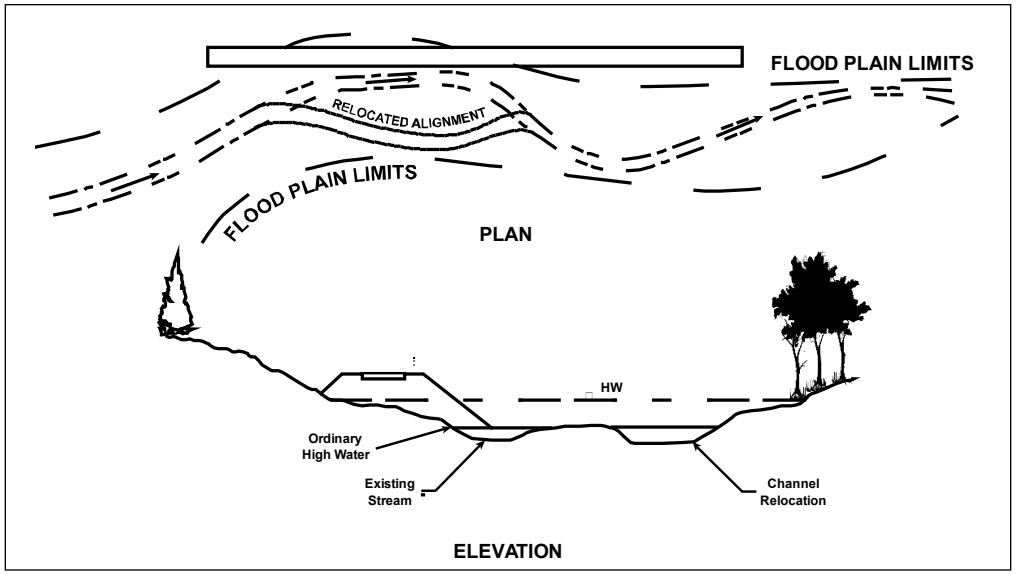


A diversion weir, appropriately designed, will divert the normal flow through the meander by way of culverts, thus eliminating the need for multiple bridge crossings. The flood channel may require some type of stabilization due to the steeper gradient and resulting scour potential; however, an economic analysis may indicate this method is less costly in initial construction and future maintenance than providing structures adequate to accommodate the full flood potential. In addition, a portion of the natural stream is preserved which otherwise would have been cut off.

Highway locations or modifications in certain terrain conditions may result in an encroachment such as is illustrated in Figure 8.29. This type of channel realignment may require a channel of sufficient section to convey both normal and flood flow within the banks formed by the roadway and the floodplain. If the low flow channel requires realignment, a pilot channel should be provided within the new channel. This pilot channel should approximate the existing channel characteristics of width, depth, gradient and bottom roughness. Where no pilot channel is provided, the average daily flow is likely to spread over a much wider section, and flow depth will be reduced in such a way

that water temperature, pool formation and sediment transport are adversely affected. These modifications may result in a braided channel condition and hamper the reestablishment of the natural aquatic environment.

Figure 8.29 Highway Encroachment on Natural Streams and Stream Relocation



When existing channels must be widened to provide for the design flood discharge, bench widening design should begin at the edge of stream 0.3 m (1 ft) (or more) above the stream bed whenever normal flow conditions permit.

If the normal flow stream depth is greater than 0.3 m (1 ft), then the District Office should make a suggestion as to the depth of bench as part of the Hydrologic and Hydraulic Report submission. The depth indicated should be such as to preclude extreme channel widening. All disturbed areas should be seeded. The application of this design procedure should reduce the problem of construction equipment encroaching into the natural channel. In addition, the existing channel acts as a sub-channel for the passage of fish during low flow periods. Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Figure 10.10.1 should be referenced to accomplish this.

Side slopes and bank cover should be considered where stream realignment is necessary. The criteria for establishing channel side slopes will depend upon the soil type, erosion potential, stream velocity, right-of-way limitations, stabilization measures to be employed and fishery considerations, if any. Streambanks should be constructed as nearly vertical as possible to reduce disturbance of fish habitat, if stability will not be a problem or if limited undercutting of the banks can be tolerated.

D. Conveyance Modification. Longitudinal encroachments on the floodplain and stream realignment usually will affect the conveyance of the channel reach. The channel capacity may be decreased or increased as a result of these changes. In addition to the modifications there also may be changes in the roughness characteristics of the bed, banks and floodplain caused by clearing and grubbing, enlarged channel section and channel lining. These changes will affect the stage-discharge relationship and may affect the flow regime and influence the balance of the stream ecosystem.

Clearing of vegetation along stream banks may remove root systems which contribute to bank stability. Clearing and grubbing reduces the bank and floodplain roughness, and contributes to higher velocities and increased erosion potential for those areas. The limited clearing of adjacent right-of-way involved with transverse encroachments or crossings usually will not cause significant effects on the conveyance of a channel. A water surface profile analysis is necessary to establish the stage-discharge relationship for channels with varying roughness characteristics across the channel. The single-section analysis method of estimating stage-discharge relationships can be subject to significant error if the typical section used does not represent the actual conditions upstream and downstream of the crossing site.

Channel enlargement or cleanout through a limited channel reach sometimes is proposed in an effort to provide additional stream capacity. If the stage of the stream at the proposed highway site is controlled by downstream conditions, there can be limited or possibly no benefits derived from localized modifications. Channel sections which are constricted or enlarged contrary to the geomorphic characteristics of the existing channel often will respond adversely to the change. This response may be a change in flow regime from meandering to a braided channel or conversion from a poised condition to an aggrading or degrading condition. *Highways in the River Environment* (FHWA, 1990) should be consulted regarding the evaluation of cross section changes for a given channel configuration and flow regime.

The use of bed and bank stabilization measures often will have a pronounced effect on channel conveyance. Channel linings usually consist of concrete paving, stone riprap, gabion mattress or sod. These measures normally are very costly and involve certain trade-offs in efficiency and economics. A paved channel may create such adverse problems as high velocity flow concentration at the terminus of the section, and loss of stream environment of fish habitat. Highway locations and designs which include the conversion of natural streams to paved channels are becoming unacceptable in many areas. PennDOT may encounter objections to permit applications for such work.

Channel slope modifications generally are not utilized to the same extent as other conveyance improvement techniques; however, the removal of dams and grade control structures and meander bend cutoffs usually will result in an increase in stream slope, and the potential stream response to these changes should be evaluated.

8.11 CHANNEL STABILIZATION AND BANK PROTECTION

Highway embankments constructed within a floodplain may require stabilization to resist erosion during flood events. Embankment stabilization may be designed and constructed with the initial roadway project where the need is obvious or the risk of damage is high. Installation of embankment stabilization can be delayed until a problem actually develops where economic considerations and the availability of materials are significant constraints and the probability of damage is low.

Highway channel stabilization measures are usually local in nature and are designed primarily to protect the highway facility from attack by a shifting channel or where the floodplain adjacent to the facility is highly erodible.

For additional guidance on bank stabilization, refer to Chapter 17, *Bank Protection*, of the Model Drainage Manual (AASHTO, 2005).

A. Stabilization Considerations. If a highway location adjacent to a stream cannot be avoided, alternative protective measures should be evaluated to determine the measure best suited to the situation. Alternatives may include channel change, roadway embankment protection, stream bank stabilization and stream training works.

Channel stabilization should be considered only when economically justified and one or more of the following basic purposes will be accomplished:

- Prevent loss or damage of the highway facility and associated improvement.
- Prevent loss or damage of adjacent property and structures.
- Reduce maintenance requirements.
- Achieve secondary benefits such as beautification, recreation and the preservation or establishment of fish and wildlife habitat.

Localized stabilization measures may not be successful if located within long reaches of unstable channel. Spot stabilization often will result in high maintenance costs and repetitive reconstruction. However, if bank erosion occurs only at isolated locations, stabilization measures at these locations probably will afford an economical solution even though a period of repetitive maintenance may follow.

Stream response to local stabilization may be a change in flow regime, or the stream may attack the unprotected bed or opposite bank. The probability of these occurrences should be considered in designing necessary stabilization measures.

B. Selection of Protective Measures. The selection of a type of appropriate measure for use at a site depends on many factors, but perhaps foremost is experience with similar facilities in existing locations. In drawing conclusions from other stream stabilization projects, the effects of differing characteristics at each site should be considered. Factors such as slope, grain size of bed load, bed and bank material, mean flood and low water discharges must be taken into account. Interviews with maintenance personnel familiar with the areas being compared will provide valuable supplemental data for use in design.

Relative costs and durability of available materials, as well as the difficulty of construction and anticipated maintenance requirements, are additional considerations in the selection process.

C. Revetments. Revetments are continuous type structures generally placed longitudinally along the stream banks or highway embankment to protect against destruction or damage by stream currents and flood flows. Revetments are generally, though not exclusively, located on the outside bank of bends where bank recession or erosion is most active as a result of impinging flow. They may be required elsewhere to protect an embankment from wave wash or flood attack. Because of conditions affecting construction, the types of materials available and differences in the duration and intensity of attack, the segment of revetment placed above the annual flood elevation may be of different design than the segment located below that elevation. The higher segment is termed upper bank protection and the lower segment, sub-aqueous protection. Both are required to prevent bank recession, and the upper bank protection may be extended to a sufficient height to protect against wave action. For smaller streams and rivers, the upper and subaqueous protections are usually of essentially the same design and are placed in a single operation.

Grading and shaping of banks on which revetments are to be placed are important. The area should be graded to slopes that will be stable when saturated and an adequate filter system incorporated to prevent loss of bank material through the protective revetment. The type of filter system used is dependent upon slope stability, bank material, the type of revetment and available filter materials.

Numerous materials such as dumped rock, portland cement concrete, sacked sand cement, soil cement, gabions and precast blocks have been used for bank protection. Filter materials may consist of sand, gravel, woven or nonwoven synthetic filter cloth, or combinations of these. Figures 8.30 through 8.32 are examples of bank protection.

Detailed descriptions and commentary on the various types of materials and placement techniques are beyond the scope of this Chapter; HEC-23, *Bridge Scour and Stream Instability Countermeasures* (FHWA, 2001a), HEC-11, *Design of Riprap Revetment* (FHWA, 1989a), HEC-15, *Design of Roadside Channels with Flexible Linings* (FHWA, 1988), and *Bank and Shore Protection in California Highway Practice* (California Division of Highways, 1960) are suggested for consideration and use by the reader.

Figure 8.30 Gabions Used for Bank Protection



Figure 8.31 Gabions Used as a Revetment

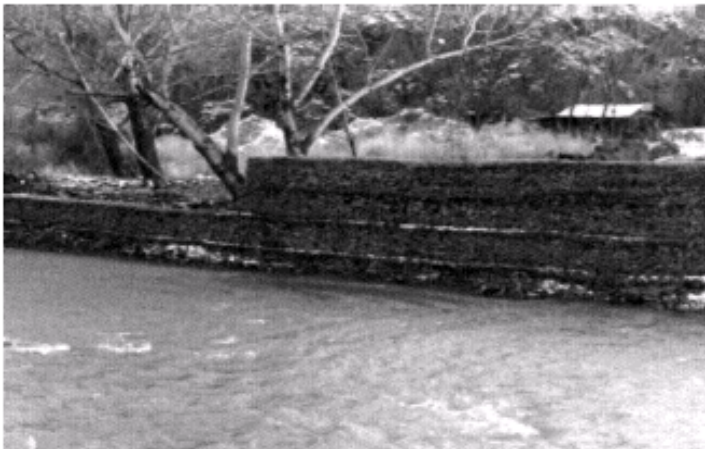
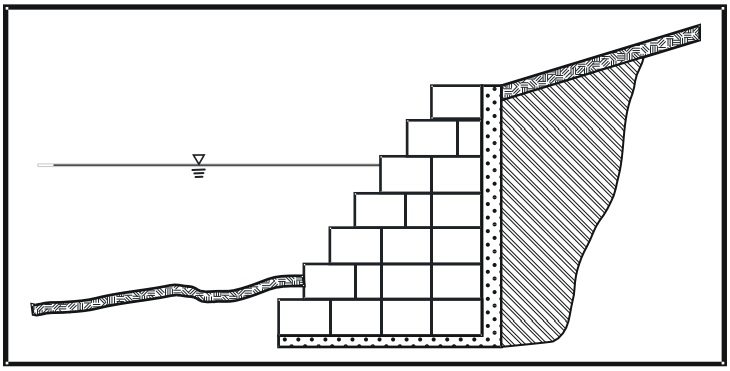


Figure 8.32 Gabions Used as a Revetment



8.12 SUBSURFACE INVESTIGATIONS

Constructed open channels designed for major streams or roadside ditches must incorporate provisions to protect the facility against scour, undercutting and erosion of the bed and banks. A thorough subsurface investigation by geologists or foundation engineers often is required to determine the type and degree of protection required for major stream channels. The scope of such an investigation will be commensurate with the importance and cost of the facility and with the potential risk associated with the channel.

Design requirements and alternatives for such features as side slopes, type of lining and filter requirements will depend upon information obtained from the subsurface investigation. These data are essential to the completion of final contract plans and specifications.

A. Reinforcement for Rigid Linings. Large channels requiring rigid lining usually are designed with structural reinforcement, but smaller concrete channels are not. The design of structural reinforcement for most rigid linings depends upon consideration of foundation conditions, groundwater conditions, shape and slope of the channel, provisions for maintenance access, cost to repair or replace, and flow conditions (hydrostatic and momentum forces).

Reinforcement for small drainage channels usually consists of some combination of welded wire mesh, reinforcing steel and load transfer dowels at joints. These types of reinforcement are normally sufficient to protect rigid linings against total destruction from cracking due to temperature changes, scour and undercutting along the channel sides and will maintain the channel structure integrity until proper maintenance repairs can be accomplished. Reinforcement also should extend into the anchor lugs and cutoff walls to prevent them from breaking under stress.

B. Buoyancy and Heave. Buoyancy becomes a problem when rigid channels are constructed in saturated soil or below the normal groundwater table. Provisions should be incorporated in the design to relieve hydrostatic pressures which tend to crack or float the lining. Otherwise, uplift pressures must be resisted by the empty weight of the channel lining, and additional lining thickness and reinforcement within tension zones may be required.

Weep holes often are constructed in channels which carry subcritical flow but are not suggested where supercritical flow is expected because negative pressures will leach out bedding materials. An underdrain system may be employed where supercritical flow channels require protection from hydrostatic pressure.

Frost heave can best be prevented with a free draining subbase or underdrain system. Additional lining thickness and/or reinforcement may be required to resist cracking where frost heave is expected.

C. Seepage Control. Riprap-lined channels often require the installation of some type of filter beneath the riprap to prevent embankment and bedding material from being withdrawn through voids in the slope protection (leaching). Loss of this underlying material will result in uncontrolled settlement of the riprap and subject the lining to attack by currents and high velocity flow. The use of geotextile is becoming common, and is currently indicated in the construction specifications. The FHWA has published numerous reports which are excellent references for design and construction data (HEC-11, HEC-15).

8.13 CONSTRUCTION RELATED HYDRAULIC CONSIDERATIONS

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

An individual well versed in the design should be present at the pre-bid and preconstruction conferences to explain special features and planned construction phasing where these considerations are necessary to 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.

Immediately prior to the commencement of construction of bank stabilization measures, the designer should inspect the site for bank movement that may have occurred prior to completion of the surveys. If evidence of bank movement exists, the design may need to be modified.

The designer should be consulted when field revisions to the design are necessary. It is important to remember that a Chapter 105 permit is usually required for these types of activities, and construction must be done consistent with the terms and conditions of the permit. Revisions to plans in the approved permit must be coordinated with the applicable resource agency(ies).

The post-construction inspection following completion of the project should document any deviations from the original plans as well as an initial assessment of the hydraulic performance.

Construction personnel should be encouraged to inform the designer of any difficulties which are encountered and to make suggestions to improve future designs.

8.14 MAINTENANCE RELATED HYDRAULIC CONSIDERATIONS

Stream channels and roadside ditches should be designed recognizing that periodic maintenance, inspection, and repair may be required. Where possible, provisions should be incorporated for access by maintenance personnel and equipment. Consideration should be given to the size and type of equipment that will be used in assessing the need for permanent or temporary access easements, entrance ramps, and gates through right of way fences.

A. Maintenance during Contract Period. Channel work on some projects may be completed several months before total project completion. The time between completion of channel work is usually longer when grading and

structures contracts are separate from the contract for paving. During this period, vegetative erosion control measures are not well established, and maintenance to correct erosion and sediment deposition in the newly constructed channels is important to achieving the results intended. PennDOT's construction contracts require maintenance by the contractors during the term of the contract and require interim protective measures to ensure that minor damage will not develop into major damage which will require costly repairs or replacement.

B. Hydraulic Related Maintenance Considerations. Damaged channels can be both expensive to repair and hazardous to traffic. A comprehensive program of channel maintenance should include periodic inspections and routine repair of these facilities and extraordinary inspections and repairs following major floods. Conditions which appear to require extensive repair or reconstruction or frequently recurring maintenance should be referred to the designer for evaluation.

The growth of weeds, brush and trees in a channel may reduce the conveyance well below design values. The channel also may reshape and realign itself in response to natural or induced morphological changes. For this reason, a channel must not be regraded simply to maintain the as built geometry. In many instances, the regrading effort will prove expensive and fruitless since the channel will only revert to a more natural shape and alignment. Major channel reconstruction should be undertaken only when the designer determines that extensive reconstruction is necessary to repair damages or increase the hydraulic capacity of the channel. This does not preclude maintenance forces from accomplishing channel cleaning and minor erosion repair.

An ideal maintenance program will include a procedure for reporting the effectiveness and efficiency of channel designs. This information helps in evaluating design procedures and practices and will supplement the survey data collected for the analyses and design of future projects.

8.15 CHAPTER 8 NOMENCLATURE

Symbol	Definition	Units
A	Flow cross sectional area	m^2 (ft^2)
A_i	Flow area of subsection i	m^2 (ft^2)
A_t	Total flow area of cross section	m^2 (ft^2)
α	Kinetic energy coefficient	dimensionless
E	Specific energy head	m (ft)
g	Gravitational acceleration	m/s^2 (ft/s^2)
H	Total energy head	m (ft)
H_c	Contraction losses	m (ft)
H_e	Expansion losses	m (ft)
h_f	Friction head loss from upstream to downstream	m (ft)
$_i$	Subsection	dimensionless
K	Conveyance	m^3/s (cfs)
K_{ave}	Average conveyance	m^3/s (cfs)
K_{as}	Conveyance downstream	m^3/s (cfs)
K_c	Contraction coefficient	dimensionless
K_e	Expansion coefficient	dimensionless
K_i	Conveyance in subsection i	m^3/s (cfs)
K_t	Total conveyance for cross section	m^3/s (cfs)
K_{vs}	Conveyance upstream	m^3/s (cfs)
n	Manning roughness coefficient	dimensionless
n_i	Subsection Manning's n-value	dimensionless
n_w	Weighted Manning's n-values	dimensionless
R	Hydraulic radius	m (ft) = A / P
P	Wetted perimeter of flow	m (ft)
P	Pressure	N/m^2 (lb/ft^2)
P_i	Subsection wetted perimeter	m (ft)
q	Discharge per unit width	m^3/s-m (ft^3/s-ft)
q_i	Discharge in subsection i	m^3/s (ft^3/s)
Q	Volumetric flow rate	m^3/s (cfs)

S	Slope of the energy gradeline	m/m (ft/ft)
S_{ave}	Average slope	m/m (ft/ft)
S_f	Friction slope	m/m (ft/ft)
S_{fave}	Average friction slope	m/m (ft/ft)
T	Top width at water surface	m (ft)
T_c	Water surface width for critical flow	m (ft)
τ_d	Maximum shear stress on channel bottom and channel side	N/m^2 (lb/ft^2)
T_w	Tailwater	m (ft)
v	Average velocity of flow (mean xsection velocity)	m/s (ft/s)
$v^2/2g$	Average velocity head	h_v, m (ft)
v_i	Average velocity in subsection	m/s (ft/s)
WS	Water-surface elevation or stage	m (ft) = z + y
x	Manning's Equation Constant	dimensionless
y	Depth of flow	m (ft)
y_c	Critical depth	m (ft)
y_o	Normal depth	m (ft)
γ_w	Unit weight of water, 9810 (62.4)	N/m^3 (lb/ft^3)
z	Elevation head	m (ft)

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CHAPTER 9

CULVERTS

9.0 INTRODUCTION TO CULVERTS

A. General - Culverts. A culvert conveys surface water through a roadway embankment or away from the highway right-of-way. In addition to this hydraulic function, it also must carry construction traffic, highway traffic, and earth loads; therefore, culvert design involves both hydraulic and structural design. The hydraulic and structural designs must be such that risks to traffic, property damage, and failure from floods are consistent with good engineering practice and economics. This chapter describes the hydraulic aspects of culvert design, construction and operation of culverts, and makes references to structural aspects only when they are related to the hydraulic design.

Culverts, as distinguished from bridges, usually are covered with embankment and are composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed or concrete riprap channel serving as the bottom. For economic and hydraulic efficiency, culverts should be designed to operate with the inlet submerged during flood flows, if conditions permit. Bridges, on the other hand, are not designed to take advantage of submergence to increase hydraulic capacity even though some are designed to be inundated under flood conditions.

At many locations, either a bridge or a culvert will fulfill both the structural and hydraulic requirements for the stream crossing. The designer should choose the appropriate structure based on the following (not necessarily in order):

- Construction and maintenance costs.
- Risk of failure.
- Risk of property damage.
- Traffic safety.
- Environmental and aesthetic considerations.
- Construction expedience.

For project sites where a the normal clear span is 24 feet or less and a box culvert is proposed to replace an existing short-span bridge, the *Joint Agency Guidance for The Analysis of Environmental Impacts and Other Issues for Short Span Structures* should be used. The guidance document is located in Appendix 9A. The document provides an overview of the various alternatives related to structure replacements and their associated construction and environmental impacts and other issues associated with construction of a box culvert versus bottomless structures, including bridges, rigid frames, and arches. This guidance is intended to form a basis for the development of an alternatives analysis suitable for submission with a permit application to the Department of Environmental Protection (PA DEP) to satisfy 25 PA Codes 105.13 and 93.4c.

Culverts are considered minor structures as compared with bridges, but they are of great importance for drainage and the integrity of the facility. Although the cost of individual culverts usually is relatively small, the total cost of culvert construction constitutes a substantial share of the total cost of highway construction. Similarly, culvert maintenance may account for a large share of the total cost of maintaining highway hydraulic features. The designer can achieve improved traffic service and reduced cost by judicious choice of design criteria and careful attention to the hydraulic design of each culvert.

The designer must consider analysis of the following items before starting the culvert design process:

- Site and roadway data.
- Design parameters, including shape, material and orientation.
- Hydrology (flood magnitude versus frequency relation).
- Channel analysis (stage versus discharge relation).

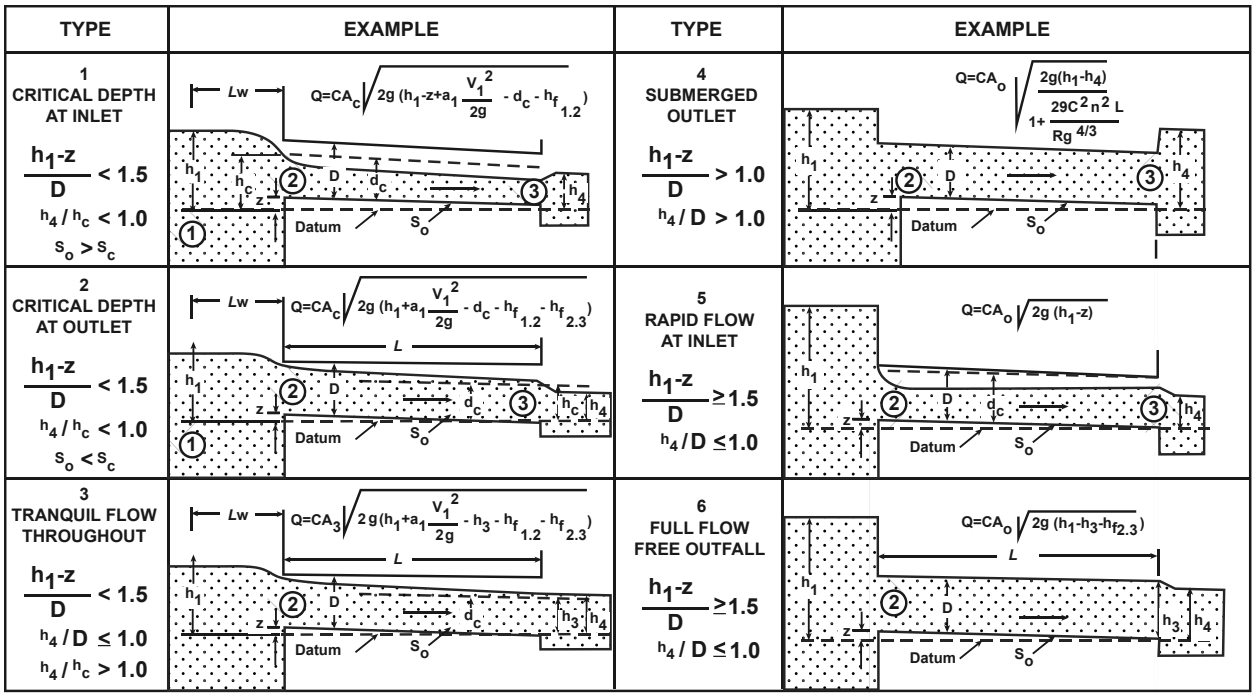
B. Concepts. Important terms utilized in culvert design are defined below.

1. 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 channel cross section geometry there is only one critical depth.

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

3. Flow Type. The USGS (1988) has established six culvert flow types which assist in determining the flow conditions at a particular culvert site. Figure 9.1 illustrates the six flow conditions.

Figure 9.1 USGS Flow Types (USGS, 1988)



4. Free Outlet. A free outlet has a tailwater equal to or lower than critical depth in the culvert. For culverts having free outlets, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

5. 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- or slope-tapered (walls or bottom tapered).

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

7. Normal Flow. Normal flow occurs in a channel reach when the discharge, velocity and depth of flow along a streamline do not change with longitudinal position. The water surface and channel bottom will be parallel. This type of flow can exist in a culvert operating on a constant slope provided the culvert is sufficiently long.

8. Slope:

- A steep slope occurs when critical depth is greater than normal depth.
- A mild slope occurs when critical depth is less than normal depth.

9. Submerged:

- A submerged outlet occurs when the tailwater elevation is higher than the crown of the culvert outfall.
- A submerged inlet occurs when the headwater submerges the inlet. This usually occurs when the headwater is greater than 1.2D, where D is the culvert diameter or barrel height.

C. General Culvert Considerations. A culvert is a structure that is usually a closed conduit or waterway that may be designed hydraulically to take advantage of submergence to increase hydraulic capacity. Culverts are constructed from a variety of materials and are available in different shapes and configurations. Culvert type selection includes the choice of material, shape and cross section, and the number of culvert barrels. Total culvert cost can vary considerably depending upon the culvert type selected. Roadway profile, terrain, foundation condition, fish passage requirements, shape of the existing channel, allowable headwater, channel characteristics, flood damage evaluations, construction and maintenance costs, and service life are some of the factors which influence culvert type selection.

1. Shapes. The shape of a culvert is not the most important consideration at most sites, so far as hydraulic performance is concerned. Rectangular, arch or circular shapes of equal hydraulic capacity generally are satisfactory. It often is necessary, however, for the culvert to have a low profile to accommodate terrain or limited fill height. Construction cost, the potential for clogging by debris, limitations on headwater elevation, fill height, and the hydraulic performance of the design alternatives enter into the selection of the culvert shape. Design and construction specifications and methods of determining maximum cover for some shapes and materials are included in PennDOT's Design Manual series.

Numerous cross sectional shapes are available. The most commonly used shapes include circular, box (rectangular), elliptical, pipe-arch, and arch. Shape selection is based on the cost of construction, upstream water surface elevation, roadway embankment height, and hydraulic performance, including limitations. Several commonly used culvert shapes are discussed in the following sections.

a. Circular. The most commonly used culvert shape is circular. This shape is preferred due to the available structural options for various fill heights. Various standard lengths of circular pipe in standard strength classes are available from local suppliers at reasonable cost. The need for cast-in-place construction generally is limited to culvert end treatments and appurtenances such as access holes.

b. Box or Rectangular. A culvert of rectangular cross section can be designed to pass large floods and to fit nearly any site condition. A rectangular culvert lends itself more readily than other shapes to low allowable headwater situations, since the height may be decreased and the total span increased to satisfy the requirement. The required total span can consist of one or multiple cells. Precast concrete and metal box sections often are used to overcome the increased construction time required for cast-in-place boxes.

c. Pipe Arch and Elliptical. Pipe arch and elliptical shapes are used in lieu of circular pipes where there is limited cover or overfill. Structural strength characteristics usually limit the height of fill over these shapes except when the major axis of the elliptical shape is laid in the vertical plane. When compared to circular sections, these shapes are more expensive for equal hydraulic capacity because of the additional structural material required.

d. Arches. Arch culverts are used in locations where vertical clearance above a waterway is a desirable feature, and where foundations are adequate for structural support. Such structures can be installed to maintain the natural stream bottom for fish passage, but the potential for failure from scour must be carefully evaluated. Structural plate metal arches are limited to use in low cover situations but have the advantage of rapid construction and low transportation and handling costs. This is especially advantageous in remote areas and in rugged terrain. Additionally, precast concrete arch systems can be fabricated in a variety of span lengths, and can be considered instead of short bridges in some instances.

e. Multiple Barrels. Culverts consisting of more than one barrel are useful in wide channels where the constriction or concentration of flow is to be kept to a minimum. A low roadway embankment offering limited cover may require the use of a series of small openings. The barrels may be separated by a considerable distance in order to maintain flood flow distribution. The practice of altering channel geometry to accommodate a wide culvert generally will result in deposition in the widened channel and in

the culvert. Where overbank flood flows occur, relief culverts with inverts at the floodplain elevation should be considered to avoid the need for channel alteration.

In the case of box culverts, it may be more economical to use multiple structures rather than a wide single span. In some locations, multiple barrels have a tendency to catch debris which clogs the waterway. They are also susceptible to ice jams and the deposition of silt in one or more barrels. Alignment of the culvert face normal to the approach flow and installation of debris control structures can help to alleviate these problems. To avoid widening the natural channel, to provide overflow (flood) relief, to support environmental preservation, and to reduce sedimentation and debris problems, it is good practice to design one barrel of the multiple-barrel system to carry low flows.

2. Material. The selection of the material for a culvert is dependent upon several factors that can vary considerably depending on location; therefore, consideration should be given to the following variables:

- Structural strength, considering fill height, loading conditions, and foundation conditions.
- Hydraulic efficiency, considering inlet geometry, Manning's roughness, cross section area, shape, and tailwater.
- Installation, local construction practices, availability of pipe bedding material, and joint tightness requirements.
- Durability, considering water and soil environment (pH and resistivity), corrosion (metallic coating selection), and abrasion.
- Cost, considering availability of materials.

Commonly used culvert materials include:

- Concrete (reinforced and non-reinforced).
- Steel (smooth and corrugated).
- Aluminum (smooth and corrugated).
- Plastic (smooth and corrugated).

The most economical culvert is one which has the lowest total annual cost over the design life of the project; however, the initial cost should not be the only basis for culvert material selection. Replacement costs and traffic delay are usually the primary factors in selecting a material that has a long service life. If two or more culvert materials are equally acceptable for use at a site, including hydraulic performance and annual costs for a given life expectancy, consideration should be given to bidding the materials as alternates.

3. Inlets. A multitude of different inlet configurations is utilized on culvert barrels. These include both prefabricated and cast-in-place installations. Commonly used inlet configurations include projecting culvert barrels, cast-in-place concrete head walls, pre-cast or prefabricated end sections, and culvert ends mitered to conform to the fill slope. Structural stability, aesthetics, erosion control, and fill retention are considerations in the selection of various inlet configurations.

The hydraulic capacity of a culvert may be improved by appropriate inlet selection. Since the natural channel is often wider than the culvert barrel, the culvert inlet edge represents a flow contraction and may be the primary flow control. The provision of a more gradual flow transition will lessen the energy loss and thus create a more hydraulically efficient inlet condition. Beveled edges are more efficient than square edges; side-tapered and slope-tapered inlets, commonly referred to as improved inlets, further reduce head loss due to flow contraction. Depressed inlets, such as slope-tapered inlets, increase the effective head on the flow control section, thereby increasing the culvert efficiency.

D. Economics. The hydraulic design of a culvert installation always includes an economic evaluation. A wide spectrum of flood flows with associated probabilities will occur at the culvert site during its service life. The benefits of constructing a large capacity culvert to accommodate all of these events with no detrimental flooding effects are normally outweighed by the initial construction costs. Thus, an economic analysis of the trade-offs may be performed with varying degrees of effort and thoroughness depending on the project scope.

- Benefits and costs. The purpose of a highway culvert is to convey water through a roadway embankment. The major benefits of the culvert are decreased traffic interruption time due to roadway flooding and increased driving safety. The major costs are associated with the construction of the roadway embankment and the culvert itself. Maintenance of the facility and flood damage potential also may be factored into the cost analysis.
- Analysis. Traditional economic evaluations for minor stream crossings have been somewhat simplistic. Culvert design flows are based on the importance of the roadway being served with little attention given to other economic and site factors. Inundation of the travelway dictates the level of traffic service provided by a waterway facility. The travelway overtopping flood level identifies the upper limit of serviceability, and it provides one of the important definitions of the term design flood. The minimum magnitudes of design floods are given in Chapter 7, *Hydrology*.
- Regulatory requirements also may apply. These may include:

  25 PA Code § 102 Erosion and Sediment Control
  25 PA Code § 105 Dam Safety and Waterway Management
  25 PA Code § 106 Floodplain Management
  25 PA Code § 111 Stormwater Management
  FAPG 23 CFR 650.115 (a)(1)(ii) Risk Assessment

9.1 DESIGN CONSIDERATIONS

The hydraulic design of a culvert consists of an analysis of the performance of the culvert in conveying flow from one side of the roadway to the other. To meet this conveyance function adequately, the design must include consideration of the variables discussed in the following sections.

A. Design Frequency. Choose the appropriate design storm frequency according to Section 7.2 and Sections 10.2.C, 10.3.C, 10.6.E and Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Table 10.6.1.

B. Headwater. Energy is required to accelerate flow through a culvert. This energy comes from an increase in depth versus the natural unobstructed water surface elevation on the upstream side of the culvert. The depth of the upstream water surface measured from the invert at the culvert entrance generally is referred to as headwater depth.

The headwater of a culvert is a function of several parameters, including the culvert geometric configuration. The culvert geometric configuration is based primarily on the allowable headwater. This geometric configuration consists of the following:

- The number of barrels.
- Barrel dimensions.
- Length.
- Slope.
- Entrance characteristics.
- Barrel roughness characteristics.

Potential damage to adjacent property or inconvenience to property owners should be of primary concern in the design of all culverts. In urban areas, the potential for damage to adjacent property is greater because of the high number and value of properties that can be affected. If roadway embankments are low, flooding of the roadway and delay to traffic are usually of primary concern, especially on highly traveled routes.

Culvert installations under high fills may present the designer an opportunity for use of a high headwater or ponding to attenuate flood peaks. If deep ponding is considered, the possibility of catastrophic failure (see DEP 25 PA Code §105) should be investigated because a breach in the highway fill could be quite similar to a dam failure.

The study of culvert headwater should include verification that watershed divides are higher than design headwater elevations. If the divides are not sufficiently high to contain the headwater, culverts of lesser depths or earthen training dikes may be used, in some instances, to avoid diversion across drainage divides. In flat terrain, drainage

divides often are undefined or nonexistent, and culverts should be located and designed for the least disruption of the existing flow distribution.

C. Site Data. For purposes of this section, site information from any source is broadly classified as survey data. The survey should provide the designer with sufficient data for locating the culvert and identifying information on all features which may be affected by installation of the culvert. Examples of such information include elevations and locations of houses, commercial buildings, croplands, roadways and utilities.

Sources of data include aerial or field survey; interviews; water resource, fish and wildlife, and planning agencies; newspapers; and floodplain zoning studies. Complete and accurate survey information is necessary to design a culvert to best serve the requirements of a site. The individual in charge of the drainage survey should have a general knowledge of drainage design and coordinate the data collection with the hydraulics engineer. The amount of survey data gathered should be commensurate with the importance and cost of the proposed structure.

The extent of survey coverage required for culvert design varies with location. In areas with relatively flat slopes, backwater effects may propagate a considerable distance upstream and require an extensive survey. The extent of ponding behind culverts located in depressed regions or areas with steep slopes may be very small and require only a limited survey.

The items which should be known either through data collection or calculation include:

- Adjacent property descriptions.
- Allowable headwater level, $HW_{max.}$
- Allowable outlet velocity, $v_{max.}$
- Aquatic wildlife requirements.
- Basin area.
- Basin hydrologic characteristics.
- Channel geometry.
- Culvert shape and material.
- Culvert slope, $S_{o.}$
- Culvert hydraulic length, L.
- Debris characteristics.
- Design discharge, Q.
- Design tailwater, TW.
- Entrance conditions.
- Existing culvert description.
- Maximum allowable depth of barrel.
- Recreational requirements.
- Site geological report.
- Site history.
- Threatened and endangered species report *(impact)*.

1. Field Review. Engineers designing drainage structures should be thoroughly familiar with the watershed site under consideration. Much can be learned from survey notes, but the most complete survey cannot adequately depict all watershed site considerations or substitute for a personal inspection. A site examination can be mutually beneficial to the designer and construction engineer by helping to improve the drainage design and reducing construction problems.

2. Drainage Area. Drainage area is an important factor in estimating the flood potential. For a detailed discussion on drainage area, refer to Chapter 7, *Hydrology*.

3. Channel Characteristics. The survey should describe the physical characteristics of the existing stream channel. For purposes of documentation and design analysis, sufficient channel cross sections (at least four), a stream bed profile and the horizontal alignment should be obtained to provide an accurate representation of the channel, including the floodplain area. These cross sections can be used to obtain the natural streambed width, side slopes, and floodplain width. Often, the proposed culvert is positioned at the same longitudinal slope as the streambed. The channel profile should extend far enough beyond the proposed culvert location to define

the slope and locate any large stream bed irregularities, such as headcutting. The designer must also use this pre-construction data to predict the consequences of constricting the natural flood plain by installing an embankment across a flood plain.

General characteristics helpful in making design decisions should be noted. These include channel roughness, Manning's n values, the type of soil or rock in the stream bed, the bank conditions, type and extent of vegetal cover, permanent or intermittent wetlands, amount of drift and debris, ice conditions, and any other factors which could affect the sizing of the culvert and the durability of culvert materials. Photographs of the channel and the adjoining area can be valuable aids to the designer and serve as excellent documentation of existing conditions.

4. Aquatic Wildlife. Survey data should include information regarding necessary steps to accommodate aquatic wildlife. Information regarding the species present and required measures to accommodate their presence may be obtained from the resource agencies. A culvert designed for fish passage is discussed in more detail in Section 9.7.D.

5. Highwater Information. Highwater marks can be can be used to check results of flood estimating procedures, establish highway grade lines, and locate hydraulic controls. Often the highwater mark represents the energy of the stream and not the water surface. Even though the highwater marks are available it often is difficult to determine the flood discharge that created them.

When highwater information is obtained, the individuals contacted should be identified and the length of their familiarity with the site should be noted. In addition, the designer should ascertain whether irregularities such as channel blockage or downstream backwater altered the expected highwater. Other sources for such data might include commercial and school bus drivers, mail carriers, law enforcement officers, and highway and railroad maintenance personnel.

6. Existing Structures. The designer should place considerable importance on the hydraulic performance of existing structures. The performance of structures upstream or downstream from the culvert site can be helpful in the design.

Data on existing structures should be collected as available to include the following:

- Date of construction.
- Major flood events since construction, dates of occurrence, and water surface elevation.
- Performance during past floods based on interviews with the sources mentioned above.
- Scour indicated near the structure.
- Type of material in streambed and banks.
- Alignment and general description of structure, including:
  - Condition (abrasion, corrosion, or deterioration).
  - Dimensions.
  - Shape and material.
  - Flowline invert elevation.
  - Is the top of the foundation exposed, if so, what is the elevation.
- Highwater elevations with datum and dates of occurrence.
- Location and description of overflow areas.
- Photographs.
- Silt and drift accumulations.
- Evidence of headcutting in stream.
- Appurtenant structures such as:
  - Energy dissipaters.
  - Debris control structures.
  - Stream grade control devices.
  - As-built plan of structure.

7. Waterway Data. Changes in the flood plain due to roadway embankments can have significant effects on streamflow. In addition to the site data discussed in Section 9.1.C., information on the waterway must also be

gathered. The waterway data includes hydraulic resistance, downstream water surface elevation (tailwater) information, and upstream storage capacity.

a. Cross Sections. A field survey provides one of the best ways of collecting stream cross section data. At least four cross sections and a channel profile should be taken to establish the stream slope, the configuration of natural channel, and the possible culvert profile. The natural streambed width, side slopes, and flood plain width may be obtained from these cross sections. The cross sectional data also will help verify the accuracy of existing topographic maps. If significant ponding is likely, additional sections may be necessary to determine the storage capacity upstream of the culvert. Likewise, additional downstream sections may be necessary to establish downstream water level (tailwater) conditions. To assess the impact of the culvert, water surface profiles may be computed beginning at some downstream cross section and carrying the computations to some point upstream. For more information on the location and extent of the data needed see Chapter 8, *Open Channels*.

Additional information on stream slope and upstream storage volume also should be obtained from the topographic maps.

b. Stream Slope. The longitudinal slope of the existing channel in the vicinity of the proposed culvert should be determined in order to establish the culvert vertical profile and to define flow characteristics in the natural stream. Often, the proposed culvert is positioned at the same longitudinal slope as the streambed. However, the culvert may have to be positioned on a different slope than the streambed, as discussed in Section 9.1.F.

c. Resistance. The hydraulic resistance of the natural channel must be evaluated in order to calculate pre-project flow conditions. This resistance usually is represented by an average Manning's "n" value. Various methods are available to evaluate resistance coefficients for natural streams including comparisons with photographs of streams with known resistance values or tabular methods based on stream characteristics. Some of these methods are discussed in:

- Chapter 8, *Open Channels*.
- HEC-RAS Hydraulic Reference Manual (U.S. Army Corps of Engineers, 2002).
- HDS-6, *River Engineering for Highway Encroachments* (FHWA, 2001b).

d. Tailwater. Tailwater is defined as the depth of water downstream of the culvert measured from the outlet invert. It is an important factor in determining culvert capacity under outlet control conditions. Tailwater may be caused by an obstruction in the downstream channel, by the hydraulic resistance of the channel, by a lake, or by a receiving stream with high water surface elevation under design assumptions. In any case, backwater calculations from the downstream control point can be performed to estimate tailwater. When hydraulic resistance of the channel controls the flow depth, normal depth approximations may be used instead of backwater calculations. Normal depth assumptions are that the channel bed slope, water surface slope, and energy grade line slope are equal and that the depth of flow does not change.

e. Upstream Storage. The stream storage capacity upstream from a culvert may have an impact upon its design. The designer can approximate upstream storage capacity from contour maps of the upstream area. However, it is preferable to obtain a number of cross sections upstream of the proposed culvert. These sections must be referenced horizontally as well as vertically. The length of the upstream reach required will depend on the expected headwater and the stream slope. The cross sections can be used to develop contour maps, or the cross sectional areas can be used to compute storage. The topographic information should extend upward from the channel bed to an elevation equal to, or greater than, the design headwater elevation in the area upstream of the culvert. Usually, it is necessary to compute backwater up to the cross section where the change in water surface elevation is equal to zero.

8. Roadway Data. The proposed or existing roadway profile affects the culvert cost, hydraulic efficiency, and alignment. Information from the roadway profile and the roadway cross sections can be obtained from preliminary roadway drawings or from standard details on roadway sections. When the culvert must be sized prior to the development of preliminary plans, a best estimate of the roadway section can be used, but the culvert design must be checked and confirmed after the roadway plans are completed.

a. Roadway Section. The roadway cross section normal to the centerline typically is available from highway plans. However, the cross section needed by the culvert designer is the section at the stream crossing where the culvert is to be located. This section may be skewed with reference to the roadway centerline. For a proposed culvert, the roadway plan, profile, and cross sectional data should be combined as necessary to obtain the desired section.

b. Culvert Length. Important dimensions and features of the culvert will become evident when the desired roadway cross section is measured or established. The dimensions are obtained by superimposing the estimated culvert barrel on the roadway cross section and the streambed profile. This superposition establishes the inlet and outlet invert elevations. If the culvert is to be depressed below the natural ground slope then the length needs to be estimated with this in mind. These elevations and the resulting culvert length are approximate since the final culvert barrel size still must be determined.

Because of the difference between the lengths of metal and non-metal end sections for pipe culverts, the length of connecting pipe can vary. To avoid showing different lengths of the connecting pipe on the plans, the designer should show on the plan only the length of the metal type pipe for all alternatives.

c. Roadway Horizontal Alignment and Vertical Profile. The roadway embankment represents the obstruction encountered by the flowing stream. The embankment can act much like a dam. The culvert is similar to the normal release structure, and the roadway crest acts as an emergency spillway in the event that the upstream pool (headwater) attains a sufficient elevation. The location of initial overtopping is dependent upon the roadway geometry. Generally, the location of overtopping should be designed to conform as closely as possible to the location of the majority of flood flow under existing conditions.

The vertical profile contained in highway plans generally follows the roadway center-line. These elevations may not represent the high point in the highway cross section. The culvert designer should determine the vertical profile which establishes roadway flooding and roadway overflow elevations. Necessary provisions must be provided to ensure that all overtopping flow is returned to the original channel.

The minimum diameter of a pipe culvert shall be 450 mm (18 in), except pipes under a 7.6 m (25 ft) or greater fill shall not be less than 600 mm (24 in). Culverts shall be provided in 150 mm (6 in) increments.

For the inverts of new pipe culverts, refer to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10 for the appropriate stream invert grade at both the inlet and outlet ends.

9. Allowable Headwater. The designer should base allowable headwater elevation on the following site characteristics:

- Regulatory requirements (e.g., DEP, FHWA, FEMA, USACOE, and USCG).
- Critical elevations on the highway itself, which should include:
  - Edge of pavement.
  - Elevation at which overtopping flow begins.
  - Low point of sub-base.
  - Top of headwall.
- Elevation above which adverse impact (i.e., flooding, standing water) from highwater might occur to adjacent developed property, such as buildings, agricultural crops, or other facilities.

The maximum allowable headwater (HW) is the depth of water, measured from the entrance invert that can be ponded during the design flood. The surrounding features, flow limitations, and roadway classification must be considered for each situation.

The surrounding features that may limit the allowable headwater include the following:

- Lowest elevation of the roadway adjacent to the ponding area.
- Flowline of the roadway ditch which passes water along the roadway to another drainage basin.
- Upstream property, such as buildings or farm crops, which will be damaged if inundated.

- Elevation established to delineate floodplain zoning.

Flow limitation factors that can affect the allowable headwater include the following:

- The debris which could plug the structure.
- Excessive ponding which would allow too much silting.
- High hydrostatic pressure which would cause seepage along the culvert backfill.

The HW/D ratio to be considered for design is the ratio of headwater depth to the diameter, height, or rise of a culvert entrance. The following are the maximum allowable HW/D ratios for the design of new culverts:

- HW/D = 2.0: Circular and elliptical (squash) pipe culverts with diameters (or equivalent diameters) of 750 mm (30 in) or less.
- HW/D = 1.5: Circular and elliptical pipe culverts with diameters greater than 750 mm (30 in) and less than or equal to 1800 mm (72 in), and other culverts with cross-sectional areas equal to or less than 2.8 m^2 (30 ft^2).
- HW/D = 1.2: Circular and elliptical pipe culverts with diameters greater than 1800 mm (72 in), and other culverts with cross-sectional areas greater than 2.8 m^2 (30 ft^2).

The headwater should be checked for the design flood, based on roadway classification, and for the 100-year flood to ensure compliance with floodplain management criteria. The maximum acceptable outlet velocity should be identified. The headwater should be set to produce acceptable velocities; otherwise, stabilization or energy dissipation should be provided where acceptable velocities are exceeded. For streams with debris issues, trash racks should be considered.

Occasional flowage easement shall normally be obtained for new flooding areas beyond the right-of-way line for the 100-year storm event for flow (Q_{100}). Q_{100} shall be used to be consistent with FEMA requirements and the Pennsylvania Floodplain Management Act. Except for the Interstate Highway, which cannot be inundated at the 50-year storm event for flow (Q_{50}), all classes of highways may be inundated at the design Q if a practicable alternative is not available.

In any event, the design discharge should not inundate the travel way for the highway design event. Additionally, where practicable for the 100-year event, the net increase in water surface at the upstream face of the culvert should not exceed 0.3 m (1 ft). If the project is located in an NFIP study area, NFIP procedures must be followed and NFIP criteria must be met. Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10 should be consulted regarding increases to the 100-year flood profile in FEMA study areas.

D. Culvert Locations. Culvert location deals with the horizontal and vertical alignment of the culvert with respect to both the stream and the highway. The culvert location is important to the hydraulic performance of the culvert, stream and embankment stability, construction and maintenance costs, and the safety and integrity of the highway.

Horizontal and vertical alignment are important in maintaining a sediment free culvert. At normal depth, deposition may occur in culverts because the sediment transport capacity of flow within the culvert is less than in the stream. The following factors contribute to deposition in culverts:

- Point bars form on the inside of stream bends, and culvert inlets placed at bends in the stream will be subjected to deposition in the same manner. This effect is most pronounced in multiple barrel culverts with the barrel on the inside of the curve often becoming almost totally plugged with sediment deposits;
- Abrupt changes to a flatter grade in the culvert or in the channel adjacent to the culvert will induce deposition. Gravel and cobble deposits are common downstream from the break in grade because of the reduced transport capacity in the flatter section.

Deposition usually occurs at flow rates smaller than the design flow rate. The deposits may be removed during larger floods, dependent upon the relative transport capacity of flow in the stream and in the culvert, compaction and composition of the deposits, flow duration, ponding depth above the culvert and other factors.

Most culvert locations approximate the natural stream bed. Other locations sometimes are chosen for economic reasons. Modified culvert slopes, or slopes other than that of the natural stream, can be used to arrest stream degradation, induce sedimentation, improve the hydraulic performance of the culvert, shorten the culvert, or reduce structural requirements. Modified slopes also can cause stream erosion and deposition; therefore, slope alterations should be given special attention to ensure that significant detrimental effects do not result from the change.

E. Stream Modification. Plan location deals basically with the route the flow will take in crossing the right of way. Regardless of the degree of sinuosity of the natural channel within the right of way, a crossing is generally accomplished by using a straight culvert either normal to or skewed with the roadway centerline.

Ideally, a culvert should be placed in the natural channel (Figure 9.2). This location usually provides good alignment of the natural flow with the culvert entrance and outlet, and little structural excavation and channel work are required. Where location in the natural channel would require an inordinately long culvert, some stream modification may be in order (Figure 9.3). Such modifications to reduce skew and shorten culverts should be designed carefully to avoid erosion and siltation problems.

Culvert locations normal to the roadway centerline are not suggested where severe or abrupt changes in channel alignment are required upstream or downstream of the culvert. Short radius bends are subject to erosion on the concave bank and deposition on the inside of the bend. Such changes upstream of the culvert result in poor alignment of the approach flow to the culvert, subject the highway fill to erosion, and increase the probability of deposition in the culvert barrel. Abrupt changes in channel alignment downstream of culverts may cause erosion on adjacent properties.

Figure 9.2 Culvert Located in a Natural Streambed

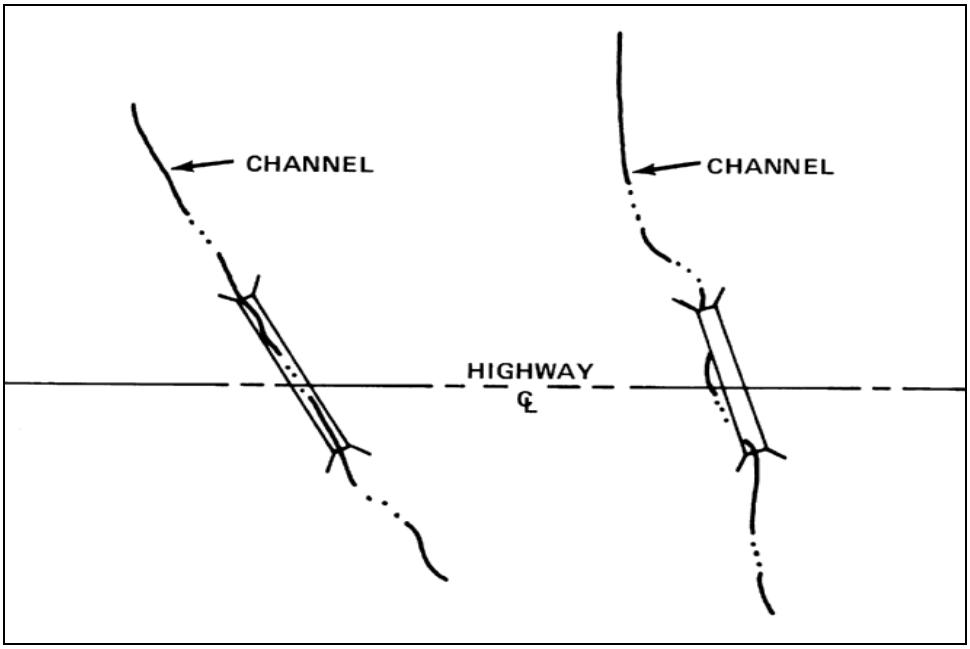
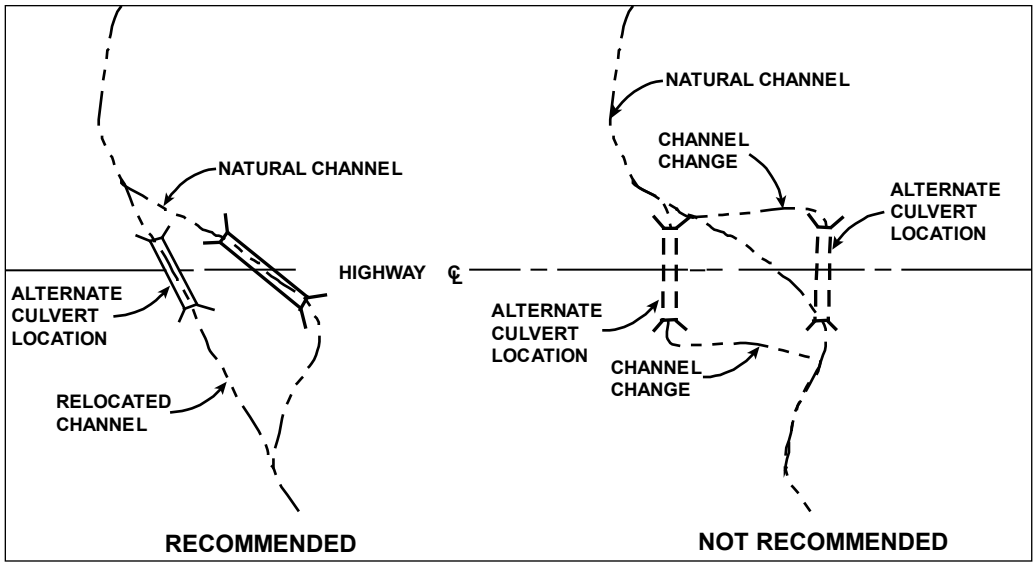


Figure 9.3 Stream Relocation Options



Culverts in live stream environments may necessitate temporary diversion channels to carry flow around the work site, as discussed in Section 9.1.G.

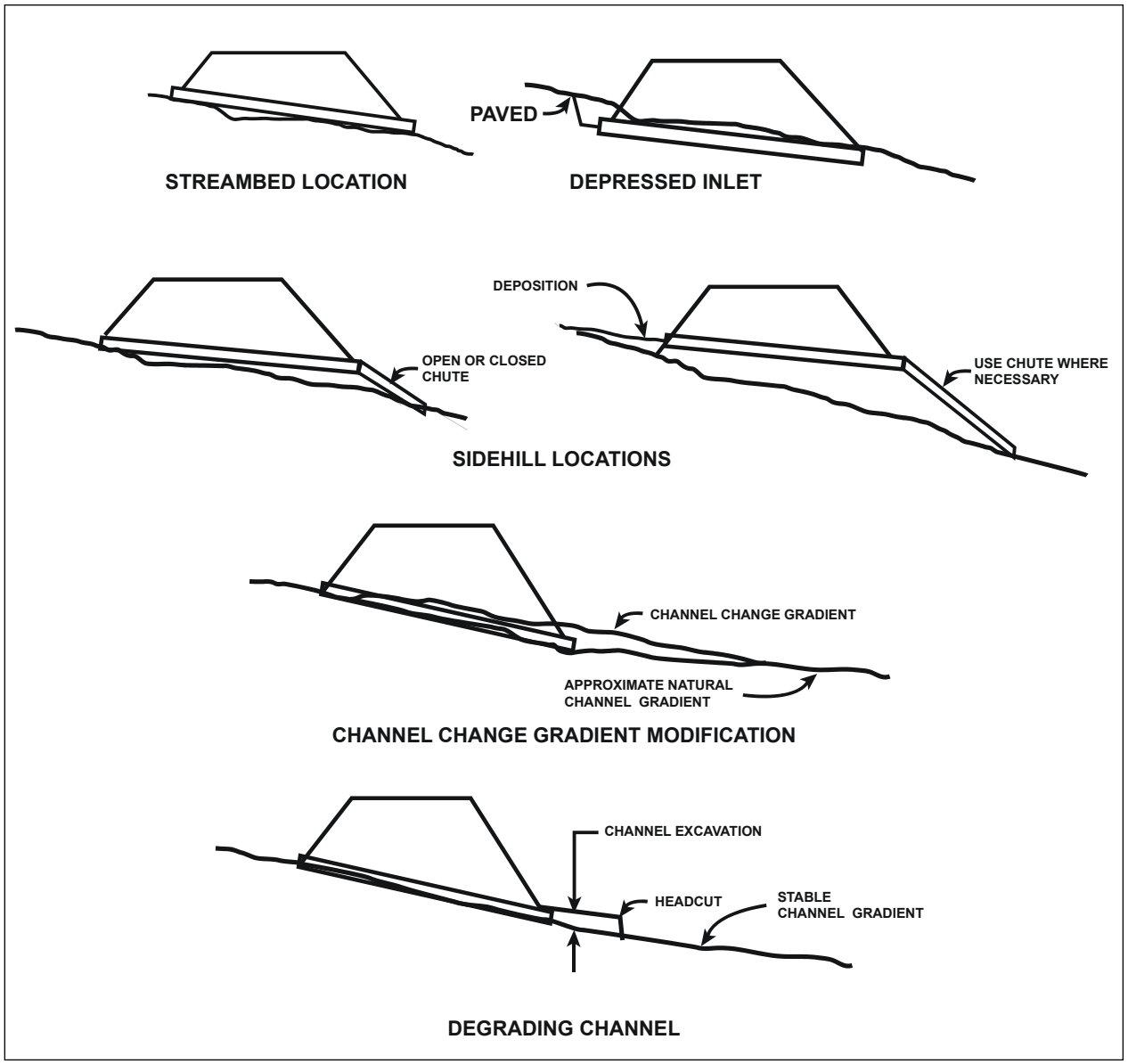
F. Culvert Profile. The establishment of the vertical profile of the culvert usually is a matter of placing the upstream and downstream flow line elevations of the culvert at the same elevations as the existing streambed or counter-sinking the entire culvert so that the upstream and downstream ends of the culvert are an equal distance below the natural streambed. In some instances, the upstream flowline may need to be lowered. Lowering the upstream flowline can provide improved hydraulic operation, but may create maintenance problems due to a higher potential for both sedimentation and scour.

The designer should avoid placing the downstream flowline of the culvert at a level higher than the roadway embankment toe of slope. Such a configuration results in an increase in the potential for erosion.

Channel changes often are shorter and steeper than the natural channel. A modified culvert slope can be used to achieve a flatter gradient in the channel so that degradation will not occur. Figure 9.4 illustrates some possible culvert profiles.

Where channel excavation is planned, culvert invert elevations can be established to accommodate drainage requirements if concurrent channel and highway construction is possible. If concurrent construction is not feasible, a joint or cooperative project should be investigated so that highway culverts can be designed and constructed to serve current highway drainage requirements as well as future needs for land drainage.

Figure 9.4 Culvert Placement Locations



G. Temporary Diversion Channels. Culvert construction in live stream environments frequently necessitates the installation of temporary diversion channels to carry the stream around the work site. The temporary diversion channels need protective linings to prevent erosion. At times it also may be necessary to develop a staged construction sequence which will permit a portion of the work to be done; stream flow is then diverted through the completed portion of the culvert while the remainder of the culvert installation is constructed. Additional information on temporary erosion and sediment control measures that can be used at a construction site may be found in Chapter 12, *Erosion and Sediment Pollution Control* and Volume III, Erosion and Sediment Control in Highway Construction of the *Highway Drainage Guidelines* (AASHTO, 2003).

H. Design and Allowable Outlet Velocity. At flood conditions, a culvert may restrict the available channel area, and flow velocities in the culvert may be higher than in the channel. These increased velocities can cause streambed scour, bank erosion in the vicinity of the culvert outlet, and can hamper fish passage. Minor problems occasionally can be avoided by increasing the barrel roughness with baffles. Energy dissipaters and outlet protection devices sometimes are required to avoid excessive scour at the culvert outlet. When a culvert is operating under inlet control and the culvert barrel is not operating at capacity, it often is beneficial to reduce the barrel slope or add a roughened section to reduce outlet velocities.

Similar to the allowable headwater, the allowable outlet velocity is a design criterion which is unique to each culvert site. The types and characteristics of soil can vary considerably from site to site. Energy dissipaters or rock stilling basins may be required in erodible soils to decrease the velocity of flow to prevent damage to the channel.

Velocities at which soils erode may vary widely. The designer should attempt to estimate the threshold of erosive velocity for each culvert location. This may be done by observing storm flows on various soil types and estimating those velocities at which erosion is occurring (see HEC-15, *Design of Roadside Channels with Flexible Linings* (FHWA, 1988), HEC-11, *Design of Riprap Revetment* (FHWA, 1989a), HDS-6, *River Engineering for Highway Encroachments* (FHWA, 2001b), HEC-23, *Bridge Scour and Stream Instability Countermeasures* (FHWA, 2001c), and HEC-20, *Stream Stability at Highway Structures, Third Edition* (FHWA, 2001d)).

The designer should exercise extreme caution when considering culvert designs with outlet velocities of greater than 4.5 m/s (14.8 ft/s). The designer should provide riprap or control devices in situations where outlet velocity poses potential erosion problems. Section 9.5 describes different velocity protection and control devices.

If the culvert has been sized properly according to allowable headwater criteria, it almost always is more economical to protect against excessive outlet velocity with riprap and/or velocity protection or control devices than to try to adjust the culvert size to reduce the excessive outlet velocity.

Velocities of less than about 0.6 m/s (2 ft/s) usually foster deposition of sediments at flow depth of 20% of culvert diameter (HEC-23 (FHWA, 2001c)). Therefore, 0.6 m/s (2 ft/s) is suggested as a minimum for culvert design and operation where sediment deposit would present a problem.

I. End Treatments. End treatments serve several different purposes, but typically they act as a retaining wall to keep the roadway embankment material out of the culvert opening. Some secondary characteristics of end treatments include improvements to:

- Performance.
- Public safety.
- Debris control.
- Flood protection.
- Piping prevention (flow through the embankment outside of the culvert).

Figure 9.5 shows sketches of various end treatment types.

Figure 9.5 Typical Culvert End Treatments



Projected end



Projected Bell Inlet



Head wall with beveled edge inlet



Head wall with wing walls

J. Safety Considerations. Cross-drainage and longitudinal drainage facilities usually are necessary in any highway project. These facilities control highway drainage and natural runoff from areas adjacent to the highway. However, due to their inherent mass and fixed nature, they can affect the safety of the motoring public.

Three practical methods are available for addressing safety issues associated with highway culverts:

- Safety treatment at culvert ends.
- Satisfaction of clear-zone requirements.
- Shielding by guide fences and/or other types of barriers.

Safety treatment of culvert ends is effective in avoiding the hazards created by protruding culvert ends; however, it also represents a significant interference with the original purpose of the drainage structure. A safety end treatment has a tendency to accumulate trash and flood debris, thus blocking flow into and out of the culvert.

Mitered end sections should be used carefully for several reasons. First, the use of mitered end sections may increase hydraulic head losses; the hydraulic performance of this type of inlet is approximately the same as a thin-edged projecting inlet. Additionally, a non-reinforced mitered end may affect the structural integrity of the culvert. With the use of mitered end sections, where practicable, the designer should incorporate safety end treatment standards, such as metal guides.

The simple step of removing the headwall and applying a mitered end section alone (see Figure 9.6) offers relatively little obstacle for passage of drift or debris.

Figure 9.6 Mitered End Treatment for Safety



K. Culvert Type Selection. Culvert type selection includes the choice of material to meet design life, shape and cross section, and number of culvert barrels. Total culvert cost can vary considerably depending upon the culvert type selection. The following are some of the factors which influence culvert type selection:

- Fill height.
- Terrain.
- Foundation condition.
- Shape of the existing channel.
- Roadway profile.
- Allowable headwater.
- Stream stage discharge.
- Frequency-discharge relationships.
- Cost.
- Service life.
- Fish passage.

Generally, the primary factors affecting culvert type are economics, hydraulic properties, durability, and strength. A summary of acceptable criteria for specifying alternate types of culvert pipes based on the type of use is presented in Table 9.1.

Table 9.1. Alternate Pipes Selection Criteria Based on Type of Installation

Types of Drainage Installations	Types of Pipe	Minimum Number of Alternates Required
Cross Drains Under All Highways Including Pipes Under Pavements	CP or RCP or CCGS or FBCGS or ECP or SLCPP	2
Parallel Storm Sewer Outside of Pavement	All pipes listed in Table Legend	3
Side Drains (Driveways, etc.)	All pipes listed in Table Legend	3
Slope Pipes	AA or CSPMC	2
Combination Storm Sewer and Underdrain and Other Special Drainage Systems	CP open joint or RCP open joint or perforated CSPMC or perforated AA or perforated CPP	3

Table Abbreviations:
PSLCGS -- Plastic (PVC) Smooth-Lined Corrugated Galvanized Steel
SSP -- Stainless Steel Pipe
CP -- Non-reinforced Concrete Pipe (36" max.)
RCP -- Reinforced Concrete Pipe
SLCGS -- Smooth-lined Corrugated Galvanized Steel
AA -- Aluminum Alloy
Ext. Str. VC -- Extra Strength Vitrified Clay (36" max.)
FBCGS -- Fiber-Bonded Corrugated Galvanized Steel Pipe
DIP -- Ductile Iron Pipe
CSPMC -- Corrugated Steel Pipe Metallic Coated
CCGS -- Coated (Polymer) Corrugated Galvanized Steel
ECP -- Epoxy-Lined Concrete Pipe
SLCPP -- Smooth-Lined Corrugated Polyethylene Pipe
CPP -- Corrugated Polyethylene Pipe (36" max.)

Table Notes:
1. Pipe alternates may be eliminated for the following engineering reasons: (1) unstable support, (2) high impact and concentrated loading, (3) high embankments, (4) limited clearance, (5) steep gradients, (6) high acidity or alkalinity of soils and water or other corrosive elements, (7) high erosive forces, or (8) for other pertinent reasons.
2. When conditions are such that the pipe requires coating, galvanized pipes shall be polymeric coated in accordance with Publication 408, *Highway Specifications*. Polymeric coated galvanized steel pipe is available only in 10 gage (3.5 mm) or lighter; if a coating is required for 8 gage (4.3 mm) or heavier, this alternate should be eliminated.

The selection of pipe alternates is dependent upon environmental factors as presented in Table 9.2. Consideration to the future land use should be given. For example, pipe placed in an area not being mined presently, but which ultimately may be mined, should be designed to handle the mine acid drainage.

Table 9.2. Pipe Selection Criteria for Corrosion Protection Based on pH and Resistivity Values

TYPE OF PIPE	COATING	WATER AND/OR SOIL pH	SOIL RESISTIVITY (OHM-M)	ABRASION COATING REQUIRED
Aluminum Alloy	Uncoated	4.0 to 8.5	> 15	Paved Invert
Concrete	Uncoated	4.0 or Greater	All	Epoxy Lined
Concrete	Vitrified Clay	< 4.0	All	None Required
Thermo-Plastic		All	All	None Required
Steel	Metallic Coated	5.5 to 8.5	> 60	Paved Invert
Steel	10 mil Polymer-Type C	5.5 to 8.5	>60	None Required

Table Notes:
1. Selection of pipe alternates and requisitions for pipe shall be supported with the pH of the effluent and pH and resistivity of the soil. For design purposes, the pH of the water at the construction site shall be determined in the field by ASTM D-1293. If the pH is below 5.5, a one (1) quart sample shall be furnished to the Materials and Testing Division (MTD) for exact identification. Testing should be done seasonally, if possible, and the worst set of conditions used in making determination of the proper type of pipe. Additionally, a six (6) to eight (8) pound (2.7 to 3.6 kg) sample of the site soil shall be sent to the MTD for determination of the soil pH and resistivity by AASHTO T-288, for further consideration of the proper pipe type.
2. All pipe designs shall vary based on location as stated in Design Manual Part 2, Table 10.5.5 and will range from a 25- to 100-year design life. If an analysis is not feasible, the designer shall assume worst-case conditions. For steel pipe, this would require a minimum of 10 gage (3.5 mm) for corrosion protection in the pH range 5.5 to 8.5 or a polymeric coating at the gage required in the appropriate fill-height table, with the exception that side drains may be supplied in 14 gage (2.0 mm). If the fill-height criteria indicates the need for heavier than 10 gage (3.5 mm), the steel pipe alternate shall be eliminated unless appropriate site analysis is provided which indicates a minimum 50-year service life.

L. Economics. The designer should select a material that satisfies hydraulic, structural, and other design criteria with the lowest overall cost. One must keep in mind that both material availability and ease of construction influence the total cost of the structure, as well as the timing of project delivery. Choosing culvert components which are readily available to construction contractors or maintenance forces may result in lower bid prices and faster completion of the project. Some common combinations are:

- Pipe (concrete, steel, aluminum, plastic):
  - Circular.
  - Pipe-arch (CMP only).
  - Elliptical.
  - Precast concrete arches.
- Structural-plate (steel or aluminum):
  - Circular.
  - Pipe-arch.
  - Elliptical.
  - Arch.
- Box culverts (single or multiple barrel):
  - Concrete box culvert.
  - Steel or aluminum box culvert.
- Long span (structural-plate [steel or aluminum]):
  - Low profile arch.
  - High profile arch.
  - Elliptical.

In those cases where Departmental design criteria specifies that alternate pipes shall be included in the plans and proposal, design computations shall be submitted for each alternate. If the design computations determine that, for one or more of the alternates, different sizes are adequate, the construction drawings and quantities should be developed for the larger size.

Alternate sizes should be indicated on the tabulation and/or summary sheet when a choice in size exists.

M. Hydraulic Properties. Each shape has distinct hydraulic properties, and each material has an associated wall roughness. Both factors influence the hydraulic operation of a structure, in addition to inlet geometry, as discussed earlier.

N. Operation. The designer should consider the feasibility of operating the roadway and culvert over the period of anticipated service life. Each of the factors listed below depends on the type of material and the shape of the culvert and therefore requires consideration.

- Ease of access for maintenance purposes.
- Repair and rehabilitation costs.
- Durability.

O. Fill Heights. Fill heights conform to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.4 and Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Appendix B.

9.2 HYDRAULIC ANALYSIS OF CULVERTS

A. General - Hydraulic Analysis of Culverts. Flow through a culvert is classified by one of two types of operation: outlet control or inlet control. The two basic culvert design criteria are allowable headwater and allowable outlet velocity. Allowable headwater is usually the most important in influencing the overall configuration of the culvert. By consideration of various parameters, the designer can arrive at the appropriate calculation procedure. In this chapter, an attempt has been made to illustrate the major conditions affecting culvert analysis. Some minor variations of the identified conditions presented here may be found, but these variations should not alter the result appreciably.

Actual culvert flow conforms to the laws of open-channel flow and closed-culvert flow. The procedures set forth in this section assume steady flow but still can involve extensive calculations which lend themselves to computer application. Computer models should be used for all final design applications; however, for a simple cross pipe to be replaced by maintenance forces or for initial planning, simplified hand methods and nomographs may be appropriate.

Culvert analysis involves computing headwater elevations for both inlet and outlet control for a given discharge. These elevations are compared, and the larger of the two is used as the controlling headwater elevation. Tailwater effects are taken into consideration when calculating these elevations. If the controlling headwater elevation overtops the roadway embankment, an overtopping analysis is done in which flow is balanced between the culvert discharge and the flow over the roadway.

For multiple conveyance systems, a headwater balancing technique is used to compute the discharge through each opening.

B. Computations for Culvert Hydraulics. The flow through a culvert is classified as either inlet or outlet control. Under inlet control, the headwater elevation is a function of only the inlet configuration, which includes the size, shape, and material as well as entrance conditions such as headwalls. Under outlet control the energy equation is solved in terms of an orifice with an empirical coefficient of discharge to account for the inlet losses. For the outlet control the energy equation is balanced between the inlet and outlet of the pipe accounting for the minor losses at the entrance and exit and the friction losses within the culvert barrel.

In order to understand the equations the following sections dealing with subcritical and supercritical flow and uniform flow are presented. For a more detailed discussion see Chapter 8, *Open Channels*.

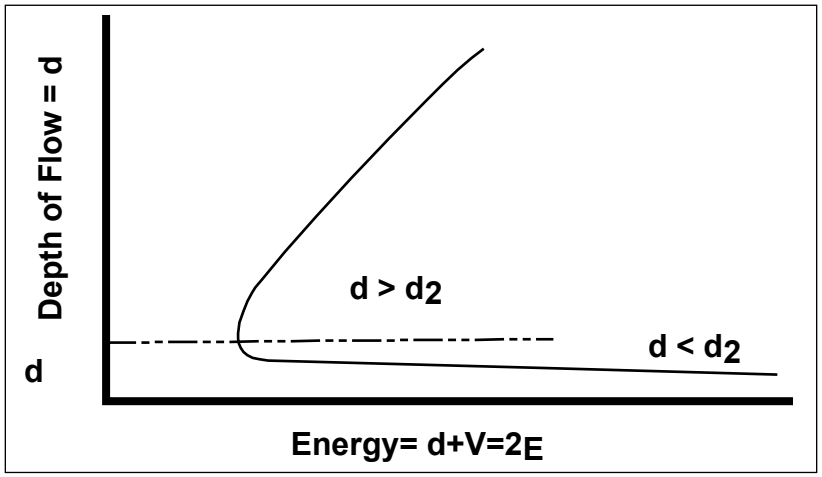
C. Supercritical versus Subcritical Flow. The specific energy, E, of a fluid is defined as the energy per unit weight of the fluid measured relative to the bottom of the channel and comprises the sum of depth of flow and velocity head.

(Equation 9.1)

$$E = d + \frac{v^2}{2g}$$

where: E = specific energy, m (ft)
d = depth of flow, m (ft)
v = average velocity of flow, m/s (ft/s)
g = gravitational acceleration, 9.81 m/s^2 (32.2 ft/s^2)

Figure 9.7 Plot of Depth of Flow versus Specific Energy for a Constant Flow Rate per Unit Width (q)



When specific energy is plotted against the depth of flow, a parabolic curve, as seen in Figure 9.7, is formed. Specific energy is at a minimum where the depth of flow is at critical depth (d_c). This critical state represents the threshold between "subcritical" flow and "supercritical" flow.

With respect to Figure 9.7, supercritical flow occurs when depth of flow is lower than critical depth and subcritical flow occurs when the depth of flow is greater than critical depth of flow. Refer to Chapter 8, *Open Channels*, for additional information regarding types of flow in channels.

It is apparent from the specific energy diagram that when the flow conditions are close to critical, a relatively large change of depth occurs with small variations in specific energy. Flow under these conditions is unstable, and excessive wave action or undulations of the water surface may occur.

When the flow is subcritical, computations of water surface elevations proceed from downstream to upstream; for supercritical flow, the computations are reversed beginning with upstream and proceeding downstream. For culvert hydraulics, inlet control often indicates that supercritical flow occurs in the culvert while outlet control indicates that flow computations are controlled by friction in the system and the flow regime is subcritical.

A determination of the appropriate flow regime is accomplished by evaluating a dimensionless value called the Froude number, F_r.

(Equation 9.2)

$$F_r = \frac{v}{\sqrt{g d_m}}$$

where: d_m = hydraulic depth, m (ft)

The hydraulic depth is calculated by dividing the cross sectional flow area (A) by the width of the free water surface (T). When $F_r > 1.0$, the flow is supercritical and is characterized as rapid. When $F_r < 1.0$, the flow is subcritical and is characterized as smooth and tranquil. If $F_r = 1.0$, the flow is said to be critical.

D. Critical Depth in Culverts. Critical depth can be illustrated best as the depth at which water flows over a weir; this depth is attained automatically where no other backwater forces are involved. This is because it is the depth at which the energy content of flow is at a minimum. Critical depth is a direct function of discharge and geometry of the culvert. The general equation for critical flow in any shape culvert or channel is:

(Equation 9.3)

$$\frac{Q_c^2}{g} = \frac{A_c^3}{T}$$

where: Q_c = critical discharge, m^3/s (cfs)
A_c = cross sectional area of flow at critical discharge, m^2 (ft^2)
T = water surface width of critical discharge, m (ft)

The formula applicable for calculating critical depth in rectangular channels is:

(Equation 9.4)

$$d_c = \sqrt[3]{\frac{q^2}{g}}$$

where: q = discharge per unit width of rectangular culvert, $m^3/s/m$ (cfs/ft)

Critical depth of circular and pipe-arch or irregular shapes can be calculated by iterative use of Equation 9.5. Equations 9.6 and 9.7 allow determination of the area, A, and top width, T, of flow in a circular pipe, respectively. For other shapes, the designer should acquire or derive relationships between depth of flow, area and top width.

(Equation 9.5)

$$\theta = \cos^{-1}\left(1 - \frac{2d}{D}\right)$$

Note: Equation 9.5 is for use in Degrees only.

(Equation 9.6)

$$A = \frac{D^2}{8}\left[\frac{\pi}{90}\theta - \sin(2\theta)\right]$$

(Equation 9.7)

$$T = D\sin(\theta)$$

where: A = section area of flow, m^2 (ft^2)
T = width of water surface, m (ft)
d = depth of flow, m (ft)
D = pipe diameter, m (ft)

E. Uniform Depth in Culverts. Uniform depth is the depth of water when the geometry, roughness characteristics, slope, and discharge are all constant for a sufficiently long reach of open channel conveyance. Uniform depth sometimes is referred to as normal depth. The characteristics of culvert flow closely approximate the culvert characteristics involved in uniform flow. Since culverts are usually hydraulically short there may not be a sufficient length of reach for attainment of normal depth.

Manning's Equation (represented by Equation 9.8) describes uniform flow and is used for the determination of uniform depth.

(Equation 9.8)

$$Q = \frac{k}{n} AR^{2/3} S^{1/2}$$

where: Q = culvert barrel discharge, m^3/s (cfs)
k = 1.0 (metric), 1.486 (U.S. Customary)
n = Manning's roughness coefficient (see Table 7.4 for suggested values)
A = cross section flow area, m^2 (ft^2)
R = hydraulic radius of the culvert, R= A/WP, m (ft)
WP = wetted perimeter, m (ft)
S = slope of water surface - taken to be the culvert slope, m/m (ft/ft)

For most shapes, a direct solution of Equation 9.10 for normal depth is not possible and an iterative solution is required. For rectangular shapes area and wetted perimeter are simple functions of flow depth. For circular pipe, area is computed using Equation 9.5 and wetted perimeter is computed using Equation 9.9. For other shapes, the designer should acquire or derive the relationship between depth of flow, area and wetted perimeter.

(Equation 9.9)

$$WP = \frac{\pi}{180} D\theta$$

Notes:
1. θ is from Equation 9.5
2. Equation 9.9 is for use in Degrees only

F. Friction Slope. The friction slope is a theoretical value which describes the slope of the energy grade line and is based upon Manning's Equation, rearranged as follows:

(Equation 9.10)

$$S_f = \left(\frac{Q_n}{xR^{2/3}A}\right)^2$$

where: S_f = friction slope, m/m (ft/ft)

G. Steep Slope versus Mild Slope. When critical depth (d_c) is higher than normal depth (d_u or d_n), the slope is considered steep. The culvert may flow completely full (pressure flow) or partly full (free surface flow). Whether the free surface flow is supercritical or subcritical depends on tailwater conditions.

In considering each factor more critical, judgment is necessary if it is kept in mind that any condition that causes turbulence and retards flow results in a greater value of "n."

Outlet velocity for bituminous paved inverts shall be determined based on a 25% reduction in Manning's roughness coefficient, "n."

When critical depth is lower than uniform depth, the slope is termed mild. Pressure flow or free surface flow may occur. Free surface flow is most likely to be subcritical within the culvert.

The shape of the free water surface is dependent on whether the culvert slope is steep or mild. The methods described in this chapter accommodate the differences in water surface shape.

H. Headwater Under Inlet Control. The design equations used to develop the inlet control nomographs presented in the HDS-5, *Hydraulic Design of Highway Culverts*, (FHWA, 2005b) are based on the research conducted by the National Bureau of Standards (NBS) under the sponsorship of the Bureau of Public Roads (now the Federal Highway Administration).

The two basic conditions of inlet control depend upon whether the inlet end of the culvert is or is not submerged by the upstream headwater. If the inlet is not submerged the inlet performs as a weir. If the inlet is submerged, the inlet performs as an orifice. Equations are available for each of the above conditions.

Between the unsubmerged and the submerged conditions, there is a transition zone for which the NBS research provided only limited information. The transition zone is defined empirically by drawing a curve between the tangent of the curves defined by the unsubmerged and submerged equations. In most cases, the transition zone is short and the curve is easily constructed.

Equations 9.11 through 9.13 present the unsubmerged and submerged inlet control design equations developed by FHWA. Note that there are two forms of the unsubmerged equation. Form (1) is based on the specific head at critical depth, adjusted with two correction factors. Form (2) is an exponential equation similar to a weir equation. Form (1) is preferable from a theoretical standpoint, but Form (2) is easier to apply. Either form of unsubmerged inlet control equation will produce adequate results.

The constants for the equations are given in Table 9.3.

Inlet Control Design Equations by FHWA presented in HDS-5 (FHWA, 2005b) are as follows:

Unsubmerged

(Equation 9.11)

Form (1)
$$\frac{HW_i}{D} = \frac{H_c}{D} + K\left[\frac{K_u Q}{AD^{0.5}}\right]^M - 0.5S^2$$

(Equation 9.12)

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

Table 9.3. Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomo-graph Scale	Inlet Edge Description	Eqn. Form	Unsubmerged		Submerged	
					K	M	c	Y
1	Cir. Concrete	1	Square edge w/hw	1	.0098	2.0	.0398	.67
		2	Groove end w/hw		.0018	2.0	.0292	.74
		3	Groove end projecting		.0045	2.0	.0317	.69
2	Cir. CMP	1	Headwall	1	.0078	2.0	.0379	.69
		2	Mitered to slope		.0210	1.33	.0463	.75
		3	Projecting		.0340	1.50	.0553	.54
3	Cir.	A	Beveled ring, 45° bevels	1	.0018	2.50	.0300	.74
		B	Beveled ring, 33.7° bevels*		.0018	2.50	.0243	.83
8	Rect. Box	1	30° to 75° ww	1	.026	1.0	.0347	.81
		2	90° and 15° ww flares		.061	.75	.0400	.80
		3	0° ww flares		.061	.75	.0423	.82
9	Rect. Box	1	45° ww flare d=.043D	2	.510	.667	.0309	.80
		2	18° to 33.70 ww flare d=.083D		.486	.667	.0249	.83
10	Rect. Box	1	90° hw w/3/4" chamfers	2	.515	.667	.0375	.79
		2	90° hw w/45° bevels		.495	.667	.0314	.82
		3	90° hw w/33.7° bevels		.486	.667	.0252	.865
11	Rect. Box	1	¾" chamfers; 45° skewed hw	2	.545	.667	.04505	.73
		2	¾" chamfers; 30° skewed hw		.533	.667	.0425	.705
		3	¾" chamfers; 15° skewed hw		.522	.667	.0402	.68
		4	45° bevels; 10°-45° skewed hw		.498	.667	.0327	.75
12	Rect. Box	1	45° non-offset ww flares	2	.497	.667	.0339	.803
	¾" chamfers	2	18.4° non-offset ww flares		.493	.667	.0361	.806
		3	18.4° non-offset ww flares - 30° skewed barrel		.495	.667	.0386	.71
13	Rect. Box	1	45° ww flares – offset	2	.497	.667	.0302	.835
	Top Bevels	2	33.7° ww flares – offset		.495	.667	.0252	.881
		3	18.4° ww flares – offset		.493	.667	.0227	.887
16-19	C M Boxes	2	90° hw	1	.0083	2.0	.0379	.69
		3	Thick wall projecting		.0145	1.75	.0419	.64
		5	Thin wall projecting		.0340	1.5	.0496	.57

Table Abbreviations:

CMP – Corrugated Metal Pipe
Cir. – Circular
hw – headwall
Eqn. – Equation
CM – Corrugated Metal
Rect. – Rectangular
ww – wingwall
Ref. – References

Submerged

(Equation 9.13)

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

where HW_i = Headwater depth above the inlet control section invert, m (ft)
D = Interior height of culvert barrel, m (ft)
H_c = Specific head at critical depth, m (ft)
Q = Discharge, m^3/s (cfs)
A = Full cross sectional area of culvert barrel, m^2 (ft^2)
S = Culvert barrel slope, m/m (ft/ft)
K, M, c, Y = Constants (see Table 9.3)
K_u = 1.811 (metric), 1.0 (U.S. Customary)

A culvert operates with inlet control when the flow capacity is controlled at the entrance by the depth of headwater and the entrance geometry, including the barrel shape, cross sectional area and the inlet edge. Sketches to illustrate inlet control flow for unsubmerged and submerged projecting entrances are shown in Figure 9.8.

For a culvert operating with inlet control, the roughness and length of the culvert barrel and outlet conditions (including tailwater) are not factors in determining culvert hydraulic capacity. The entrance edge and the overall entrance geometry have much to do with culvert capacity in this type of flow; therefore, special entrance designs can improve hydraulic capacity and result in a more efficient and economical culvert.

Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. Inlet control is most likely to occur when the culvert configuration is hydraulically steep ($d_c > d_u$). The control section of a culvert operating under inlet control is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical. Depending on conditions downstream of the culvert inlet, a hydraulic jump may occur in the culvert.

Under inlet control, hydraulic characteristics downstream of the inlet control section do not affect the culvert capacity. The upstream water surface elevation and the inlet geometry represent the major flow controls. The inlet geometry includes the barrel shape, the cross sectional area, and the inlet edge.

It is important to note that, although inlet control conditions are more prevalent when the culvert slope is greater than the critical slope, inlet control can occur under mild slope conditions. Such a condition is likely when the tailwater is lower than critical depth in the culvert exit of a short culvert.

A fifth-degree polynomial equation was developed by FHWA, based on regression analysis, to model the inlet control headwater for a given flow. The regression equation was developed for the range of inlet heads from one-half to three times the culvert rise. Analytical equations, based on minimum energy principles, are matched to the regression equations to model flows that create inlet control heads outside of the regression data range.

For $0.5 \leq \frac{HW_{ic}}{D} \leq 3.0$, Equation 9.14 applies:

(Equation 9.14)

$$HW_{ic} = [a + bF + cF^2 + dF^3 + eF^4 + fF^5]D - 0.5DS_0$$

where:
- HW_{ic} = inlet control headwater, m (ft)
- D = rise of culvert barrel, m (ft)
- a to f = regression coefficients for each type of culvert (Table 9.4)
- S_0 = culvert slope, m/m (ft/ft)
- F = function of the average outflow discharge being routed through a culvert; culvert barrel rise; and, for box and pipe arch culverts, the width of the barrel, W, as shown in Equation 9.15

(Equation 9.15)

$$F = 1.8113\frac{Q}{WD^{3/2}}$$

where:
- W = width or span of culvert, m (ft)

Figure 9.8 Inlet Control Conditions
(Condition A – Inlet unsubmerged; Condition B – Inlet submerged; Condition C – Outlet submerged)

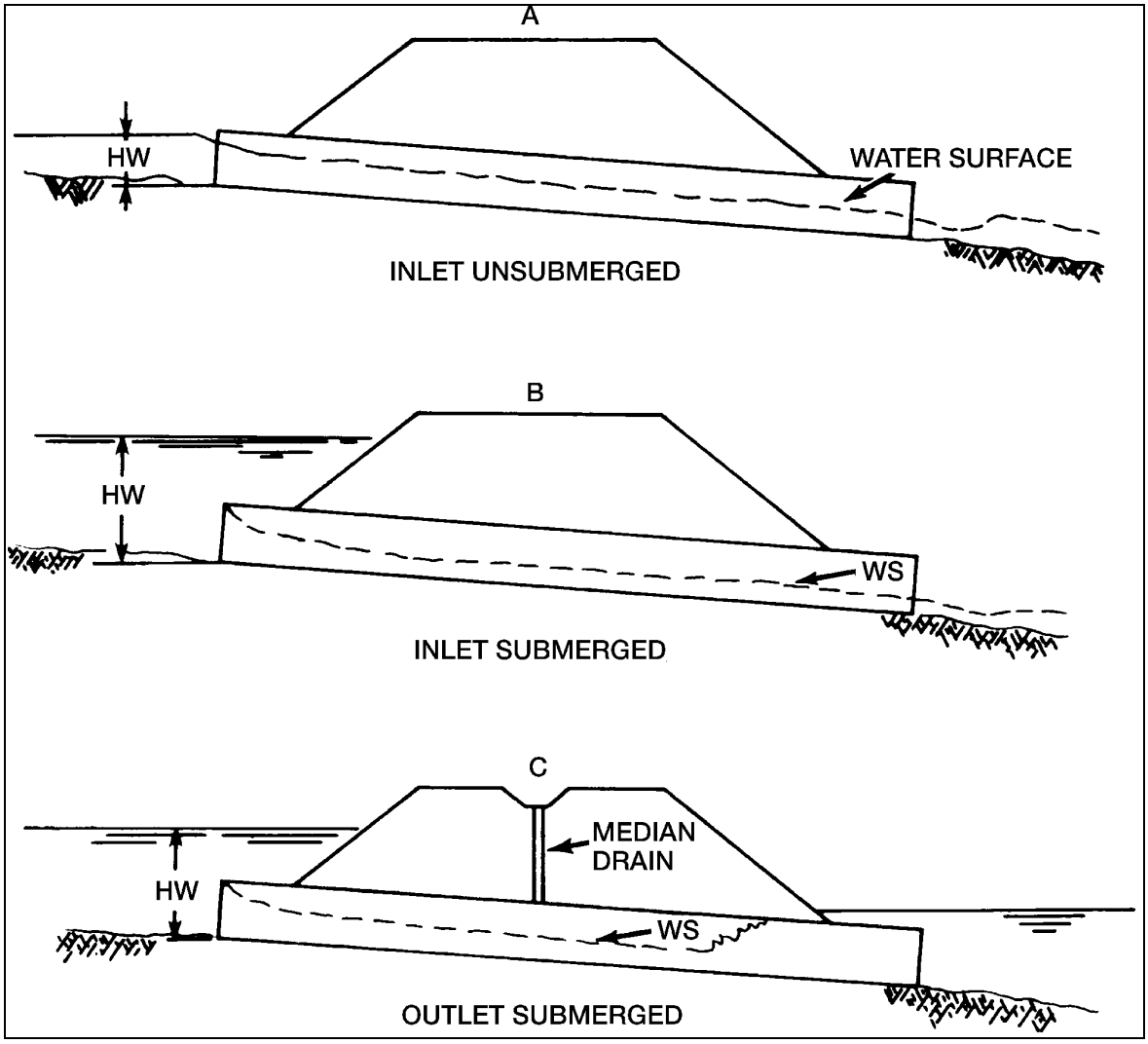


Table 9.4. Regression Coefficients for Inlet Control Equations

Shape & Material	Entrance type	a	b	c	d	e	F
RCP	Square edge w/hw	0.087483	0.706578	-0.2533	0.0667	-0.00662	0.000251
	Groove end w/hw	0.114099	0.4653562	-0.2336	0.059772	-0.00616	0.000243
	Groove end proj.	0.108786	0.662381	-0.2338	0.057959	-0.00558	0.000205
	Beveled Ring	0.63343	0.766512	-0.316097	0.08767	-0.00984	0.000417
	Improved (flared) inlet	0.2115	0.3927	-0.0414	0.0042	-0.0003	-0.00003
CMP	Headwall	0.167433	0.53859	-0.14937	0.039154	-0.00344	0.000116
	Mitred	0.107137	0.757789	-0.3615	0.123393	-0.01606	0.000767
	Proj.	0.187321	0.567719	-0.15654	0.044505	-0.00344	0.00009
	Improved (flared) inlet	0.2252	0.3471	-0.0252	0.0011	-0.0005	-0.00003
Box	30-70° Flared ww	0.072493	0.507087	-0.11747	0.02217	-0.00149	0.000038
	Parallel to 15° ww	0.122117	0.505435	-0.10856	0.020781	-0.00137	0.0000346
	Straight ww	0.144138	0.461363	-0.09215	0.020003	-0.00136	0.000036
	45° ww w/top bevel	0.156609	0.398935	-0.06404	0.011201	-0.00064	0.000015
	Parallel hw w/bevel	0.156609	0.398935	-0.06404	0.011201	-0.00064	0.000015
	30° skew w/chamfer edges	0.122117	0.505435	-0.10856	0.020781	-0.00137	0.000034
	10-45° skew w/bevel edges	0.089963	0.441247	-0.07435	0.012732	-0.00076	0.000018
Oval B > D	Square edge w/hw	0.13432	0.55951	-0.1578	0.03967	-0.0034	0.00011
	Groove end w/hw	0.15067	0.50311	-0.12068	0.02566	-0.00189	0.00005
	Groove end proj.	-0.03817	0.84684	-0.32139	0.0755	-0.00729	0.00027
Oval D > B	Square edge w/hw	0.13432	0.55951	-0.1578	0.03967	-0.0034	0.00011
	Groove end w/hw	0.15067	0.50311	-0.12068	0.02566	-0.00189	0.00005
	Groove end proj.	-0.03817	0.84684	-0.32139	0.0755	-0.00729	0.00027
CM pipe-arch	Headwall	0.111281	0.610579	-0.194937	0.051289	-0.00481	0.000169
	Mitred	0.083301	0.795145	-0.43408	0.163774	-0.02491	0.001411
	Proj.	0.089053	0.712545	-0.27092	0.792502	-0.00798	0.000293
Struct pl pipe arch	Proj. - 450mm corner pl	0.089053	0.712545	-0.27092	0.792502	-0.00798	0.000293
	Proj. - 780mm corner pl	0.12263	0.4825	-0.00002	-0.04287	0.01454	-0.00117
CM Arch (flat bottom)	Parallel hw	0.111281	0.610579	-0.1949	0.051289	-0.00481	0.000169
	Mitred	0.083301	0.795145	-0.43408	0.163774	-0.02491	0.001411
	Thin wall projecting	0.089053	0.712545	-0.27092	0.792502	-0.00798	0.000293

Table Abbreviations:

CMP – Corrugated Metal Pipe
RCP – Reinforced Concrete Pipe
hw – headwall
pl – plate
CM – Corrugated Metal
Proj. – Projecting
ww – wingwall

For HW_i/D >3.0, an orifice equation, Equation 9.16, is used to estimate headwater. The potential head is determined from the centroid of the culvert opening, which is approximated as the sum of the invert elevation and one-half the rise of the culvert.

(Equation 9.16)

$$HW_i = \left[\frac{Q}{k}\right]^2 + \frac{D}{2}$$

where: HW_i = inlet control headwater depth, m (ft)
Q = design discharge, m^3/s (cfs)
k = orifice equation constant ($\sqrt{2gAC}$)
D = rise of culvert, m (ft)

The effective area, A, and orifice coefficient, C, are implicit.

The coefficient, k, is determined by rearranging Equation 9.16 using the discharge that creates a HW/D ratio of 3 in the regression equation (i.e., the upper limit of the Equation 9.11):

(Equation 9.17)

$$k = 0.6325\frac{Q_{3.0}}{D^{1/2}}$$

where: $Q_{3.0}$ = discharge in m^3/s (cfs) at which $HW_i/D = 3$

Generally, for Department designs, it is not considered efficient to design culverts for $HW_i/D < 0.5$. However, if such a condition is likely ($HW_i/D < 0.5$), an open channel flow minimum energy equation (weir equation) is used with the addition of a velocity head loss coefficient. The minimum energy equation, with the velocity head loss adjusted by an entrance loss coefficient, generally describes the low flow portion of the inlet control headwater curve. However, numerical errors in the calculation of flow for very small depths tend to increase the velocity head as the flow approaches zero. This presents little or no problem in most single system cases since the flows that cause this are relatively small.

In many of the required calculations for the solution of multiple culverts, the inlet control curve must decrease continuously to zero for the iterative calculations to converge. Therefore, in computer models, modifications to this equation have been made to force the velocity head to continually decrease to zero as the flow approaches zero.

Refer to the "Charts" in HDS-5 (FHWA, 2005b) for graphical solution of headwater under inlet control. The fifth degree polynomials are an approximation of the original research performed by FHWA. Rather than have the engineer solve using Equation 9.14, FHWA developed the charts for hand computations. Those charts have since been modified to be equivalent to the Equations 9.11 through 9.13, which were developed from the same data as the fifth degree polynomials. Within the accuracy of reading the charts the solutions are equivalent to those equations.

I. Headwater Under Outlet Control. Outlet control occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. Outlet control is likely only when the hydraulic grade line inside the culvert at the entrance exceeds critical depth. Therefore, outlet control is most likely when the flow in the culvert is on a mild slope ($d_n > d_c$). It is also possible to experience outlet control with a culvert on a steep slope ($d_n < d_c$) and a high tailwater such that subcritical flow or full flow exists in the culvert.

The headwater resulting from flow through a culvert in outlet control is a function of the following:

- Discharge.
- Culvert section geometry.
- Culvert roughness characteristics.
- Length of the culvert.
- Profile of the culvert.
- Entrance geometry (to a minor extent).
- Tailwater level (possibly).

For practical purposes, when a culvert is under outlet control, the headwater can be adjusted by modifying either the culvert geometry or roughness.

Outlet control headwater HW_{oc} depth (from the flowline of the entrance) is computed by balancing energy between the culvert exit and the culvert entrance as indicated by Equation 9.18.

(Equation 9.18)

$$HW_{oc} + h_{va} = h_e + h_{vi} + \sum h_f - S_o$$

where: HW_{oc} = headwater depth due to outlet control, m (ft)
h_{va} = velocity head of flow approaching the culvert entrance, m (ft)
h_{vi} = velocity head at entrance as calculated using Equation 9.19, m (ft)
h_e = entrance head loss as calculated using Equation 9.20, m (ft)
h_f = friction head losses as calculated using Equation 9.21, m (ft)
S_o = culvert slope, m/m (ft/ft)

When applying Equation 9.18, the velocity at the entrance often is assumed to be negligible (i.e. $h_{vi} = 0$) so that the headwater and energy grade line are coincident at the upstream face of the culvert. This is conservative for most Department needs. Some conditions may warrant consideration of the approach velocity, in which case the approach velocity head could be subtracted. Possible scenarios under which it may be necessary to consider the approach velocity include:

- Estimating the impact of a culvert on FEMA designated flood plains.
- Designing or analyzing a culvert used as a flood attenuation device where the storage volumes are very sensitive to small changes in headwater.
- A culvert that has an effective flow area similar to the approach channel section so that approach velocities and through culvert velocities are similar.

The velocity head at any location in the culvert is computed using Equation 9.19. The velocity at the entrance (v_i) is used to compute the velocity head at the entrance.

(Equation 9.19)

$$h_v = \left[\frac{v^2}{2g}\right]$$

where: v = flow velocity in culvert, m/s (ft/s)
g = the gravitational acceleration, 9.81 m/s^2 (32.2 ft/s^2)

The entrance loss, h_e, is dependent on the velocity of flow at the inlet, v_i, and the entrance configuration, which is accommodated using an entrance coefficient, C_e.

(Equation 9.20)

$$h_e = C_e\left[\frac{v_i^2}{2g}\right]$$

where: C_e = entrance loss coefficient
v_i = flow velocity inside culvert inlet, m/s (ft/s)

Values of C_e are selected by the designer from the values in Table 9.5 (entrance loss coefficients) based on culvert shape and entrance condition. The velocity, v_i, is equal to the outlet velocity (v_o) for submerged conditions. For free surface flow, the designer should determine the velocity at the entrance using backwater calculations through the culvert.

The outlet depth, H_o, is established based on the following conditions:

- For steep slope: if the Manning's Equation capacity of the culvert is lower than the discharge (i.e., $d_u > D$), the culvert is expected to flow full and the outlet depth, H_o, is taken as the higher of the barrel depth, D, and the tailwater depth, TW. Otherwise (i.e., $d_u < D$), the outlet depth is set to the tailwater depth when the tailwater exceeds critical depth (d_c). If the tailwater is below critical depth in a steep-slope culvert, outlet control headwater is unlikely.
- For a culvert on a mild slope, if critical depth exceeds the barrel depth ($d_c > D$), the outlet depth (H_o) is taken as the higher of the barrel depth and the tailwater depth. Otherwise (i.e., $d_c < D$), the outlet depth is taken as the higher of critical depth and the tailwater depth.

J. Outlet Control Headwater Due to Full Flow in Culverts. The friction slope is constant over the length of the barrel that is flowing full. The frictional headloss, h_f, is computed using Equation 9.21.

(Equation 9.21)

$$h_f = S_f L$$

where: h_f = head loss due to friction in the culvert barrel, m (ft)
S_f = friction slope, m/m (ft/ft) (See Equation 9.10)
L = length of culvert containing full flow, m (ft)

Table 9.5. Entrance Loss Coefficients (C_e)

Concrete Pipe	**C_e**
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square-edge	0.5
Rounded (radius 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Corrugated Metal Pipe or Pipe-Arch	**C_e**
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Reinforced Concrete Box	**C_e**
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	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

** "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.*

Full flow at the outlet occurs when the outlet depth (H_o) is equal to, or higher than, the barrel depth, D. Refer to the discussion above regarding the outlet depth.

For a completely submerged culvert, as shown in Figure 9.9, Equation 9.18 is used directly to determine the outlet control headwater. Figure 9.9 includes all the energy conditions associated with a straight, prismatic culvert. The

magnitude of the energy loss at the exit, h_o, does not affect the outlet control headwater under the assumption that the hydraulic grade line inside the culvert is equal to the tailwater depth or critical depth (whichever controls). In some publications, the velocity head at the entrance is termed exit loss.

If the friction slope is less than the culvert slope, it is possible that full flow may not occur along the entire length of the culvert. Free surface flow begins at the point of intersection of the hydraulic grade line and the soffit of the culvert barrel as shown in Figure 9.9(C). If this condition occurs, the outlet control headwater is determined using the free surface flow approach described below with the starting depth (d_1) equal to the barrel rise (D) and starting at the location along the barrel at which free surface flow begins. The total friction losses ($\sum h_f$) include the head loss due to full flow friction (h_{ff}) and the friction losses associated with partially full or free-surface flow (h_{fp}). The energy loss at the exit, h_o, does not affect the outlet control headwater under the assumption that the hydraulic grade line at the culvert exit is equal to the tailwater elevation.

The distance from the outlet at which free surface flow begins (L_f) is determined using the geometric relationship shown in Equation 9.22.

(Equation 9.22)

$$x = \frac{h_o + H_o - D}{S_p - S_f}$$

If x is greater than the culvert length, or S_f is greater than S_o, the entire length of the culvert is full.

When the outlet is not submerged, if the culvert is long enough and the flow high enough, full flow will begin within the culvert. The point at which full flow begins is determined using the procedures described in Section 9.2.K. Figure 9.9(C) illustrates this condition. Since the hydraulic grade line has already been established as being at a depth of D, the entrance control headwater is determined using Equation 9.23.

(Equation 9.23)

$$HW_{oc} = D + S_f L_f - S_o L_f + h_e + h_{vi} - h_{va}$$

where: L_f = length from beginning of full flow in culvert to culvert entrance in m (ft)

This constitutes an implicit use of Equation 9.15 since the backwater calculation accommodates the friction losses (h_{fp}), and change in elevation for the free surface flow length (L - L_f). As discussed earlier, the velocity head associated with the flow approaching the culvert entrance often is ignored.

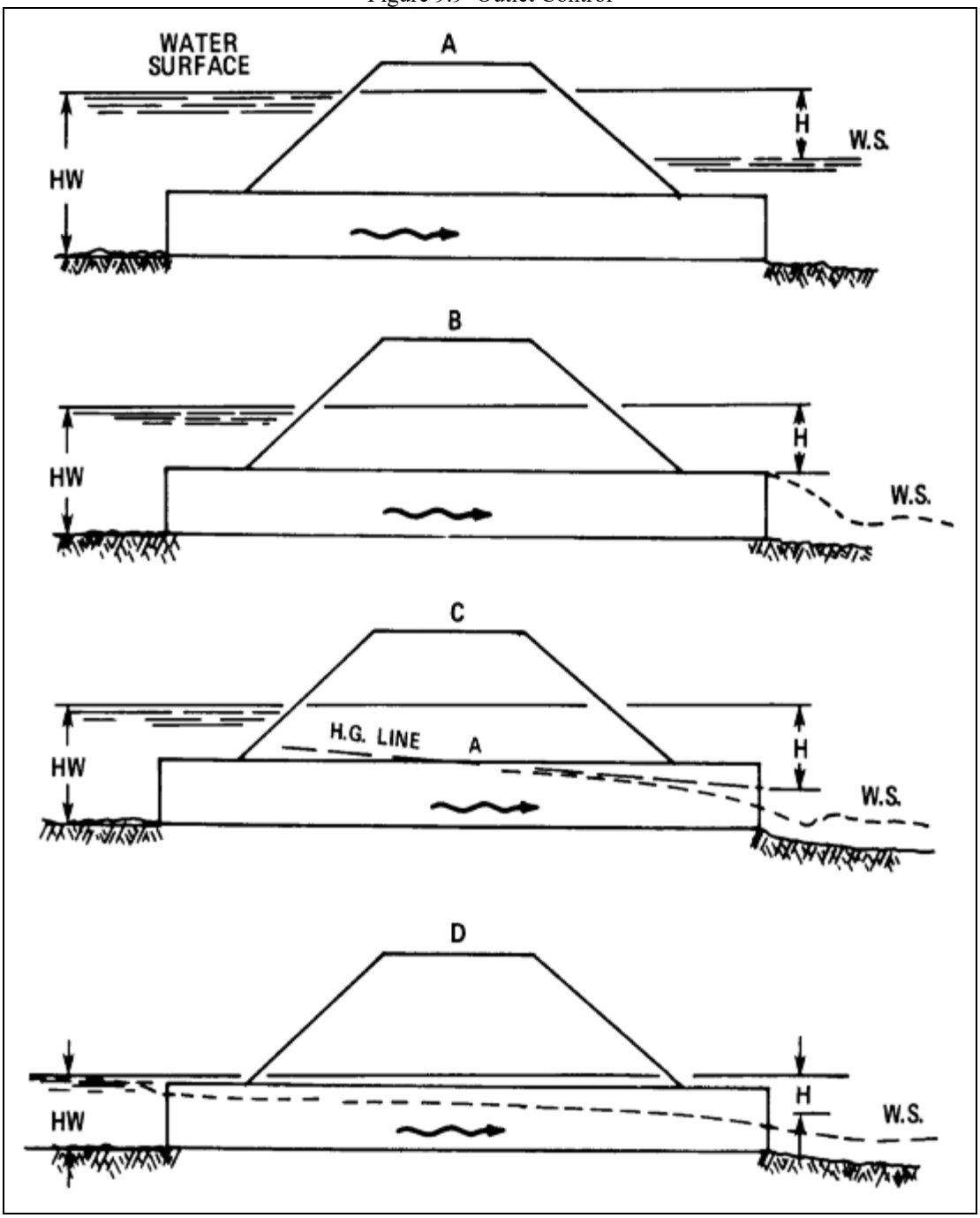
K. Outlet Control Headwater Due to Free-Surface Flow in Culverts. If free surface flow is occurring in the culvert, the hydraulic parameters are changing with flow depth along the length of the culvert. One of two combinations of free surface flow is likely to occur in the culvert:

1. Entire culvert maintains free surface flow, as shown in Figure 9.9(D)

2. Free surface at exit but backwater reaches barrel rise within culvert, establishing full flow for the remaining portion, as shown in Figure 9.9(C)

For each of the above conditions, it is necessary to calculate the backwater profile based on a starting depth, d_i. The following discusses the selection of appropriate starting depths.

Figure 9.9 Outlet Control



By definition, a free-surface backwater from the outlet end of a culvert may only affect the headwater when subcritical flow conditions exist in the culvert. Subcritical, free-surface flow at the outlet will exist if either the culvert is on a mild slope with a hydraulic grade line (H_o) lower than the outlet soffit or the culvert is on a steep slope with a hydraulic grade line higher than critical depth at culvert outlet and lower than the outlet soffit. Therefore, when free surface flow exists at the outlet, the starting depth should be taken as the higher of critical depth (d_c) and the tailwater depth (TW).

As indicated in Section 9.2.J., if full flow exists at the outlet and free-surface flow begins upstream in the barrel, the backwater calculations should begin at the barrel rise (d_1 = D) and continue to the inlet to get the depth at the inlet, d_i.

The Direct Step Method, which is discussed in Section 9.2.N., is appropriate for determining water surface profiles in culverts. The calculations begin at the outlet and proceed in an upstream direction until either the end of the culvert is reached (at which $d_i = d_2$), or the calculated depth (d_2) reaches the barrel rise (D).

When subcritical free surface flow exists at the inlet, the outlet control headwater is calculated using Equation 9.24.

(Equation 9.24)

$$HW_{oc} = d_i + h_e + h_{vi} - h_{va}$$

This is applying Equation 9.15 implicitly because the backwater calculation process accommodates the friction losses ($\sum h_f$) and the change in elevation (S_oL). The approach velocity head (h_{va}) often is ignored.

If the backwater reaches the barrel depth before reaching the end of the culvert, the remaining length, L_f, will flow full and the outlet control headwater is computed using Equation 9.23.

L. Slug Flow. When the flow becomes unstable, a phenomenon termed slug flow may occur. In this condition the flow varies from inlet control to outlet control and back again in a cyclic pattern under the following circumstances:

- Flow is indicated as supercritical but the tailwater level is relatively high.
- Uniform depth and critical depth are relatively high with respect to the culvert barrel depth.
- Uniform depth and critical depth are within about 5% of each other.

The methods discussed in this chapter accommodate the potential for slug flow by assuming the higher of inlet and outlet control headwater.

M. Determination of Outlet Velocity. The outlet velocity, v_o, is dependent on the culvert discharge (Q) and the cross sectional area of flow at the outlet (A_o).

(Equation 9.25)

$$v_o = \frac{Q}{A_o}$$

The variable d_o is assigned as the depth with which to determine the cross sectional area of flow at the outlet.

For outlet control, the depth, d_o, is set equal to the higher of critical depth (d_c) and tailwater depth (TW) as long as the value is not higher than the barrel rise (D) as shown in Figure 9.9. If the culvert will flow full at the outlet, usually due to a high tailwater or a culvert capacity lower than the discharge, d_o is set to the barrel rise (D) so that the full cross sectional area of the culvert is used.

For inlet control, under steep slope conditions, the designer may estimate the depth at the outlet using one of the following approaches:

1. Employ a step backwater method (such as the Direct Step Method discussed in Section 9.2.N.) starting from critical depth (d_c) at the inlet and proceeding downstream to the outlet. If the tailwater is lower than critical depth at the outlet, calculate the velocity resulting from the computed depth at the outlet. If the tailwater is higher than critical depth, a hydraulic jump within the culvert is possible. Section 9.2.O. discusses a means of estimating whether the hydraulic jump occurs within the culvert. If the hydraulic jump does occur within the culvert, determine the outlet velocity based on the outlet depth, $d_o = H_o$.

2. Assume uniform depth at the outlet. If the culvert is long enough and tailwater is lower than uniform depth, uniform depth will be reached at the outlet of a steep slope culvert. For a short, steep culvert with tailwater lower than uniform depth, the actual depth will be higher than uniform depth but lower than critical depth, so this assumption will be conservative in that the estimate of velocity will be somewhat higher than the actual velocity. If the tailwater is higher than critical depth, a hydraulic jump is possible and the outlet velocity could be significantly lower than the velocity at uniform depth. Outlet velocities should be checked if a hydraulic jump is present since outlet protection may no longer be necessary.

N. Direct Step Backwater Method Applied to Culverts. The Direct Step Backwater Method uses the same basic equations as the Standard Step Backwater Method (see Section 8.5), but is simpler to compute because no iteration is necessary. In the Direct Step Method, the designer chooses an increment (or decrement) of water depth (□d) and computes the distance over which the depth change occurs. The accuracy is dependent on the size of □d. The method is appropriate for prismatic channel sections such as occur in most culverts. It is useful for estimating supercritical profiles and subcritical profiles. The following provides some direction on its application for free surface flow in culverts.

1. Choose a starting point with a known starting water depth (d_1). This starting depth is dependent on whether the profile is supercritical or subcritical.

- For a mild slope ($d_c < d_u$) and free surface flow at the outlet, begin at the outlet end. Select the higher of critical depth (d_c) or tailwater depth (TW). This is not to imply that supercritical flow will not occur in a culvert on a mild slope; however, most often, the flow will be subcritical when mild slopes exist. A check of this assumption may be necessary.
- For a steep slope ($d_c > d_u$), where the tailwater exceeds critical depth but does not submerge the culvert outlet, begin at the outlet with the tailwater as the starting depth.
- For a steep slope in which tailwater depth is lower than critical depth, begin the water surface profile computations at the culvert entrance starting at critical depth and proceeding downstream to the culvert exit. This implies inlet control in which case the computation may be necessary to determine outlet velocity but not headwater.
- For a submerged outlet in which free surface flow begins within the barrel, use the barrel depth, D, as the starting depth. Begin the backwater computations at the location where the hydraulic grade line is coincident with the soffit of the culvert.

The following steps assume subcritical flow on a mild slope culvert for a given discharge, Q, through a given culvert of length L, at a slope S_o.

2. Calculate the following at the outlet end of the culvert based on the selected starting depth (d_1):

- Cross section area of flow, A.
- Wetted perimeter, P.
- Velocity, v (= Q/A).
- Velocity head, h_v, using Equation 9.19.
- Specific energy, E, using Equation 9.1.
- Friction slope, S_f, using Equation 9.10.
- Assign the subscript 1 to the above variables (A_1, P_1, etc.).

3. Choose an increment or decrement of flow depth, Δd; if $d_1 > d_u$, use a decrement (negative Δd); otherwise use an increment. The increment, Δd, should be such that the change in adjacent velocities is not more than 10%.

4. Calculate the parameters, A, P, v, E, and S_f at the new depth, $d_2 = d_1 + \Delta d$, and assign the subscript 2 to these (e.g., A_2, P_2, etc.).

5. Determine the change in energy, ΔE, using Equation 9.26.

(Equation 9.26)

$$\Delta E = E_2 - E_1$$

6. Calculate the arithmetic mean friction slope using Equation 9.27.

(Equation 9.27)

$$S_f = \frac{\left(S_{f2} + S_{f1}\right)}{2}$$

7. Using Equation 9.28, determine the distance, ΔL, over which the change in depth occurs.

(Equation 9.28)

$$\Delta L = \frac{\Delta E}{S_o - S_f}$$

8. Consider the new depth and location to be the new starting positions (assign the subscript 1 to those values currently identified with the subscript 2) and repeat steps 3 to 7, summing the incremental lengths, ΔL, until the total length, $\sum L$, equals or just exceeds the length of the culvert. The same increment may be used throughout or the designer may modify the increment to achieve the desired resolution. Such modifications are necessary when the last total length computed far exceeds the culvert length and when high friction slopes are encountered. If the computed depth reaches the barrel rise (D) before reaching the culvert inlet, skip step 9 and determine the outlet control headwater using Equation 9.23.

9. The last depth (d_2) established is the depth at the inlet (d_i) and the associated velocity is the inlet, v_i. Calculate the headwater using Equation 9.24.

The procedure for subcritical flow ($d > d_c$), but steep slope ($d_c > d_u$) is similar, with the following exceptions:

- Choose a decrement in depth, Δ d.
- If the depth, d, reaches critical depth before the inlet of the culvert is reached, the headwater is under inlet control (Section 9.2.H.) and a hydraulic jump may occur in the culvert barrel. Refer to Section 9.2.O. for discussion of the hydraulic jump.
- If the depth at the inlet is higher than critical depth, determine the outlet control head water using Equation 9.24 as discussed in Section 9.2.K. A hydraulic jump may occur within the culvert. Refer to Section 9.2.O. for discussion of the hydraulic jump.

The procedure for supercritical flow ($d < d_c$) and steep slope is similar, with the following exceptions:

- Begin computations at critical depth at the culvert entrance and proceed downstream.
- Choose a decrement of depth, d.
- If the tailwater is higher than critical depth, a hydraulic jump may occur within the culvert. Refer to Section 9.2.O. for discussion of the hydraulic jump.

O. Hydraulic Jump in Culverts. For a given discharge in any channel, when water flows at a depth which is less than critical depth (supercritical flow), there is a "sequent" (or "conjugate") depth in subcritical flow such that the sum of the forces due to momentum and hydrostatic pressure at respective cross sections will be the same. With a proper configuration, the water flowing at the lower depth in supercritical flow can "jump" abruptly to its sequent depth in subcritical flow. This is called a hydraulic jump. With the abrupt increase in flow depth, there is a corresponding increase in cross sectional area of flow and a decrease in average velocity.

The balance of forces is represented using a momentum function, as appears in Equation 9.29.

(Equation 9.29)

$$M = \frac{Q^2}{gA} + A\bar{d}$$

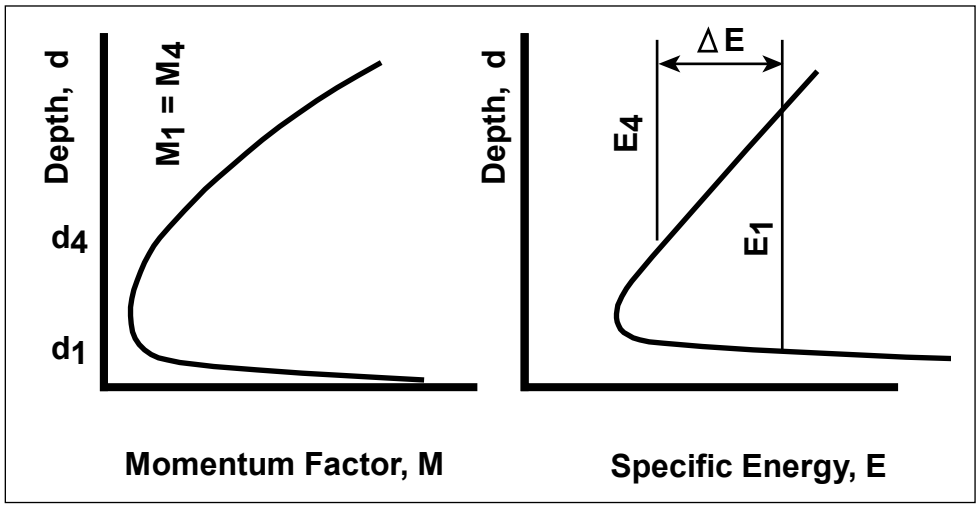
where:
- M = momentum function
- Q = discharge, m^3/s (cfs)
- A = section area of flow, m^2 (ft^2)
- $\bar{d}$ = height from water surface to centroid of flow area, m (ft)

The term $A\bar{d}$ represents the moment of the area about the water surface. Assuming no drag forces or frictional forces at the jump, conservation of momentum maintains that the momentum function at the approach depth, M_1, is equal to the momentum function at the sequent depth, M_s.

Figure 9.10 provides a sample plot of depth and momentum function and an associated specific energy plot. By comparing the two curves at a supercritical depth and its sequent depth it can be seen that the hydraulic jump

involves a loss of energy. Also, the momentum function defines critical depth as the point at which minimum momentum is established.

Figure 9.10 Momentum Function and Specific Energy



The potential occurrence of the hydraulic jump within the culvert is determined by comparing the outfall conditions with the sequent depth of the supercritical flow depth in the culvert. The conditions under which the hydraulic jump is likely to occur depend on the slope of the culvert.

Under mild slope conditions ($d_c < d_u$), two typical conditions could result in a hydraulic jump:

1. The backwater profile in the culvert caused by the tailwater is higher than the sequent depth computed at one location in the culvert.

2. The supercritical profile reaches critical depth upstream of the culvert outlet.

Under steep slope conditions, the hydraulic jump is likely only when the tailwater is higher than the sequent depth.

P. Sequent Depth for Rectangular Culvert. A direct solution for conjugate depth, d_s, is possible for free surface flow in rectangular culverts using Equation 9.30.

(Equation 9.30)

$$d_s = 0.5d_1\left(\sqrt{1+\frac{8v_1^2}{gd_1}}-1\right)$$

where: d_s = sequent depth, m (ft)
d_1 = depth of flow, m (ft)
v_1 = velocity of flow at depth d_1, m/s (ft/s)

Q. Sequent Depth for Circular Culvert. A direct solution for conjugate depth in a circular culvert is not feasible. However, an iterative solution is possible by selecting a trial sequent depth, d_s, and applying Equation 9.31 until the calculated discharge is equal to the design discharge.

(Equation 9.31)

$$Q^2 = \frac{g\left(A_s \overline{d_s} - A_1 \overline{d_1}\right)}{\frac{1}{A_1} - \frac{1}{A_s}}$$

where:
- Q = discharge, m^3/s (cfs)
- A_s = area of flow at sequent depth, m^2 (ft^2)
- $A_s \overline{d}_s$ = moment of area about surface at sequent depth, m^3 (ft^3)
- $A_1 \overline{d}_1$ = moment of area about surface at supercritical flow depth, m^3 (ft^3)

R. Sequent Depth for Other Shapes. Equation 9.31 is applicable to other culvert shapes. The moment of area about the surface, $A\overline{d}$, is dependent on the shape of the culvert and depth of flow. The designer should acquire or derive a relationship between flow depth and second moment of area.

S. Roadway Overtopping. Where water flows both over the roadway and through a culvert, a definition of hydraulic characteristics requires a flow distribution analysis. This is a common problem where a discharge of low probability of occurrence (e.g., 500-year return period) is applied to a facility which may have been designed for a higher probability (e.g., 10-year return period). For example, a complete design involves the application and analysis of a 100-year discharge to a hydraulic facility which may have been designed for a much smaller flood.

In such a case, the headwater may exceed the low elevation of the roadway, causing part of the water to flow over the roadway embankment while the remainder flows through the structure. The headwater components of flow form a common headwater level. An iterative process is necessary to establish this common headwater.

The following is one iterative approach that is reasonable for hand computations and computer programs.

Step 1 Initially assume that all the runoff (analysis discharge) passes through the culvert and determine the headwater using the procedures outlined in Section 9.3.D. If the headwater is lower than the low roadway elevation, no roadway overtopping occurs and the analysis is complete; otherwise, proceed to Step 2.

Step 2 Record the analysis discharge as the initial upper flow limit and zero as the initial lower flow limit. Assign 60 or 40% of the analysis discharge to the culvert and the remaining portion to the roadway as the initial apportionment of flow.

Step 3 Using the procedures outlined in Section 9.3.D., determine the headwater and the apportioned flow for the culvert.

Step 4 Compute the roadway overflow (discharge) associated with the headwater level determined in step 3 using the Weir Equation (Equation 9.32).

(Equation 9.32)

$$Q = k_t C L H_h^{1.5}$$

where:
- Q = discharge, m^3/s (cfs)
- k_t = over-embankment flow adjustment factor for friction
- C = discharge coefficient (for roadway overtopping use 1.66 (metric), 2.6 (U.S. Customary))
- L = horizontal length of overflow, m (ft). This length should be perpendicular to the overflow direction. For example, if the roadway curves, the length should be measured along the curve.
- H_h = average depth between headwater and low roadway elevation, m (ft)

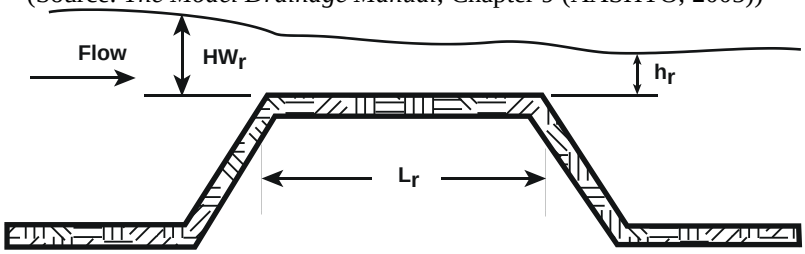
The value H_h assumes that the effective approach velocity is negligible (less than 1.5 m/s (4.92 ft/s)), similar to the culvert headwater procedure. For estimation of maximum headwater, this is a conservative assumption; however, under some conditions, such as the need to provide adequate detention storage, the designer may need to consider the approach velocity head ($v^2/2g$). That is, replace H_h with $H_h + v^2/2g$ in Equation 9.32.

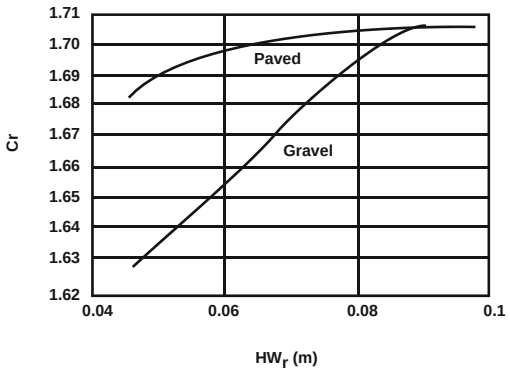
Tailwater will affect the over-embankment flow if its excess (H_t) over the highway is lower than critical depth of flow over the road, which is approximately 0.67 H_h. For practical purposes, H_t/H_h may approach 0.8 without any correction coefficient. For H_t/H_h values above 0.8, use Figures 9.11(a) and 9.11(b) to determine k.

For most cases of flow over highway embankments, the section over which the discharge must flow is parabolic or otherwise irregular. In such cases, it becomes necessary to divide the section into manageable increments and to calculate individual weir flows for the incremental units, summing them for total flow.

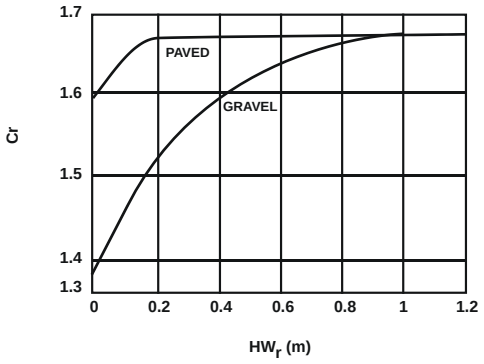
If the tailwater is sufficiently high, the flow over the roadway can be affected. In fact, at high depth, the flow over the road may become open channel flow and weir calculations are no longer valid. At extremely high depth of roadway overtopping, it may be reasonable to ignore the culvert opening and compute the water surface elevation based on open channel flow over the road.

Figure 9.11(a) Metric Discharge Coefficients for Roadway Overtopping
(Source: *The Model Drainage Manual*, Chapter 9 (AASHTO, 2005))



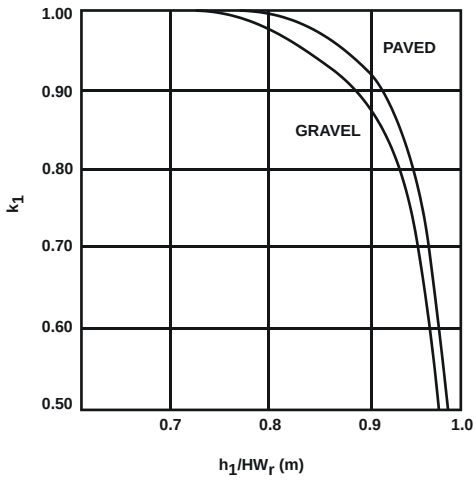


A) Discharge Coefficient for $HW_r/L_r > 0.15$



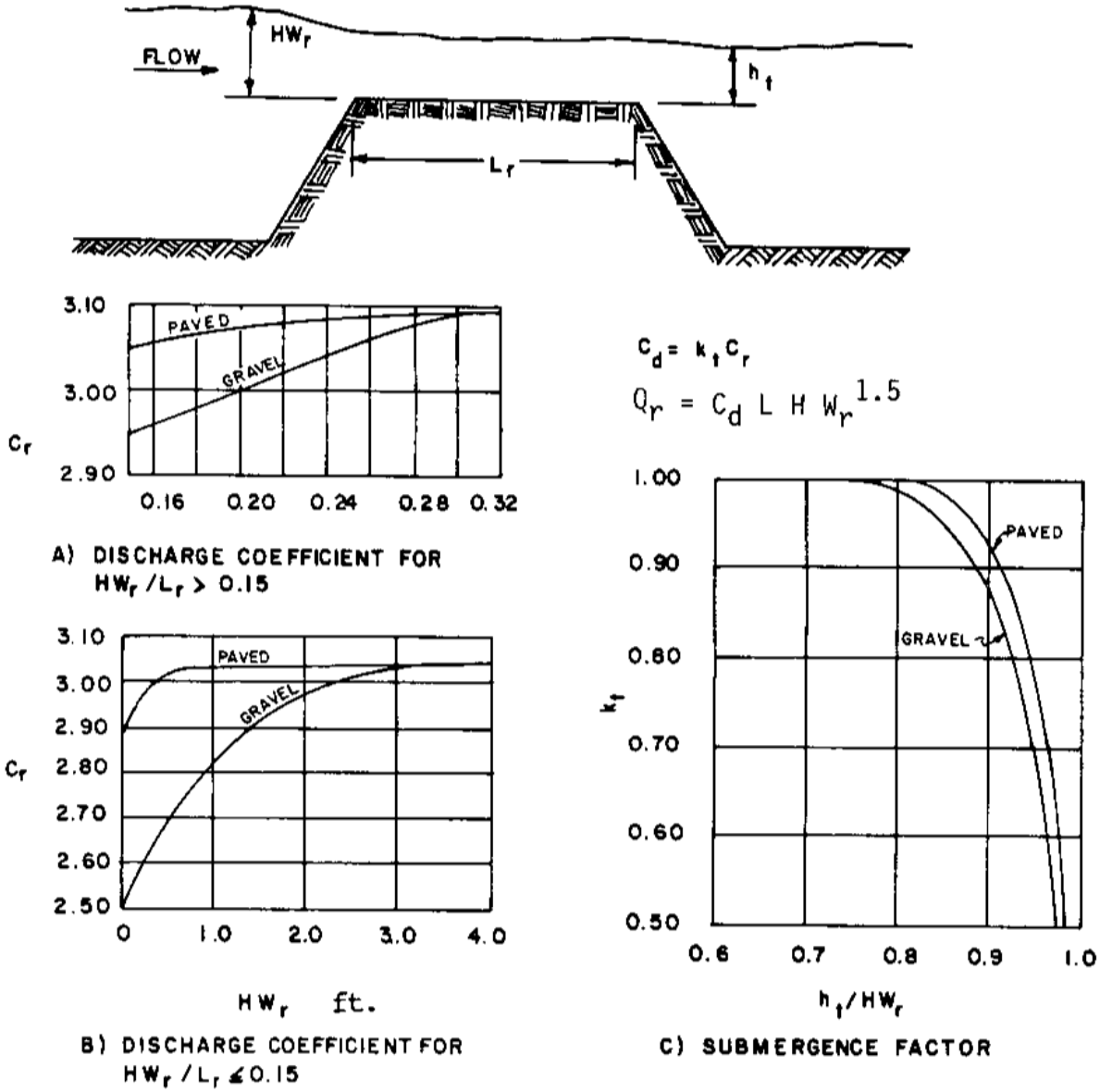
B) Discharge Coefficient for $HW_r/L_r \leq 0.15$

$C_d = k_1 Cr$
Cr = Coefficient of Free Discharge
k_1 = Adjustment Factor for Submerged Water Flow
(TW is Higher Than Roadway Elevator)
$Q_r = C_d L HW_r^{1.5}$



C) Submergence Factor

Figure 9.11(b) U.S. Customary Drainage Coefficients for Roadway Overtopping
(Source: *The Model Drainage Manual*, Chapter 9 (AASHTO, 2005))



Step 5 Add the calculated roadway overflow to the culvert flow. If this is greater than the analysis discharge, record the current culvert flow apportionment as the current upper flow limit and set the new culvert flow apportionment at a value halfway between the current upper and lower flow limits. If the calculated total is less than the analysis discharge, record the current culvert flow apportionment as the lower flow limit and set the new culvert flow apportionment at a value halfway between the current upper and lower flow limits.

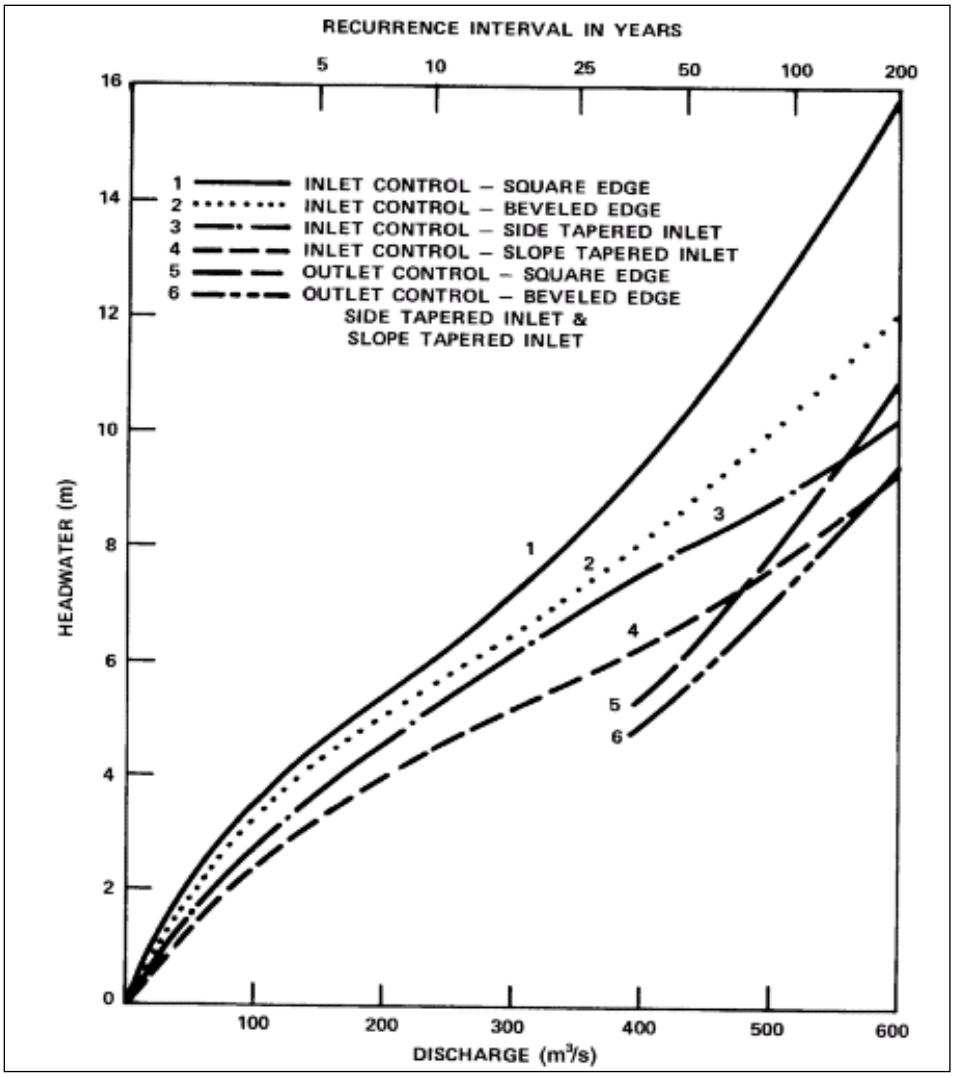
Step 6 Repeat Steps 3 to 5, using the culvert flow apportionment established in Step 5, until the difference between the current headwater and the previous headwater is less than a reasonable tolerance. For computer programs, consider a tolerance of about 3 mm (0.1 in). Consider the current headwater and current assigned culvert flow and calculated roadway overflow as the final values.

T. Performance Curves. For any given culvert, the control (outlet or inlet) might vary with the discharge. Figure 9.12 shows sample plots of headwater versus discharge for inlet and outlet control. The envelope (shown as the bold line) represents the highest value of inlet and outlet headwater for any discharge in the range. This

envelope is termed a performance curve. In this example, inlet control prevails at lower discharges and flow transitions to outlet control as the discharge increases. The flatter portion represents the effect of roadway overflow.

The performance curve is generated by performing culvert headwater computations for increasing values of discharge. Such information is particularly useful for performing risk assessments and for hydrograph routing through detention ponds and reservoirs.

Figure 9.12 Typical Performance Curve



9.3 CULVERT DESIGN/ANALYSIS PROCEDURE

A. General - Culvert Design/Analysis Procedure. The following basic steps are necessary. The procedure contained in HDS-5, *Hydraulic Design of Highway Culverts* (FHWA, 2005b), as noted below, shall be used for the design of pipe culverts:

- Define the location, orientation, shape, and material for the culvert to be designed. In many instances, more than one shape and/or material may be considered.
- With consideration of the site data, establish a maximum limit for barrel depth.
- Based upon the discharges of interest, associated tailwater levels, and allowable headwater level, define an overall culvert configuration to be analyzed (as part of the iterative design process of trial and error).
- Determine the flow type (supercritical or subcritical) to determine headwater and outlet velocity.
- Reconsider and refine the culvert configuration if necessary.
- Treat any excessive outlet velocity separately from headwater. Energy dissipators may be warranted to reduce the outlet velocity to acceptable limits.

B. Multiple Barrels. As a general rule, if there will be more than one barrel in the culvert, use shapes of uniform geometry and roughness characteristics. The flow distribution for uniform multiple barrels is then a simple equal distribution of flow through each barrel. However, if the inverts are set at different elevations, then the flow distributions are not equal and computer modeling using HY-8 (FHWA, 1997) or HEC-RAS should be used for the analysis.

C. Overview of Culvert Hydraulic Design. The procedure for culvert hydraulic design is usually an iterative process. The following must be done until all design criteria are satisfied economically:

- Analyze a trial configuration.
- Compare the results with the design criteria.
- Adjust the configuration.
- Perform another analysis.

The flow charts shown in Figures 9.13 and 9.14 provide guidance throughout the culvert design process. The flow chart may not address every nuance which the designer may encounter as the culvert is designed. However, it does lend guidance to the computational process for the vast majority of culvert design situations.

Figure 9.13 Flow Chart A – Culvert Design Procedure

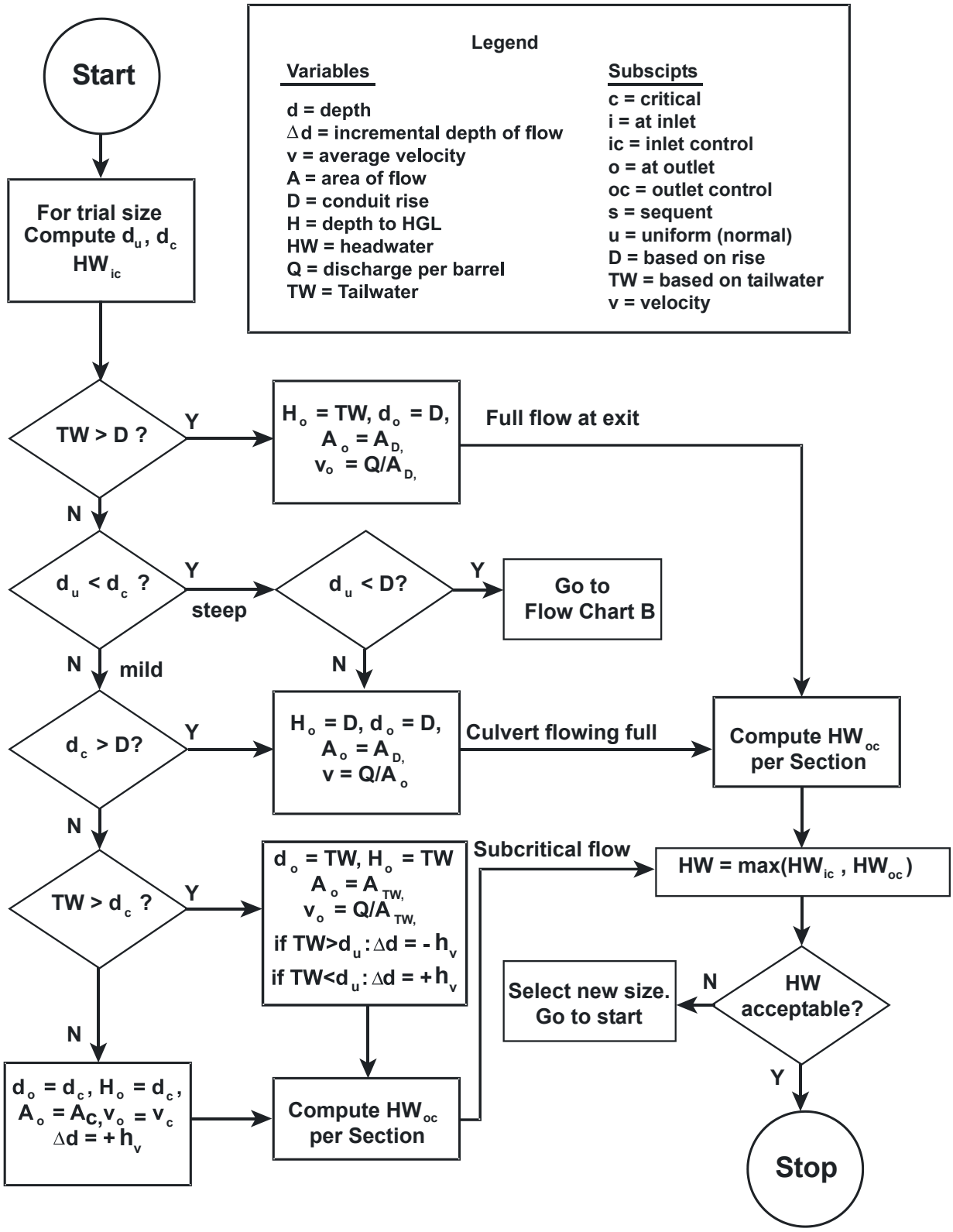
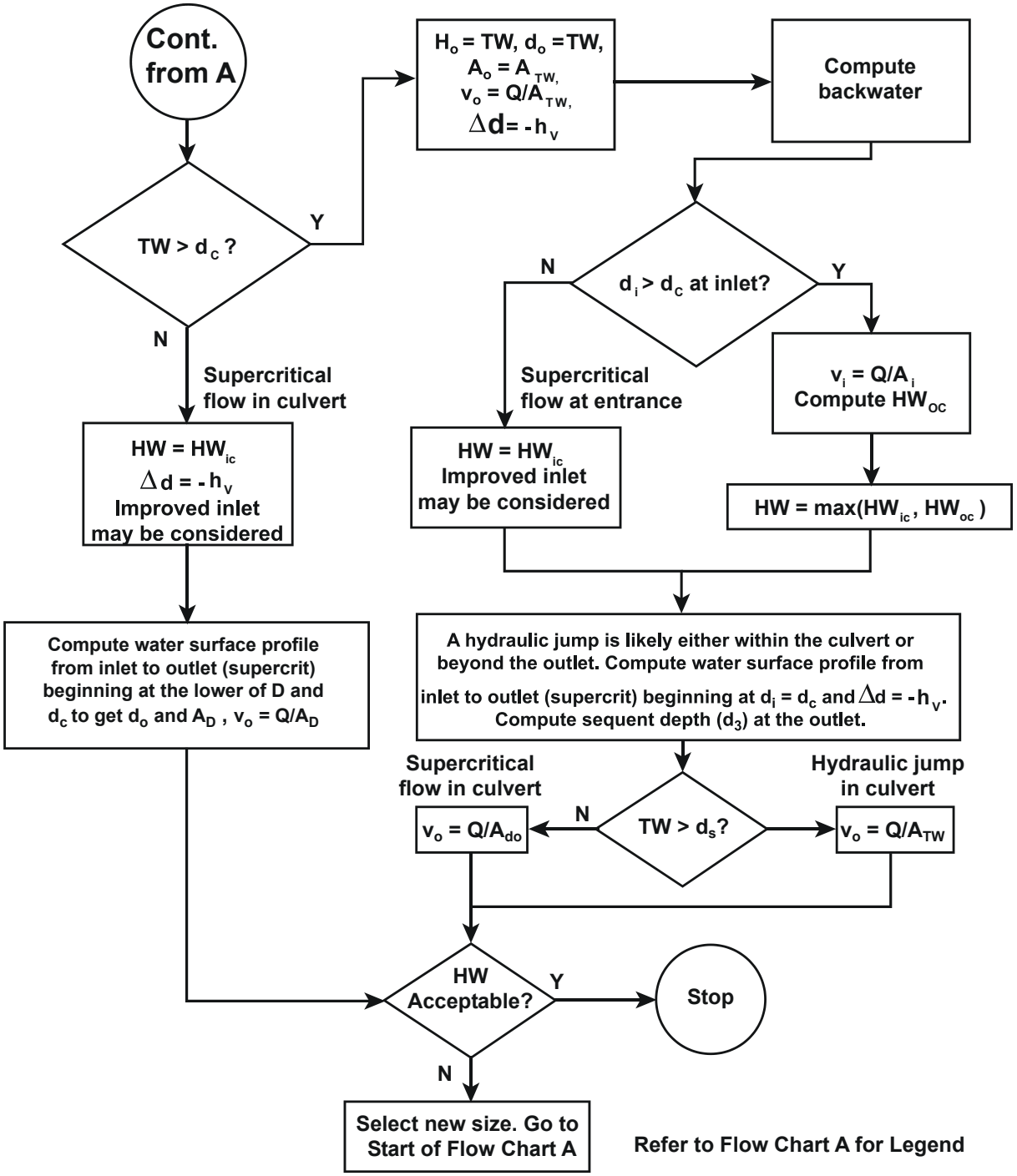


Figure 9.14 Flow Chart B – Culvert Design Procedure (continued)



While the important criterion of allowable outlet velocity is considered, it has little or no influence on the culvert barrel configuration in the design process. Any problem with excessive outlet velocity should be treated separately in most cases (e.g., the use of energy dissipators). The reader is referred to Chapter 12, *Erosion and Sediment Pollution Control* and Publication 72M, *Roadway Construction Standards* for guidance on designing outlet protection.

As mentioned previously, the designer must choose design storms for which the culvert will be analyzed. Storms with larger return periods should be checked to see if the performance of the proposed structure is within acceptable tolerance, and to check for compliance with the regulatory criteria.

D. Design Procedure for Culverts. The following is a step-by-step culvert design procedure for a standard culvert configuration. That is, straight in profile and, if multiple barrels are used, the barrels are parallel and of equal size. Variations from normal culvert design are covered elsewhere in this chapter. Any of the configurations considered in the iterative process of design will influence a unique flow type. Each new iteration requires a determination of whether there is inlet or outlet control.

1. Establish an initial trial size assuming inlet control.

Determine the maximum practical rise of culvert (D_{max}) and the maximum allowable headwater depth (HW_{max}). Determine a trial head using Equation 9.33.

(Equation 9.33)

$$h = HW_{max} - \frac{D_{max}}{2}$$

where: h = allowable effective head, m (ft)
HW_{max} = allowable headwater depth, m (ft)
D_{max} = maximum culvert rise, m (ft)

Use Equation 9.34 (a form of the orifice equation) to determine the required area, A, for the design discharge, Q. This assumes an orifice coefficient of 0.5 which is reasonable for initial estimates only.

(Equation 9.34)

$$A = k\frac{Q}{h^{0.5}}$$

where: A = approximate sectional area required, m^2 (ft^2)
Q = design discharge, m^3/s (cfs)
k = 0.45 (Metric), 0.25 (U.S. Customary)

Decide on the culvert shape.

For a box culvert:

a. Determine the required width, W, as A/D_{max}.

b. Round W to the nearest value which yields a whole multiple of standard box widths.

c. Divide W by the largest standard span S for which W is a multiple. This yields the number of barrels, N.

d. At this point, the determination has been made that the initial trial configuration will be N - S D_{max} L.

For a circular pipe culvert:

a. Determine the ratio of area required to maximum barrel area as:

(Equation 9.35)

$$\frac{4A}{\pi D_{max}^2}$$

b. Round this value to the nearest whole number to get the required number of barrels, N.

c. At this point, the determination has been made that the initial trial size culvert will be N - DL circular pipe.

For other shapes, provide an appropriate size such that the cross section area is approximately equal to A.

2. Determine the design discharge per barrel as Q/N. This assumes that all barrels are of equal size and parallel profiles. The computations precede using one barrel with the appropriate apportionment of flow.

3. Perform a hydraulic analysis of the trial configuration. Generally, the designer should employ a computer program or spreadsheet.

For the trial configuration, determine the inlet control headwater (HW_{ic}), the outlet control headwater (HW_{oc}), and outlet velocity (v_o) using Flow Chart A shown in Figure 9.13. Flow Chart A references Flow Chart B, which is shown in Figure 9.14. Table 9.6 provides references for some of the variables required.

Table 9.6. Quick References for Culvert Variables

Variable	Reference Section (s)
Critical depth (d_c)	9.2.D.
Uniform depth (d_u)	9.2.E.
Outlet depth (H_o)	9.2.J.
Outlet velocity (v_o)	9.2.M.
Full flow headwater (HW_{oc})	9.2.J.
Free surface flow headwater (HW_{oc})	9.2.K.
Inlet control headwater (HW_{ic})	9.2.H.
Friction Slope (S_f)	9.2.F.
Sequent depth (d_s)	9.2.P., 9.2.Q. & 9.2.R.

4. Evaluate trial design. At this step in the design process, a headwater and outlet velocity have been calculated for the design discharge through a trial culvert configuration.

a. If the calculated headwater is equal to or is not appreciably lower than the allowable headwater (an indication of culvert efficiency), the design is complete. A good measure of efficiency is to compare the calculated headwater with the culvert depth D. If the headwater is less than the depth, the configuration may not be efficient.

b. If the calculated headwater is considerably lower than the allowable headwater or lower than the culvert depth D, a more economical configuration may be possible. The trial culvert configuration should be changed by reducing the number of barrels, span widths, diameter, or other geometric or material changes. The calculations must then be repeated; go back to step 2.

- If the calculated headwater is equal to or is not appreciably lower than the allowable headwater, and the culvert is operating in inlet control, an improved inlet may be in order.
- If the operation is not inlet control, then the culvert geometry design is complete.
- If the calculated headwater is greater than the allowable headwater, the trial culvert configuration should be changed to increase capacity by adding barrels, widening spans, and increasing diameter. Regardless of the changes made here, the calculations must be repeated. Go back to Step 2.

If the culvert is operating with inlet control, the possibility exists for improving the entrance conditions with the aim of reducing the overall cost of the structure. This may be done by investigating the design of a flared (or tapered) inlet and associated structure. The design procedures for improved inlets are discussed in Section 9.4, Improved Inlets.

Due to the cost of the improved inlet, a careful economic comparison should be made between the design with a normal entrance and the design with an improved inlet.

The culvert for which the calculated headwater is satisfactory may have an excessive outlet velocity. The definition of an "excessive" outlet velocity is normally an engineering judgment based on local conditions (HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, FHWA, 2006).

In comparison to adjusting the culvert barrel configuration, it is usually more economical to provide riprap, sills, or a stilling basin at the outlet end to control any excessive velocity.

Any outlet control or protective device which may be required is considered part of the hydraulic design of the culvert. It is normal for a properly designed culvert to have an outlet velocity which is greater than the natural stream velocity (see Section 9.5).

5. Develop a performance curve and check the 100-year flood conditions using the procedures outlined in Section 9.2.

An overall hydraulic performance analysis for the designed culvert will indicate headwater and outlet velocity characteristics for a wide range of discharges. Through analysis, consider applying potential headwater levels and outlet velocities with respect to their probabilities of occurrence to the designed culvert. Such application can lead to a determination of potential risks and costs associated with the long-term operation of the culvert.

For any culvert design, the minimum additional analysis required will be the application of the 100-year discharge to the culvert. The design is considered complete if the results of the headwater and outlet velocity represent an acceptable risk and conform with regulatory requirements.

A performance curve can be used for further evaluation of culvert risks. Culvert performance analysis consists of a definition of headwater and outlet velocity for each of a variety of discharges.

If a performance curve is desired, apply a range of discharges both lower and greater than the design discharge to make this determination. For example, if the culvert was designed for the 50-year discharge, apply the 5-, 10-, 25- and 100-year discharges with the corresponding tailwater for each discharge. With each of these discharges and tailwaters, determine the resulting headwater level and outlet velocity. A tabulation or plot of the resulting performance curves then can be made.

Life cycle costs generally are not needed to determine cost-effective designs. Culverts are expected to last without maintenance for the design life specified. However, the following long-term cost components (reduced to an annual cost on the basis of the anticipated service life) may be considered if a life cycle cost analysis is requested under an unusual circumstance.

- Initial cost of the culvert.
- Expected cost of damage to the roadway.
- Expected cost of damage to the culvert and associated appurtenances.
- Expected cost of damage to the stream (approach and exit).
- Expected cost of damage to upstream and downstream private or public property.
- Expected cost of traffic detours.
- Debris removal.

The cost of traffic detours may be the most important since the extension of detoured vehicles, detour distance, and cost of operation per vehicle mile can amount to extremely heavy expense. This is particularly true if there is a large average daily traffic rate.

9.4 IMPROVED INLETS

A. Conditions for Improved Inlets. If the culvert is operating under inlet control, it may be economical to apply an improved inlet. An improved inlet serves to "funnel" the flow into the culvert so that the point of control is removed from the face of the inlet to a throat located downstream from the face. The normal contraction of flow is included in the transition from the face to the throat of the inlet. If the culvert is operating under outlet control, side tapered and slope tapered improved inlets are not effective and should generally not be considered. However, beveled inlets can be useful in the circumstance of outlet control.

Several factors must be weighed in the use of improved inlets:

- Improved inlets offer the potential advantage of increasing the capacity of existing inadequate culverts. This is important where an inlet control culvert serves a watershed which has changed in character from rural to urban, or otherwise has experienced an increase in the design discharge rate.
- For inlet control, the improved inlet design process typically will allow a reduction in the culvert size. It is important to check that this reduction in culvert size does not cause the culvert to revert to outlet control.
- Improved inlets usually are costly; therefore, it is important to develop a complete economic analysis of such designs. The analysis should weigh the additional costs associated with the installation and maintenance of the improved inlet against the savings affected by reduced culvert size.
- Available design procedures require improved inlets to be placed with the face normal to the direction of flow. That is, the procedures do not accommodate improved inlets when the face of the inlet is skewed relative to the stream channel.
- Where heavy debris loads are anticipated, improved inlets can become serious maintenance problems. This is because of a tendency for some debris to pass the face of the inlet and become lodged in the throat.

The typical types of improved inlets are:

- Beveled edges.
- Depth transition.
- Side-tapered inlet.
- Slope-tapered inlet.

For design procedures for improved inlets, refer to HDS-5, *Hydraulic Design of Highway Culverts*, (FHWA, 2005b).

B. Beveled Edges. Beveled edges effectively reduce the contraction downstream of the culvert face, resulting in a more efficient conveyance of water by the available barrel area (see Figure 9.15). Generally, little or no enlargement of the culvert inlet is required to gain the hydraulic advantage of beveled edges; thus, structural problems are minor. Beveled edges may be implemented at little additional expense and are effective for culverts operating under either inlet or outlet control. The effect of beveled edges can be a significant improvement in culvert capacity and/or reductions in the subtended headwater. They may be easily adapted to either pipe or box culverts. Table 9.4 provides polynomial coefficients for some beveled entrance conditions for use in Equation 9.13 (inlet control headwater).

Figure 9.15 Beveled Entrance



C. Top-Tapered Transition. A simple transition of depth in a rectangular box culvert may improve the hydraulic efficiency. If the box culvert is operating under inlet control, it follows that the barrel of the culvert is more hydraulically efficient than the entrance geometry. The barrel depth may be reduced in the transition from the original depth of the inlet to a minimum of 0.3 m (1 ft) greater than the uniform depth of flow. The transition length should be a minimum of 6 m (20 ft) (see Figure 9.16).

Figure 9.16 Top-tapered Box Culvert



This method is arbitrary and should be used carefully only when the culvert is definitely operating in inlet control. In terms of design and construction, the method is effective, economical, and simple to perform.

This method may be preferred when designing a multiple barrel box culvert. Other inlet improvement methods are not feasible for multiple barrel box culverts because of the need to taper or flare the side walls of the barrels.

D. Side-Tapered Inlet. Side-tapered inlets involve a widening of the face area of the culvert by tapering the sidewalls. Such inlets have two possible control sections: the face and the throat. Control should be maintained at the throat for design discharge in order to realize significant cost savings in the culvert barrel. This type of improvement is similar in operation to the flared inlet for pipes discussed in the segments that follow.

E. Slope-Tapered Inlet. The slope-tapered inlet incorporates the efficient flow characteristics of side-tapered inlets with a concentration of more of the total available culvert fall at the throat control section. Slope-tapered improvements are not practical for pipe culverts because of their complexity and lack of availability.

Some of the drawbacks of slope-tapered inlets are described as follows:

- Slope-tapered inlets have a tendency to allow sediment deposition. This can result in maintenance problems.
- Slope-tapered inlets imply a reduced slope of the culvert. A reduced slope often leads to a change of hydraulic flow type in the culvert from supercritical to subcritical. In such cases, the application of the improved inlet may be ineffective.
- Because of the lowering of the upstream end of the culvert, the use of slope-tapered inlets can result in increased costs of structural excavation.

F. Flared Entrance Design for Circular Pipe. In certain instances, if a circular pipe culvert of sufficient barrel length is operating under inlet control, a flared entrance as an inlet improvement may serve to increase the hydraulic capacity with a corresponding savings in the initial cost of the culvert barrel. A sufficient barrel length would be such that the reduced cost of the smaller diameter barrel more than offsets the additional cost of the flared inlet or pipe liner.

A flared entrance for a pipe culvert is practical only when steep slope, inlet control conditions exist.

For any circular pipe culvert operating under inlet control, there is a possible use for a flared inlet to reduce the size of the barrel. The design procedure outlined in Section 9.3.D. is applicable. For the inlet control headwater, Table 9.5 provides coefficients for concrete and corrugated metal circular pipe with flared inlets for use in Equation 9.20. The following conditions should be noted:

- If the culvert is on a mild slope ($d_c < d_u$), a flared inlet is not likely to be effective.
- If the inlet analysis procedure indicates outlet control, a flared inlet is not an efficient application.
- Trial size is verified when all three of the following criteria are met:
    - $HW_{ic} < AHW$
    - $d_u < d_c$
    - $HW_{oc} < HW_{ic}$
- If the trial size is verified, compare costs with a culvert designed without a flared inlet. Calculate the culvert outlet velocity in accordance with the procedure outlined for an inlet control culvert.
- The flared inlet unit should not be cut to a skew even if the culvert is skewed with respect to the roadway.
- If the trial size is not verified, then simply design the culvert without a flared inlet in accordance with the usual procedure given in Section 9.3.D.

9.5 VELOCITY PROTECTION AND CONTROL DEVICES

A. General - Velocity Protection and Control Devices. While allowable headwater influences the overall configuration of the culvert, the allowable outlet velocity is the governing criterion in the selection and application of various downstream fixtures and appurtenances.

If the designer considers the outlet velocity to be excessive, several possible solutions which may minimize the negative effects of velocity are available. The excessive velocity may be accommodated, reduced, or controlled.

- Minor configuration changes in the culvert barrel may reduce an excessive velocity to a more acceptable exit velocity. For situations involving excessive outlet velocities in culverts operating under inlet control, it is possible to roughen the culvert or even change geometry of the culvert and yet not affect the headwater characteristics.

- Historically, the most widely used control has been the use of riprap which covers the channel area immediately downstream from the culvert outlet.
- An efficient, but usually expensive, countermeasure is an energy dissipator. Some energy dissipators have an analytical basis for design, while others are intended to cause turbulence in unpredictable ways. With turbulence in flow, energy is dissipated and velocity can be reduced.
- Certain special culvert types may be used successfully to minimize excessive outlet velocities. One such special culvert type is the broken-back culvert.

Velocity control appurtenances for culverts may be classified broadly as either protection devices or control devices.

B. Velocity Protection Devices. A velocity protection device does not necessarily reduce excessive velocity but does protect threatened features from damage. Such devices usually are economical and effective in that they serve to provide protection from the flow for specific sensitive features.

1. Riprap. Riprap, when used as an outlet velocity protection measure, should be applied to the channel area immediately downstream of the culvert outlet for some distance, possibly to the edge of right-of-way. This limit may be tempered by engineering judgment based on the severity of the velocity and the potential for erosion or scour. Rip-rap is rock of various sizes as specified in Section 850 of Publication 408, *Highway Specifications*.

2. Pre-formed Outlet. A very effective protection device consists of a pre-formed scour hole in the area threatened by excessive outlet velocities. These holes should be lined with some type of riprap.

3. Channel Recovery Reach. Similar to a pre-formed outlet, a channel recovery reach provides a means for the flow to return to an equilibrium state within the natural, unconstricted stream channel. The recovery reach should be well protected against the threat of scour or other damage.

C. Velocity Control Devices. A velocity control device serves to effectively reduce an excessive culvert velocity to an acceptable level. The design of some control devices is based on theoretical analysis while, for others, the specific control may be unpredictable. In increasing order of their probable expense, some velocity control devices are:

- Natural hydraulic jumps (most control devices are intended to force a hydraulic jump).
- Sills.
- Roughness baffles.
- Impact basins.
- Stilling basins.
- Specialized energy dissipators.

Most velocity control devices rely on the establishment of a hydraulic jump. Since an excessive outlet velocity from a culvert usually is the result of the culvert being on a relatively steep slope, the depth downstream of the culvert exit usually is not great enough to induce a hydraulic jump. However, some mechanisms may be available to provide a simulation of a greater depth necessary to create a natural hydraulic jump.

D. Other Control Devices. Other controls are described in the FHWA publication, HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, 2006).

9.6 SPECIAL HYDRAULIC CONSIDERATIONS

In addition to the hydraulic considerations discussed in the preceding sections, other factors may be considered in order to assure the integrity of culvert installations and the highway.

A. Anchorage. The forces acting on a culvert inlet during high flows are variable and highly indeterminate. Vortices and eddy currents cause scour which can undermine the culvert inlet, erode the embankment slope and make the inlet vulnerable to failure. Flow is usually constricted at the inlet, and inlet damage (Figure 9.17) or lodged drift can accentuate this constriction. The large unequal pressures resulting from this constriction are, in

effect, buoyant forces which can cause entrance failures, particularly on a corrugated metal pipe with mitered, skewed or projecting ends.

Scour for a particular drainage feature or constriction may be assessed using the guidance provided in BD-632M, BD-633M and RC-30M.

Figure 9.17 Damage to Culvert Inlets Due to Hydraulic Forces and Drift



Anchorage at the culvert entrance helps to protect against these failures by increasing the dead load on the end of the culvert, thus protecting against bending damage, and by protecting the fill slope from the scouring action of the flow. End anchorage can be in the form of slope paving, concrete headwalls, or grouted stone, but the culvert end must be anchored to the end treatment to be effective. In some locations, prefabricated metal end sections also should be anchored to increase their resistance to failure.

Culvert ends need anchorage at many locations. Sectional rigid pipe is susceptible to separation at the joints when scour undermines the ends. Commercially available tiebars can be used to prevent separation of concrete pipe joints. Metal culvert ends projected into ponds, tidal waters or through levees are susceptible to failure from buoyant forces if tide gates are used or if the ends are damaged by debris. Figure 9.18 shows a culvert which failed from buoyant forces at the inlet end.

Figure 9.18 Culvert and Roadway Fill Failure from Buoyant Forces: Culvert Carried Downstream



B. Piping. Piping is a phenomenon caused by seepage along a culvert barrel which removes fill material, forming a void similar to a pipe, hence the term "piping" (Figure 9.19). Fine soil particles are washed out freely along the void, and the erosion inside the fill ultimately may cause failure of the culvert or the embankment. Piping also may occur through open joints into the culvert barrel.

The possibility of piping can be reduced by decreasing the velocity of the seepage or by decreasing the quantity of seepage flow. Methods of achieving these objectives are discussed in the following sections.

Figure 9.19 Void From Piping Along Culvert Barrel



C. Joints. In order to decrease the velocity of the seepage flow, it is necessary to increase the length of the flow path and thus decrease the hydraulic gradient. The most direct flow path for seepage and thus the highest hydraulic gradient is through open pipe joints. Therefore, it is important that culvert joints be as watertight as practical. If piping through joints could become a problem, flexible, long-lasting joints should be specified as opposed to mortar joints.

D. Anti-seep Collars. Piping should be anticipated along the entire length of the culvert when ponding above the culvert is planned. Anti-seep or cutoff collars increase the length of the flow path, while decreasing the hydraulic gradient and the velocity of flow, and thus the probability of pipe formation. Anti-seep collars usually consist of bulkhead type plates or blocks around the entire perimeter of the culvert. They may be of metal or of reinforced concrete, and, if practical, dimensions should be sufficient to key into impervious material. Figure 9.20 shows anti-seep collars installed on a culvert under construction.

Figure 9.20 Anti-seep Collars



E. Weep Holes. Weep holes sometimes are used to relieve uplift pressure. Filter materials should be used in conjunction with the weep holes in order to intercept the flow and to prevent the formation of piping channels. The filter materials should be designed as underdrain filter so they will not become clogged and so piping cannot occur through the pervious material and the weep hole. Geotextile filter material should be placed over the weep hole in order to keep the pervious material from being carried into the culvert.

Weep holes may not be required in culverts, and their use is becoming less prevalent. For guidance on when to use weep holes, see Design Manual Part 4 and BD-632M. If additional drainage of the fill behind the culvert wall is believed necessary, a separate underdrain system may be installed.

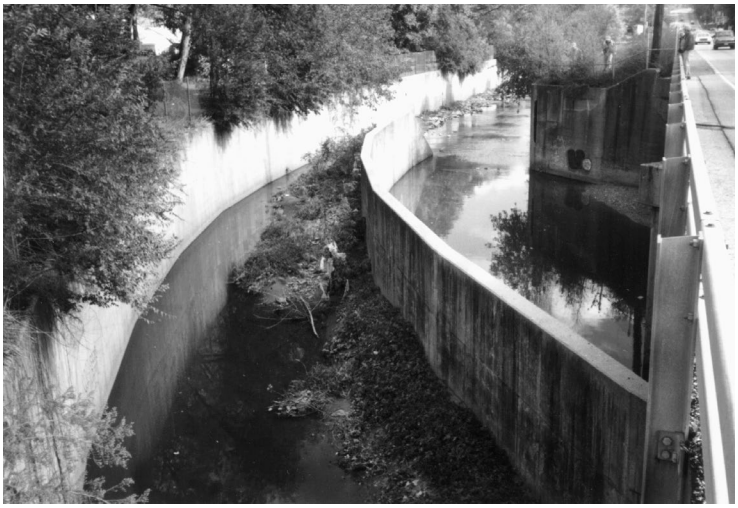
F. Junctions and Bifurcations. It sometimes is necessary to combine the flow of two culverts into a single barrel. The junction should be designed so that a minimum amount of turbulence and adverse effect on each branch will result. This is accomplished by considering the flow momentum in each branch and other variables such as the timing of peak flows, e.g., low flow in one branch and high flow in the other. Supercritical flow velocities add to the complexity of the problem. Chow (1970) and Behlke and Pritchett (1966) address the subject of junctions for supercritical flow. In critical locations, laboratory verification of junction design is advisable.

If a bifurcation in flow is necessary or desirable, it is suggested that the flow division be accomplished outside the culvert barrel. Problems with clogging by debris and the desired proportioning of flow between branches can be handled much more easily outside of the culvert.

G. Training Walls. Where supercritical flow conditions prevail in a curved approach to a culvert, training walls may be needed to align flow with the culvert inlet and to equalize flow rates in the barrels of multiple barrel culverts. In locations where overtopping of the channel or culvert, or inefficient operation, could result in catastrophic failure, laboratory verification of the training wall design is advisable.

Training walls also may be required at culvert outlets to align flow with the downstream channel if this alignment cannot be accomplished in the culvert barrel. Design of the training wall shown in Figure 9.21 was verified by laboratory testing, and the wall has been proven by operation during floods.

Figure 9.21 Training Walls



H. Sag Culverts. A sag culvert, often called an inverted siphon, is not a siphon because the pressure in the barrel is not below atmospheric. Sag culverts of pipe or box section are used extensively to carry irrigation water under highways. They are used infrequently for highway drainage and should be avoided on intermittent or alluvial streams because of problems with siltation and stagnation.

Hydraulically, a sag culvert operates with outlet control, and losses through the culvert can be computed by the procedures used for conventional culverts.

I. Irregular Alignment. At some locations, it may be desirable to incorporate bends, either in plan or profile, in the culvert alignment. When irregular alignment is advisable or desirable, bends should be as gradual and as uniform as is practical to fit site conditions. Changes in alignment may be accomplished either by curves or angular bends. When large changes are necessary, mild bends, e.g., 15° at intervals of 15 m (50 ft), should be used. Passage of debris should be considered in selecting the angle, interval and number of bends used to accomplish the change in alignment.

If the culvert operates with inlet control, bend losses do not enter into the headwater computation. If it operates with outlet control, typically, bend losses will be small. In critical locations, they should be calculated and added to the usual losses. Bend loss coefficients can be calculated using Equation 9.36. More information on bend losses can be found in Bureau of Reclamation (1987) and AASHTO (2005) publications.

(Equation 9.36)

$$h_b = 0.0033(\Delta)\left(\frac{V_o^2}{2g}\right)$$

where: h_b = bend loss coefficient
Δ = angle of curvature, Degrees
V_o = initial velocity

J. Cavitation. The phenomenon known as cavitation occurs as a result of local velocity changes at surface irregularities which reduce the pressure to the vapor limit of the liquid. Tiny vapor bubbles form at the point of lowest pressure and are carried downstream into a zone of higher pressure where they collapse. As the countless bubbles collapse, extremely high local pressure is transmitted radially outward at the speed of sound, followed by a negative pressure wave which may lead to a repetition of the cycle. Boundary materials in the vicinity are subjected to rapidly repeated stress reversals and may fail through fatigue (Rouse, 1949). Surface pitting is the first sign of such a failure.

Cavitation is seldom a problem in highway culverts because of relatively low velocities and because flow rates are not sustained for a long period. Abrasion damage sometimes is mistaken for cavitation damage.

K. Tidal Effects and Flood Protection. Where areas draining through culverts are affected by tide or flood stages, flap gates may be desirable to prevent backflow. Sand, silt, debris or ice will cause these gates to require considerable maintenance to keep them operative. Head losses due to the operation of flap gates may be computed using loss coefficients furnished by the manufacturer.

9.7 MULTIPLE USE CULVERTS

Culverts often serve purposes in addition to drainage. There are cost advantages of multiple use culverts, but one purpose or the other is often served inadequately. The cost advantages of multiple use culverts should be weighed against the possible advantages of separate facilities for each use.

A. Utilities. It is sometimes convenient to locate utilities in culverts, particularly if jacking, boring or an open cut through an existing highway can be avoided by such a location. The space occupied in the culvert is usually relatively small, and the obvious effects on culvert hydraulic performance can be insignificant. Consideration of this multiple use, however, should include recognition of the flood flow and debris hazard to the utility and the probability of reduced culvert capacity from debris caught on the utility line. Also, increased stream scour often occurs at pipelines at the upstream and downstream ends of culverts. This multiple use generally is not suggested if separate facilities are practicable.

B. Stock and Wildlife Passage. Culverts can serve both for drainage and for stock and wildlife passes. Culvert size may be determined either by hydraulic requirements or by criteria established for the accommodation of the stock or game which will use the structure.

C. Land Access. Culverts often serve both as a means of land access and drainage, particularly on highways with controlled access. This use is common in areas where land use on both sides of the highway is under common control. The culvert size generally will be determined by the physical dimensions of the equipment or vehicles which will make use of the facility. Scour protection not considered necessary for hydraulic reasons may be required at the outlet to facilitate access to the culvert. Where a low flow culvert is placed at a lower elevation than the multiple use culvert, precautions against headcutting from the stream to the outlet of the multiple use culvert may be necessary. Good drainage at the culvert ends is necessary for culverts used for land access.

D. Fish Passage. Many resource agencies have established design criteria for fish passage through culverts. These include maximum allowable velocity, minimum water depth, maximum culvert length and gradient, type of structure, and construction scheduling.

Several types of culvert installations have been used satisfactorily for fish passage. These include:

- Open Bottom Culverts. Culverts supported on spread footings to permit retention of the natural stream bed. The culvert size must be adequate to maintain natural stream velocities at moderate flows, and the foundation must be in rock or scour resistant material (Figure 9.22).

Figure 9.22 Culvert on Spread Footings to Retain Streambed for Fish Passage



- Oversized or Countersunk Culverts. Oversized culverts with the bottom of the culvert placed below the stream bed so that gravel will deposit and develop a nearly natural stream bed within the culvert (Figure 9.23). Sometimes, baffles are necessary to hold gravel and rock in place, especially at stream grades > 4%.

Figure 9.23 Culvert Invert Placed Below Streambed
Baffles used to hold gravel in place and provide natural stream bed for fish passage



- Special Treatment. In wide, shallow streams, one barrel of a multiple-barrel culvert can be depressed to carry low flow, or weirs can be installed at the upstream end of some barrels to provide for fish passage through other barrels at low flow.

- Timing. When fish passage is required, consideration must be given to the time of the year that the culvert will be installed. Fishery agencies usually will provide dates when spawning will occur in order to limit stream disturbance during this period.

For additional information regarding the design of fish passages, refer to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.11.

9.8 DEBRIS CONTROL

Accumulation of debris at a culvert inlet can result in the culvert not performing as designed. The consequences may be damages from inundation of the road and upstream property.

The designer has three options for coping with the debris problem: retain the debris upstream of the culvert, attempt to pass debris through the culvert, or use a bridge (Reihsen & Harrison, 1971).

If the debris is to be retained by an upstream structure or at the culvert inlet, frequent maintenance may be required. If debris is to be passed through the structure or retained at the inlet, a relief opening should be considered, either in the form of a vertical riser or a relief culvert placed higher in the embankment (Figure 9.24).

Figure 9.24 Vertical Riser for Relief



It often is more economical to construct debris control structures after problems develop since debris problems do not occur at all suspected locations.

A. Debris Control Structure Design. The design of a debris control structure must be preceded by a thorough study of the debris problem. Among the factors to be considered are:

- Type of debris.
- Quantity of debris.
- Expected changes in type and quantity of debris due to future land use.
- Streamflow velocity in the vicinity of the culvert entrance.
- Access requirements for maintenance.
- Availability of flow storage area and volume.
- Annual maintenance plan including debris removal.
- Assessment of damage due to debris clogging, if protection is not provided.

For more information on debris control structures, refer to HEC-9, *Debris Control Structures Evaluation and Countermeasures* (FHWA, 2005a).

B. Maintenance. Provisions for maintenance access are necessary for debris control structures. For high embankments, this may be difficult. If access to the debris control structure is not practical, a parking area for mechanical equipment, such as a crane, may be necessary in order to remove debris without disrupting traffic.

Many debris barriers require cleaning after every storm. The frequency of maintenance should be considered in selecting the debris control structure. If a low standard of maintenance is anticipated, the designer should choose to pass the debris through the structure.

9.9 SERVICE LIFE

Commonly used culvert materials are durable at most locations, but some soil and water environments are hostile. Service life must be a consideration in material selection and culvert design. Processes which affect the service life of culvert materials are corrosion, abrasion, and freeze-thaw cycles. Measures to increase service life are sometimes costly. The total expected annual cost should be considered when designs are prepared. Periodic culvert replacement may be a practical alternative. Driveway culverts, for instance, are generally easy to replace, and traffic service would not be a problem when replacement becomes necessary. Culverts under highways with high traffic volumes or under high fills are more difficult and costly to repair or replace. In such cases, more precaution against failure from a hostile environment is warranted.

Many of the conditions which affect service life can be evaluated and service life estimated prior to the selection of culvert material. The type and degree of protection needed can then be determined using Beaton & Stratfull (1962); Berg (1965); Braley (1951); FHWA (1991); Haviland et al., (1968); Lowe & Koeph (1964); Nordin & Stratfull, (1965); and, Peterson (1973). One of the most reliable methods available to the designer is to examine existing culverts in the same stream channel or in similar streams in the same area.

Additional information pertaining to service life of materials are provided in PennDOT's Design Manual publications.

A. Abrasion. Abrasion is the erosion of culvert materials by the bed load carried by streams (Figure 9.25). The principal factors to be considered are the frequency and duration of runoff events which transport significant amounts of abrasive materials, the character and volume of the bed load, and the resistance of the culvert material to abrasion. In some locations, culverts can be protected from abrasion by use of debris control structures to remove the abrasive sediment load from the flow.

Provision for abrasive wear can be made by the use of invert paving. The paving is to be reinforced and must be at least 5 cm (3 in) thick if it is to be counted on for increased durability.

Figure 9.25 Loss of Culvert Material from Abrasion

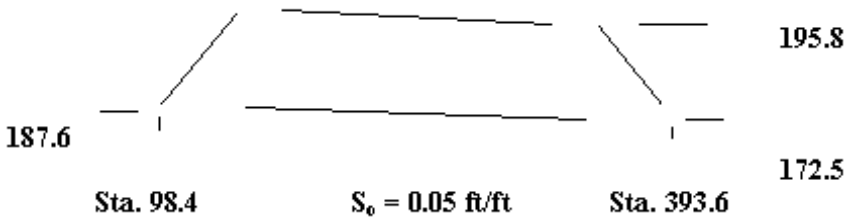


9.10 EXAMPLE

The following example problem follows the Design Procedure Steps:

Step 1 Assemble Site Data and Project File

1. Site survey Project file contains:
- USGS, site and location maps.
- Roadway profile.
- Embankment cross section.



Site visit notes indicate:
- No sediment or debris problems.
- No nearby structures.

2. Studies by other agencies — none

3. Environmental, risk assessment shows:
- No buildings near floodplain.
- No sensitive floodplain values.
- No FEMA involvement.
- Convenient detours exist.

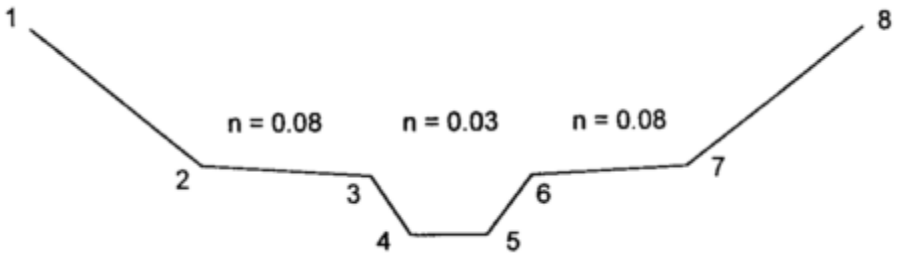
4. Design criteria:
- 50-year frequency for design.
- 100-year frequency for check.

Step 2 Determine Hydrology

1. USGS Regression equations yield:
- Q_{50} = 11.33 m^3/s (400 cfs)
- Q_{100} = 14.16 m^3/s (500 cfs)

Step 3 Design Downstream Channel

Cross section of channel (Slope = 0.05 ft/ft)



Point	Station, m (ft)	Elevation, m (ft)
1	3.7 (12.1)	54.86 (180.0)
2	6.7 (21.9)	53.34 (175.0)
3	9.8 (32.1)	53.19 (174.5)
4	10.4 (34.1)	52.58 (171.5)
5	11.9 (39.0)	52.58 (171.5)
6	12.5 (41.0)	53.19 (174.5)
7	15.5 (50.8)	53.34 (175.0)
8	18.6 (61.0)	54.86 (180.0)

The rating curve for the channel calculated by normal depth yields:

Q, m^3/s (cfs)	TW, m (ft)	V, m/s (ft/s)
2.83 (100)	0.43 (1.41)	3.39 (11.1)
5.66 (200)	0.63 (2.06)	4.18 (13.7)
8.50 (300)	0.76 (2.50)	4.87 (15.9)
11.33 (400)	0.85 (2.79)	5.34 (17.5)
14.16 (500)	0.93 (3.05)	5.73 (18.8)

Step 4 Summarize Data On Design Form

(See Figure 9.26)

Step 5 Select Design Alternative

Shape: box Size: 2135 mm x 1830 mm (7ft x 6 ft)
Material: concrete Entrance: beveled

Step 6 Select Design Discharge

$Q_d = Q_{50} = 11.33\ m^3/s$ (400 cfs)

Step 7 Determine Inlet Control Headwater Depth (HW_i)

Use inlet control nomograph - Chart 10

1. D = 1.83 m (6.0 ft)
2. Q/B = 11.33/2.13 = 5.32 m^2/s (400 / 7 = 57.1 ft^2/s)
3. HW/D = 1.33 for 19 mm (0.75 in) chamfer
HW/D = 1.27 for 45 bevel
4. HW_i = (HW/D)D = (1.27) 1.83 = 2.32 m ((1.67) 6.0 = 7.62 ft) (Neglect the approach velocity.)

Step 8 Determine Outlet Control Headwater Depth at Inlet (HW_{oi})

1. TW = 0.85 m (2.79 ft) for Q_{50} = 11.33 m^3/s (400 cfs)
2. d_c = 1.43 m (4.70 ft) from Chart 14
3. $(d_c + D)/2$ = (1.43m + 1.83m)/2 = 1.63 m, ((4.70 + 6.0)/2 =5.35 ft)
4. h_o = the larger of TW or $(d_c + D/2)$
$h_o = (d_c + D)/2$ = 1.63 m (5.35 ft)
5. K_E = 0.2 from Table 2
6. Determine (H) - use Chart 15
- K_E scale = 0.2
- culvert length (L) = 90 m (295 ft)
n = 0.012 same as on chart
- area = 3.90 m^2 (41.95 ft^2)
- H = 0.85 m (2.79 ft)

7. $HW_{oi} = H + h_o - S_oL$ = 0.85 + 1.63 - (0.05) 90 = -2.02 m (2.79 + 5.35 - (0.05) 295 = -6.61 ft)
HW_{oi} is less than 1.2D, but control is inlet control. Outlet control computations are for comparison only.

Step 9 Determine Controlling Headwater (HW_c)

- $HW_c = HW_i$ = 2.32 m (7.60 ft) > HW_{oi} = -2.02 m (-6.61 ft)
- The culvert is in inlet control.

Step 10 Compute Discharge over the Roadway (Q_r)

1. Calculate depth above the roadway:
$HW_r = HW_c - HW_{ov}$ = 7.61 - 8.50 = -0.27 m (-0.89 ft)
2. If $HW_r = 0$, $Q_r = 0$

Step 11 Compute Total Discharge (Q_t)

$Q_t = Q_d + Q_r$ = 11.33 m^3/s (400 cfs) + 0 = 11.33 m^3/s (400 cfs)

Step 12 Calculate Outlet Velocity (V_o) and Depth (d_n)

INLET CONTROL

1. Calculate normal depth (d_n):

$$Q = (1.00/n)\, A\, R^{2/3}\, S^{1/2} = 11.33\ m^3/s\ (400\ cfs) \quad (1)$$
$$= (1.00/0.012)(7.0\times d_n)[7.0\times d_n/(7.0+2d_n)]^{2/3}(0.05)^{0.5} = 11.33 m^3/s\ (400\ cfs) \quad (2)$$

After rearranging Equation (2) above,

$(7.0\times d_n)[7.0\times d_n/(7.0+2d_n)]^{2/3}$ = .608 m (14.40 ft)
Try d_n = .60 m (1.96 ft), .675 (15.98) > .608 m (14.40 ft)
Use d_n = .55 m (1.80 ft), .596 (14.2) ≈ .608 m (14.40 ft)

2. A = (2.13m)(0.55m), (7ft)(1.80ft) = 1.17 m^2 (12.6 ft^2)

3. $V_o = Q/A$ = 11.33m^3/s /1.17m^2 = 9.68 m/s, (400/12.6 = 31.75 ft/s)

Step 13 Review Results

Compare alternative design with constraints and assumptions. If any of the following are exceeded, repeat steps 5 through 12:

- Barrel has 2.59-1.83 = 0.76 m, (8.5-6.0 = 2.5 ft) of cover.

- L = 90 m (295 ft) is OK, since inlet control.
- Headwalls and wingwalls fit site.
- Allowable headwater (2.59 m), (8.50 ft) > 2.32 m (7.60 ft) is OK.
- Overtopping flood frequency > 50-year.

Step 14 Plot Performance Curve

Use Q_{100} for the upper limit. Steps 6 through 12 should be repeated for each discharge used to plot the performance curve. These computations are provided on the computation form, (see Figure 9.17).

- No flow routing, a small upstream headwater pool exists.
- Consider energy dissipators since V_o = 9.68 m/s (31.75 ft/s) > 5.34 m/s (17.5 ft/s) in the downstream channel.
- No sediment problem.
- No fishery.

Step 15 Documentation

Report prepared and background filed.

Figure 9.26 Chart 17 and Performance Curve for Design Example (Metric)

PROJECT: EXAMPLE PROBLEM
AASHTO Chapter 9

STATION: TEST 0 + 00
SHEET 1 1

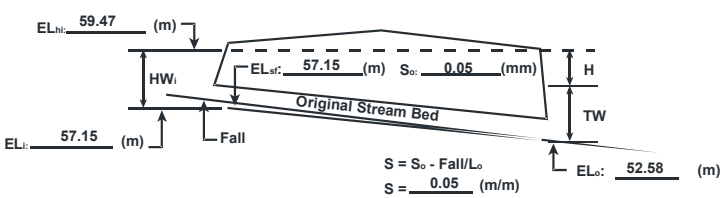
CULVERT DESIGN FORM
DESIGNER / DATE: MJM OF 7/24/06
REVIEWER / DATE: ____ OF ____

HYDROLOGICAL DATA
SEE ADDITIONAL SHEETS
■ METHOD: USGS
☐ DRAINAGE AREA: ____ ☐ STREAM SLOPE
☐ CHANNEL SHAPE:
☐ ROUTING: ____ ☐ OTHER:

DESIGN FLOW & TAILWATER

R.I.(YEARS)	FLOW (m^2/s)	TW (m)
50	11.33	0.85
100	14.16	0.93

ROADWAY ELEVATION: 59.74 (m)



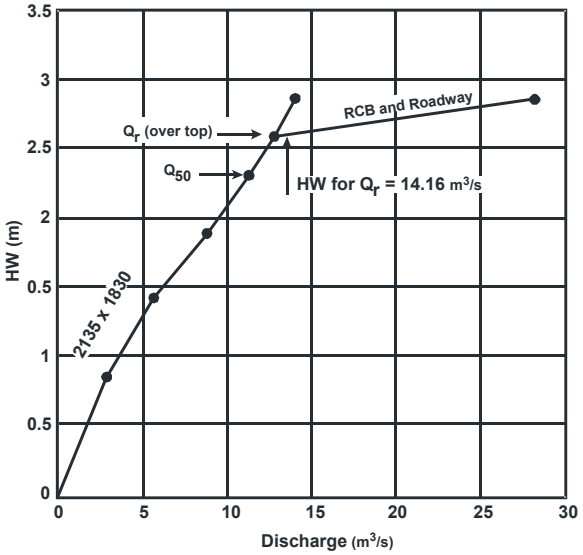
CULVERT DESCRIPTION MATERIAL-SHAPE-SIZE-ENTRANCE	TOTAL FLOW Q (m^3/s)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW/D (2)	INLET CONTROL HW	INLET CONTROL FALL (3)	INLET CONTROL ELhi (4)	OUTLET CONTROL TW (5)	OUTLET CONTROL d_c	OUTLET CONTROL $\frac{d_c + D}{2}$	OUTLET CONTROL h_o (6)	OUTLET CONTROL k_e	OUTLET CONTROL H (7)	OUTLET CONTROL ELho (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
	11.33	5.32	1.27	2.32	0	59.47	0.85	1.43	1.63	1.63	0.2	0.85	55.06	59.47	9.68	59.49 ≤ 59.47 ok
	14.16	6.65	1.56	2.87	0	60.02	0.93	1.65	1.74	1.74	0.2	1.31	55.63	60.02	10.36	60.02 > 59.74 calc=Q_r
Performance Curve	2.83	1.33	0.46	0.85	0	58.00										
	5.66	2.66	0.78	1.43	0	58.58										
	8.50	3.99	1.01	1.85	0	59.00										

TECHNICAL FOOTNOTES
(1) USE Q/NB FOR BOX CULVERTS
(2) HW/D = HW/D OR HWe/D FROM DESIGN CHARTS
(3) FALL = HWi - (ELhd - ELsf): FALL IS ZERO FOR CULVERTS ON GRADE
(4) EL = HWi + ELi (INVERT OF INLET CONTROL SECTION)
(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL
(6) h_o = TW or $(d_c + D)/2$ (WHICHEVER IS GREATER)
(7) $H = [1+k_c+(19.63\, n^2 L)/R^{1.33}]\, V^2/2g$
(8) ELho = ELo + H + h_o

SUBSCRIPT DEFINITIONS
a. Approximate
f. Culvert Face
hd. Design Headwater
hi. Headwater in Inlet Control
ho. Headwater In Outlet Control
i. Inlet Control Secton
o. Outlet
sf. Streamed at Culvert Face
tw. Tailwater

COMMENTS / DISCUSSION:
Use 14.16 (m^3/s) in culvert
$Q_p = C_d\, L\, HW_r^{1.5} = 1.67(60)(0.27)^{1.5} = 14.06\ m^3/s$
$Q_7 = 14.16 + 14.06 = 28.22\ m^3/s$

CULVERT BARREL SELECTED
SIZE 2135 x 1830 (mm x mm)
SHAPE RCB
MATERIAL Concrete n 0.012
ENTRANCE Beveled



9.11 CHAPTER 9 NOMENCLATURE

Symbol	Definition	Units
A	Area of cross section of flow	m^2 (ft^2)
A_c	Cross sectional area of flow at critical discharge	m^2 (ft^2)
AHW	Allowable HW	m (ft)
B	Barrel width	m (ft)
D	Culvert diameter or barrel height	m (ft)
d	Depth of flow	m (ft)
d_m	Hydraulic depth	m (ft)
d_c	Critical depth of flow	m (ft)
d_n	Normal Depth	m (ft)
d_u	Uniform Depth	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 (subscript indicates section)	m (ft)
K_E	Entrance loss coefficient	dimensionless
L	Length of culvert	m (ft)
n	Manning's roughness coefficient	dimensionless
P	Wetted perimeter	m (ft)
Q	Rate of discharge	m^3/s (cfs)
Q_c	Critical discharge	m^3/s (cfs)
q	Discharge per unit width	$m^3/s/m$ (cfs/ft)
R	Hydraulic radius (A/P)	m (ft)
S	Slope of culvert	m/m (ft/ft)
S_f	Friction Slope of culvert	m/m (ft/ft)
S_o	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	N/m^2 (lb/ft^2)

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CHAPTER 9, APPENDIX A

JOINT AGENCY GUIDANCE FOR THE ANALYSIS OF ENVIRONMENTAL IMPACTS AND OTHER ISSUES FOR SHORT SPAN STRUCTURES

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Date: June 17, 2008

Subject: Analysis of Environmental Impacts for Short Span Structures

To: PennDOT District Executives, PA DEP Regional Directors, PFBC Divisions

From:

Brian Thompson, PE
Director
PennDOT Bureau of Design

John Hines
Executive Director
PA DEP

Leroy Young
Director of Fisheries
PFBC

The Pennsylvania Department of Transportation (PennDOT), The Department of Environmental Protection (PA DEP), and The Pennsylvania Fish and Boat Commission (PFBC) have worked jointly to develop a document entitled *Joint Agency Guidance for The Analysis of Environmental Impacts and Other Issues for Short Span Structures*. This program support document has been developed by a joint taskforce of PennDOT, PADEP, and PFBC representatives from PennDOT Bureau of Design and District Offices, PADEP Central Office and Regional Offices, and PFBC. Attached is a copy of the Joint Guidance dated June 3, 2008.

This document is intended to form a basis for the development of an alternatives analysis suitable for submission with a permit application to PA DEP. This document applies to replacement of existing culverts and bridges with box culverts that meet the following criteria:

- Single cell opening with a normal clear span of 24 feet or less
- Depressed culvert bottom with fish baffles in accordance with PennDOT Design Manual Part 2, Chapter 10 and PennDOT Standard Drawing BD-632M
- No significant reduction in existing waterway opening
- No regulated activity is authorized that is likely to directly or indirectly affect a State or Federal species of special concern

Project sites with one or more of the following physical conditions require additional agency coordination to determine if a box culvert is an acceptable alternative:

- Culvert lengths greater than 100 feet;
- Exposed continuous bedrock present under the proposed structure; or
- High stream gradients greater than 4%

This document provides the essential elements that would be used in an alternatives analysis for the use of box culverts as waterway structures. The resulting analysis may need to be adjusted to meet site specific conditions including any additional non-typical issues that may be associated with the project. The document provides discussion and documentation of the following:

- Permanent Stream Impacts
- Temporary Stream Impacts
- Fish Passage
- Right-of-Way Impacts
- Geotechnical Issues
- Schedule
- Construction Cost

Use of the Program Support Document

For projects where a **General Permit** applies and a box culvert meeting the criteria discussed in the guidance will replace a short-span bridge the following is recommended:

- Compare the project site conditions to the alternative analysis documentation cited in the guidance and document any non-typical issues that may be associated with the planned project
- In the project description of the general permit application reference that the project has been evaluated in accordance with the *Joint Agency Guidance for The Analysis of Environmental Impacts and Other Issues for Short Span Structures* and provide documentation of any non-typical issues if applicable.

For projects that **do not qualify for a General Permit**, and a box culvert meeting the criteria discussed in the guidance will replace a short-span bridge the following is recommended:

- Compare the project site conditions to the alternative analysis documentation. Use the document as a basis for the development of the alternatives analysis as follows:
  - If the guidance is sufficient as the alternatives analysis for the structure and there are no other project issues that must be detailed in the alternatives analysis – extract the text from the Tier 1 and Tier 2 analysis sections of the document and add the specific project information. Save this as the project specific Alternatives Analysis and attach it to the Alternatives Analysis Section of the permit application
  - If the site has non-typical issues, use the guidance as a basis for the alternatives analysis. Extract the text from the Tier 1 and Tier 2 analysis sections of the document and add the specific project information and include the discussion of the non-typical issues. Save this as the project specific Alternatives Analysis and attach it to the Alternatives Analysis Section of the permit application.
  - Any non-typical issues should be addressed or discussed with the DEP regional office and the Fish and Boat Commission if applicable at the Pre-Application Meeting.

Attached is a copy of the Joint Guidance document dated June 3, 2008. For PennDOT the publication will become part of PennDOT's Drainage Manual Chapter 9-Appendix A when it is issued this year. For PA DEP this document will be available online on the Chapter 105 website

and should be part of PA DEP's desk manual for review of permit applications. For PFBC the document will be available by contacting the Division of Environmental Services Section.

If you have any questions please contact Harold Rogers at PennDOT at 717-787-3767 or Jeff Means at PA DEP at 717-772-5643 or PFBC Division of Environmental Services at 814-359-5100.

Attachment
cc: Richard H. Hogg, PE Reading File
Highway Administration Bureau Directors
PennDOT District ADE's Design
PennDOT District Bridge Engineers
PennDOT District H&H and Permit Coordinators
Kelly Heffner, PA DEP
Jeffrey Means, PA DEP
PA DEP T-21 Staff
Dave Spotts, PFBC

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JOINT AGENCY GUIDANCE FOR THE ANALYSIS OF ENVIRONMENTAL IMPACTS AND OTHER ISSUES FOR SHORT SPAN STRUCTURES



Photo Source: Pennsylvania Fish and Boat Commission

PENNDOT BRIDGE QUALITY ASSURANCE DIVISION

PA DEPARTMENT OF ENVIRONMENTAL PROTECTION

PENNSYLVANIA FISH AND BOAT COMMISSION

June 3, 2008

JOINT AGENCY GUIDANCE FOR THE ANALYSIS OF ENVIRONMENTAL IMPACTS AND OTHER ISSUES FOR SHORT SPAN STRUCTURES

INTRODUCTION

This document provides an overview of the various alternatives related to structure replacements and their associated construction and environmental impacts and other issues associated with construction of a box culvert versus bottomless structures, including bridges, rigid frames, and arches. This guidance is intended to form a basis for the development of an alternatives analysis suitable for submission with a permit application to the Department of Environmental Protection (PA DEP) to satisfy 25 PA Codes 105.13 and 93.4c.

Close coordination and agency involvement should routinely be conducted early in the project development process to identify potential aquatic resource issues. In most situations the box culvert baffle designs provide for fish and other aquatic life to migrate through the structure. However, there may be physical conditions such as culvert lengths greater than 100 feet; exposed continuous bedrock present under the proposed structure; or high stream gradients greater than 4% that provide challenges to ensure aquatic life passage. Other projects may be within watersheds that are classified as Exceptional Value (Chapter 93); contain Threatened &Endangered species; or support migratory fish populations and may also present unique aquatic challenges. In order to address these concerns, on site field scoping meetings with PADEP, Pennsylvania Department of Transportation (PennDOT), Pennsylvania Fish and Boat Commission (PFBC), and the US Army Corps of Engineers (USACE) are strongly encouraged to identify and resolve anticipated environmental impacts prior to permit submittal.

CRITERIA FOR CONSIDERATION

Project types include replacement of existing culverts and bridges. Box culverts meeting the following criteria are covered under this joint guidance:

- single cell opening with a normal clear span of 24 feet or less
- depressed culvert bottom with fish baffles in accordance with PennDOT Design Manual Part 2, Chapter 10 and PennDOT Standard Drawing BD-632M
- no significant reduction in existing waterway opening

In addition to satisfying the criteria listed in the water obstruction and encroachment permit that is applicable to the project, any proposed structure must also meet the following environmental criteria.

- No regulated activity is authorized that is likely to directly or indirectly affect a State or Federal species of special concern or a species proposed for such designation, or which is likely to destroy or adversely modify the critical habitat of such a species, as identified under the Federal Endangered Species Act of 1973; Title 30, Chapter 75 of the PA Fish and Boat Code; Title 17, Chapter 25, Conservation of Wild Plants; and Title 31, Chapter 133 Game Wildlife Code.
- No regulated activity may substantially disrupt the movement of those species of aquatic life indigenous to the watercourse, stream or body of water, including

those species which normally migrate through the area. Breeding areas for migratory waterfowl must be avoided to the maximum extent possible.

This document provides the foundation of an alternative analysis for the use of culverts, subject to the criteria identified above, as waterway structures. The guidance will only need to be adjusted to meet the site specific conditions including any additional non-typical issues that may be associated with the project.

TIER 1: ALTERNATIVES ANALYSIS

This tier 1 alternatives analysis of no-build, rehabilitation, and replacement/new alignment is provided and necessary to document project need per PA DEP requirements.

No-Build Alternative: The no-build alternative has the benefit of no environmental impacts to the streambed; however the no-build alternative cannot address existing structure and roadway deficiencies or safety concerns and usually does not meet the project need. Since the no-build alternative typically does not meet the project need, the no-build alternative is not a viable alternative.

Rehabilitation Alternative: The rehabilitation alternative addresses the project conditions that justify the need to rehabilitate an existing structure rather than replace it. The rehabilitation alternative will typically result in temporary impacts to water quality and stream flow, may be cost prohibitive compared to a new structure, and sometimes does not meet the project need. Additionally, if the rehabilitation is to an existing open-bottom structure typical of the old slab bridges, then the rehabilitation alternative will require abutment protection measures in accordance with current design criteria for bridge replacement structures.

Replacement/New Alignment Alternative: The replacement/new alignment alternative considers different structure types to replace an existing structure or construct a new structure on a new alignment. For the purpose of this document, the replacement/new alignment alternative analysis will focus on comparison of the impacts associated with construction of a box culvert versus bottomless structure types, including bridges, rigid frames, and arches. Detailed evaluations of permanent and temporary stream impacts, fish passage, right-of-way impacts, geotechnical issues, time schedule, and construction costs are provided as follows.

TIER 2: ALTERNATIVES ANALYSIS

After the tier 1 analysis is completed, if replacement or new alignment is the selected alternative, then a tier 2 analysis must be included. The tier 2 analysis should include discussion of the following:

- Permanent Stream Impacts
- Temporary Stream Impacts
- Fish Passage
- Right-of-Way Impacts
- Geotechnical Issues

- Schedule
- Construction Cost

Permanent Stream Impacts

Box Culvert: Box culvert construction will result in permanent streambed impacts due to stream channel shading, rock protection, placement of a concrete bottom in the stream, and the potential to create a "perched" condition at the outfall. To address these issues, personnel from the PFBC, PA DEP, and PennDOT developed box culvert designs for fish passage in Pennsylvania and the designs can be found within PennDOT's Design Manual Part 2 and Standard Drawing BD-632M. The invert of the culvert bottoms are depressed one foot below natural streambed elevation and baffles are incorporated into the culvert bottom, which are configured based upon existing stream width and percent stream slope. The intent of the depressed culvert bottoms and baffles is to promote the deposition of natural streambed material to collect within the culvert and allow aquatic life to emigrate through the structure. These designs require a 5-foot length of rock lining at the upstream and downstream ends of the box culvert for protection against scour and have a minimal area of stream impacts due to rock placement (Appendix 1).

In 2005, the PFBC conducted field views of 24 culverts with these new designs in 11 counties and concluded that all of the structures provided adequate fish passage and no scour was observed around the inlets or outlets. Photos 1-2 show sediment build-up in single cell culverts.





Photo 1

Photo Source: PA Fish and Boat Commission

Bridge: Bridge construction, including bridges, rigid frames, and open bottom arches, will result in permanent streambed impacts due to stream channel shading and rock protection. PennDOT Design Manual Part 4 requires rock riprap at abutments for protection against scour. Protection against scour is vital to the stability and therefore safety of the structure. The extent of riprap into the channel is dependent on the foundation material as follows:

> *Spread Footings on Soil*: Footings must be placed at least 6 feet below the streambed and excavation must be in accordance with OSHA regulations. For bridge clear spans up to 24 feet, the entire streambed between abutments may require permanent rock lining. Refer to Appendix 2.
>
> *Spread Footings on Erodible Rock*: Footings must be placed approximately 4 feet or more below streambed and excavation must be in accordance with OSHA regulations. For bridge clear spans up to approximately 24 feet, the entire streambed may require permanent rock lining. Refer to Appendix 3.
>
> *Spread Footings on Non-Erodible Rock*: Footings must be placed 3½ feet or more below streambed and excavation must be in accordance with OSHA requirements. For a 15 foot span, approximately two-thirds of the streambed requires permanent rock lining. For a 20 foot span, approximately one-half of the streambed requires permanent rock lining. For a 24 foot span, approximately 40 percent of the streambed requires permanent rock lining. Refer to Appendix 4.

Riprap requirements that are outlined in DM-4, Chapter 7 indicate a minimum of 1-foot in front of the footing for riprap placing. This is a minimum and based on the rock size this value is typically around 3 to 4 feet in order to allow for the specified Riprap to fit. For instance the D_{max} for R-6 is 24 inches and the D_{max} for R-8 is 42 inches. Scour design is required by Article 2.6.4.4.2 of the AASHTO LRFD Bridge Design Specifications, and PennDOT's scour design policy has been accepted by the Federal Highway Administration (FHWA). Additionally, PennDOT's scour protection complies with the FHWA issued Technical Advisory T 5140.23 – Evaluating Scour at Bridges on October 28, 1991, which states the following:

> *New bridges over tidal and non-tidal waterways with scourable beds should withstand the effects of scour from a superflood (a flood exceeding the 100-year flood) without failing (i.e., experiencing foundation movement of a magnitude that requires corrective action). Hydraulic studies should include estimates of scour at bridge piers and evaluation of abutment stability.*

HEC-18 is cited for the details regarding scour analyses.

Temporary Stream Impacts

Box Culvert: Box culvert construction will have temporary impacts to the stream. The excavation work involves disturbance to the streambed and banks. The required excavation depth is approximately 3 feet below streambed to allow for a 12-inch depressed bottom, 12 inches of bedding, and the bottom slab of the culvert. Impacts to

fish and macroinvertebrate life would occur during construction when temporary pipes or a diversion channel are commonly used to convey stream flow.

Bridge: Bridge construction will have temporary impacts to the stream. The excavation work involves disturbance to the streambed and banks. The required excavation depth for soil foundations is 6 feet or greater below streambed. Footings on rock may be placed at depths less than or greater than 6 feet below streambed depending on the depth to bedrock, but not to a depth less than 3½ feet. Abutments for short span bridges may be constructed sequentially rather than concurrently to allow for diversion of stream flow. The use of sequential construction, along with more complex construction methods, results in a longer construction period for bridges than that of culvert construction. This longer construction time results in a longer period of temporary impact to fish and macroinvertebrate life.

Use of Temporary Shoring versus typical excavation

One method of limiting temporary impacts to the stream channel is to use temporary shoring in the lieu of standard excavation practices in the placement of the abutments for the bridge and open bottom structures. There are several significant factors that impact the applicability of shoring for many projects including:

- Rising sheet pile cost
- Difficult to utilize shoring under phased construction
- Tie back may be required in some cases
- Impacts on stability of scour protection and potential loss of riprap as shoring deteriorates

In addition to logistic issues with using shoring at many sites, Shoring is significantly more expensive. A rough cost estimates (in %) for shoring abutments of footings over typical construction excavation is between 68% to 95% higher installation cost.

Fish Passage

Box Culvert: The inverts of box culverts are typically depressed 12 inches below the streambed elevation. Baffles are placed in the culvert and aprons in accordance with PennDOT Design Manual Part 2 and Standard Drawing BD-632M to allow for fish passage. The use of depressed box culverts with baffles has been evaluated in studies completed in Pennsylvania and other states that have high quality streams supporting trout populations (McClellan 1980, Shervinskie 1998, and Bates 2003). These studies have concluded that the use of depressed culverts with alternating baffles does allow fish passage and does not eliminate macroinvertebrate populations.

Bridges: The rigid frame, open bottom arch, and bridge require the use of rock lining for scour protection. The limits of rock lining may encompass the entire streambed beneath the bridge; therefore, there may be no natural streambed material under the bridge. The extents of the rock lining required in these short-span structures may also prevent the formation of a low flow channel and reduce the normal flow depth.

Right-of-Way Impacts

Box Culvert: Box culverts result in the most compact structure because of the efficiency of the system, structurally and hydraulically. The walls and top slab of culverts are the thinnest structural elements used in roadway structures. The compactness of the box culvert typically results in the least amount of right-of-way impact. Additionally, for projects with roadway overtopping, any changes to the roadway profile can significantly impact the upstream water surface elevations. Therefore the use of a box culvert rather than a bridge may eliminate upstream flooding impacts because the existing roadway profile will be maintained.

Bridge: Bridge construction typically includes a superstructure and abutments. The superstructure is typically comprised of precast beams, a cast-in-place concrete deck, and bridge barriers. Superstructure depths are 17 inches and greater and abutment walls are a minimum thickness of 18 inches. Rigid frames and open bottom arches commonly require additional fill depth on the structure. The additional superstructure depth for bridges and fill for rigid frames and open bottom arches (versus a box culvert) may require an increase in roadway grade or lengthening of the span. Depending on hydraulic needs, the span for a bridge, rigid frame, or open bottom arch may be longer than that of a culvert. An increase in fill depth, roadway grade, or span length involves more right-of-way impact.

Geotechnical Issues

Box Culvert: The pressure transferred to founding soils from box culverts is typically less than that of a bridge due to the box configuration and efficiency of the culvert structure. Therefore box culverts can be constructed on many soils without the need for soil improvement or deep foundations such as piles or caissons.

Bridge: Bridge construction involves additional geotechnical testing to determine the type of footings that must be used with a bridge, rigid frame, or open bottom arch. Soils with a low bearing capacity would make a bridge, rigid frame, or open bottom arch with spread footing susceptible to differential settlement. Differential settlement could lead to maintenance issues including cracking in the superstructure, leaking joints, or more frequent repaving of approach roads. Where differential settlement exceeds tolerable values, pile foundations may be used for a bridge structure at a significantly greater expense and longer construction time.

Time Schedule

Box Culvert: Construction of box culverts requires the least amount of construction time. Precast box culvert systems are typically used, allowing for the shortest construction duration. This minimizes temporary disturbance to the stream and adjacent stream bank. Impact to the traveling public is also minimized.

Bridge: Bridge construction involves footing construction and other cast-in-place elements, resulting in the longest construction duration. Rigid frame and open bottom

arches, if precast, require construction of cast-in-place footings followed by erection of precast elements. The construction period for rigid frames and arches is longer than that for a precast box culvert but less than that for a bridge.

Construction Cost

Box Culvert: Precast box culverts are typically the least cost structure due to efficiency of the structural system, construction duration, and use of precast elements.

Bridge: Bridge construction involves construction of many cast-in-place components (footings, abutment stemwalls, concrete deck, etc.) and is more costly than box culvert construction.

A comparison of estimated costs for bridges, Con Span Open Bottom Structures and Box Culverts was developed to show an estimate of the difference in cost between the different structure types. The table shows a comparison of costs for a box culvert versus open bottom structures on spread footing and open bottom structures on piles.

Cost Estimates (in %) for a Bridge, Con Span and Box Culverts:

The estimates below are based an analysis of costs from twenty short span structures. These estimates will change depending on the characteristics of a given project but where provided to give an estimate of the magnitude of the cost differences between the different project types.

- Bridge versus Box culvert
  Percent decrease in cost for the Box Culvert = 40%

- Con Span (on spread footings) versus Box culvert
  Percent decrease in cost for the Box Culvert = 20%

- Con Span (piles) versus Box culvert
  Percent decrease in cost for the Box Culvert = 40%

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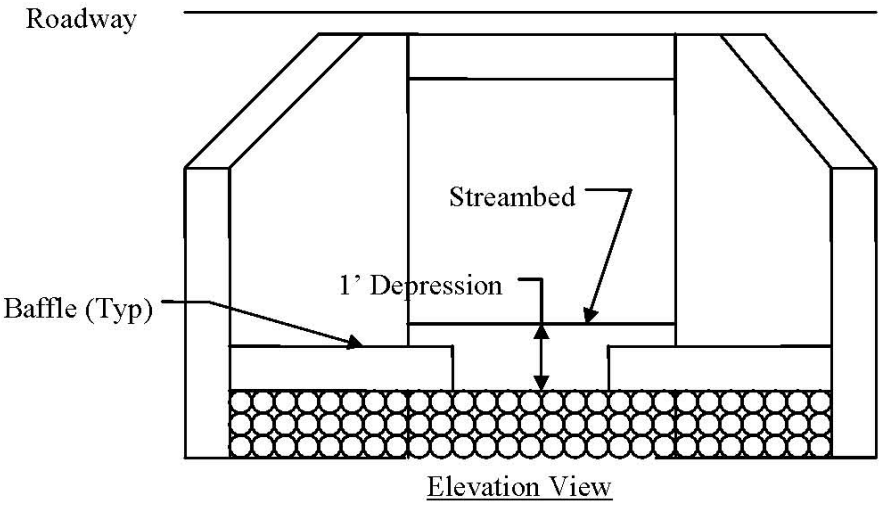
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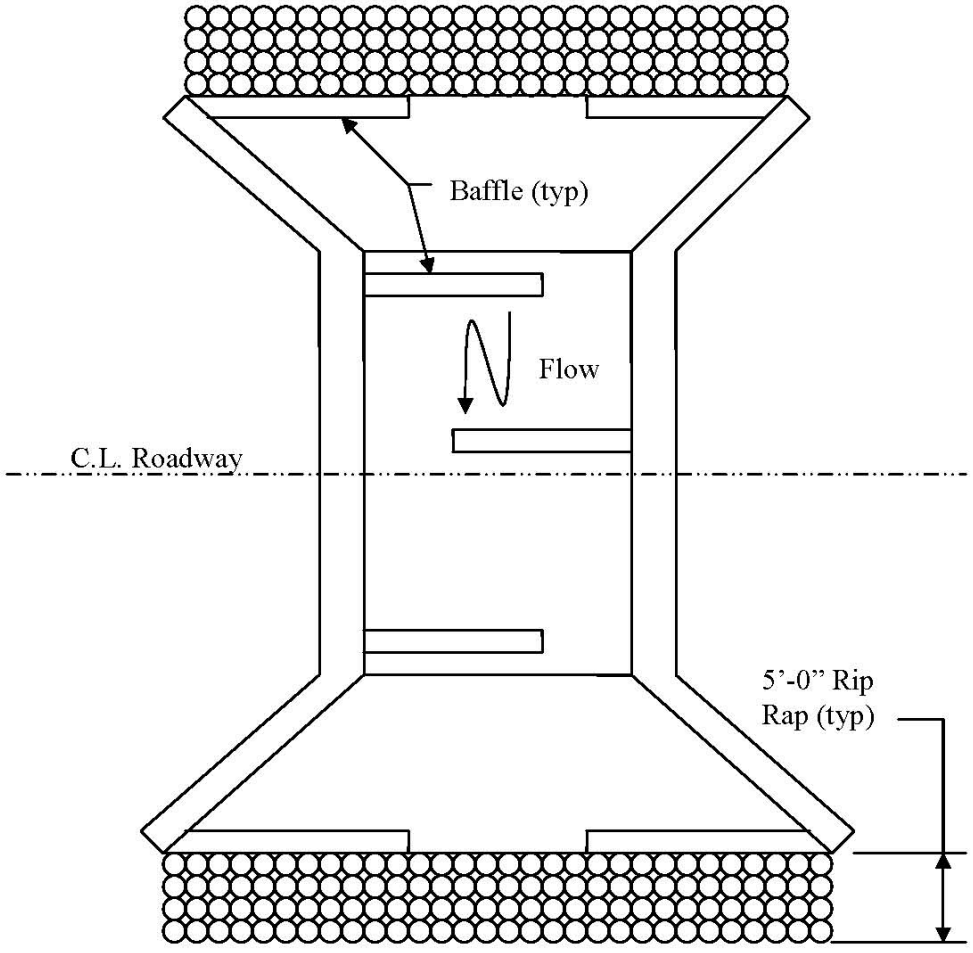
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APPENDIX 1

BOX CULVERT STREAM IMPACTS



Elevation View

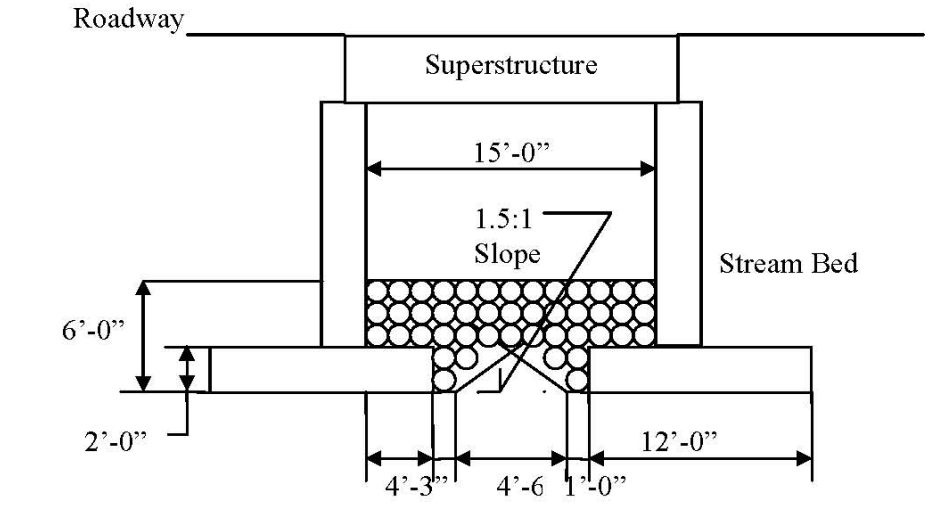


Plan View

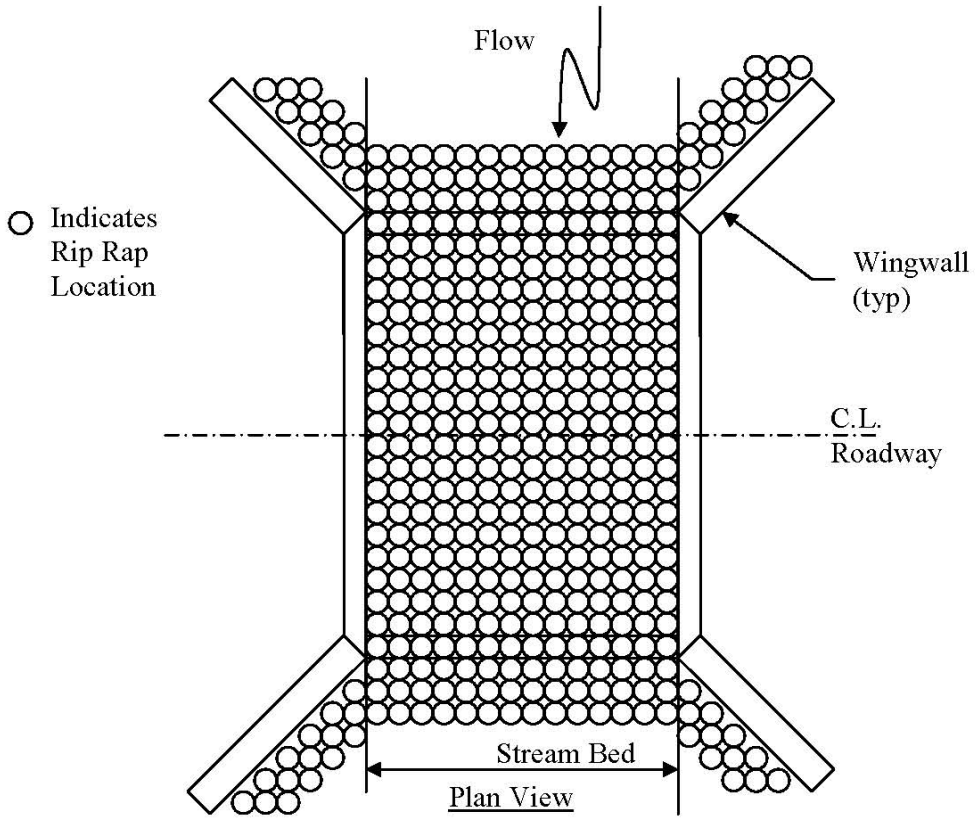
Typical Box Culvert

APPENDIX 2

BRIDGE STREAM IMPACTS
SPREAD FOOTING ON SOIL

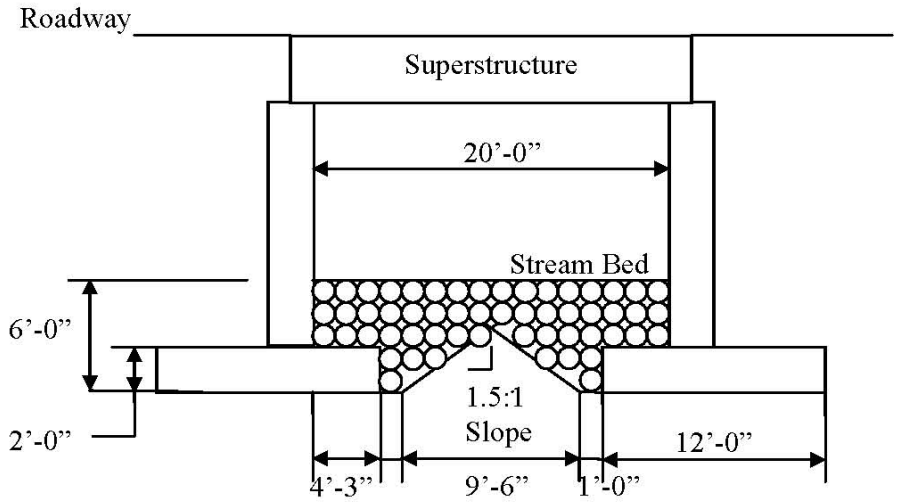


Elevation View

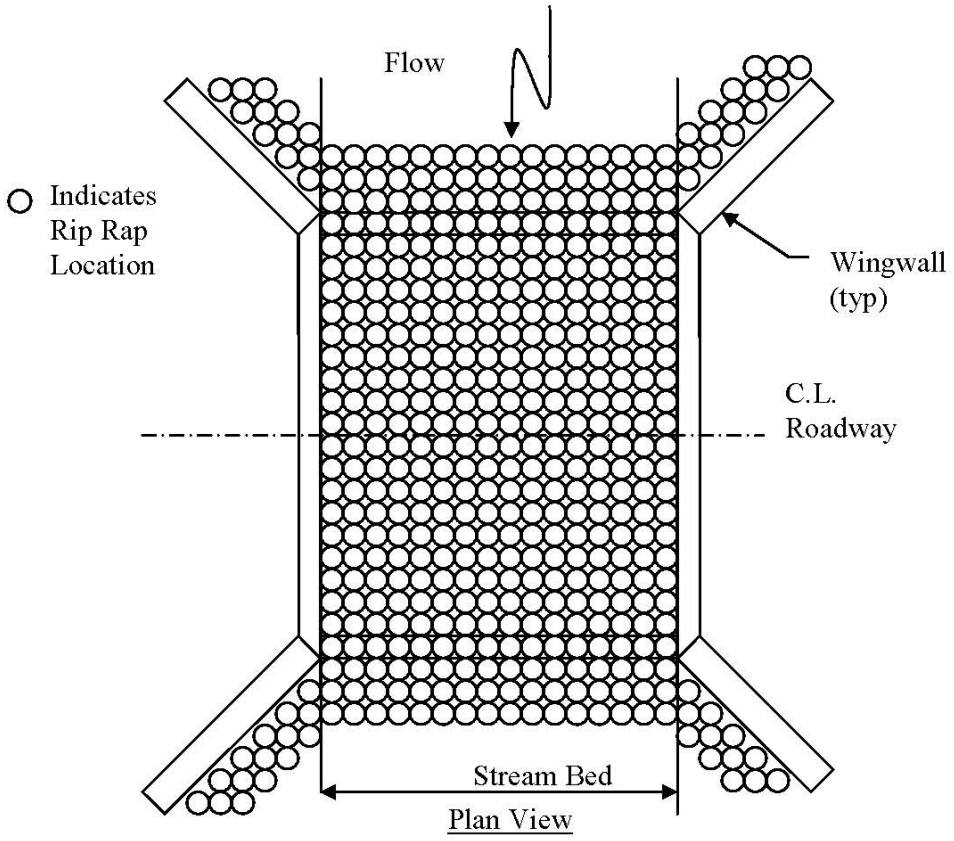


Plan View

Spread Footing on Soil, Span Length 15'-0"

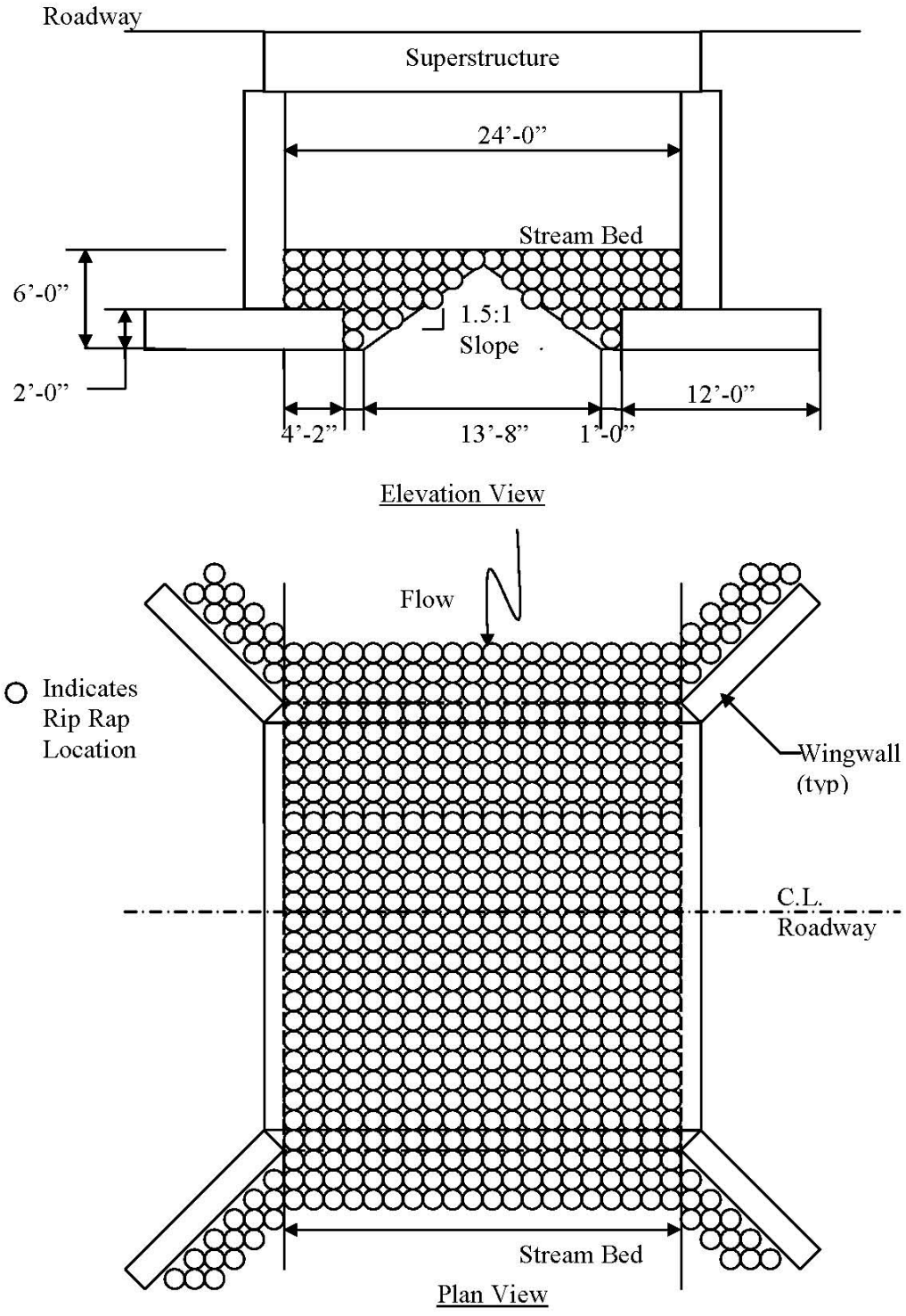


Elevation View



Plan View

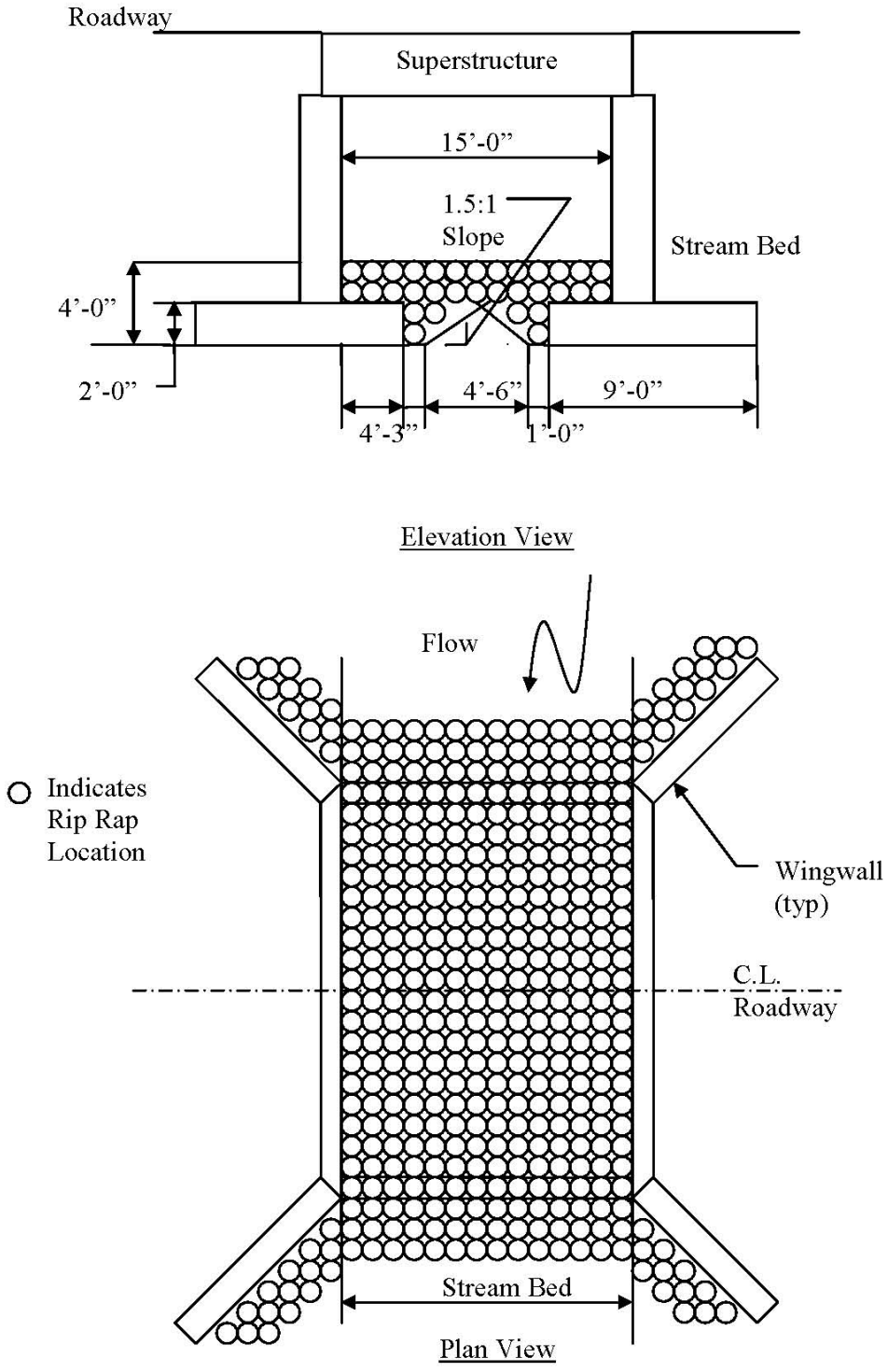
Spread Footing on Soil, Span Length 20'-0"



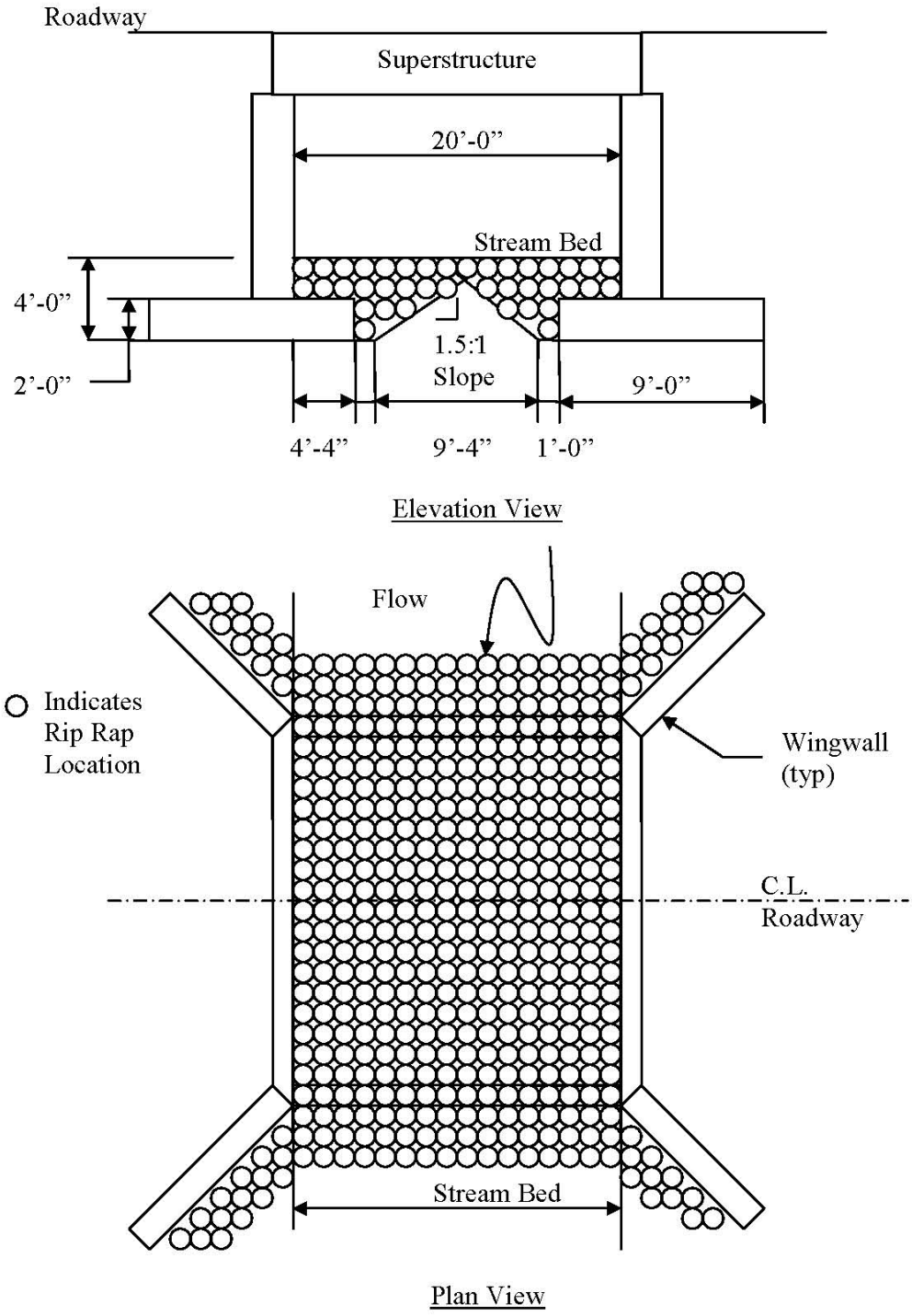
Spread Footing on Soil, Span Length 24'-0"

APPPENDIX 3

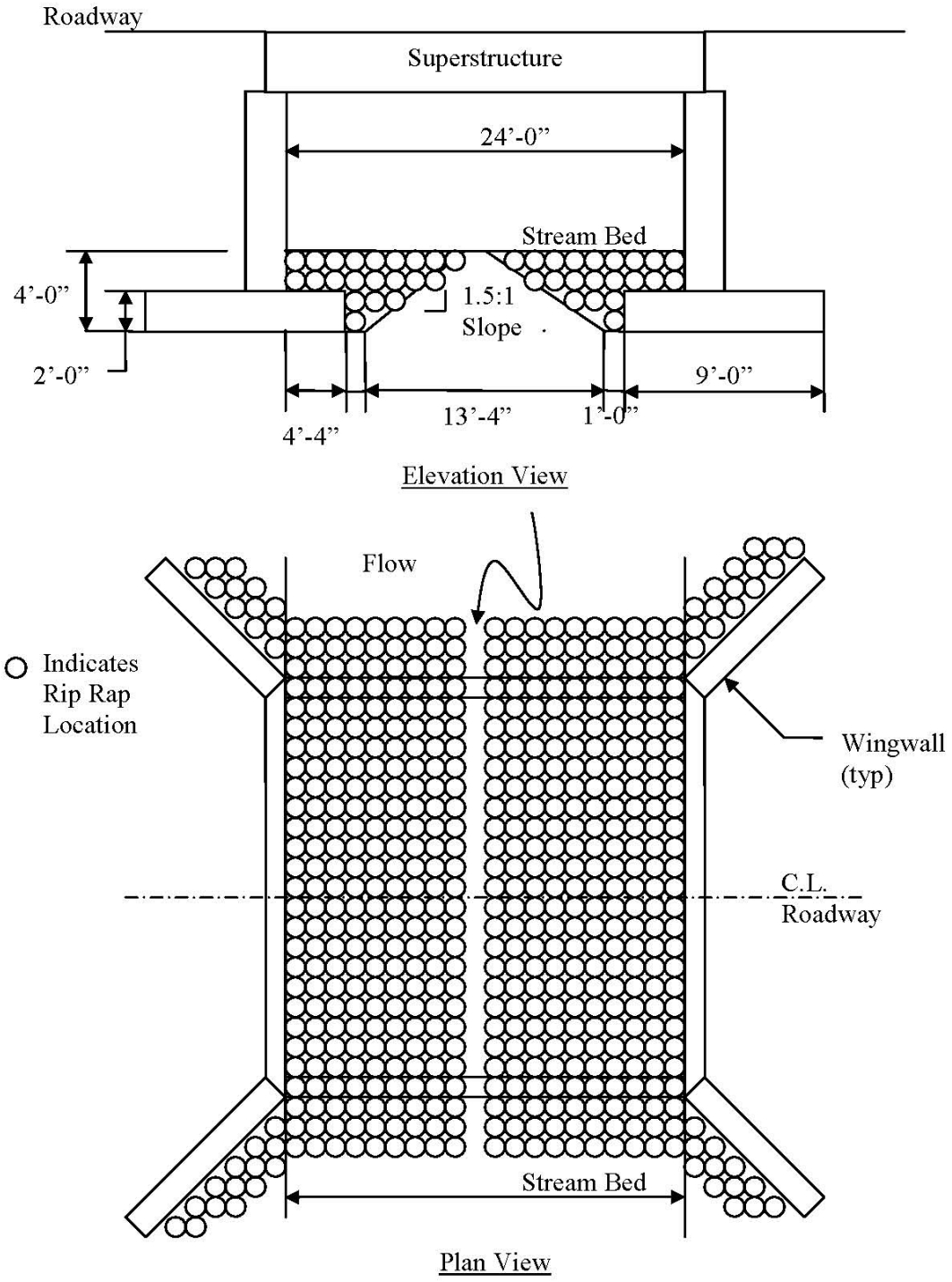
BRIDGE STREAM IMPACTS
SPREAD FOOTING ON ERODIBLE ROCK



Spread Footing on Erodible Rock, Span Length 15'-0"



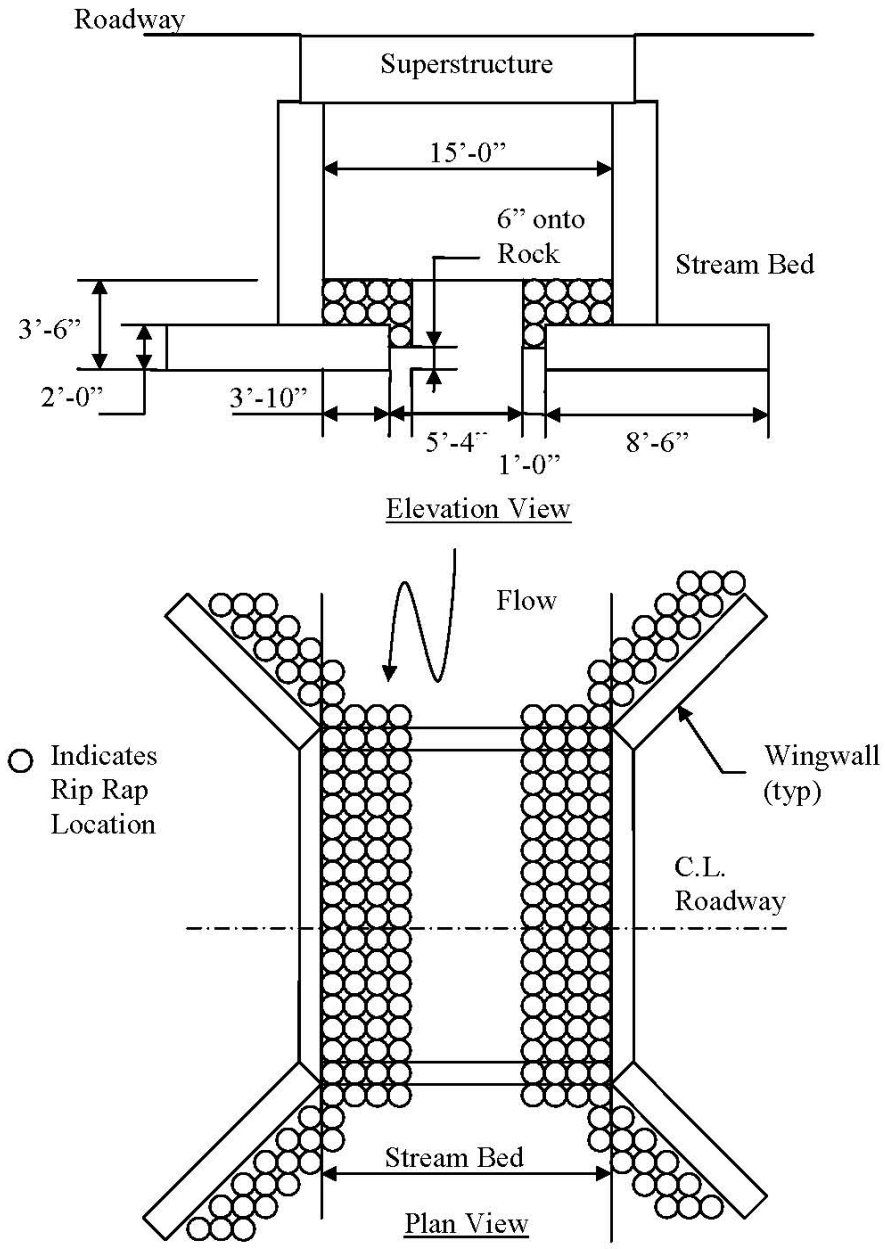
Spread Footing on Erodible Rock, Span Length 20'-0"



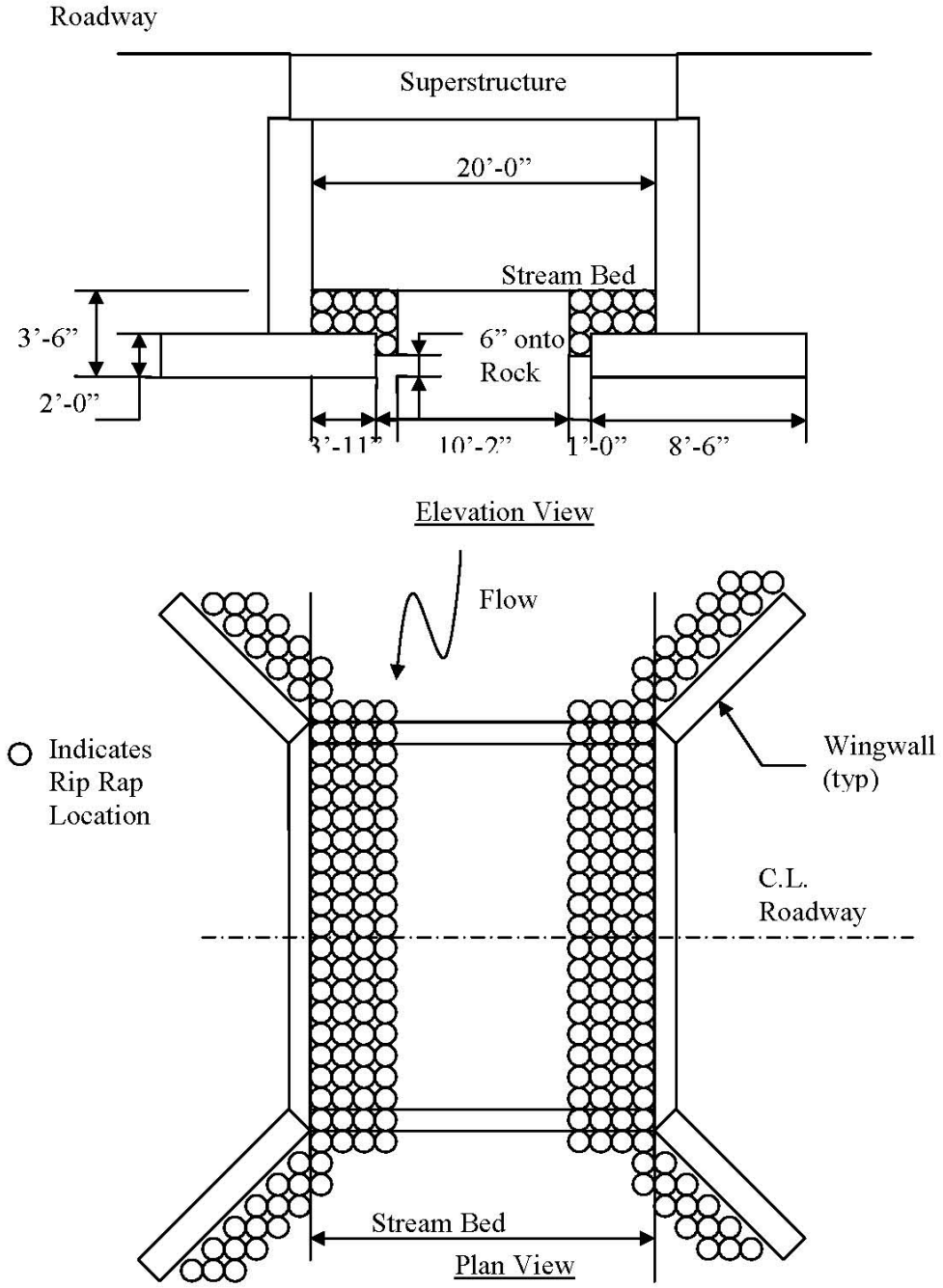
Spread Footing on Erodible Rock, Span Length 24'-0"

APPENDIX 4

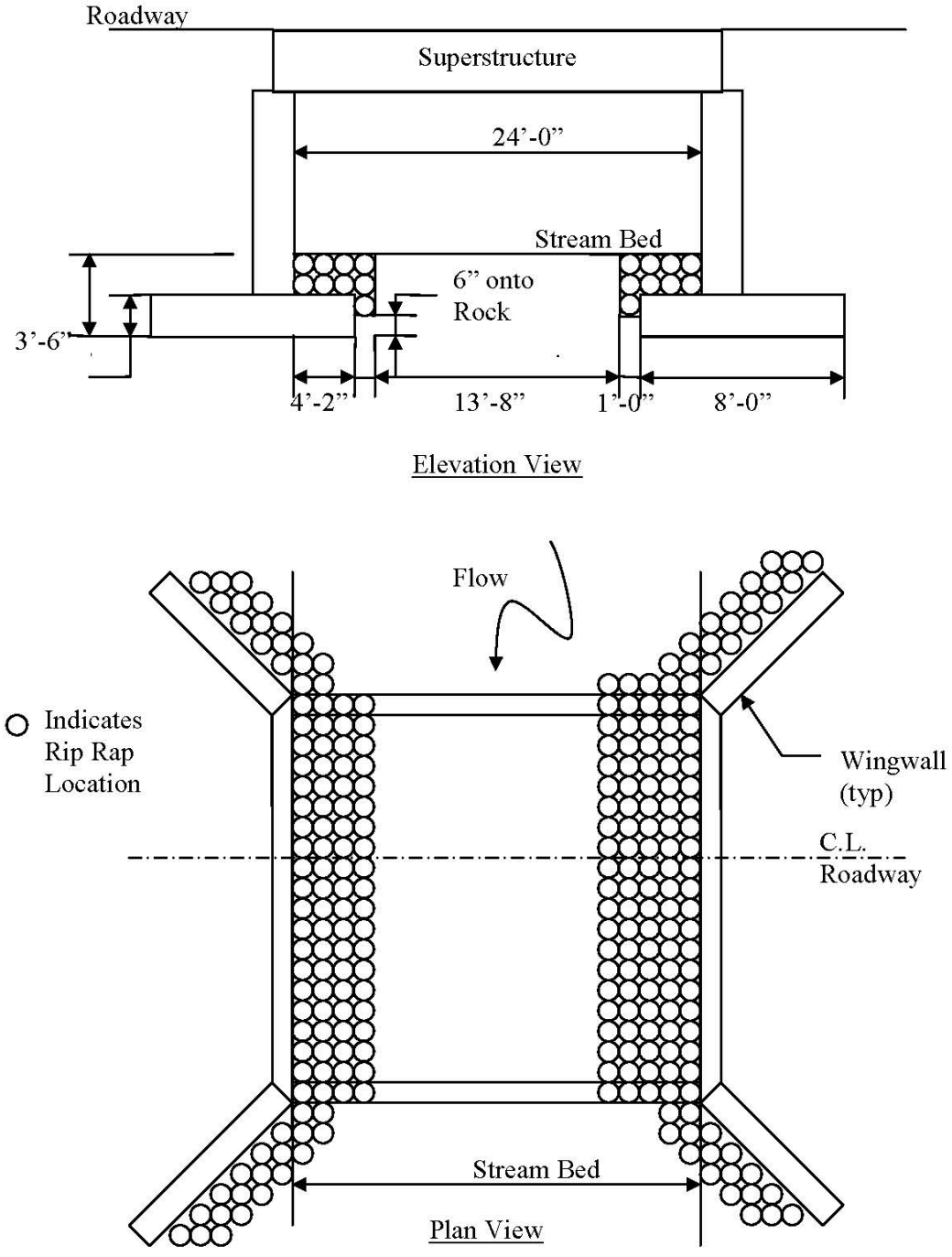
BRIDGE STREAM IMPACTS
SPREAD FOOTING ON NON-ERODIBLE ROCK



Spread Footing on Non-Erodible Rock, Span Length 15'-0"



Spread Footing on Non-Erodible Rock, Span Length 20'-0"



Spread Footing on Non-Erodible Rock, Span Length 24'-0"

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CHAPTER 10

BRIDGE HYDRAULICS

10.0 INTRODUCTION TO BRIDGES

A. General - Bridges. Bridges serve a variety of highway purposes including the elimination of conflicts with traffic and other modes of transportation, such as rail, marine, air and pedestrian. Bridges enable watercourses to maintain the natural function of flow conveyance and sustain aquatic life. Bridges are also important and expensive highway-hydraulic structures and are vulnerable to failure from flood-related causes. In order to minimize the risk of failure, the hydraulic requirements of stream crossings must be recognized and considered carefully.

Features which are important to the hydraulic performance of a bridge include the approach fill alignment, skew and profile; bridge location, skew and length; span lengths; bent and pier location and design; and foundation and superstructure configuration and elevations. These features of a highway-stream crossing are usually the responsibility of location, design and bridge engineers; however, the integrity and safety of the facility are often as dependent upon competent hydraulic design as on competent structural and geometric design.

Only structures designed hydraulically as bridges are treated in this chapter, regardless of length. This discussion of bridge hydraulics considers the total crossing, including approach embankments and structures on the flood plains.

B. Design Requirements. Hydrologic and hydraulic analyses are required for all bridge projects over waterways that may adversely affect the floodplain even if no structural modifications are necessary. Typically, this should include an estimate of peak discharge (sometimes complete runoff hydrographs), comparing water surface profiles for existing and proposed conditions, consideration of potential stream stability problems, and consideration of scour potential. Hydrologic and hydraulic analyses generally should not be required for minor modifications such as bridge widening that does not affect the water surface profile.

10.1 PLANNING AND LOCATION CONSIDERATIONS

A. Overview of Planning and Location Considerations. Generally, a stream crossing location is selected during the planning of the corridor alignment phases of a highway project. The final location should be selected only after detailed survey information has been obtained and preliminary hydraulic studies have been completed. Although they are not the sole consideration in bridge location, hydraulic aspects should receive major attention in the initial planning of a highway alignment. The location and alignment of the highway can either magnify or eliminate hydraulic problems at the crossing.

National objectives, such as U.S. Coast Guard navigation clearance requirements for marine traffic, also must be considered in site selection. Another national objective is to reduce the rate of annual increase in flood damage losses by restricting the use of flood plains. This means that highways in the vicinity of streams must conform with the Federal Emergency Management Agency (FEMA) National Flood Insurance Program (NFIP) requirements (E.O.11988). Additionally, the preservation of wetlands must be considered in the selection of stream crossings during preliminary design.

B. Location and Orientation. Some of the factors to consider in the selection and orientation of bridge alignments are as follows:

- The safety of the highway user.
- Vertical and horizontal highway profiles.
- Hydraulic performance.
- Construction and maintenance costs.
- Foundation conditions.
- Highway capacity.
- Navigation requirements.
- Stream regime.

- Environmental considerations.
- Flood hazard and risk.
- Structural efficiency and performance.

Obviously, the most favorable hydraulic conditions cannot always be achieved when other considerations are in conflict. Increases in additional construction and maintenance costs are sometimes necessary to achieve a satisfactory balance among competing factors including hydraulic performance. However, the selected design always should ensure that the integrity of the hydraulic design and the safety of the bridge are not compromised.

Hydraulic considerations in site selection are numerous because of the many possible flow conditions that may be encountered at the crossing and because of the many water-related environmental factors. Flow may be in an incised stream channel, or the stream may have floodplains which are several kilometers (miles) wide. Floodplains may be clear or heavily vegetated, symmetrical about the stream channel or highly eccentric, clearly defined by natural topography or man-made levees, or indeterminate. Flow may be uniformly distributed across the floodplains or concentrated in swales in the overbank areas. Flow direction often varies with the return period of the flow so that a bridge substructure oriented for one flow would be incorrectly oriented for another. Flow direction in overbank areas is often unrelated to that in the main or low flow channel. In some instances, the floodplains convey a large proportion of the total flow during extreme floods and the stream channel conveys only a small proportion.

Not all of the above will apply to each stream crossing or bridge location, but many of the most important site considerations are hydraulic or water-related. Crossing location alternatives often do not include the most desirable site from the hydraulic design viewpoint, but the difficulties involved often can be reduced by careful hydraulic analysis.

Incorporation of roadway approaches which will accommodate overflow may be necessary for some configurations. Such overflow reduces the threat to the bridge structure itself. Of course, the flow of traffic is interrupted, and the potential costs associated with such interruption and potential damage to the roadway embankment and bridge integrity should be considered by the designer.

C. Structure Type. The final selection of structure type should not be made prior to the completion of detailed surveys and necessary scour, hydrologic and hydraulic studies. Where final structure-type selection is necessary to satisfy the requirements of the environmental assessment, public hearings or right-of-way acquisition, all available information should be furnished to the engineer and the decision made in consideration of all factors which influence the selection. Selection of the final type, size, and location of structures is at the sole discretion of the engineer.

D. Environmental Considerations. This chapter does not include a definitive discussion of the environmental considerations in site selection and bridge design. Because of the many and complex considerations involved, discussion here is limited to a broad approach regarding the environmental concerns that must be addressed in the selection and design of a stream crossing. A *Field Checklist for the Preliminary Design Permit Coordination* was developed by a joint taskforce of PennDOT, PA DEP, and PFBC is included in Appendix 10A, *Field Checklist for the Preliminary Design Permit Coordination*. The use of the checklist is highly encouraged for PennDOT projects. The checklist was developed to facilitate early coordination and documentation for projects to help streamline and expedite the permitting process.

The environmental effects of construction activities may be classified as the hydrologic and hydraulic, physical, chemical, cultural, historical, archeological, aesthetic and biological aspects.

Hydrology at a crossing is unlikely to be a factor in site selection, but the hydraulics may be an overriding concern. Hydraulic considerations include the effects of the crossing on velocities, water surface profiles, velocity and flow distribution, scour, bank stability, sediment transport, aggradation and degradation of the channel, and the supply of sediment to the stream or water body. The engineer must evaluate the potential effect of the crossing on these factors as well as the potential effects of the crossing site on the environment.

Effects of a highway on the chemical quality of surface waters are not ordinarily a major consideration in site selection, though it is possible that contaminants in the form of minerals or from a sanitary landfill would be exposed in one location and not at an alternate site. There is some concern for chemical quality at crossing sites, particularly near public water supply intakes, due to the risk of toxic material spills. The probability of such spills

should be considered in weighing factors that influence site selection. The use of deicing salts, fertilizers, growth inhibitors and other chemicals usually would not vary appreciably among alternate sites, but the adverse effects of normal usage of such materials may vary among alternatives and, therefore, may be a factor in site selection. A formal evaluation of these issues will occur during the Water Quality Certification under Section 401 of the Clean Water Act.

Aesthetic considerations include effects on the visual, odor and taste qualities of surface waters. The aesthetic quality of surface waters should be considered in site selection where potable water supplies, water contact sports and fisheries are involved. The visual quality most often affected by highways is temporary turbidity during construction.

Biological considerations in site selection include the effects on habitat and ecosystems in the floodplain, and aquatic ecosystems in the stream and associated wetlands. It is advisable that biologists or environmental managers assess this aspect of site selection, but much of the information necessary for a valid assessment of the biological effects and the alternatives available for mitigation must come from the designer. This data evaluation includes the following:

- Alternatives to avoid, minimize, or mitigate impact to wetlands.
- Alternatives to avoid, minimize, or mitigate impacts to cultural resources.
- Alternatives to avoid or minimize encroaching on or crossing streams.
- Effects of circulation of fresh or brackish water in marshes and estuaries.
- Feasibility of providing mitigating measures for the loss of invertebrate population.
- Effects on shaded areas and resting areas for fish.

E. Stream Characteristics. All streams change with time, and the rate and manner in which they will change can be estimated. Planning engineers should be very conscious of stream morphology and be aware that methods are available for quantifying natural changes and changes that can occur as the result of stream encroachments and crossings. Chapter 8, *Open Channels*, includes an introduction to the concepts of stream morphology. *Highway Drainage Guidelines: Hydraulic Analysis and Design of Open Channels* (AASHTO, 2000d), contains more comprehensive discussion and is recommended as a convenient summary of stream response to highway construction. HDS-6, *River Engineering for Highway Environment*, (FHWA, 2001c) is a document on river mechanics, containing valuable material on the qualitative and quantitative analysis of river system response to natural and construction-induced change. The designer should attempt to ensure that highway work within a stream environment does not induce significant change in the stream morphology.

F. Replacement, Repair and Rehabilitation. The decision to replace, repair, or rehabilitate a bridge often is made in the planning and location phase of highway project development. Bridges may be replaced for a number of reasons, including the following:

- Functional obsolescence.
- Structural inadequacies or deterioration.
- Structural damage from collision.
- Alignment and geometric inadequacies.
- Flood-related damage, damage due to scour and debris impact.
- Inadequate clearances for navigation.
- Plans for water resources projects.
- Flood control.
- Inadequate hydraulic capacity.

The hydraulic adequacy of an existing crossing should be examined critically before a decision is made to replace the bridge in-kind, widen or undertake major rehabilitation. The purpose of the examination is to determine if the existing crossing will provide adequately for changed traffic service requirements, to reevaluate flood hazards and risks, and to reexamine the hydrology and hydraulics. Flood experience at an existing crossing is extremely important in analyzing the hydraulic adequacy of stream crossings. The performance of existing crossings during floods is valuable input to the analysis, and it constitutes information that usually is not available at new crossings.

Some problems experienced at a crossing may be due to the occurrence of rare floods rather than the hydraulic inadequacy of the existing crossing. On the other hand, a stream crossing that has served well over a long period of time does not assure its hydraulic adequacy. The odds are 2-to-1 that a 20-year-old bridge has not experienced a 2% exceedance probability flood event and over 4-to-1 that a 1% exceedance probability flood has not occurred during the existence of the bridge. Hydrologic changes may be due to flood control by reservoirs, channelization or levees, development in the watershed which increases the runoff and peak flows, changes in land use practices, and other causes. The hydraulics of the stream may have been changed by channelization, levees, development in the floodplain, different land use in the floodplain, changes in the stream regime, and other causes.

The stability of the stream itself can change with time from natural or construction-induced causes. A stream which was relatively stable at the time a bridge was constructed may be highly unstable when the bridge needs to be rehabilitated or reconstructed. Aggradation, degradation, or lateral instability should be considered when a decision is made to rehabilitate or reconstruct a bridge.

Methods used to analyze the hydrology and hydraulics at bridge sites continue to improve. The adequacy of the analysis for the original crossing design should be examined before undertaking major reconstruction or replacement. In many cases the method used in the original analysis is no longer an appropriate method. The analysis for these crossings should be recalculated using an appropriate method. Additionally, the risk of failure of the existing structure should be reconsidered, including the following:

- Increased traffic volumes.
- Changed traffic service requirements.
- Increased highway construction and maintenance costs.
- Liability for damages to property that could be attributed to the highway crossing.

10.2 COORDINATION WITH OTHER AGENCIES

A. General - Coordination with Other Agencies. Numerous local, state, and federal agencies have vested interests in surface waters. These agencies represent interests in the following areas:

- Water rights.
- Flood control.
- Drainage.
- Natural resource conservation.
- Navigation and maintenance of channels.
- Recreation.
- Flood plain management.
- Safety of flood plain occupancy.
- Aquatic habitat for fish and wildlife.
- Preservation of wetlands.
- Regulation of construction for the protection of environmental values.
- Gaging stations and apparatus for environmental data collection.

Other local, state, and federal agencies have vested interests in historic and archaeological preservation of cultural resources which include an interest in historic bridge structures and archaeological resources. Early coordination with these agencies will reveal areas of mutual interest and offer opportunities to conserve public funds and to resolve conflicts between PennDOT's plans, plans for water resource development, and requirements for resource protection and preservation.

The following is a list of agencies commonly involved with bridge planning and location:

- U.S. Army Corps of Engineers (USACE).
- Federal Emergency Management Agency.
- Conservation Districts.
- River Basin Commissions.
- Pennsylvania Department of Environmental Protection (PA DEP).

- U.S. Coast Guard.
- U.S. Geological Survey (USGS).
- Municipal Water Authorities.
- Local government agencies (counties, townships, cities, towns, and boroughs).
- Federal Highway Administration (FHWA).
- Department of Community and Economic Development.
- Pennsylvania Fish and Boat Commission.
- Pennsylvania Game Commission.
- Environmental Protection Agency (EPA).

B. Water Resource Development Projects. Water resource development projects often require the relocation or reconstruction of existing highways and can interfere with the location or design of proposed highway-stream crossings. Many water resource development projects are planned or authorized for periods of years or even decades before construction begins. Others are never built and may even be permanently stopped by court decisions or regulatory agency actions.

Where stream crossing locations are chosen to take advantage of, or to accommodate, planned water resource development projects (such as reservoirs or stream channel modifications), it should be recognized that the water resource agency's plans may never come to fruition and that the highway facility must be designed for both existing and future site conditions. Planning and constructing a highway facility at a future water resource project site must be studied carefully. The excess cost of building the facility to accommodate a planned water resource project must be considered in selecting the stream crossing site. Some alternatives available to the department include:

- Cost-sharing with the water resource agency.
- Constructing the highway and stream crossing without consideration of the planned water resource development project.
- Choosing an alternate location, if practicable, which would not be impacted by the planned water resource development project.

C. FEMA Designated Flood Plains. Many highway bridge crossings involve flood plains in the NFIP administrated by FEMA. FEMA criteria often influence the design of a bridge over a waterway; therefore, it is important that FEMA requirements are considered in the planning phase of a project and accommodated in the design. Early coordination with the community's NFIP administrator is essential to identify and avert potential problems.

10.3 DATA COLLECTION

For purpose of this section, site information is broadly classified as data collection. Sources of data include aerial and field surveys; interviews; water resources, fish and wildlife, and planning agencies; newspapers; and flood hazard delineation studies. Complete and accurate survey information is necessary to design a crossing which will meet the requirements of the site. The individual in charge of the field hydraulics survey should have a general knowledge of drainage design and coordinate data collection with the hydraulics engineer. The amount of survey data collected and the detail of the data should be commensurate with the complexity of the hydraulics, the severity of stream stability problems, the importance and cost of the structure, and the risk of damage to the highway and of causing damage to other properties and values.

The bridge hydraulics data collection in this section should be coordinated with data collection efforts discussed in Chapter 6, *Data Collection.*

A. Topographic Features. The survey data collected should provide sufficient information for structural and hydraulic engineers to select the location of the crossing, make trial layouts, and conduct hydraulic studies. All significant physical features and cultural resources in the vicinity of the proposed crossing site should be identified and located, particularly those features which could be adversely affected. Features such as residences, commercial and industrial establishments, croplands, wetlands, roadways, railroads, utilities, wells and other facilities can influence design. Their locations and elevations should be established by the survey.

The extent of survey coverage required for the hydraulic design of a highway-stream crossing is related to the topography and stream slope. Backwater above bridges may extend a considerable distance upstream in streams with relatively flat slopes, and features which may be affected by the backwater should be located and identified.

B. Land Use and Development Resources. Present and future land use and cultural resources in the vicinity of a stream affect both the hydraulics of the stream and the design of the highway-stream crossing. Flow distribution, velocities, and the stage-discharge relationships are influenced by development and land use. The potential for floating debris (such as lumber, house trailers or debris from timbering operations) is largely dependent on land use in the watershed and development in the floodplains.

One of the objectives in the location and design of highway-stream crossings is the avoidance of damage to private property from highway-caused flooding. The practicability of avoidance generally can be evaluated on the basis of the probability of the highway causing incremental flood damage, the economic cost of the probable damage, and the cost to avoid and/or mitigate the damage should be considered. Information on land use, such as crops, parks and recreational development, and the elevation, use and value of structures that may be affected should be collected.

C. Hydrologic Data. Data requirements for hydrologic analyses are largely dependent upon the methods used to estimate flood flows. Commonly needed hydrological data include information on flood flows, drainage basin characteristics, highwater during past floods, flood history at existing structures, channel geometry, and precipitation. A more detailed and complete discussion of data needs for hydrologic analysis are contained in Chapter 7, *Hydrology*.

D. Flood Data. Flood flow data and streamflow records are available for many locations on streams in the US. Data collected by the USGS and other agencies are published periodically in the surface water records which are available at local offices of the USGS. Data also are available from other sources, such as universities and local and state governmental agencies. Railroad and state highway maintenance files often contain valuable information on flood stages. Newspaper and magazine accounts of floods may contain information from which stages and water surface profiles can be reconstructed. Flood marks and other positive evidence of unusual flood events are valuable data, especially where no gaging station records are available or records are short. The USGS publishes open file reports which document unusual flood events. Flood hazard reports and flood insurance studies compiled for FEMA may contain information on floods that have 10-, 50-, 100-, and 500-year return periods.

The USGS has established a service, the National Water Information System (NWIS), to provide assistance in identifying, locating, and acquiring available water data through a national network of local assistance centers. These centers provide access by computer terminal to information about the availability of streamflow data, groundwater levels, sediment discharges, and the quality of surface and groundwaters from stations operated by more than 300 agencies. Information regarding NWIS can be obtained from local offices of the USGS.

Records should be examined for inconsistencies and evidence of changes in the stream hydrology. Discrepancies in records obtained from different sources may be the result of inadequate accuracy standards of one source. Changes in the hydrology of a stream may be the result of changes in land use or urbanization in the watershed, channelization, flood control, or dam projects. A chronological plot of annual maximum flood peaks may reveal trends in the hydrology of a stream that would not otherwise be detected. Records which show some changes in the stream hydrology are said to lack stationarity. Stationarity should exist in the data used for the hydrologic analysis, and conditions should be reasonably representative of existing or present watershed conditions. For assistance with evaluating the stationarity of a stream gage record, contact the Pennsylvania District of the USGS.

A considerable period of time usually elapses between the conception of a highway project and its construction. During this period, important information can be collected if flood events occur. Correlation of stage at the bridge site with a stream gaging station upstream or downstream will help to verify the stage-discharge relationship by providing one known point on a curve that is otherwise based entirely on computations. If there is no gaging station on the stream, stage and discharge data should be collected to aid in both the hydrologic and hydraulic analyses. This requires preparation in advance so that resources can be quickly mobilized when a flood occurs. An agreement can be made with the USGS, or another similarly qualified agency, to collect flood data at bridge sites. This data often can be used to improve designs.

E. Highwater Information. Reliable highwater data can provide invaluable information for establishing the stage and discharge of past floods, for locating existing hydraulic controls, and for establishing highway profiles. Obtaining more than one highwater mark and its exact location for the same flood event is recommended. Several dependable highwater marks are required to compute flood discharge by the slope-area method.

It is extremely important that experienced personnel be used to identify and evaluate highwater marks because the apparent evidence of highwater can be deceiving to the uninitiated. Highwater marks should be flagged and surveyed as soon as practicable after a flood because they may disappear within weeks in heavily vegetated areas. If an unusual flood has not occurred for several years, the highwater marks located by even experienced personnel are likely to represent a relatively small flood. Highwater stages may be misleading since they are sometimes caused by ice, log jams, confluences, or land use which has subsequently disappeared. Such stages may be on the order of a meter (3 feet) off the normal stage for the same discharge. Examination of aerial photographs taken during the flood, or more than one indirect measurement taken at reaches some distance apart, can assist in identifying these stages as abnormal.

Information on highwater elevations can be obtained by observing debris and mud lines on tree trunks and bridge abutments, wash-lines and fine-debris lines on banks and bridge approach fills, wisps of grass or hay lodged in tree limbs and fences, and evidence of erosion and scour. Interviews with residents, commercial and school bus drivers, mail carriers, law enforcement officers, highway and railroad maintenance personnel, and others who might have an opportunity to observe unusual floods may yield additional information. The date of the flood occurrence, the name and address of the observer, and the stage and location of the observation should be recorded. The observed frequency of occurrence should be noted since reliable information that a stream reaches a certain elevation every two or three years provides important frequency information for the designer. A few hours spent in interviewing several people who are familiar with the flood history of a stream can result in substantial savings in construction, liability, and future maintenance, and can lead to improvements in the design.

Important highwater information includes:

- Major flood events since construction and dates of occurrence.
- Flood heights upstream and downstream of the bridge (flood stages within bridge openings generally provide little useful information).
- Observed differences in water surface elevations upstream and downstream of the structure at as many locations as the information is available; dates and flood magnitudes should be included, as available.
- Observation on flow distribution, overbank flow, flow directions, and velocities.
- Direction of flow relative to piers and relative to the low-water channel.
- Observed drift size and quantities.
- Clearance and freeboard.
- Duration of flooding.
- Damage to the highway, slope protection, stream control measures, bridge and other property.
- Magnitude of flood relative to other notable floods.
- Photographs of the structure, flood events, stream and any other feature that will aid in the design of the proposed bridge.
- Bridge design details including deck profile, superstructure design, pier design and orientation, and bridge rail design.

F. Existing Structures. Structures in the vicinity of a proposed bridge may have experienced unusual floods or floods which were sufficiently large to provide useful information. Information on floods which have occurred since the construction of bridges may be obtained from highway maintenance files, from residents of the area, and from highwater marks.

Data at existing structures should include the highwater information noted above and as much of the following as is available or is practical to obtain:

- Date of construction.
- Location relative to proposed structure.
- Hydrologic and hydraulic design data, assumptions, and calculations.
- Cross section under bridge from as-built plans.

- Present cross section under bridge.
- Type and size of materials in stream bed and banks.
- Condition of structure.
- Sediment deposits, scour and erosion.
- Evidence of headcutting in stream.
- Roadway profile extending to the extremities of the floodplain.

G. Channel Characteristics. Survey data are required to analyze the streamflow characteristics and stream morphology at bridge sites. In both analyses, aerial photographs are useful in identifying types of vegetation, sizes and locations of sandbars, thalwegs, stream controls, geological formations, existing stream bank protective works, and old meander channels. A series of aerial photographs taken over a period of years can be used to determine the pattern and estimate the rate of movement of meander bends. Aerial photographs also can be used to determine whether the stream is straight, meandering or braided, and to detect evidence of stream degradation or aggradation.

Cross sections of the stream channel and floodplains are required to establish the stage-discharge relationship and conveyance. Sufficient cross sections should be obtained to provide an accurate representation of the channel and floodplains. If a stream control section, such as constriction or confluence of dense vegetal cover, exists downstream of the crossing site, cross sections should be obtained so that a water surface profile can be computed beginning at the control section. Cross sections should be extended laterally to include the total floodplain for the design and larger floods. The cross sections should be normal to expected flood flow directions and not necessarily normal to the stream channel. The number of sections required is dependent upon flow conditions at the site.

Data on land use, vegetative cover, and steam bed material should be obtained to assess roughness characteristics for use in conveyance computations. Photographs of the channel and floodplains and descriptions are necessary for use in the analysis, and a site inspection by the designer may be necessary to ensure a good estimate of roughness coefficients.

Other characteristics necessary to make design decisions should be noted. These include soil types in the stream bed, banks and overbank areas, and stream bed material gradation, if possible. Features such as rock outcrops and meander plugs, or deposits of cohesive materials in old channel bendways due to a cutoff should be noted (AASHTO, 2000b). Evidence of drift and debris size and volume, ice conditions, bank caving, waterfalls, headcuts, and other conditions which would affect abutment and pier location, orientation, and type should be recorded. Photographs of the channel and stream bed, preferably in color, can be valuable aids to the designer and serve as excellent documentation of existing conditions.

H. Environmental Data. The need for environmental data in the engineering analysis of a stream crossing stems from the obligation to investigate possible impacts due to specific design configurations. In those cases where an environmental assessment has been completed earlier in project development, part or all of this evaluation already may have been accomplished. Where an environmental assessment has not been made, the data developed for planning and location of the crossing is often of value in the engineering-environmental analysis.

The engineer and environmental specialist working as a team need information on water quality standards for the stream. Some of this information is available in the water quality standards and criteria published in Chapter 93, Water Quality Standards by the PA DEP. Physical, chemical and biological data for many streams also are available from state and federal water pollution control agencies, USGS, and from municipalities, water districts and industries which use the stream as a supply source.

A description of existing water circulation patterns and definition of the types and extent of potentially affected wetlands are necessary for the team to assess the effects of each bridge-fill configuration. Data on circulation, tides, water velocity, water quality, and wetlands may be available from the USGS, U.S. Fish and Wildlife Service, the USACE, universities, and marine institutes, as well as other state, federal and local agencies and organizations.

Information on fish and fish habitat often is necessary in order to evaluate proposed channel modifications and to design replacement habitat. Fish and fish habitat information is available from state and federal fish and game agencies, such as the Pennsylvania Fish and Boat Commission, the U.S. Fish and Wildlife Service, and the Pennsylvania Game Commission. An analysis of the material in the stream bed and banks, as well as proposed fill materials, may provide essential data for projects in critical water-use areas, such as near municipal or industrial water supply intakes.

It may be necessary for the highway agency to collect data at critical sites, if the required information is unavailable from other sources.

Data needs may be summarized as follows:

- Information necessary to define the environmental sensitivity of the crossing; e.g., water use, water quality and wetlands information.
- Information necessary to determine the most practicable, environmentally compatible design; e.g., circulation patterns and sediment transport data.
- Information necessary to define the need for mitigation measures; e.g., fish habitat, sediment analysis, water use, and quality standards, and to design these measures if necessary.

I. Site Plan. A general site plan should be prepared to show the physical features of the site. Although a contour map of a larger area generally is required for these purposes, a contour map of the site is helpful in defining flow directions, and in making decisions on span lengths and abutment and pier locations and orientation. The site plan should be prepared as noted in Publication 13M, Design Manual, Part 2, *Highway Design*, Section 10.7.

J. Field Reviews. Field reviews are highly desirable 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. A field review also is useful in confirming that additional site data are necessary. The selection of roughness coefficients, the evaluation of apparent flow directions and concentrations, and first-hand observation of land use and floodplain development are the factors that most often need to be confirmed by field inspection. Consultation with construction and maintenance engineers regarding site conditions often will provide information regarding factors that they have found to be important to their responsibilities.

10.4 BRIDGE HYDRAULIC CONSIDERATIONS

A. General - Bridge Hydraulic Considerations. Design criteria are the real means for placing accepted policies into action. These policies become the basis for the selection of the final design configuration of the stream-crossing system. After establishing the flood frequency and the stage-discharge curve according to the principles described in Chapter 7, *Hydrology* and Chapter 8, *Open Channels*, the type of cross-drainage facility can be chosen. Usually this choice is between a bridge or a culvert.

In some cases the choice may not be especially clear. It may be useful to evaluate both types of facilities and to make a choice based on performance and economics. If the stream crossing is wide with multiple concentrations of flow, a multiple opening facility may be necessary. The hydraulic analysis of a highway stream crossing for a particular flood frequency involves the following general considerations related to the hydraulic analyses for the location and design of bridges:

- Backwater associated with each alternative vertical profile and waterway opening should not significantly increase flood damage to property upstream of the crossing.
- Effects on flow distribution and velocities; the velocities through the structure(s) should not damage either the highway facility or increase damages to adjacent property.
- Existing flow distribution should be maintained 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.
- Freeboard at structure(s) designed to pass anticipated debris and ice.
- Risks of damage.
- Stream instability countermeasures.
- Minimal disruption of ecosystems and values unique to the floodplain and stream.
- Highway level of service compatible with that commonly expected for the class of highway.
- Design choices should support costs for construction, maintenance and operation, including probable repair and reconstruction and potential liability that are affordable.

B. Design Storm. The flood-frequency analysis for a stream crossing is discussed in Chapter 7, *Hydrology*. For purposes of this section, the hydrologic analysis consists of establishing peak flow-frequency relationships for the crossing and flow-duration hydrographs as necessary for the site analysis. Stage-discharge relationships are a part of the hydraulic analysis, and the hydraulic analysis for design includes an evaluation of the effects of the highway crossing for a range of flow rates.

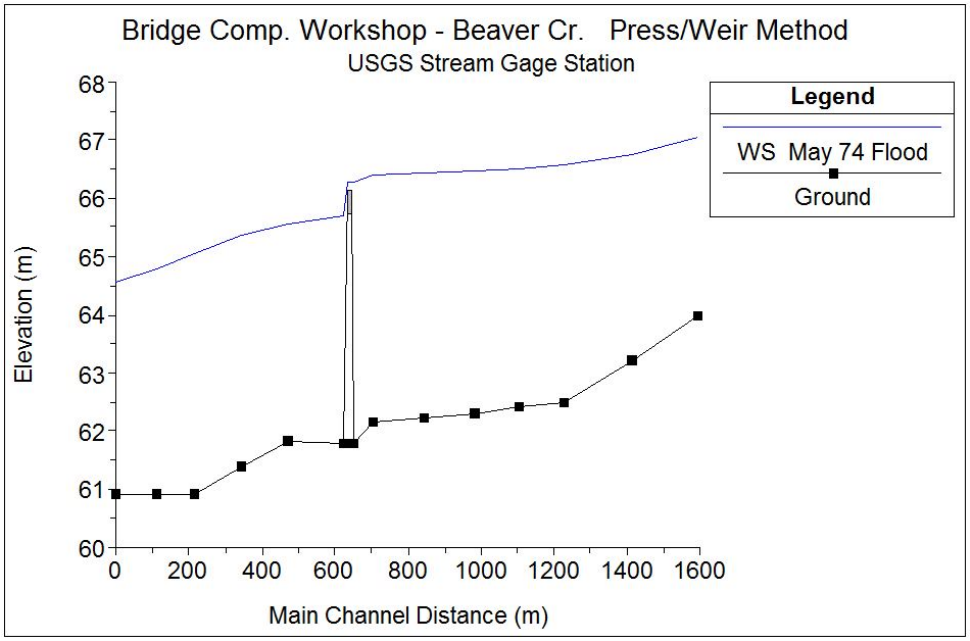
The flood selected for design may differ from the flood of a preselected exceedance probability. Relatively rare floods (i.e., floods larger than the design flood) may need to be considered in the hydraulic analysis. For these reasons, and because of uncertainties in the hydrologic analysis, the hydrologic analysis should define peak flow-frequency relationships for a wide range of events for use in the subsequent hydraulic analysis of the crossing. The hydraulic analysis may need to consider the ordinary highwater and the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year flood events. It is easier to develop the discharges for all of these at one time during the hydrologic phase of the analysis.

C. Flow Near Bridges. When flood flow encounters a restriction in the natural stream, adjustments take place in the flow regime in the vicinity of the restriction. Flow is contracted through the bridge and then must expand as it exits the bridge. An exchange of energy between potential energy and kinetic energy is required to maintain the contraction and expansion of flow. In addition, energy is required to overcome friction and disturbances associated with piers and abutments. This exchange of energy is reflected by an increase in the depth of flow upstream of the encroachment, termed backwater, as shown in Figure 10.1.

In subcritical flow conditions, the backwater tails off upstream until the undisturbed water surface elevation is reached. The distance upstream over which backwater occurs is dependent on the channel conditions and flow conditions, see Chapter 8, *Open Channels*. The maximum backwater tends to occur a short distance upstream of the bridge, as shown in Figure 10.1. The relatively steep water surface gradient between the maximum backwater and the opening is termed the drawdown area.

In a stream channel with supercritical flow conditions, it is possible to have a constriction, such as a bridge, that does not affect the upstream flow conditions. However, if the constriction is severe enough, it could cause a change in flow regime such that a subcritical flow backwater occurs upstream of the bridge with a hydraulic jump forming upstream of the bridge at the transition from supercritical to subcritical flow.

Figure 10.1 Backwater at a Stream Crossing



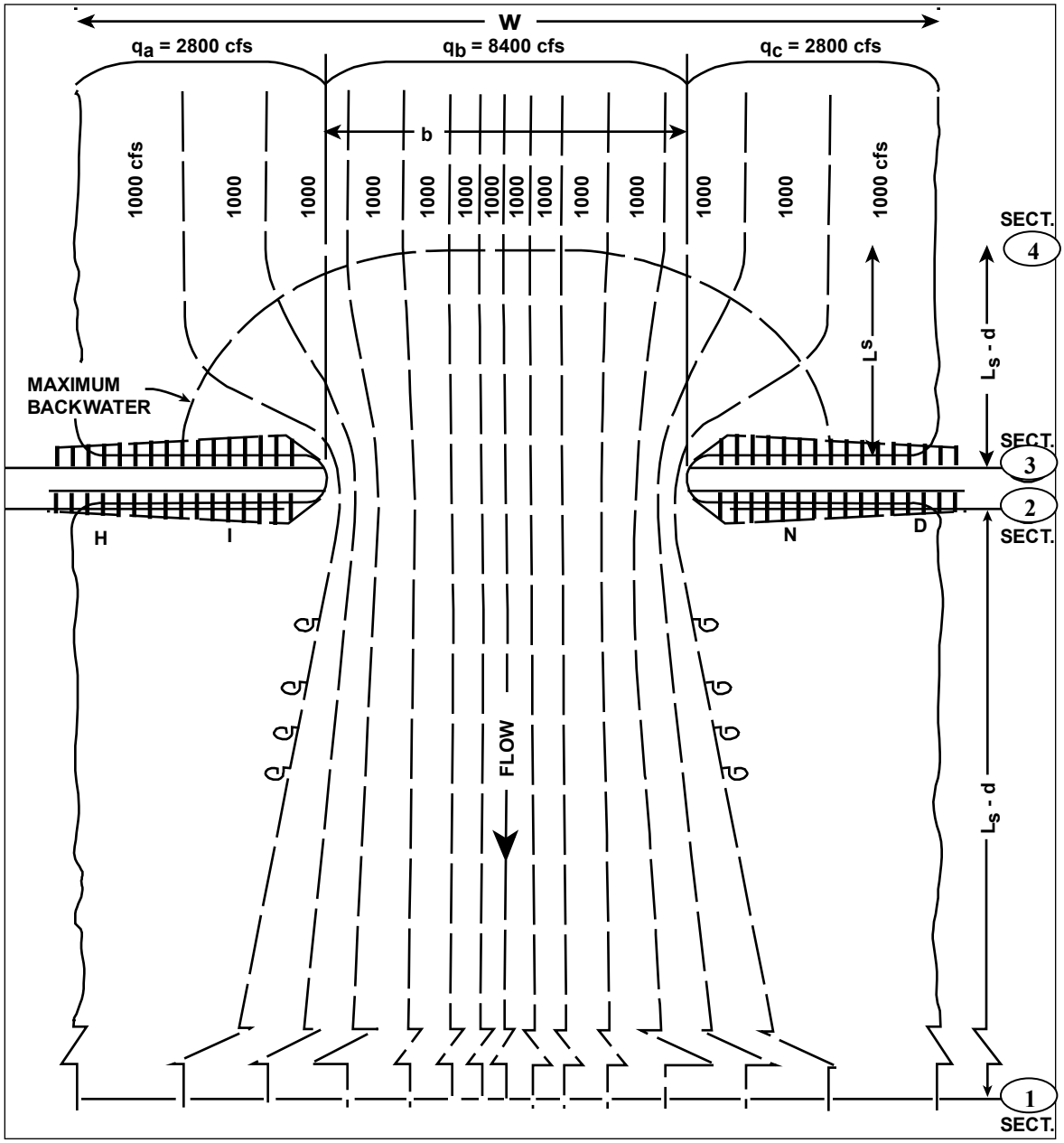
As subcritical flow moves toward the bridge opening, the velocity increases as the flow lines begin to converge and become constricted between the bridge abutments, see Figure 10.2. This increase in velocity can result in scour along the embankment and through the bridge. At the bridge abutments, the high velocities can cause severe

turbulence and eddies. Piers in the waterway create additional local turbulence and vortices. Turbulence, eddying, and vortices often result in scour.

The hydraulic design of a bridge over a waterway involves establishing a location, bridge length, orientation, roadway profile, and bridge profile such that the risks associated with backwater and increased velocities are not excessive.

D. Allowable Backwater Due to Bridges. For design storm conditions, the allowable backwater should be established by the designer based on consideration of design policies, regulatory requirements, and the risk associated with potential flood-related damage to the highway and adjacent properties.

Figure 10.2 Typical Flow Directions through Bridge Opening

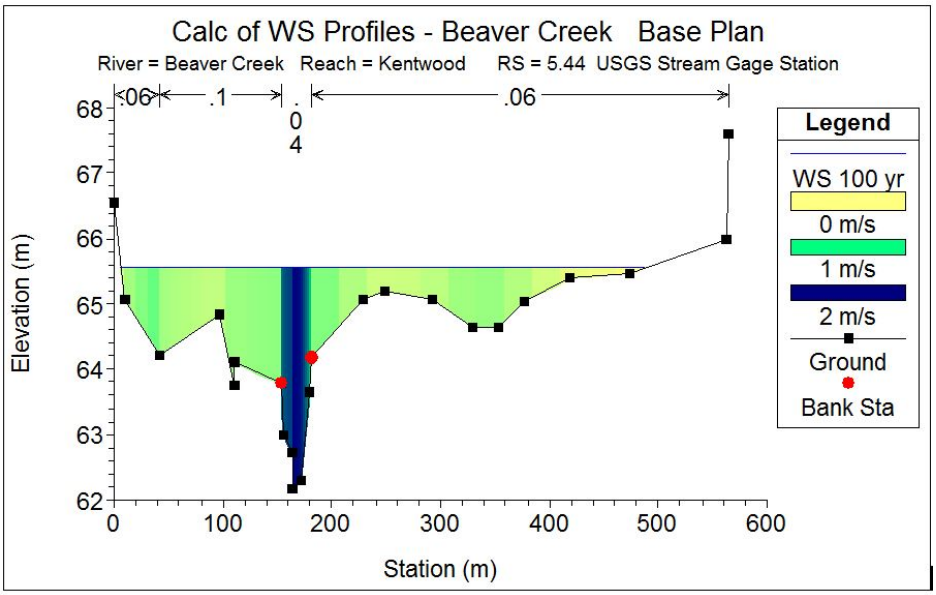


Analysis of the backwater associated with the 100-year event must conform with FEMA's NFIP requirements, where applicable. Other considerations include (a) the Federal Aid Policy Guide 23 CFR 650, (b) 25 PA Code § 105 and § 106, and (c) PennDOT policy regarding mitigation of occasional inundation. Generally, if the risk associated with the backwater is deemed to be excessive, the designer should consider one or more of the following actions:

- Consider other design alternatives to reduce the backwater.
- Perform a detailed economic analysis of various design options.
- Design channel improvements.
- Determine the incremental area of high risk that will be inundated and arrange purchase of flood easements or acquisition of the affected property.

E. Flow Distribution. Flow distribution is the proportion of the total flow in the stream that is conveyed by each of the various portions of the cross section. Velocity distribution is the average velocity in each of the subsections and may not be indicative of locally high or low velocities. Hydraulic computations; however, are usually sufficiently accurate for design purposes. Figure 10.3 shows the flow distribution for the main channel and floodplains of a stream at a given discharge. Flow distribution usually will change with changes in stage and discharge and should be estimated for the various flow rates of interest in the design of the crossing.

Figure 10.3 Flow Distribution (1m = 3.28ft)



The analysis of flow distribution will reveal sections where the flow rates are relatively high and sections which are relatively ineffective in conveying flow. This information is necessary to properly locate bridges or other openings on the floodplains, determine bridge lengths, locate overflow sections in approach roadways, and to evaluate the need for and location of spur dikes and other protective and preventive features to be incorporated into the design.

Flow distribution will be disturbed during some floods by any stream crossing that uses a combination of fill and bridge within the flood plain; however, flow distribution should be preserved to the extent practicable in order to:

- Avoid disruption of the stream-side environment.
- Preserve local drainage patterns.
- Minimize damage to property by either excessive backwater or high local velocities.
- Avoid concentrating flow in areas that were not subjected to concentrated flow prior to construction of the highway facility.
- Avoid diversions for long distances along the roadway embankment.

Flow distribution is determined by converting the conveyance of each subsection to discharge. The results should be examined carefully and, when possible, compared with observed floods to determine whether the computed flow

distribution is reasonable. There may be topographic features, vegetative cover, development or other physical features upstream or downstream of the cross section which would make the computations invalid.

Generally, the disturbance of flow distribution can be minimized by establishing bridge openings at the areas of high conveyance. For many situations, one-dimensional analysis techniques will suffice for determining optimum bridge locations. Complex sites, such as those at a bend, and skewed crossings can be analyzed with one-dimensional models only by using a great deal of intuition, experience, and engineering judgment to supplement the quantitative analysis. Unfortunately, complex sites are encountered frequently in stream crossing design. The development of two-dimensional methods of analysis greatly enhances the capabilities of hydraulic engineers to deal with these complex sites.

F. Bridge Scour and Stream Degradation. A scour analysis is required for all new bridges, replacements, and widenings. Scour analysis procedures are discussed in Section 10.9.

If a scour analysis indicates high depths of potential contraction scour, it may be more cost effective to provide a wider structure than that required by the amount of flow through the structure. On the other hand, it may be more cost effective to design foundations and armoring to withstand local scour depths. The designer must decide which alternative is more desirable.

Bridge foundations must be designed to adequately withstand anticipated scour as noted in Publication 15M, Design Manual, Part 4, *Structures,* Policies and Procedures, Chapter 7.

Stream stability issues, such as potential vertical and horizontal degradation, may warrant accommodations in the design of the bridge. If the channel is vertically degrading, it is likely that as the channel deepens the channel banks will slough, resulting in widening. Also, where significant meandering is occurring, meanders tend to migrate downstream and increase in amplitude. For possible measures, such as river training techniques, see HEC-20, *Stream Stability at Highway Structures* (FHWA, 2001d), and HEC-23, *Stream Instability Counter Measures* (FHWA, 2001a).

G. Freeboard and Low Chord. The lowest level of the bridge superstructure is called the low chord elevation. Navigational clearance and other reasons notwithstanding, the low chord elevation is equal to the sum of the design water surface elevation (highwater) and the freeboard.

If you do not have constraints that limit the height of your profile, consider adding freeboard for passage of debris. Generally for bridge replacement structures, the low chord should not be lower than that of the structure being replaced unless geometric constraints exist. The bridge opening generally should not be reduced where hydraulics is a controlling factor.

10.5 STREAM CROSSING DESIGN

The process by which a highway-stream crossing design is developed has changed significantly over the last few years. Fortunately, hydraulic engineering technology is available to meet the challenge of changed construction economics as well as the needs created by increased traffic demands and the concerns for highway user safety, the environment, and increasing flood losses caused by floodplain development.

Procedures for the design of a stream crossing generally should follow the sequence indicated below:

1. Hydrologic Analysis. Estimate of the stream discharge at the site for a range of flood exceedance probabilities. All existing flood control measures, land use and anticipated changes in the watershed should be considered in the hydrologic analysis.

2. Stream and Floodplain Analysis. The estimate of streamflow characteristics at the crossing site for the range of discharges being considered. These estimates include stages, flow distributions, velocities, stream morphology, sediment transport and the influence of land use and development.

3. Identification of Criteria for Design. Certain criteria or standards, by which a design is judged to meet the objectives of a crossing design. These criteria may override risk and economic considerations in some

crossing designs, and may include standards imposed by legislation, design policy, or the importance of the highway (e.g., national defense or for emergency vehicle access), and preclude politically, environmentally or socially unacceptable solutions such as placing an embankment in a significant marsh or wetland, or leaving a community isolated during floods.

4. Analysis of Design Alternatives. The analysis of a highway-stream crossing involves an engineering, environmental, and economic evaluation of various design alternatives. The objective of the analysis is to achieve a design which will have the least expected cost to society considering expected losses and capital costs within the constraints imposed by the criteria for design.

The hydraulic analysis of the stream is discussed in Section 10.4. This section deals primarily with the design procedure beginning with the identification of criteria for design.

A. Design Criteria for Highway-Stream Crossing System. Hydraulic engineers and other highway engineering and environmental personnel are encouraged to think of a highway-stream crossing as a system consisting of the stream and its floodplains, the bridge(s) provided to pass floods of an estimated exceedance probability, and the roadways on the floodplains which may be overtopped when that flood is exceeded.

It is customary to arrive at the design of the highway components of the stream-crossing system individually based on input from the several disciplines involved. The common objective of all disciplines, however, must be to provide a safe facility for traffic, prudent expenditure of public funds, and a design which minimizes damage to property and the environment, to the extent practicable.

First and foremost, all floods which occur during the life of the facility will pass the crossing site. With unlimited funds for construction, any crossing can be designed for a very small probability of damage or traffic interruption. It is good economics and good engineering, however, to weigh the capital costs of measures taken to avoid damage and traffic interruption against the probability of future costs. In order to assess the future costs, risks associated with floods that are smaller than, as well as in excess of, the customary design flood must be evaluated, including the largest flood that must pass through the highway structure(s) and larger floods that will overtop the highway. Thus, all components of the highway-stream crossing are a part of the system that must be considered in the hydraulic design.

The hydraulics of the stream component of the highway-stream crossing system are discussed in Section 10.4. Highway and bridge components are discussed in the following sections.

1. Design Strategies/Alternatives. Highway profile and alignment ordinarily are considered somewhat independently of bridge waterways except for the consideration given to providing some elevation differential between the conventional design highwater and the roadway surface. As part of a highway-stream crossing system, both profile and alignment are controls on the magnitude of the maximum flood that will pass through the waterway opening(s) provided. The bridge waterway(s) and the roadway alignment and profile together determine the capacity of the system to pass floods without overtopping of one or both components. The effects of the roadway profile on the adequacy of the waterway opening(s) should be considered in establishing the profile and in designing the bridge waterway(s).

The stage-discharge relationship for the stream and backwater associated with the design alternative are the hydraulic considerations for establishing the highway profile. The horizontal and vertical alignment of the highway are factors that must be considered in establishing the waterway design. The vertical alignment will be considered first in this discussion.

Several profile alternatives are available for consideration, dependent upon site topography, traffic requirements and flood damage potential. The alternatives range from crossings which are designed to overtop frequently to crossings which are designed to overtop rarely or never. The alternative selected will depend upon the criteria established for the design, construction costs, risks, stage-discharge-duration relations, flow distribution and scour consideration.

In Figure 10.4, vertical sag curve profile, the bridge is near the low point in a sag-vertical curve profile. Examples of the use of this profile configuration are the use of low bridges in rolling terrain for low traffic roads which are frequently overtopped and high bridges in rugged terrain which probably will never be

threatened by flood. A distinctive feature of this profile is the certainty that the bridge structure will be submerged when any overflow of the roadway occurs. If possible, bridges on vertical sag curves should be avoided because drift can accumulate in the superstructure and on the bends and accentuate general and local scour. Also, large drift can produce high impact forces on the structure, possibly causing structural failure, especially if scour has affected the foundations. If a vertical sag curve cannot be avoided, and there is even a small probability of overtopping, it is advisable to avoid curbs and use open-type railing with this profile in order to minimize damage for high velocity flow around the ends of parapets. Bridges on vertical sag curves also are undesirable as far as deck drainage is concerned because ponding will occur on the deck if inlets have insufficient capacity to intercept the flow or become clogged with ice or debris.

Figure 10.5, crest-vertical curve profile, illustrates a profile which may be used where the valley width is sufficient to utilize a profile which allows the roadway to be overtopped without the superstructure of the bridge being submerged. Variations of this profile may be used in locations where the stream channel is located on one side of the floodplain, i.e., an eccentric crossing, and the profile allows overtopping of the approach roadway on only one side. The difference between the low point in the roadway profile and the low chord in the bridge superstructure can be varied, within geometric constraints, to meet requirements for maintaining free surface flow and to accommodate passage of ice, debris and drift.

A third profile alternative is shown in Figure 10.6, level profile. Variations in this profile include a slight crest-vertical curve on the bridge to establish a camber in the superstructure. With this profile, all floods with stages below the profile elevation of the roadway and bridge deck will pass through the waterway opening provided. The disadvantages of a sag-vertical curve profile are applicable, and the same cautions should be exercised. With either profile configuration, severe contraction scour will be likely to occur under the bridge and downstream for a short distance when the superstructure is partially or totally submerged. The velocity of flow and depth of the superstructure may impose large hydraulic forces on the bridge superstructure. The accumulation of debris or ice on the upstream side of the structure can increase the effective depth of the superstructure, impose larger hydraulic forces on the bridge superstructure, and increase scour depths. Since no relief from these forces is afforded, crossings on zero gradients and in sag-vertical curves are more vulnerable than those with profiles which provide an alternative to forcing all water through the bridge waterway.

Figure 10.4 Vertical Sag Curve Profile

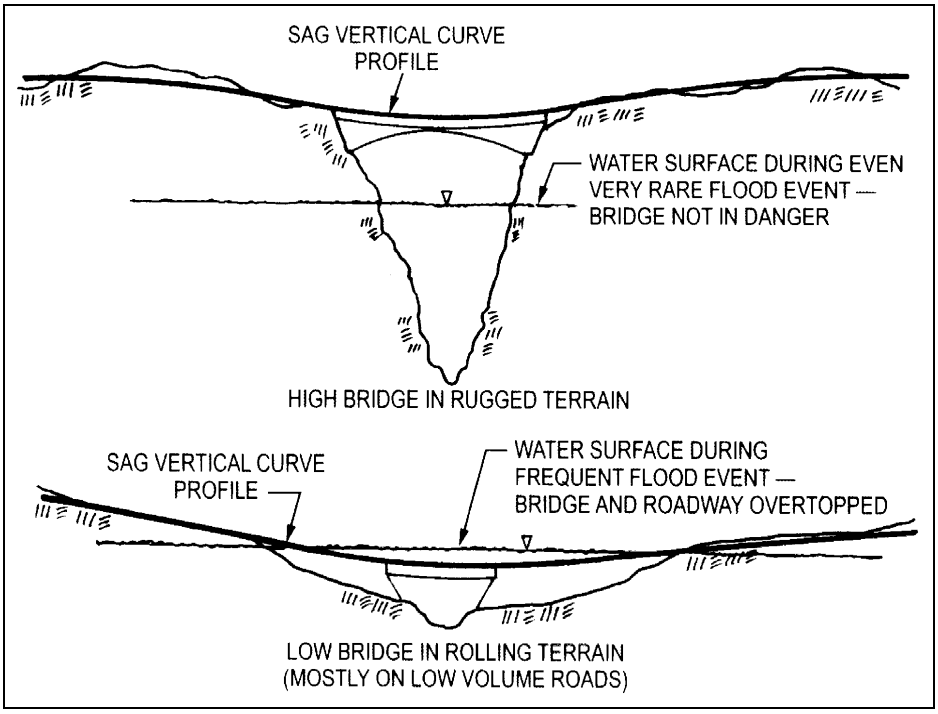


Figure 10.5 Crest-Vertical Curve Profile

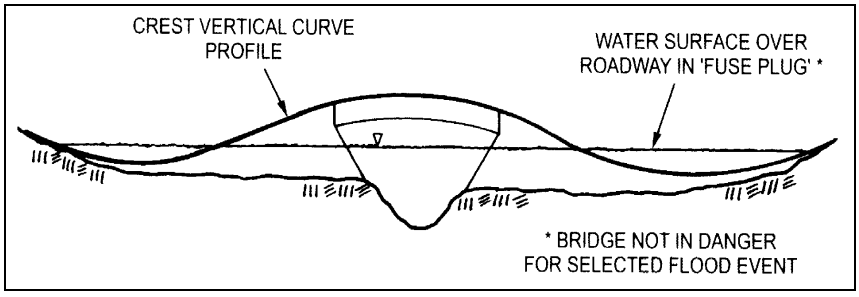
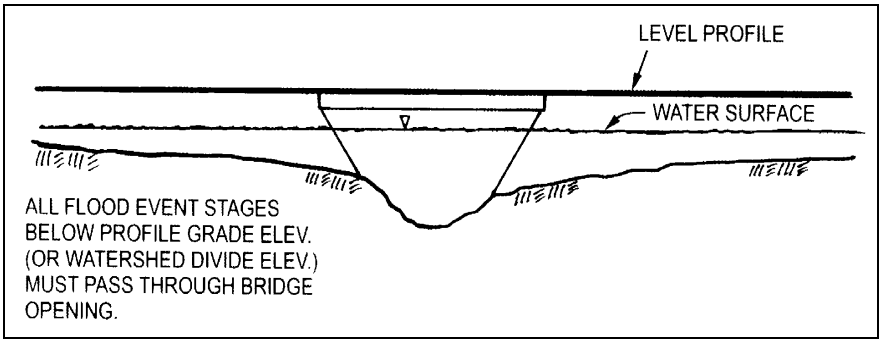


Figure 10.6 Level Profile



The horizontal alignment of a highway at a stream crossing also must be considered in selecting waterway opening location(s) and design, and the crossing profile. Potential lateral migration of the stream could threaten the stability of the structure, see Figure 10.7.

Water surface elevations during peak flow along a skewed highway-stream crossing are not equal because of the gradient in the water surface profile. The elevations upstream of the crossing will differ, sometimes considerably, depending upon the water surface gradient, land use, the floodplain width and the severity of the crossing skew. Water surface differentials between the upstream side of the crossing and the downstream side will be much greater at points A and B than the backwater above the bridge, as illustrated, in Figure 10.8.

The apparent effect of the crossing on water surface elevations is sometimes highly exaggerated because of the difference in water surface elevations from one side of the road to the other. Figure 10.9 is an example in which the roadway was cut in a location equivalent to point A in Figure 10.8, because of the differential across the road during an extreme flood. It may be advisable, in many locations, to provide waterway opening(s) at locations A and/or B, Figure 10.8, in order to avoid large head differences from the upstream to the downstream side of the roadway. Again, this is dependent upon several considerations including the land use upstream and downstream, water surface gradient, the severity of the skew, backwater from the highway crossing, and the length of the roadway encroachment in the floodplain.

Large head differentials from the upstream side to downstream side of skewed crossings can be explained by referring to Figure 10.8. Flood waters upstream of the embankment near location B will be diverted toward the structure since there will be a gradient from B. Downstream, water must flow from the bridge toward B in order to fill the area downstream of the roadway on the left floodplain. Thus, the water surface elevation upstream at B will be higher than at the bridge and downstream at B will usually be lower than downstream of the bridge.

Figure 10.7 Lateral Scour at a Bridge

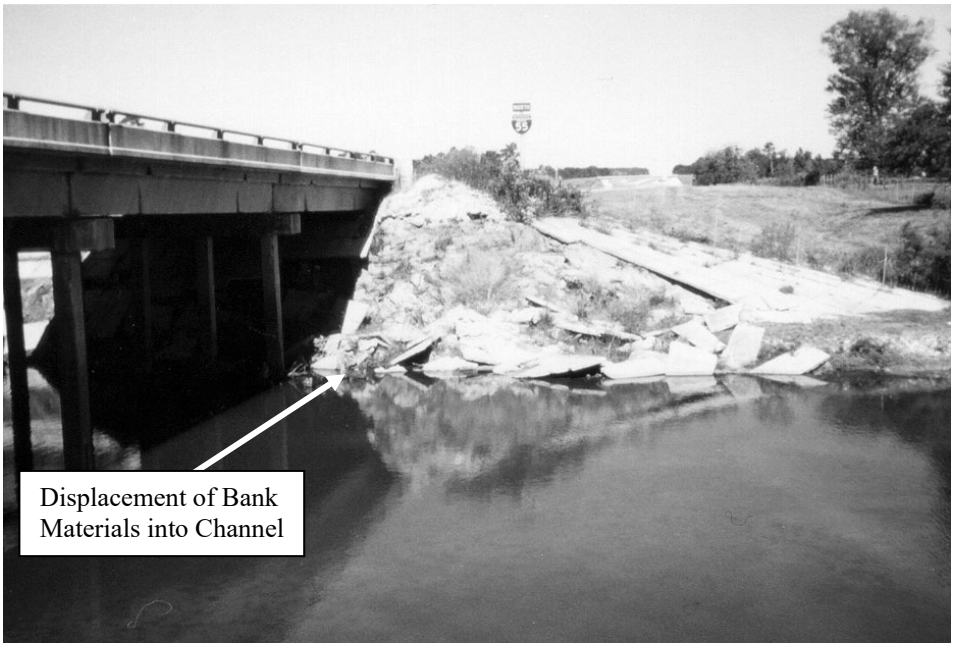


Figure 10.8 Upstream and Downstream Water Surface Elevations at a Skewed Crossing

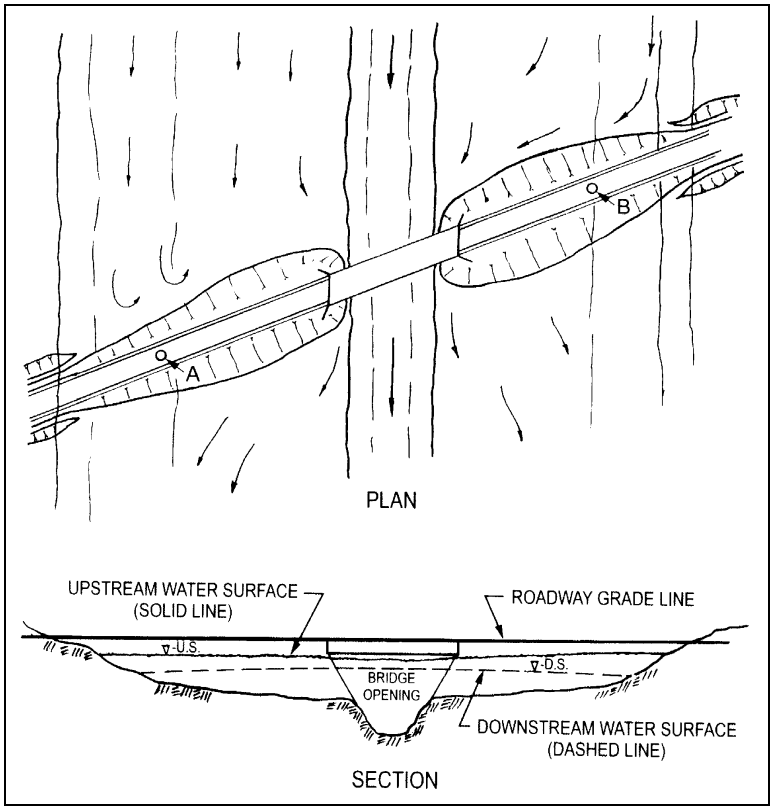


Figure 10.9 Roadway Cut through the Embankment Caused by Head Differentials Across the Road at a Skewed Crossing



Similarly, the water surface elevation on the upstream right floodplain near location A will be approximately equal to the water surface elevation at the bridge plus the velocity head at the bridge. Thus, there will be no flow from A to the bridge along the roadway embankment. On the downstream side, flood waters will flow from the bridge toward A and, dependent on the velocity head at the bridge, will be lower downstream of A than downstream of the bridge. Therefore, the water surface differential from upstream to downstream will be greater at both A and B than at the bridge.

One-dimensional methods (step backwater) may be inadequate to provide a quantitative analysis of water surface elevations up- and downstream of a skewed highway crossing of a stream. Finite element and finite difference models are two-dimensional methods that can be applied in some complex situations. These models enable designers to study the water surface elevations in cross sections rather than in profile only and can identify locations where undesirable head differentials could occur. These are more complex models which require more site information and a greater length of time to use. The accuracy provided by the two-dimensional model is not justified in most cases; thus a one-dimensional model is used, tempered with sound engineering judgment.

Discussion of the hydraulic analysis of stream crossings is provided in Section 10.5.B. Structural features that can have a significant effect on the performance of the system are discussed later in this section.

2. Waterway Openings. The design flood for a PennDOT structure is based on the roadway classification. The design flood may be adjusted based on horizontal and vertical alignment of the approach roadways, preservation of the natural flow distribution of the stream, environmental impacts, and/or avoidance on creating undue hazards to the highway and other properties.

Concerns other than hydraulic requirements which influence waterway opening location and size include clearances for navigation, roadway geometries, terrain, soil stability at abutments, access to adjacent properties, intersections and interchanges with other roads, separations for other roads or railroads, wetlands, economics, and numerous others. Discussion here will be limited to hydraulic and economic considerations in locating and sizing waterway openings.

(a) Location. The choice of location of a waterway opening at a stream crossing site with limited floodplain widths is not difficult because it is readily apparent that one opening will suffice or that it is not physically practicable to use more than one opening. Similarly, when approach roadways are not significantly higher than the floodplains, auxiliary waterway openings on the floodplains are either unnecessary or culvert-size openings for local drainage are all that is necessary.

The location of waterway openings in the highway-stream crossing system is more complex for designs for rare floods and at sites with extensive floodplains. In this discussion, it is assumed that an opening will be provided at the principal channel of the stream and that available options include a wider opening at that location and auxiliary opening(s) on the floodplain or some combination of these options.

Several factors influence decisions on the location of waterway opening(s) to provide for flood passage. Basic objectives in choosing the location(s) of auxiliary opening(s) include maintenance of flow distribution and flow directions to the extent practicable, provision for relatively large flow concentrations in the floodplains, avoidance of diversion of floodplain flow along roadway embankments for long distances, and considerations of backwater and scour damage to the highway and other property. Site conditions, economics, budgetary constraints, and the horizontal alignment of the highway limit the extent to which these objectives can be met. The objectives are complementary in that the purpose in maintaining flow distribution and direction is to minimize damage to the floodplain environment and to avoid excessive backwater and scour. Providing for large flow concentrations achieves similar purposes, as does avoiding long distance diversion along roadway embankments.

Other site-specific factors which influence opening location are local drainage, the possibility of causing a cutoff in a meander bend, other transportation facilities in the vicinity, floodplain use, and the horizontal and vertical alignment of the highway.

The need to provide for local drainage occurs where an area on the floodplain will not drain after the highway is constructed and where the highway alignment intersects a tributary stream upstream of its

confluence with the main stream (Figure 10.10). In either case, diversion along the highway fill to the principal stream channel or providing an auxiliary opening on the floodplain is possible. Diversion along the highway embankment can create maintenance problems by increasing the gradient in the tributary channel and by providing a more efficient channel for floodplain flow at the toe of the highway embankment. Headcutting can occur which will endanger the embankment and upstream property, and a delta can form in the principal stream channel, causing it to change course (Figure 10.11).

An opening on the floodplain for a tributary stream or local drainage is the most desirable alternative if additional costs for construction are warranted. Other factors, such as flow concentration in the floodplain or diversion of floodplain flow for a long distance along the highway embankment, may influence the decision as well. Although the location of the opening on the floodplain will be influenced by the need to accommodate for local drainage or a tributary, any opening on the floodplain will be subjected to flood flows of the principal stream. The size of the opening may be influenced more by the amount of floodplain flow from the large stream than by flow from the tributary.

A highway located in the bend of a river, as in Figure 10.12, presents a particularly difficult problem in the location of auxiliary openings. Flow in the floodplain is likely to be concentrated across the mouth of the bend, as illustrated. Because the distance across the bend is shorter than in the channel, the water surface slope is steeper and conveyance in the floodplain is increased relative to conveyance in the channel. An auxiliary opening at this location which severely constricts the flow will cause general scour which could result in a cutoff of the bend, depending upon the width across the bend, the length of the bend, land use and soils. If an opening is necessary, the design should be conservative to guard against the possibility of scouring velocities. Other measures also may be advisable. Spur dikes can make the opening more hydraulically efficient, and armoring the flow line may protect against scour in some instances. Additional auxiliary openings can be used in a long bend to reduce the degree of flow constriction, and the opening can be located away from the mouth of the bend in some instances. Also, elimination of the bend by channel realignment may be possible at some locations. Where the effects of an auxiliary opening at a bend could be especially damaging, physical modeling may be warranted.

Figure 10.10 Diversion and Bridge Alternatives for Tributary Stream in the Floodplain

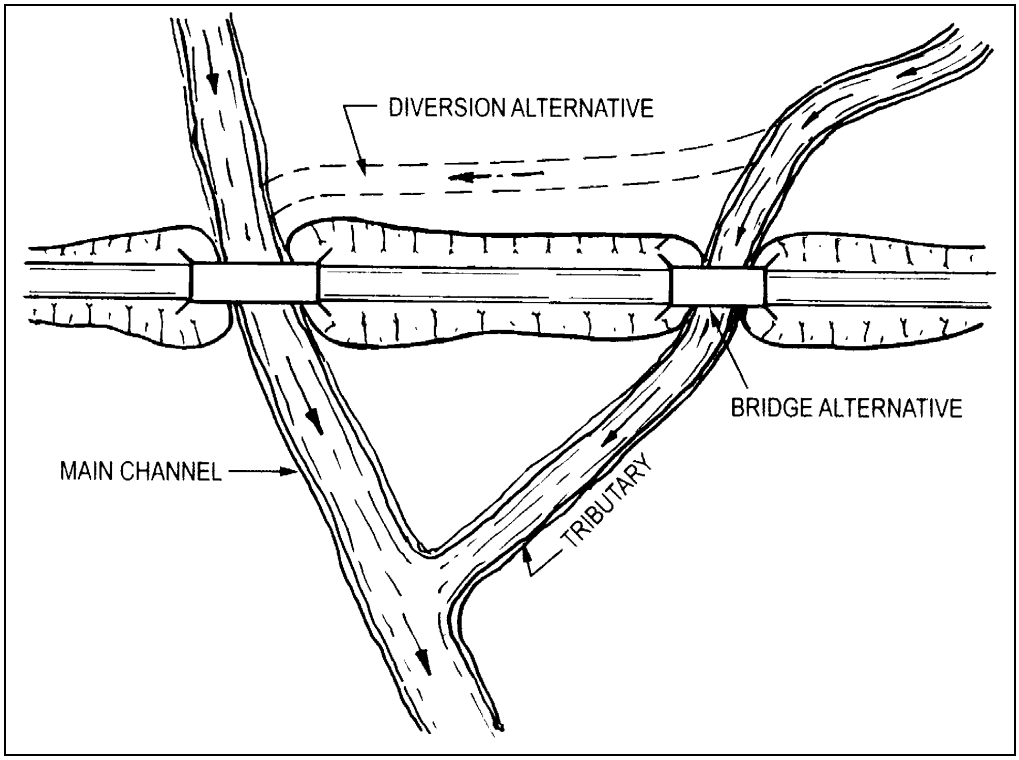
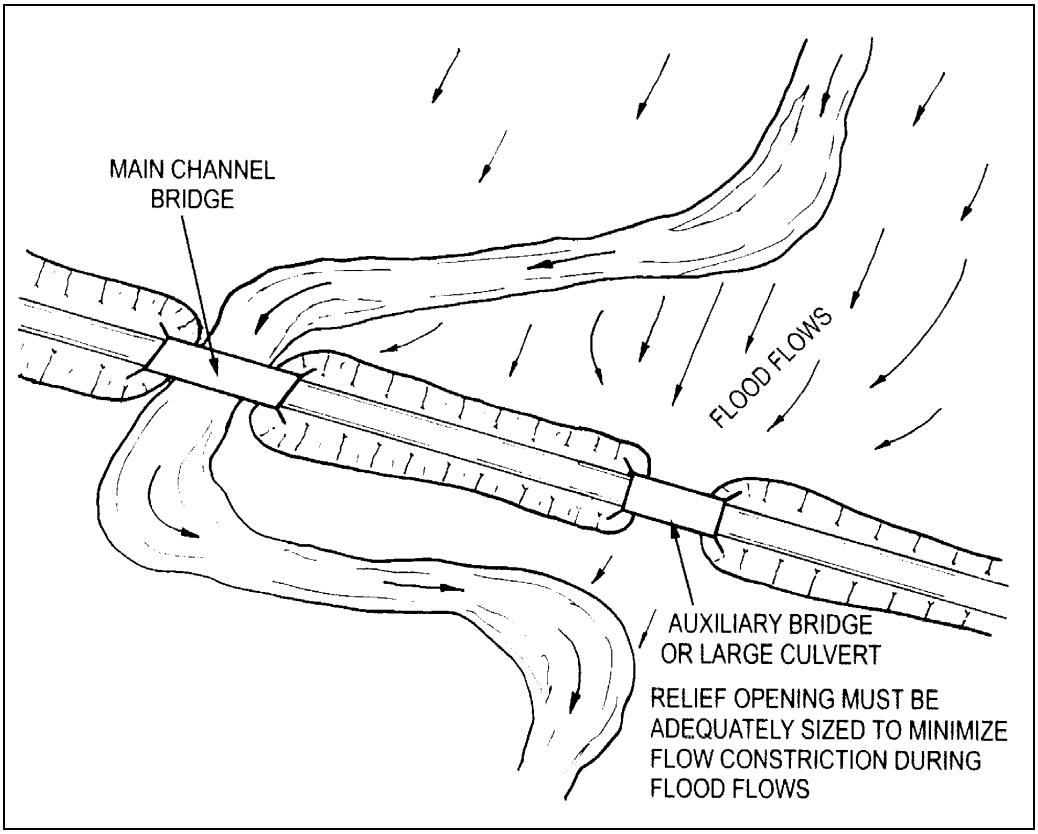


Figure 10.11 Delta Formed in Principal Stream by Divided Tributary



Figure 10.12 Stream Crossing in a Bend



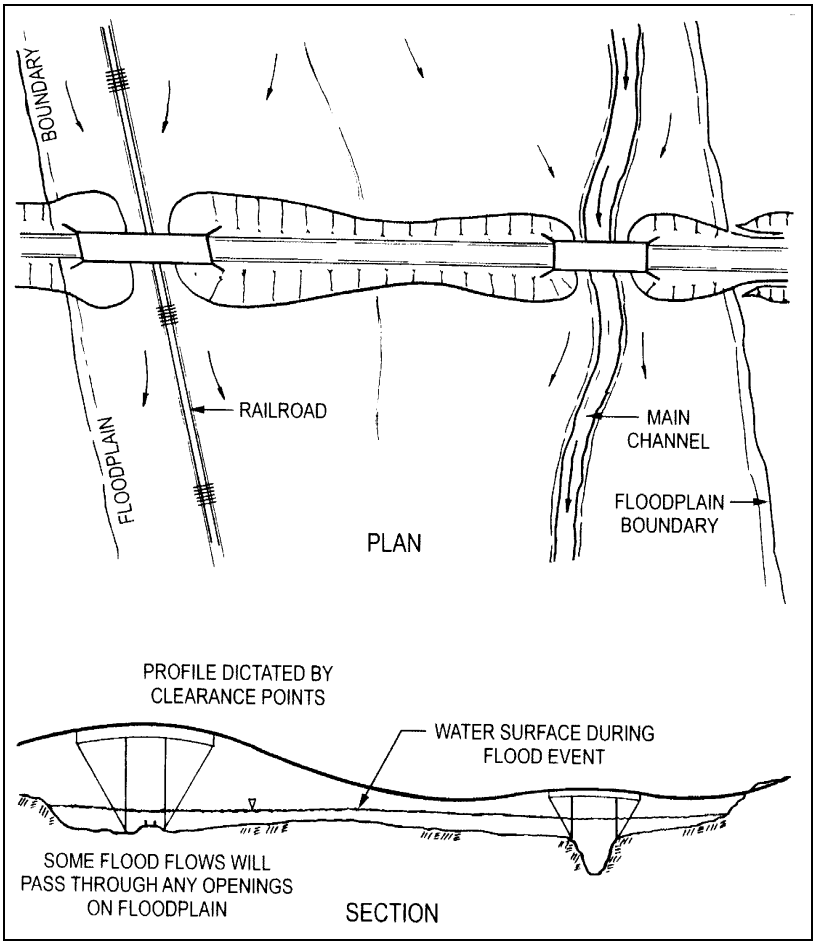
Often, there are existing crossings in the vicinity of the proposed crossing which have altered the flow distribution. In order to keep the effects of a new crossing on existing flow distribution to a reasonable minimum, considerable weight should be given to the location of existing opening(s), but the size of proposed opening(s) should be based on hydraulic requirements. Where existing auxiliary opening(s) do not accommodate significant flow, decisions on opening(s) in the new crossing should be based on the hydraulic requirements of the crossing system.

Other transportation facilities on the floodplain may require grade separation. This opening on the floodplain will pass flood waters, and roadway elevations required for separation may significantly alter the magnitude of the flood that must be passed through the proposed highway openings. Consideration should be given to the flow constriction at the separation which may cause damage to the other facility (Figure 10.13).

Floodplain uses which can influence opening location include development and the need for access across the highway right-of-way. Auxiliary waterway openings may need to be located to accommodate these uses and in recognition of the effects of development on flow distribution and tolerable backwater.

Auxiliary waterway opening(s), or relatively low roadway profiles in bridge approach areas, may be necessary in order to avoid large differences in water surface elevations between the upstream and downstream side of skewed highway crossings of streams with wide floodplains. An opening located at either A or B in Figure 10.8 should be sized so that head differentials will be minimized and severe scour will not develop.

Figure 10.13 Grade Separation for another Transportation Facility in the Floodplain Will Serve as an Auxiliary Waterway Opening



(b) Size. The size of a waterway opening is limited by the boundaries comprised of the stream bed and/or floodplain, the roadway embankment ends at each side, and the superstructure of the bridge at the top. Many characteristics of a crossing site influence the selection of waterway opening size. These characteristics are used in defining criteria for judging the acceptability of alternative designs of the crossing system as discussed in the introduction of this section. It is possible that a multitude of roadway profile and waterway opening alternatives would satisfy the criteria established from characteristics of the site. As an example, if criteria for the crossing include a severe limitation on backwater, this limitation can be satisfied either by using a small opening and a low roadway profile or by bridging all of the stream cross section. It is probable, however, that in most locations, alternatives which are somewhere between these two extremes will also satisfy the criteria established for the site and will prove to be more prudent insofar as the expenditure of public funds is concerned.

The performance of a waterway opening is dependent not only on the boundaries defined by the terrain, the bridge superstructure, and the embankment at each end of the bridge, but also by water surface elevations. The flood which will flow through the opening without disrupting traffic is determined by the above physical boundaries of the opening and the profile of the crossing.

Both alternative roadway profiles and waterway openings are practicable for many highway-stream crossings. Where this freedom is available, the probability of overtopping is a design decision which can be made considering the economic consequences of the decision. Bridge structural components, foundations, waterway opening size, and approach roadways should be designed so that the selected stream crossing system results in optimal or near-optimal use of public funds. Capital costs for construction; risks of damage to approach roadways; risks of damage to the bridge from buoyancy, drag, impact loads and scour; the costs of traffic interruption; and risks to other properties should be considered in determining the economic consequences of selecting a design from available alternatives.

The design of many other stream crossing systems is constrained by social, political, or environmental concerns; engineering considerations, such as geometries; multiple-use purposes, such as navigation, livestock passage and land access; economic considerations other than optimal use of public funds; policy decisions, such as minimum standards; and topographic controls. Where such constraints are imposed, the alternative crossing design which meets the constraints at the least cost in public funds should be selected. Economic concerns must be considered in selecting an alternative. In either case, foundations, bridge structural components, the size of the waterway opening, protective and preventive measures, and the stream crossing profile should be predicated on capital costs and risk costs for all floods which substantially contribute to those costs.

Waterway openings are most severely taxed by the largest flow that must pass through the opening. This flow rate is approximated at incipient overtopping of the highway-stream crossing system; therefore, waterway openings should be sized considering the probability of such an occurrence and the associated risks of damage.

(c) Auxiliary Openings. The need for auxiliary waterway openings, or relief openings, arises on streams with floodplains. The terminology adopted here, i.e., auxiliary openings, is intended to be consistent with the concept of a highway-stream crossing system in which each component has a specific role. The purpose of openings on the floodplain is to pass a portion of the flood flow in the floodplain when the stream reaches a certain stage. It does not provide relief for the principal waterway opening in the sense that an emergency spillway at a dam does, but it has predictable capacity during flood events.

The location of auxiliary openings was discussed above. The required size of auxiliary openings is not extensively researched, although an effort has been undertaken by the State of Mississippi to study the hydraulic performance of existing multiple bridge systems.

A method currently in use for determining the size of auxiliary openings is described here. While some accuracy may be lost due to the use of one-dimensional models to analyze two-dimensional flow, the results are sufficient for most applications.

Auxiliary openings on the floodplain generally are assigned a portion of the total streamflow based on conveyance calculations. Conceptually, the flow will separate at an assumed or a real divide and

continue to the appropriate opening. For a normal crossing system at a straight reach of the stream, this flow divide can be accomplished approximately as assumed by sizing all waterway openings so that backwater above each is approximately the same. If any opening is sized so as to create more backwater than another, the divide will not be as planned because the highwater surface at that opening will cause diversion toward another opening.

The complexity in analysis with one-dimensional models comes with crossings which are not normal to the flow direction, with bends and sinuosity in the stream system, and with flow directions which vary along the crossing and with stages in the stream. For this reason, it has been recommended (above) that auxiliary openings in skewed crossings at either location A or B, Figure 10.8, should be conservatively sized as judged by the best available method of analysis. Use of the Surface Water Modeling System should be considered for the evaluation of such crossings.

(d) Replacement Bridges. Investments in replacement bridges constitute an increasing proportion of capital expenditures for highway construction. A wealth of experience may be available at the site of existing bridges relative to the hydrology and morphology of the stream. This experience and modern hydrologic, hydraulic and economic analysis technology should be fully exploited when the replacement crossing system is selected. In particular, floodplain usage may have changed near the crossing site since the original construction indicating a need to reevaluate the risk to private property. Traffic volume increases and changes in traffic character may indicate a need to reassess traffic service requirements. Changes in the relative costs of construction, maintenance and flood damage repair of various components of the crossing system may indicate that an alternative that differs from the existing crossing system should be selected.

Many existing bridges have withstood substantial floods, and studies may indicate that no change in the stream crossing system is warranted. In such cases, the fact should not be overlooked that the replacement of short spans with longer spans or a truss with a girder design will result in a reduction in the waterway opening if the replacement structure has a deeper superstructure and the same vertical alignment). As a result of the replacement, the risk of backwater damage will be increased, the probability of overtopping will be increased, and there will be a greater hazard of ice and debris damage. If the existing level of traffic service is to be maintained, insofar as interruption by flooding is concerned, and the risks of flood damage to the highway and other property are not to be increased, the grade of the bridge deck should be raised to compensate for the deeper superstructure of the replacement bridge.

3. Structural Alternatives. A myriad of structural alternatives is 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.

(a) 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 structural alternative should be selected on the basis of economics. Other considerations which may influence structure-type selection are listed in Table 10.1 and discussed in subsequent sections.

Table 10.1 lists many of the advantages and disadvantages of bridges and culverts. Those considerations which are associated with the use of culverts are discussed in *Highway Drainage Guidelines: Hydraulic Design of Culverts* (AASHTO, 2000b). Culvert(s) in combination with bridge(s) are used in numerous highway-stream crossings, either to pass flow in a floodplain or to provide for local drainage in the floodplain. Where culverts or small bridges are used in the floodplain in conjunction with a bridge, the potential scour as a result of the head differential from upstream to downstream and the long duration of the hydrograph should be considered, see Figure 10.14. As an example, a culvert at location A or B in Figure 10.8 would have a high outlet velocity because of the large head differential across the roadway, and severe scour could occur at the outlet.

Table 10.1 Bridge or Culvert

Bridges	
Advantages	*Disadvantages*
• Less susceptible to clogging with drift, ice and debris • Waterway increases with rising water surface until water begins to submerge superstructure • Scour increases waterway opening • Flowline is flexible • Minimal impact on aquatic environment and wetlands • Widening does not usually affect hydraulic capacity	• Require more structural maintenance than culverts • Spill slopes susceptible to erosion and scour damage • Piers and abutments susceptible to failure from scour • Susceptible to ice and frost formation on deck • Bridge railing and parapets hazardous as compared to recovery areas • Deck drainage may require frequent maintenance cleanout • Buoyant, drag and impact forces are hazards to bridges • Susceptible to damage from stream meander migration
Culverts	
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 above the culvert before other roadway areas • 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 • 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

Figure 10.14 Scour at a Culvert in the Floodplain



(b) Piers. Economy of construction usually plays a large role in the determination of spans, pier locations and orientation, and substructure and superstructure design. It is necessary that construction costs always be a factor in the structural design of a bridge in order to make use of economically available structural materials, but the cost of construction is only one part of the total economic cost of a stream crossing system. Hydraulic considerations, maintenance costs, and expected cost of failure also should be considered in making decisions on the type, size, and location of the system.

The number of piers in any channel should be limited to a practicable minimum, and piers in the channel of small streams should be avoided, if practical. Piers properly oriented with the flow do not make large contributions to bridge backwater. But, they do contribute to general scour. In some instances, severe scour has developed immediately downstream of bridges because of eddy currents and because piers oriented improperly occupy a significant area in the channel. Lateral as well as vertical scour also occurs at some locations.

Piers should be aligned with flow direction at flood stage in order to minimize the opportunity for drift to be caught in piling or columns, to reduce the contraction effect of piers in the waterways, to minimize ice forces and the possibility of ice dams forming at the bridge, and to minimize backwater and local scour (Highway Research Board, 1970). Pier orientation is difficult where flow direction changes with stage or time. Circular piers, or some variation thereof, are probably the best alternative if orientation at other than flood stage is critical.

Piers located on a bank or in the stream channel near the bank are likely to cause lateral scouring of the bank. Piers located near the stream bank in the floodplain are vulnerable because they can cause bank scour. They also are vulnerable to failure from undermining by meander migration. Piers which must be placed in locations where they will be especially vulnerable to scour damage should be founded at elevations safe from undermining (Figure 10.15).

Figure 10.15 Piers on Bank Undermined by Meander Bend Migration



Pier shape is also a factor in local scour (Highway Research Board, 1970). A solid pier will not collect as much debris as a pier bent or a multiple-column bent. Rounding or streamlining the leading edges of piers helps to decrease the accumulation of debris and reduces local scour at the pier.

Where ice is a problem, piers can be battered and armored to facilitate breaking up ice flows which otherwise would crush against the leading edge of the pier (Watt & Podolny, 1976). Armoring the leading edge is also a necessary countermeasure in many streams that transport large material as bed load (Figure 10.16).

Figure 10.16 Pier Damaged by Boulders in Bed Load



(c) Abutments. Abutments may be of vertical walls or the fill slope may spill through the end span forming a "spill-through" abutment, or a combination of these types of abutments may be used. The type of abutment used has little effect on total backwater except where the flow section is severely contracted by a short bridge. Selection of abutment type is based largely on cost, the type of foundation material, and stability considerations.

The principal concerns of hydraulic engineers for abutments are orientation and security from scour-related failure. Concerns for security from scour usually are resolved by protective and preventive measures which are discussed in Section 10.5.C. Orientation is usually the same as for piers in adjacent bents; the effects of skew are discussed elsewhere in this section.

(d) Foundations. The foundation is one of the elements of a bridge which is most vulnerable to attack by floods. Examination of individual boring logs and plots of the profiles of various subsurface materials are important to the prediction of potential scour depths as well as to estimation of the bearing capacity of the soils.

Five basic types of foundations are used for stream crossings: (1) spread footings, (2) pile or drilled shaft trestle bents, (3) footings on piles or drilled shafts, (4) pedestals, and (5) caissons, see Figures 10.17 to 10.20. Spread footings are dependent upon the bearing capacity of the material in which they are founded for support. They may be used if founded below the level of anticipated scour or where sound rock is relatively shallow.

Figure 10.17 Spread Footings
(after Highway Research Board, 1970)

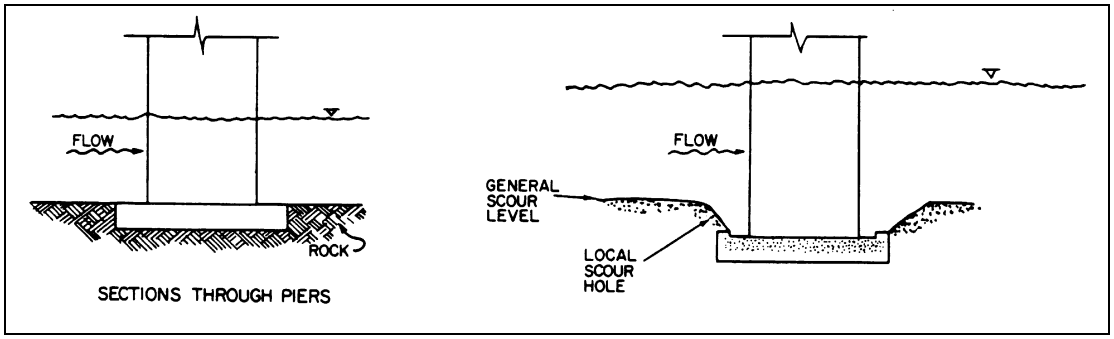


Figure 10.18 Drilled Shaft Foundation
(after Highway Research Board, 1970)

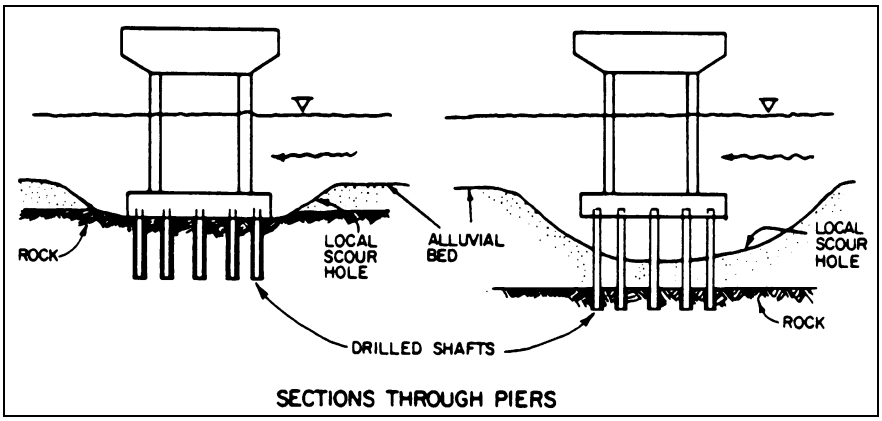


Figure 10.19 Typical Pile Foundation
(after Highway Research Board, 1970)

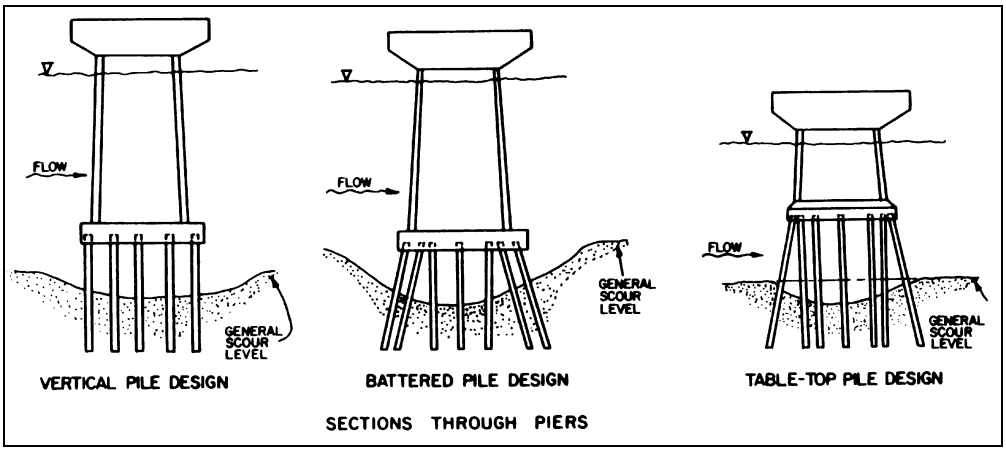
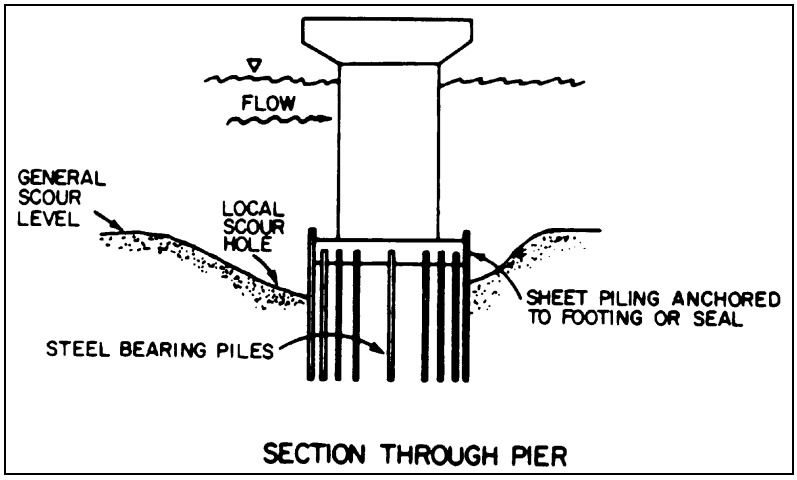


Figure 10.20 Protected Pile Foundation
(after Highway Research Board, 1970)

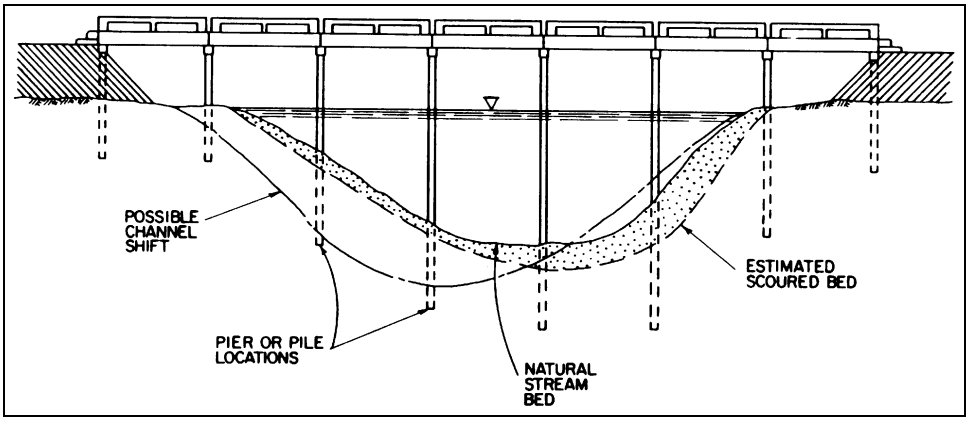


Piling or drilled shafts, whether in a trestle-type bent or under a footing, usually depend upon the surrounding material for skin friction and lateral stability. In some instances, they can be carried to rock or other dense material for load-carrying capacity. Tip elevations for piling or drilled shafts should be based on estimates of potential scour depths as well as bearing in order to avoid losing lateral support and load-carrying capacity during floods. Pile bearing capacity derived from driving records has little validity during floods if the material through which the piles were driven is scoured away.

Caissons are used in large rivers and usually are sunk to dense material by excavation inside the caisson. Founding depths are such that scour usually is not a problem at the pier after construction is completed. Severe contraction scour has developed downstream of some such bridges, however, because of contraction of the flow from the large piers.

Attention should be given to potential scour and the possibility of channel shifts in designing foundations for bridges on floodplains and spans approaching the stream channel. The thalweg in the channel should not be considered to be in a fixed location when establishing founding elevations. The history of a stream and a study of how active it has been can be very useful in making decisions on pile and drilled shaft tip elevations (Figure 10.21).

Figure 10.21 A Changing Channel can Undermine Foundation
(after Neill, 1973)



(e) Superstructures. Hydraulic forces that should be considered in the design of a bridge superstructure include buoyancy, drag, and impact from ice and debris. Decisions on superstructure type should be influenced by the profile configuration, the probability of submergence, expected problems with ice and debris, flow velocities, and the usual economic, structural and geometric considerations.

(1) Buoyancy. When the superstructure of a bridge is submerged or partially submerged, the new effective weight of the superstructure is the weight of the structure less the weight of the volume of water displaced. This will differ from the submerged weight of the superstructure if air is trapped under the deck or if the superstructure is of box girder design. Where solid parapets are used, the large volume above the deck may displace water if the bridge is at the bottom of a sag-vertical curve, or on a small gradient on the approach roadways. In some cases, this effect can be reduced if drain slots are provided in the parapets.

The effect of trapped air under the bridge deck can be reduced by providing holes through the deck between each girder. Superstructures are usually anchored to piers to counter buoyant forces and resist drag forces created by flowing water.

(2) Drag Forces. The drag forces on a submerged or partially submerged superstructure may be calculated by the following equation.

(Equation 10.1)

$$F_d = C_d \rho H \frac{v^2}{2}$$

where: F_d = drag force per unit of bridge length, N/m (lb/ft)
C_d = coefficient of drag
ρ = density of water, kg/m^3 (slug/ft^3)
H = depth of submergence of superstructure, m (ft)
v = velocity of flow, m/s (ft/s)
Note: Slugs = 32.2 pounds mass

The coefficient of drag, C_d, can be taken as 2.0 to 2.2 based on usual Reynolds numbers in natural streams and the usual shape of bridge superstructures (Streeter, 1971).

The density of fresh water is usually taken as:

$$\rho = 1000 \text{ kg/m}^3 = (1.94 \text{ slugs/ft}^3)$$

It is apparent from Figure 10.22 that combined buoyant and drag forces need to be dealt with in design. Structural and hydraulic engineers should be aware of these forces in the selection of profile alternatives, and of the probability of overtopping and the implications of parapet and superstructure alternatives. If warranted by the probability of submergence, superstructures should be adequately anchored and vented.

Figure 10.22 Girder Spans Displaced by Buoyant and Drag Forces



(3) Ice Forces. Ice forces are considered in the design of bridge piers according to procedures described in Section 3.9.2 of the LRFD, *Specifications for Highway Bridges* (AASHTO, 1998 and subsequent). Inherent in this section of the bridge specifications is an assumption that the bridge superstructure will not be subjected to impact forces from floating ice, static pressure from thermal movements or from ice jams, or uplift from adhering ice in water of fluctuating levels. It is reasonable that bridge superstructures would rarely be subjected to uplift from adhering ice or pressure from thermal movements since these forces normally are associated with relatively large bodies of water and superstructures normally should be high enough to be unaffected.

Ice jams and floating ice forces can be imposed on superstructures depending upon the highway-stream crossing profile configuration and the probability of overtopping during the spring breakup of ice on the stream.

The bridge specifications recommend that static ice pressures shall be given consideration, but a method for calculating forces associated with ice jams is not prescribed or recommended. A method for computing dynamic forces on piers from floating ice is included by values used in the equation developed from data at piers and may not be applicable for computing forces on superstructures.

Figures 10.23 and 10.24 are examples of ice damage to bridge superstructures. Research is needed to determine the static and dynamic loads that can be expected from ice loads on superstructures under various conditions of ice strength and streamflow. The probability of ice flows occurring simultaneously with streamflow at stages which would impose ice forces on the bridge superstructure is probably small in many locations.

Figure 10.23 Span Displaced by Force of Ice



Figure 10.24 Girder Twisted by Force of Ice



In addition to the forces imposed on bridge substructures and superstructures by ice loads, there are other hydraulic implications of ice flows. As an example, an ice jam at a bridge in or near a community could cause flooding of the community by water backed up behind the ice dam.

Figure 10.25 is an example in which the sluicing action of streamflow under an ice jam caused a foundation failure. It is clear that the ice jam was the cause of the failure since the scour occurred under the ice at the upstream side of the bridge. Streamflow during the event which caused failure had a probability of exceedance in any one-year period of about 0.20. The joint probability of the ice flow and streamflow that occurred during the event is likely to be much smaller than 0.20 unless significant ice forms on the stream every year and the annual maximum streamflow occurs every year with the spring breakup of ice. Even though the joint probability may be small, the relatively small additional cost of riprap scour protection, a deep foundation and/or a longer opening to protect against such a failure could be a good investment at many locations.

Ice Engineering (USACE, 1980) is a good source of information on how ice jams are formed, where they are likely to form, how to prevent them, and how to combat them.

(4) Debris Forces. Floating debris may consist of logs, trees, house trailers, automobiles, storage tanks, lumber, houses, and many other items representative of floodplain usage upstream of the bridge. Because of the variety of debris involved, it would be necessary to compute impact forces on a worst case basis for the debris that could originate in the basin using an assumption for the negative acceleration of the debris on impact in the equation:

(Equation 10.2)

$$F = M\, dv/dt = \frac{Mv^2}{2S}$$

where:
- F = the impact imparted by the debris, N (lb)
- M = the mass of the debris, kg (slug)
- dv/dt = the change in velocity of the debris with respect to time
- v = velocity of the debris on impact, m/s (ft/sec)
- S = stopping distance, m (ft)

Instantaneous stopping of the debris would indicate an infinite impact force; therefore, it is apparent that yielding of both the debris and the bridge component is possible. For research on the maximum loads for debris loading, see NCHRP Report 445, *Debris Forces on Bridges* (Parola, A., et. al., 2000).

Drag forces on the superstructure also can be increased significantly by a buildup of debris. The buildup increases not only the effective depth of the superstructure, but the drag coefficient also may be increased because of the increased resistance to flow.

In addition to forces imposed on bridges by debris, perhaps the most common hazard is partial or total clogging of the waterway opening. Partial clogging can result in a channel shift or a sluicing action similar to that from the ice in Figure 10.25. Total clogging can result in failure of the bridge or a shift in the channel location from under the bridge.

Figure 10.25 A Foundation Failure from Scour under an Ice Jam during a Small Flood



4. Channel Modification. A primary objective in the design of a highway-stream crossing system should be to disturb the stream as little as is practical. Channel modifications should be made only where modification is necessary to achieve compatibility between the highway and stream and to accommodate streamflow with a minimum of interruption to the stream and its environment.

Highway-associated channel modifications involve only short reaches of a stream in almost all instances. For an in-depth discussion of channel modification, refer to Chapter 8, *Open Channels*.

Modifications for the purpose of increasing channel capacity are not commonly encountered at highway-stream crossings because of the limited effectiveness of such modifications. Channel capacity improvements usually must be carried long distances downstream if the stage-discharge relationship at the site is to be altered. Because of flow controls downstream, the stage-discharge relationship at a crossing will not be altered a discernible amount by a short reach of modified channel.

Stream channels are sometimes widened through the waterway opening of a bridge. As indicated, this essentially will have no effect on the natural stage-discharge relationship of the stream, but by increasing the waterway opening under the bridge, backwater at high flood stages may be decreased. However, short reaches of stream channels which have been enlarged may return to the natural cross section by deposition within the enlarged reach unless it is regularly maintained. At many locations, it may be advisable to consider additional bridge length rather than to rely on an enlarged channel section under the bridge. When considering widening a stream reach, any environmental effects must also be taken into consideration.

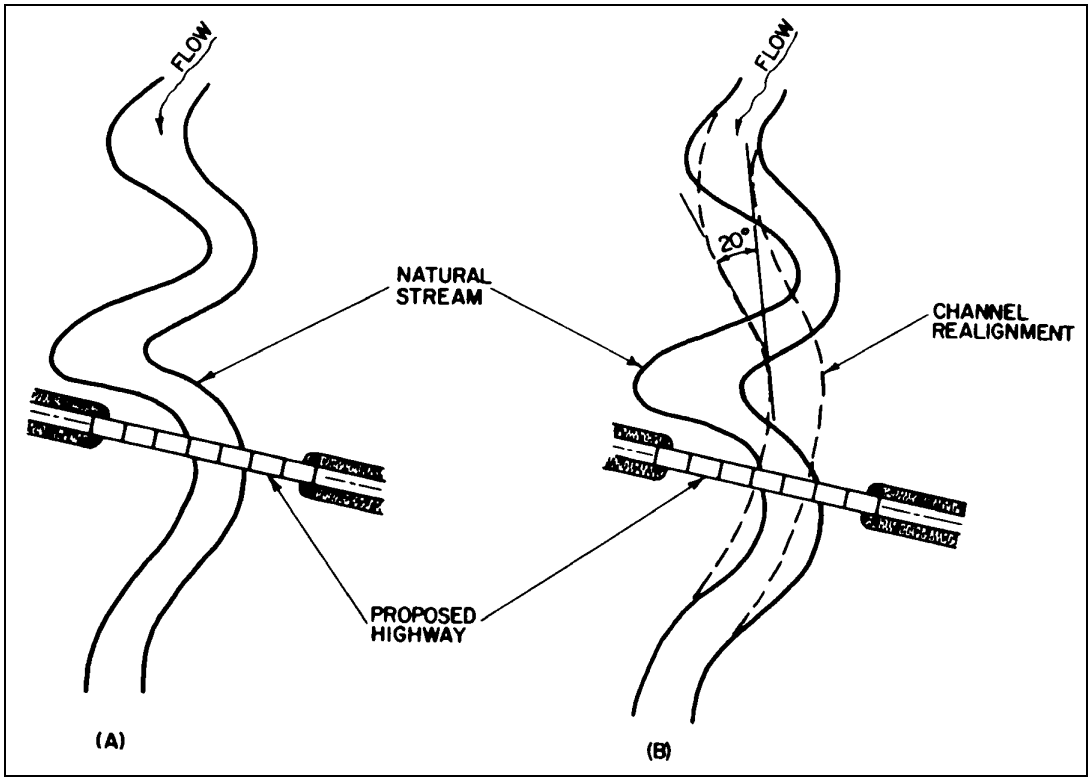
An enlarged section through the bridge may be more successful if excavated so that the bed load will not be deposited in the excavated area and the enlarged area will be available when needed to convey flow through the bridge (Figure 10.26). However, concerns for the stream and floodplain environment and for bank stability may preclude the use of such an enlarged section on some streams. When existing fishable streams must be widened to provide additional conveyance capacity, widening should occur above the normal flow elevation within the stream and be confined to the floodplain above the normal water surface elevation within the stream. For a complete description of recommended alterations to fishable streams please refer to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10.

In some circumstances, it may be advantageous to realign a channel in order to create better flow alignment with a waterway opening. These circumstances might arise at crossing sites where other controls make it difficult or impractical to select an alternative site or different highway alignment (Figure 10.27). Modification of the channel alignment usually can be accomplished successfully if some general principles and guidelines are followed even though generalized criteria applicable to all streams are difficult to formulate. It should be noted that mitigation and/or enhancement measures such as those discussed in *Highway Drainage Guidelines: Hydraulic Analysis and Design of Open Channels* (AASHTO, 2000d) should be considered in developing the design of any channel modification. It also should be noted that the channel modification as illustrated in Figure 10.27 does not resolve any problems with the skew of the highway crossing with flow direction in the floodplain.

Figure 10.26 Channel Enlargement which Preserves the Section of the Low Flow Channel



Figure 10.27 Channel Realignment



The general rules which should be followed in making channel alignment modifications in alluvial channels pertain to the radii of bends, sinuosity, slope, system response and stream power. The radii of bends in the realigned channel should be made about equal to the mean radii of bends in an extensive reach of the river. The angle between a line drawn tangent to the inside of two successive bends and the bank line between bends should be about 20 degrees. This allows sufficient crossing length for the thalweg to move from one side of the channel to the other. The sinuosity, P, multiplied by the channel slope will be unchanged by the alignment modification. Thus, if subscript 1 represents the original channel and subscript 2 the modified channel, then:

(Equation 10.3)

$$P_1S_1 = P_2S_2$$

The slope, S_2, in the modified channel should be chosen to satisfy Equation 10.4 where S_2 is in m/m (ft/ft) and Q is in m^3/s (cfs).

(Equation 10.4)

$$S_2Q^{1/4} < 0.0007 \text{ (0.0017 in U.S. Customary Units)}$$

If this equation is not satisfied, there is a possibility of the channel becoming braided. In order to avoid causing a braided channel, the sinuosity, P_2, should be increased in order to reduce the slope, S_2.

Stream power in the new channel should be about the same as in the old channel (AASHTO, 2000d). Equating stream power in the old and new channel provides guidance for establishing the width, W_2, in the channel modification in the form of the equation:

(Equation 10.5)

$$W_2S_1 = S_2W_1$$

and

(Equation 10.6)

$$\frac{W}{S} \approx const$$

Thus, if the slope $S_2 > S_1$, then W_2 can be greater than W_1 so long as the increase in width is moderate.

Maintenance of the realigned channel may require riprap protection on the outside bank at bends and removal of deposits after floods so that new meander patterns will not form. For a more detailed discussions of channel modifications, see *Highway Drainage Guidelines: Hydraulic Analysis and Design of Open Channels* (AASHTO, 2000d). In addition, *Restoration of Fish Habitat in Relocated Streams* (FHWA, 1979a) contains excellent guidance for the design and construction of relocated channels and describes measures which will lead to rapid recovery of aesthetic and fishery values in new channels.

B. Analysis of the Stream Crossing System. Analysis of the stream crossing system involves the use of hydraulic engineering principles and techniques and engineering economics to select an alternative design. The design must provide the traffic service required at a minimum cost and meet the criteria established for protection of the stream environment, as well as social, political, and other economic and engineering concerns. This section addresses the hydraulic, environmental and economic analyses of stream crossings.

The hydraulic analysis of a highway-stream crossing system involves determining the backwater associated with each alternative profile and waterway opening(s), the effects on flow distribution and velocities, and estimating the scour potential. The environmental analysis involves an assessment of the impacts of the crossing alternative on the stream environment and the consideration of provisions for practicable mitigation and enhancement measures.

The hydraulic, environmental and economic analyses must be of the total crossing facility including all roadway items, all waterway openings, and the environmental implications and risks associated with each alternative.

1. Hydraulic Performance of the Crossing System. The hydraulic performance and backwater at various stream stages generally are the first measures used to judge the acceptability of an alternative highway-stream crossing system design.

The introduction of a constriction such as a highway crossing in a stream channel with supercritical flow conditions will not cause backwater above the constriction. Constriction results in the conversion of potential energy to kinetic energy, and, therefore, there is no rise to the water surface elevation upstream of the introduced constriction as long as the constriction is minor and does not cause a hydraulic jump to occur. However, there will be a rise in the water surface elevation through the constriction.

A highway facility which constricts a stream with subcritical flow conditions will cause higher water surface elevations upstream of the crossing than prevailed during floods prior to construction of the highway facility. The higher water surface elevations represent the amount of kinetic energy converted from potential energy to overcome losses comprised principally of contraction and expansion losses. Other losses include those from piers, abutments, eccentricity, superstructures, if submerged, and friction from longer flow paths. Friction losses can be significant if the floodplain constriction is relatively severe and the resistance to flow in the floodplain is high, as in wooded areas (Schneider et al., 1977).

The increase in water surface elevation upstream of the highway facility is referred to as backwater (AASHTO, 2000a). It is measured upstream of the waterway opening above the theoretical water surface elevation prior to construction of the crossing (Figure 10.28). Backwater above a highway crossing will cause incremental depth and duration of flooding and an increased area of inundation for a given flood magnitude. The incremental flooding associated with various flood magnitudes should be considered in evaluating the risks associated with alternative designs of the stream crossing system (Schneider & Wilson, 1980; Corry et al., 1980).

The hydraulic performance of a stream crossing system, such as the one illustrated in Figure 10.29, will be similar to that illustrated in Figure 10.28. The performance of a crossing system which includes an auxiliary opening is also illustrated. Figure 10.29 is schematic only and is not intended to represent the relative increase in hydraulic performance provided by an auxiliary opening or the relative decrease in backwater that would be provided.

The flow through waterway openings in a highway-stream crossing system is greatly influenced by the profile elevation of the roadway (Schneider & Wilson, 1980). If the backwater at a bridge is relatively small, the capacity of the waterway opening at incipient overtopping is relatively insensitive to changes in opening size for practicable alternatives in bridge length. Conversely, practicable alternative roadway profiles may

substantially change the flow rate which must pass through the waterway opening at incipient overtopping. However, the total performance of a highway-stream crossing system is sensitive to both waterway opening and profile elevation. The effects on velocities, flow distribution, stream environment, scour and construction costs and risk costs are highly responsive to the total design of the crossing.

The hydraulic design of a highway-stream crossing system is an iterative process in which alternative waterway opening designs are tried for each profile alternative. Each alternative opening and profile will have an associated hydraulic capacity and probability that the opening capacity will be exceeded.

2. Backwater. Backwater, or the increment of increased flood depth upstream of a highway crossing of a stream, often is used as the sole criteria for judging the adequacy of an alternative waterway opening. The amount of backwater permissible is sometimes established for application statewide without regard for risks to property, which are site specific, and differences in flow conditions at various crossing sites. The advent of the NFIP of the FEMA resulted in widespread adoption by various jurisdictions of such standard measures of acceptability. Some jurisdictions have established rules that require waterway openings adequate for floods with an exceedance probability of 1% in any period of one year with backwater not in excess of a stated amount. Such rules require the same standards of stream crossing design regardless of the warranted level of traffic service or the risk of property damage.

Figure 10.28 Backwater at a Highway-Stream Crossing

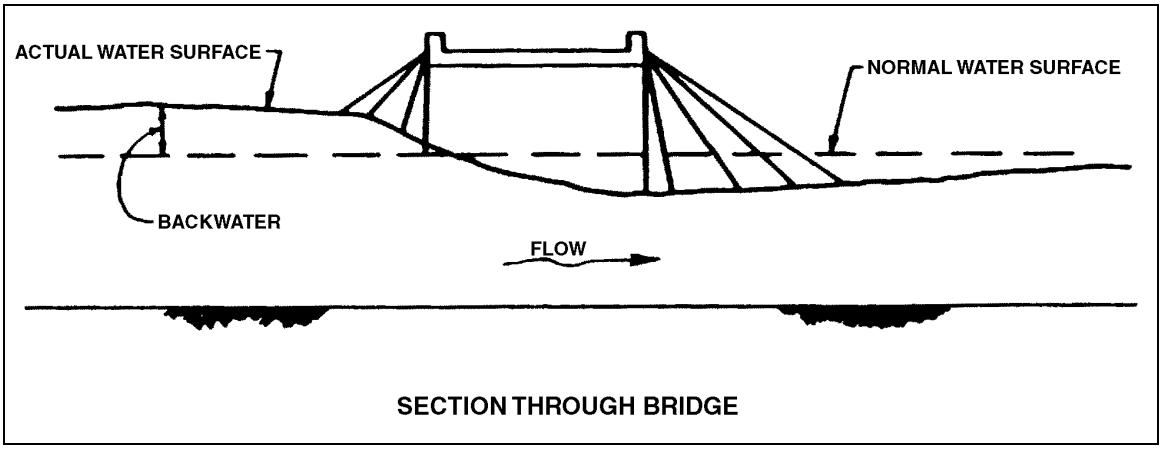
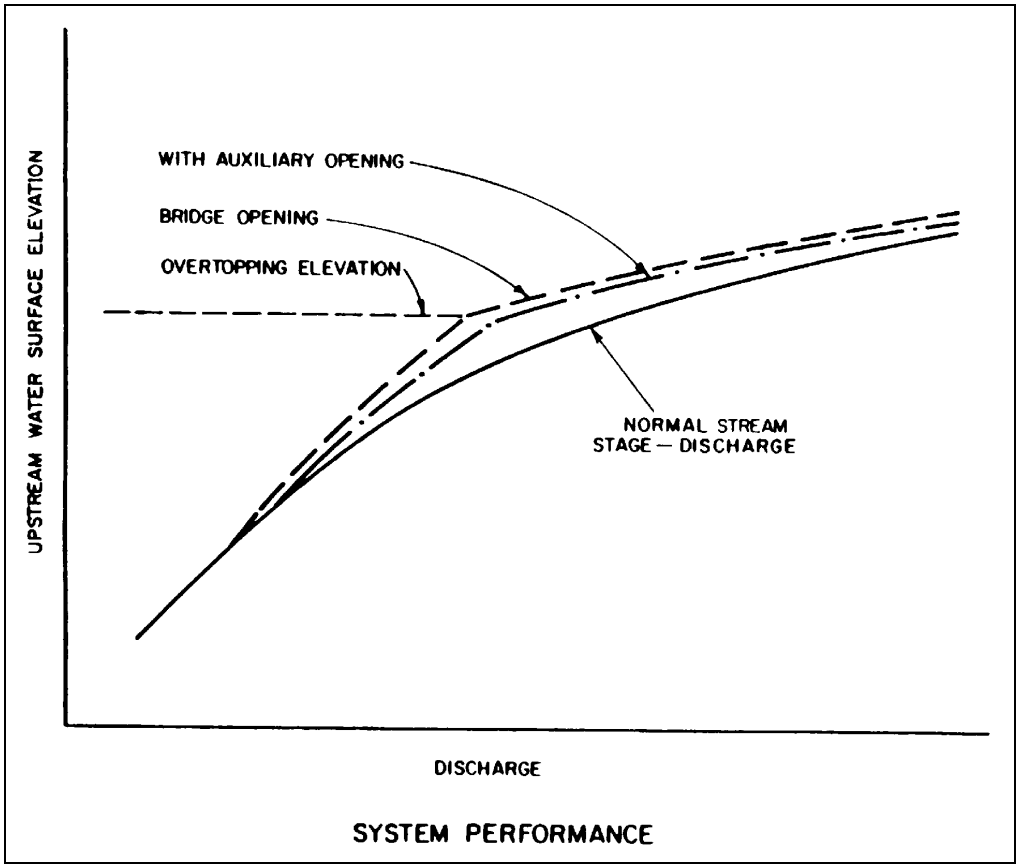


Figure 10.29 Hydraulic Performance of a Highway-Stream Crossing



Backwater should not be used as the sole criterion for judging the acceptability of an alternative design. Instead, it is an aid that can be used in selecting the waterway opening and crossing profile, and to assess the risk costs of incremental flooding caused by the crossing facility.

Backwater associated with each profile and opening alternative can be computed by methods given in HDS-1, *Hydraulics of Bridge Waterways* (Bradley, 1978), *Two Dimensional Finite-Element Hydraulic Modeling of Bridge Crossings* (FHWA, 1988), *Bridge Waterways Analysis Model* (FHWA, 1990a), *Computation of Backwater and Discharge at Width Constrictions of Heavily Vegetated Flood Plains* (Schneider et al., 1977), HEC-2, *Water Surface Profile* (USACE, 1983), and *Hydraulic Reference Manual for HEC-RAS* (USACE, 1995b). Overflow over the roadway can be computed using methods in HDS-1, *Hydraulics of Bridge Waterways* (Bradley, 1978) and *Discharge Characteristics of Embankment Shaped Weirs* (Kingsvater, 1964).

3. Flow Distribution. Flow distribution will be disturbed during some floods by any highway-stream crossing system which utilizes a combination of fill and bridge within the floodplain. Flow distribution should be preserved to the extent practical, however, in order to avoid disruption of the streamside environment (U.S. Water Resources Council, 1978), to preserve local drainage patterns, to minimize damage to property by either excessive backwater or high local velocities, and to avoid concentrating flow in areas which were not subjected to concentrated flow prior to construction of the highway facility. Many crossing sites will present other good reasons for minimizing redistribution of existing flow, such as those at which an auxiliary opening on the floodplain will be more hydraulically efficient than an extension of the opening at the stream channel.

Flow distribution is a consideration at crossing sites where significant flow is distributed over a relatively wide floodplain during most floods. Minimal disruption of flow distribution can be achieved by providing openings to avoid diversions for long distances along the roadway embankment, at locations where relatively large flow concentrations occur, and by providing a crossing stream profile which will be overtopped at appropriate locations. An opening which is provided to maintain flow distribution must be sized to avoid diversion caused by backwater creating a hydraulic gradient toward another opening. This can be accomplished where the

crossing is normal to flow direction by adjusting opening sizes so that backwater above each opening is approximately equal. Complex sites, such as those at a bend, as in Figure 10.13, and skewed crossing, as in Figure 10.14, can be analyzed with one-dimensional models only by using a large measure of intuition, experience and engineering judgment to supplement the quantitative analysis. Unfortunately, complex sites are encountered frequently in stream crossing design. Development of two-dimensional methods of analysis greatly enhances the capabilities of hydraulic designers to deal with these complex sites.

4. Velocity. Velocities in the waterway opening(s) of most stream crossings are higher than velocities in the natural stream because of the contraction of the flow by approach fills, bridge piers and, in some cases, bridge superstructures. In some instances, flow will pass through critical in the contraction and return to subcritical downstream if the contraction is severe or velocities in the stream are near critical.

The use of an average velocity for all waterway openings as a criterion for design is not advisable. The acceptability of the average velocity in a waterway opening is dependent upon site characteristics such as natural stream velocity, bed materials, soil types, foundation materials, and risk considerations from backwater and scour as well as practical countermeasures to reduce the risk of damage. Other site-specific considerations, such as existing crossings, also may influence decisions regarding acceptable average velocities.

Scour computations completed with average velocity may result in under estimation of the scour that occurs at a particular location. This is because the velocity within a channel in the area where scour occurs may be higher than the average velocity across the entire channel. As the selection of the appropriate design velocity varies depending on the design objective, designers are encouraged to reference HEC-18, *Evaluating Scour at Bridges, Fourth Edition* (FHWA, 2001b) for selection of appropriate design velocities for scour computations and reference HEC-23, *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Second Edition* (FHWA, 2001a) to select appropriate velocities for countermeasure design.

5. Scour. Potential scour can be a most significant criterion in the analysis of a stream crossing system. The design of a crossing system involves an acceptable balance between (1) a waterway opening that will not create undue damage by backwater or suffer undue damage from scour, and (2) a crossing profile sufficiently high to provide the required traffic service.

Scour at waterway openings occurs as natural scour, contraction scour and local scour.

Natural Scour:
- Is a long-term aggradation and degradation of the stream bed.
- May be lateral migration.
- May be the natural trend of the stream or the result of some modification to the stream or watershed.

Contraction Scour:
- Occurs when the flow area of a stream at flood stage is decreased from normal, either by a natural contraction or by a bridge.
- Occurs due to increased velocities in the contracted section.

Local Scour:
- Involves removal of material from around piers, abutments, spurs and embankments.
- Occurs due to high velocities and flow disturbances such as eddies and vortices (Figures 10.30 and 10.31).

Figure 10.30 Schematic Representation of Scour at a Cylindrical Pier

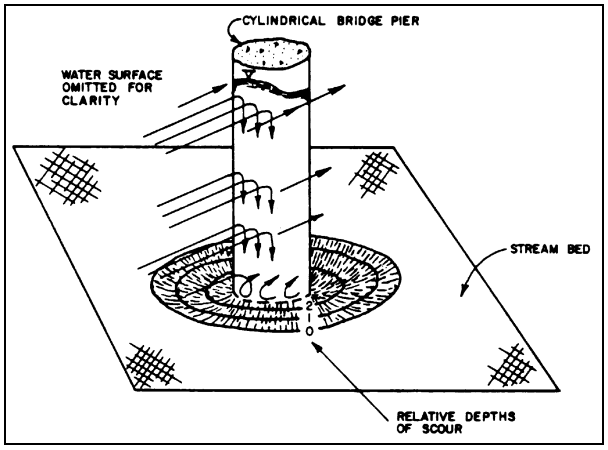
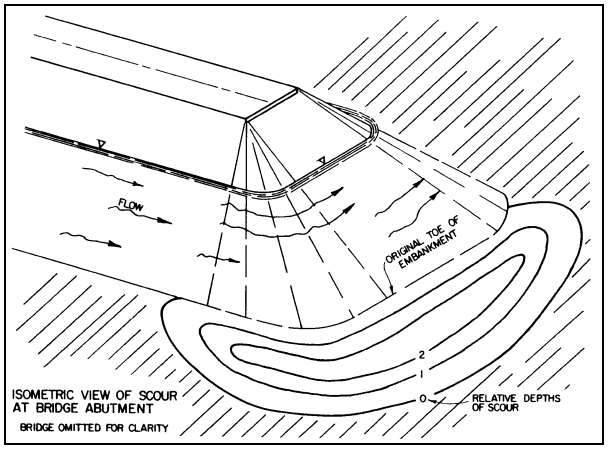
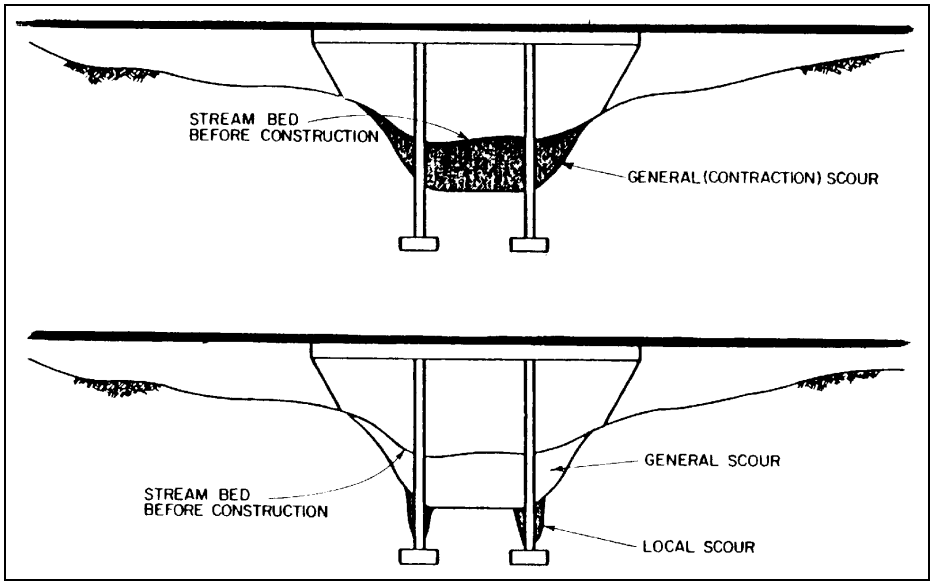


Figure 10.31 Schematic Representation of Scour at an Embankment End



The effects of natural, contraction, and local scour are generally additive where they occur at the same location, except that the existence of one may affect the magnitude of another (Figure 10.32). General degradation in the stream also may be occurring naturally or as a result of development. Aggradation and degradation should be considered in assessing the potential effects of natural, general and local scour because changes in flow conditions, sediment transport or debris can significantly change the scour potential at the water opening.

Figure 10.32 Combined Effect of General and Local Scour (after Neill, 1973)



The rates of scour in different materials and under different flow conditions depend on erosive power in the flow, erosional resistance of the material, and a balance between sediment transported into and out of the section. In erosion-resistant materials, final, worst case, or equilibrium scour may not be reached in any one flood event but may develop over a series of events.

The resistance of fine-grained, cohesive materials to erosion depends on a number of physiochemical and environmental factors, since bonding between particles must break down before erosion can occur. Weak sandstones and weakly cemented sands and gravels may become cohesionless because the cementing material may dissolve. Laminated materials, such as hard shales, may tend to delaminate during floods (Neill, 1973).

A channel bed may be underlain by strata of varying resistance to scour. Where a comparatively resistant layer overlies more easily eroded material, it may be advisable to take precautions against scour penetration by careful design and construction of piers, by adoption of protective measures, and by avoiding severe contraction of the flow. Erosion-resistant strata may limit the potential depth of scour, and careful evaluation of bed stratigraphy may reveal opportunities for savings in foundation costs if the structure is designed and constructed to preserve the integrity of the resistant strata.

(a) Contraction Scour. The depth and area of contraction, or general scour at a waterway opening may be affected by any or all of the following factors (Neill, 1973):

- Slope, natural alignment and shifting of the channel.
- Type and amount of bed material in transport.
- Nature and occurrence of flood events.
- Accumulations of ice or other debris.
- Construction and/or realignment of flow due to the stream crossing.
- Layout and geometry of training works.
- Geometry and orientation of piers.
- Classification, stratification and consolidation of bed and subbed materials.
- Placement or loss of riprap and other protective materials.
- Natural or man-made changes in flow or sediment regimes.
- Failures, such as collapse of a nearby structure.
- Growth of vegetation in the channel or floodplain.

The HEC-18, *Evaluating Scour at Bridges* (FHWA, 2001b), HEC-20, *Stream Stability at Highway Structure, Third Edition* (FHWA, 2001d), HEC-23, *Bridge Scour and Stream Instability*

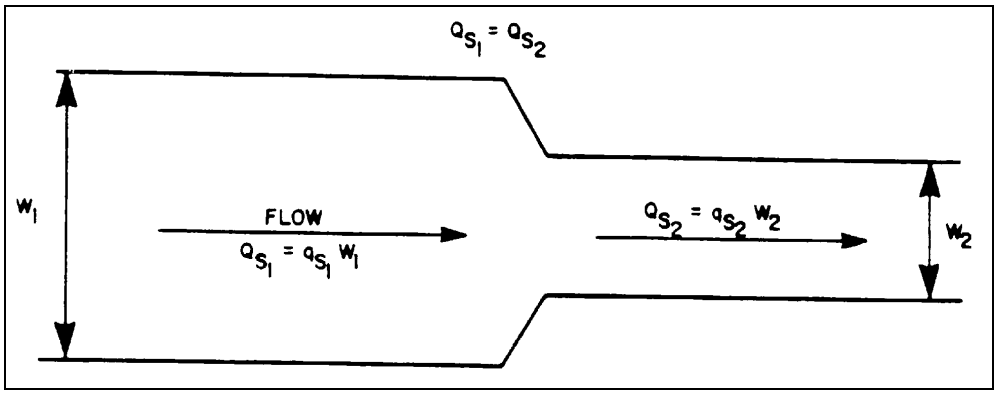
Countermeasures: Experience, Selection, and Design Guidance, Second Edition (FHWA, 2001a), and HDS-6, *River Engineering for Highway Encroachments* (FHWA, 2001c) contain the state-of-knowledge and practice for dealing with scour at highway bridges and the procedures for designing new, replacement and rehabilitated bridges to resist scour.

Numerous studies of scour at bridge waterways have been conducted, and, as yet, no definitive work applicable to all situations is advisable. The peculiarities and applicability of each method should be understood when scour potential is estimated.

General scour occurs at contractions because of increases in velocity and bed shear stress and a corresponding increase in stream power. Scour continues so long as more bed material is transported from the contracted section than is transported into the section.

For the case where flow is confined to the stream channel, Figure 10.33 can be used to illustrate the equilibrium condition. At equilibrium, the total sediment transport through the contracted section, Q_{S2}, is equal to the total sediment transported into section 2 from section 1, Q_{S1} (Highway Research Board, 1970).

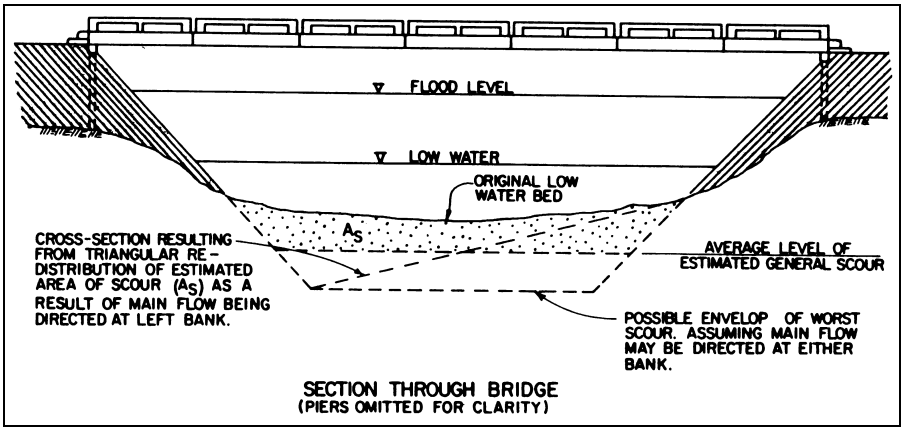
Figure 10.33 Sediment Transport at a Control Section (Highway Research Board, 1970)



Equilibrium can be reached by erosion of the banks at section 2, by scour in the stream bed, or by a combination of stream bed and bank erosion. The stream will attempt to reach equilibrium by bank erosion so that $W_1 = W_2$ where piers or embankment ends form a contracted section in the channel of a stream which transports heavy bed loads or has an erosion-resistant bed.

Where stream banks are armored, as with riprap, the equilibrium area of general scour at a contracted section can be conservatively estimated by assuming stream bed scour and a trapezoidal cross section that will make the average velocity through the waterway opening equal to the estimated average channel velocity outside the constriction (Neill, 1973). It should be noted, however, that in bends the scour may assume a triangular rather than trapezoidal shape with the deepest scour near the outer bank. In some cases, it may be desirable to assume an envelope of worst scour, assuming that the deepest scour can shift (Figure 10.34).

Figure 10.34 Envelope of Worst General Scour (after Neill, 1973)



An equation for computing contraction scour where all overbank flow is forced into the channel section by the contraction and the channel itself is contracted, either by piers or by the embankment ends, is provided in *Synthesis of Highway Practice 5, Scour at Bridge Waterways* (Highway Research Board, 1970) and *Scour at Bridge Crossings* (Laursen, 1958). The equation is based on a balance of sediment supply and sediment transport capacity. It will not be quoted here because the applicability can best be assessed by study of material presented in the above references. Inherent in the procedure is an assumption that scour will be limited to the stream bed. Experience with such contractions indicates, however, that some of the general scour will occur as bank erosion when the stream channel is constricted by embankment or piers.

General scour does not occur as a result of contraction by embankment ends where abutments are set back from the stream banks a sufficient distance to avoid diverting overbank flow into the channel. However, general scour can occur at such waterway openings if piers occupy a significant portion of the channel.

General scour at auxiliary openings on floodplains differs from scour in the channel because the sediment transport from upstream is usually small (Laursen, 1962 & 1963).

(b) Local Scour. Local scour occurs at embankment ends, piers, spur dikes, and similar obstructions in the flow. It is caused by the vortex of fluid which results from the pileup of water on the upstream edge and subsequent acceleration of flow around the obstruction. Scour continues so long as sediment transported out of the scour hole exceeds the sediment transported into the hole. As the depth of the hole is increased, the strength of the vortex is reduced and scouring ceases as sediment transport rates become equal.

Local scour at abutment ends and at piers located in the floodplain away from the stream bank should be considered clearwater scour because no bed load is present. The depth of maximum scour at piers located in a stream will depend on the presence or absence of dunes. If dunes are present, the depth of scour will vary with time. Sediment transport in the trough of a dune is small, and local scour will be greater than when the rest of the dune passes the local scour hole and sediment supply is large. Maximum scour depths, as determined from laboratory tests, are commonly 30% greater than equilibrium scour.

The shape of the upstream part of a local scour hole at piers is a truncated cone with the cone angle approximating the angle of repose of the sediment. Downstream slopes are flatter where the roller mixes with other flow and a bar is formed downstream of the hole. The lateral extent of the scour hole can be estimated from the angle of repose of the material and the depth of scour.

Methods for estimating local scour depths presented in *Highway Drainage Guidelines*: *Hydraulic Analysis for the Location and Design of Bridges* (AASHTO, 2000e), *Scour Around Circular Bridge Piers at High Froude Numbers* (FHWA, 1979b), *Stream Stability at Highway Structures* (FHWA, 1991), and *Synthesis of Highway Practice 5, Scour at Bridge Waterways* (Highway Research Board, 1970) are

recommended. It also should be noted that appropriate adjustments to computed scour depths should be made for pier shape.

Scour at Bridge Waterways: A Review (Anderson, 1974), *Scour Around Bridge Piers* (Hopkins et al., 1980), and HEC-18, *Evaluating Scour at Bridges, Fourth Edition* (FHWA, 2001b) are recommended for review of formulas for estimating scour at bridge piers and discussion of state-of-practice of scour estimation at piers.

(c) Natural Scour. A long reach of a stream in flood can be considered to be a series of contractions and expansions. During the rising flood, scour occurs in the contractions, deposition occurs in the expansions; during the falling stages, the opposite occurs. Thus, the stream bed can be extremely erratic and changeable from natural scour and filling in the channel.

Natural scour also occurs as meander bends progress downstream. As a meander moves, the outside bank and bed at the bend is scoured and filling occurs on the inside of the bend. Significant scour occurs at the outside of bends during floods even where the bend can be considered to be in a relatively stable location. Even in straight reaches, natural scour occurs with the migration of bed forms and shifting of the thalweg.

Wherever possible, estimates of maximum natural scour should be based on soundings at the site or at comparable sites on the same stream during floods which result in representative scour. Such measurements are usually difficult to obtain because of hazards associated with making soundings during floods and because the floods of interest are relatively rare occurrences. Natural scour may be estimated at existing bridge sites by reviewing stream bed elevation changes found in bi-annual inspection reports.

Borings, density logging and other geotechnical methods of investigating the sub-bed stratigraphy have been suggested as methods of inferring past scour levels. The lowest level of recent scouring may be detected by differences in density, the line of demarcation between the recently deposited material and the older bed material. Since the interpretation of geotechnical data is necessary, the estimation of maximum scour by these methods is subject to considerable uncertainty (Neill, 1973).

The *Guide to Bridge Hydraulics* (Neill, 1973) suggests a method for estimating natural scoured depths taken from the *Standard Specifications and Code of Practice for Bridges* (Indian Roads Congress, 1966). The method is for unconstricted alluvial rivers, and mean scoured depths of flow are estimated with the aid of an empirical regime formula.

Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Second Edition (FHWA, 2001a) and *Highway Drainage Guidelines: Hydraulic Analysis for the Location and Design of Bridges* (AASHTO, 2000e) contain procedures for estimating the length and depth of scour in stream bends.

(d) In-Stream Borrow Areas, Commercial Mining, and Dredging for Navigation and Flood Control. In-stream borrow is often used as a source of good quality fill material by highway contractors. Commercial mining of sands and gravels in streams also is common because the material is clean, often well-graded, and the supply is replenished by the stream. Borrow pits, either upstream or downstream of a highway-stream crossing, can cause scour at the bridge (Figure 10.35). Scour occurs upstream of the borrow because of the increased gradient of the stream bed. The bed load of the stream will be deposited in the borrow area, and scour occurs downstream as the stream regains its bed load.

In-stream borrow may not pose a scour hazard to a bridge, depending upon the amount of material removed from the stream, the effects of the borrow area on flow directions, the location of the borrow area, the size of the stream, and the sediment transport capacity of the stream. Many borrow areas have been filled without detriment to nearby bridges during a moderate rise in large streams which carry a large sediment load (Figure 10.36).

Figure 10.35 In-stream Borrow Area (after Highway Research Board, 1970)

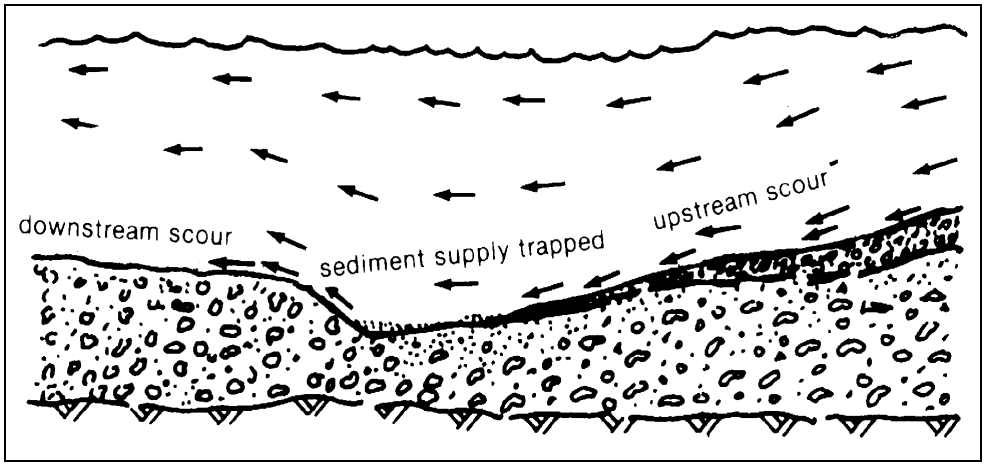


Figure 10.36 Borrow Area Filled after One Moderate Rise in the Stream



In-stream mining for aggregates and dredging for navigation and flood control can be extremely damaging where so much material is removed from the stream bed that all of the incoming sediment supply is trapped and degradation of long reaches occurs (Figures 10.37(a) and 10.37(b)). At some locations, dredging may be necessary, or commercial mining cannot be terminated either by legal action or by purchase. In these cases, measures to stabilize the stream bed elevation and the stream bank may be necessary, or pier and abutment foundations must be set below the expected future elevation of the stream bed.

(e) Combined Effects of Natural, Local, and Contraction Scour. Two methods for estimating total scour at a crossing site are provided in Section 10.9.C. and by Chapter 10 of the *Model Drainage Manual* (AASHTO, 2005). Method 1 is used when stream bed armoring is of concern, more accurate contraction scour estimates are necessary, or deposition is expected and is a primary concern. Method 2 is used when armoring is not a concern or insufficient information is available to permit evaluation or where more accurate estimates are not necessary.

Figure 10.37(a) Degradation in a Stream Mined for Sand and Gravel



Figure 10.37(b) Degradation in a Stream Mined for Sand and Gravel



There are large uncertainties in scour estimates. The following factors should be taken into account when using the final estimate of total scour for the design of piers and abutments (Neill, 1973):

- The reliability of hydrologic, hydraulic and geotechnical data.
- The probability of exceedance of design floods.
- The foundation type and its susceptibility to failure in the event that estimated scour is exceeded.
- The consequences of total or partial pier failure.

- Agency experience on the same or comparable streams.

6. Stream Environment. The environmental impacts of a stream crossing and measures for mitigating impacts and enhancing the stream and floodplain environment are an integral part of the analysis of a highway-stream crossing system. *A Guide for Wildlife Protection and Conservation for Transportation Facilities* (AASHTO, 1976), *Highway Drainage Guidelines: Hydrology* (AASHTO, 2000a), *Floodplain Management Guidelines for Implementing Executive Order 11988* (U.S. Water Resources Council, 1978), and *Water Quality Manual* (Shirley et al., 1976) are recommended for evaluating possible impacts on stream and streamside environments and mitigation and enhancement measures that should be considered. Preceding *Highway Drainage Guidelines* (AASHTO, 2000) contain a great deal of material relative to the protection of streams, and a future Guideline will deal specifically with water quality considerations. *Restoration of Fish Habitat in Relocated Streams* (FHWA, 1979a), *Highway Drainage Guidelines*, Volume II, *Hydrology* (AASHTO, 2000a) and *Stream Corridor Restoration Guide* (NRCS, 1998) are suggested as guides for the restoration of habitat in modified stream channels.

Hydraulic engineers most commonly have been concerned with flood flows in the design of stream crossings. The importance of low flows and the effects of alterations to low flows have been recognized in recent years. *Restoration of Fish Habitat in Relocated Streams* (FHWA, 1979a) and HDS-6, *Highways in the River Environment* (FHWA, 2001c) contain a wealth of information on methods of stream rehabilitation.

Wetlands often are associated with floodplains, and special attention is warranted so as to cause minimal effects on the functions of wetlands. *Ecological Effects of Highway Fills on Wetlands: Research Report* (Transportation Research Board, 1979b) and *Ecological Effects of Highway Fills on Wetlands: User's Manual* (Transportation Research Board, 1979c) are suggested as guides for use in analyzing the effects on wetlands associated with stream crossings.

Sections 10.1.D. and 10.3.H. contain information on environmental planning and data acquisition which is valuable in establishing criteria for designs to protect the water environment.

7. Economic Analysis. The objective in the design of a stream crossing system is to select the alternative which satisfies all engineering and environmental criteria and has the least total cost to construct, maintain and operate. This objective also is applicable to other components of a highway. It presumes, however, that least total cost design also will satisfy political and social considerations and emergency vehicle and emergency evacuation needs, and that budget allocations are unlimited. In the real world, political and social considerations and emergency needs affect decisions daily. Budgetary constraints exist in that all highway needs cannot be met instantly, and every decision regarding the selection of an alternative design affects the timing of projects to fill needs of lesser priority. Alternative selection also has budgetary implications in that the highway agency accepts the maintenance and operation costs, repair and reconstruction risks, and liability associated with the alternative. The decision, therefore, should be based on the best information that can be developed regarding capital costs and probable future costs. Where this information is available, the cost to the public of choosing other than an optimum design can be identified and dealt with rationally.

When standard recurrence intervals are prescribed for design, the engineering and economic analyses consist of selecting the profile and waterway opening alternative which will meet that standard at the least total expected cost and conducting studies to determine whether a higher standard is warranted for that particular crossing. It also may be desirable to extend the analyses to other exceedance probabilities to provide information on the incremental costs of designing for a different probability of exceedance.

Freeboard above stated recurrence interval floods and fixed crossing profiles both require determination of the largest flood that will flow through the waterway opening(s). This is necessary in order to be able to estimate expected future costs of flood damages to the highway, incremental damages to other property from backwater, and the expected future costs of traffic interruption and detours. The economic analysis consists of selecting the alternative which meets the constraint and results in the least total economic cost. Documentation of the analysis should include the economic costs of designing with the constraints imposed by the fixed gradeline and freeboard.

Hydraulic Design of Bridges with Risk Analysis (Schneider & Wilson, 1980) and HEC-17, *The Design of Encroachments on Flood Plains Using Risk Analysis* (Corry et al., 1980) are recommended for techniques of

economic analysis of stream crossing systems. Selected references listed in HEC-17, *The Design of Encroachments on Flood Plains Using Risk Analysis* (Corry et al., 1980) are recommended for further reading and study.

C. Protective and Preventive Measures. Numerous measures are used at stream crossings to protect the highway facility from flood damages. These may be categorized as measures intended to prevent such damages, such as riprap to prevent or inhibit stream bank erosion, and measures intended to protect the highway facility from flood damages, such as deep pier foundations to protect a bridge against severe scour. Measures generally in use are more easily explained by discussing elements of the crossing rather than by the categories of protective and preventive measures.

The principles of economic analysis can be extended to measures used to protect the crossing system to the extent that damages are predictable. For example, in the study of a crossing system, a question may arise regarding the economic feasibility of providing slope protection for the downstream side of the highway fill. Considering capital costs for slope revetment, the cost to replace lost pavement and embankment, and the cost of traffic delays while repairs are made, revetment may prove to be justified for the alternative having a 10% probability of suffering severe damage and not justified for the alternative with a 4% probability of suffering extensive damage from overtopping. Similar logic can be applied to slope protection at embankment ends, the use of spur dikes, stream bank protection, pier foundations (see the following sections), and numerous other appurtenances and components of the stream crossing system.

Actions of alluvial streams often are unpredictable, especially regarding the time which will pass before an anticipated event will occur, such as the passage of a meander bend. Considering the large investments that usually are involved in river training and control, it is sometimes prudent to adopt a "wait-and-see" strategy. Knowledge of the stream and its response to floods is the best guide for determining when protective works should be installed. When protection is needed, whether at the time of construction of the crossing system or at a later date, the cost of providing the control measures should be compared with the costs of traffic delays and repairing damages that would be incurred.

1. Pier Foundations. Preventive and protective measures both are used at piers to avoid damage by scour, which is the primary flood-related concern at piers. Preventive measures generally are intended to minimize the flow disturbance caused by the pier and thus to minimize local scour at the pier, or they are intended to inhibit or arrest local scour at the pier at an acceptable depth.

Protective measures consist of various types of foundations at depths secure from failure by scour. Protective and preventive measures both are used at some locations, particularly where cofferdams are used and scour during construction could greatly exceed anticipated scour after the cofferdams are removed.

Stream degradation and general and natural scour at the crossing site should not be overlooked whether preventive, protective or both measures are used to secure the pier against scour. The effect of piers in contracting the flow and causing general scour should be considered in selecting span lengths. Debris and ice accumulations upstream of a bridge can cause great exaggeration of general scour even during relatively minor floods (see Figure 10.15). Submergence of the bridge superstructure also will contract flow and cause increased general scour. The depth of any bridge superstructure, including parapets, that can become submerged at any time is very important to the security of the bridge. Open rails, no curbs, and a substructure that is as slender as possible should be used where a chance of submergence exists.

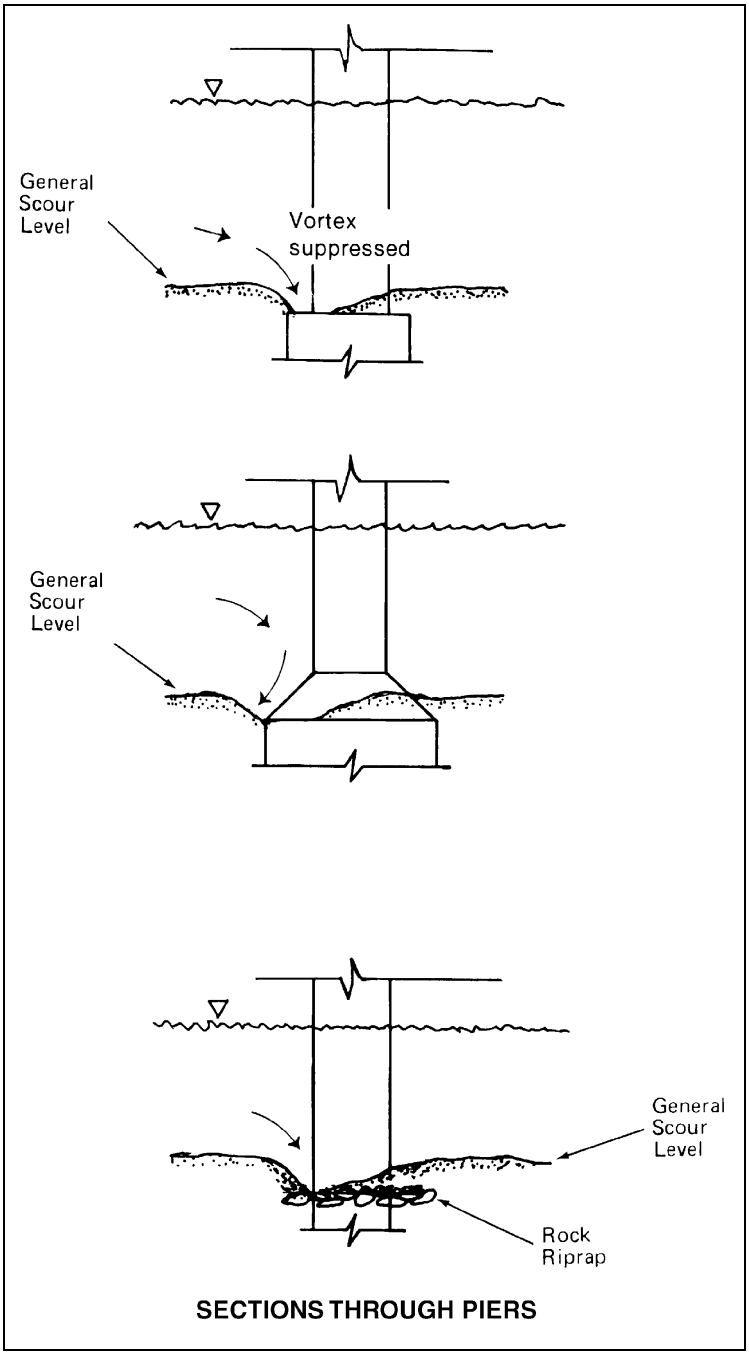
While it may not be possible to subject the design of pier footings to a rigorous economic analysis because of uncertainties in scour predictions, a good case can be made for extreme conservatism in foundation design (Highway Research Board, 1970; Laursen, 1970). The capital costs of providing a foundation secure against scour usually will be small when compared to the risk costs of scour-related failure.

(a) Preventive Measures at Piers. The preventive measures most commonly recommended to minimize flow disturbance and resultant scour are circular piers and a circular nose on an elongated pier oriented with the flow direction. *Synthesis of Highway Practice 5, Scour at Bridge Waterways* (Highway Research Board, 1970), and *Scour at Bridge Crossings* (Laursen, 1958) provide information on the effects of pier shape and pier orientation on local scour depths.

Pier foundations also may be considered in the category of measures used to minimize flow disturbances and resultant local scour. Figure 10.38 illustrates three foundation treatments that are intended to suppress the vortex and thereby reduce the sediment transport capacity out of the scour hole (Neill, 1973). An increased width of pier projected normal to flow direction in the stream tends to increase local scour; therefore, measures illustrated in Figure 10.38 should be below the expected general scour level. The foundation or riprap provided to suppress the vortex should be extended to the expected limits of the vortex action in order to minimize the chance of undermining. *Guide to Bridge Hydraulics* (Neill, 1973) suggests that the measure used to suppress the vortex should extend out from the pier for a distance at least equal to the diameter or width of the pier shaft, and up to twice this distance is acceptable.

A very conservative approach is recommended in estimating the extent of general or natural scour for the use of vortex suppression measures. A conservative approach also is recommended in the design of the suppression measure and judging its probable effectiveness. These measures will not control degradation, general or natural scour, and may fail to control local scour. They may even make it worse if the enlarged section of the pier is exposed to the main current of the stream.

Figure 10.38 Measures to Suppress Vortex and Reduce Local Scour (after Neill, 1973)



Other preventive measures that have been used usually are intended to minimize the effects of general or natural scour at the bridge and to protect the bridge from progressive stream degradation. These measures usually consist of hardening the perimeter of the waterway at the bridge with riprap, soil cement or concrete. At some locations, drop structures or grade-control structures of concrete, sheet piling, gabions or grouted riprap are used in or downstream of the bridge waterway opening to halt the progression of stream degradation. All of these measures are susceptible to failure by piping and by stream degradation which exceeds anticipated degradation to the extent that the downstream side of the structure is undermined. Water bypassing the structure because of erosion of the stream banks into which the structure is tied also can cause failure of grade-controlled structures.

Generally, flexible drop structures are less susceptible to failure from undermining and piping because of their self-healing qualities. Rock must be large enough to prevent displacement by high velocity flow or wire enclosed to form an articulated structure. The effects of piping can be controlled with gravel or geotextile filters. An apron should be provided downstream of the structure to prevent undermining by scour. Structures intended to protect bridges from damage by degradation often involve substantial capital investment, and they should be designed to withstand the static and dynamic hydraulic forces that will be imposed on them. Failure of the drop structure could involve the loss of investment in the bridge it was intended to protect as well as the investment in the drop structure. Physical modeling of design alternatives may be warranted in many instances. Figure 10.39 shows the construction of a drop structure of grouted riprap between concrete walls. Figure 10.40 shows a more conventional structure of sheet piling and rock riprap.

Caissons are used in large alluvial rivers such as the Mississippi, Missouri, Arkansas and Red Rivers, where extreme scour is expected. In many cases, woven mattresses of willow or other materials are used for scour prevention during and after construction. The rock used on the woven mattress must be large enough to resist rolling during extreme floods if the mattress is intended to afford protection after construction is completed.

(b) Protective Measures at Piers. Several foundation types are used at locations where protection against scour is provided by deep foundations. Spread footings located below anticipated general and local scour are illustrated in Figure 10.17. These generally are used in relatively stable streams whose beds are naturally armored with boulders and cobbles, and in streams where deeper foundations are extremely difficult to construct because of boulders below the stream bed. Spread footings also are used where the pier can be founded on sound rock.

Serious problems and failures have occurred at numerous locations where spread footings were used in seemingly stable streams and the footings supposedly were placed at secure depths, or the difficulties in driving piles or using drilled shaft foundations were seemingly insurmountable. Figure 10.18 illustrates such a location.

Figure 10.39 Drop Structure of Concrete Walls and Grouted Riprap



Figure 10.40 Grade Control Structure of Sheet Piling and Rock Riprap



Shales, sandstones, and other soft rocks can be eroded, go into solution, or delaminate when exposed. Problems have developed at bridges founded on these erodible rocks. Drilled shafts or drilled piers offer a practical solution to the provision of a foundation secure against scour in these rocks as well as in most gravels and clays where piling cannot be driven. Figure 10.18 shows typical drilled shaft designs.

Piling driven deep below anticipated scour depths affords protection against scour where the piles are of sufficient length to support the structure after scour. The piling should have reserve bending strength, or battered piles should be utilized to resist the forces of ice and debris in the scoured condition. Load-bearing capacity based on skin friction will be different after scour occurs. Large piling sizes or concrete filled pipe piles which depend on point-bearing capacity will increase the ability of the piling to withstand scour in many cases. Figure 10.19 shows typical pile foundation designs.

Some protection of pile footings may be provided by sheet piling surrounding the footing. The pile cap should be located below expected natural or general scour or the enlarged section will cause exaggerated local scour because of the large area subjected to the main current of the stream. The pile cap should be extended to suppress the vortex action at the pier; allowing the sheet pile to project above the cap also may help to suppress the vortex. Figure 10.20 shows a sheet pile-protected pile footing.

2. Abutments and Approach Fills. Embankments projecting into floodplains are susceptible to scour damage from flow concentrations at the abutments, along the embankment where efficient channels are formed by right-of-way clearing and borrow excavation, and by flow over the roadway. Measures commonly used to inhibit scour or protect the embankment from damage by scour include revetments of concrete, rock, sacked cement and sand or soil cement, sheet pile retaining walls, spur dikes, toe dikes, guide banks, and other dikes of various descriptions used to impede flow along the projecting embankment.

(a) Protective and Preventive Measures Along Embankments. Embankments that encroach on flood plains are most commonly subjected to scour and erosion damage by overflow and by flow diverted along the embankment to waterway openings. Damage also can occur from the redistribution of flow in the floodplain downstream of the waterway.

The incidence of damage from flow along approach fills probably is highest in wooded floodplains where the rights-of-way are cleared of all trees and where borrow areas are established upstream of the embankment. Damage to approach fills usually is not severe, but scour from the flow contraction at the abutment will be greater than would otherwise occur. Preventive measures along the embankment, therefore, usually are intended to provide for security at the abutment rather than for the approach fill.

The most commonly recommended preventive measures are abstention from right-of-way clearing beyond the fill slope and use of borrow spaces which will not cause damage to the highway facility during floods. Other measures which have been used are pervious dikes of timber or finger dikes of earthen material spaced along and normal to the approach fill to impede flow along the embankment. Spur dikes and toe dikes at the abutment also serve this purpose. In wooded floodplains, dikes used to impede flow or align flow with the waterway opening should extend into wooded areas. Riprap or other revetment is not usually necessary to protect the fill from lateral flow except where the fill is of cohesionless material or the crossing is on a relatively severe skew with the floodplain. Where there is adequate rainfall, plating the fill slope with topsoil and a good vegetative cover usually will provide adequate protection for fills of cohesionless material.

The measure commonly used to protect fills projecting into floodplains against erosion and scour from overtopping is to establish a "freeboard" in the crossing profile above design flood elevations. There is a probability, however, that the design will be exceeded and a probability that the highway facility will be overtopped. The practice of ignoring these probabilities may lead to design imbalance in that floods which exceed the design flood are forced to pass through the waterway opening. The frequent consequences of floods significantly in excess of the design flood being forced through the waterway opening are failure of abutments and end spans of bridges from scour and destruction of approach fills at the abutment. The alternative to "freeboard" is a crossing profile which has an acknowledged probability of overtopping and waterway opening designed considering the risk of damage from the flood that must flow through the opening. If practicable, overflow should occur at a location or locations removed from the bridge in order to minimize the chance of damage to the bridge. Where the probability of overtopping

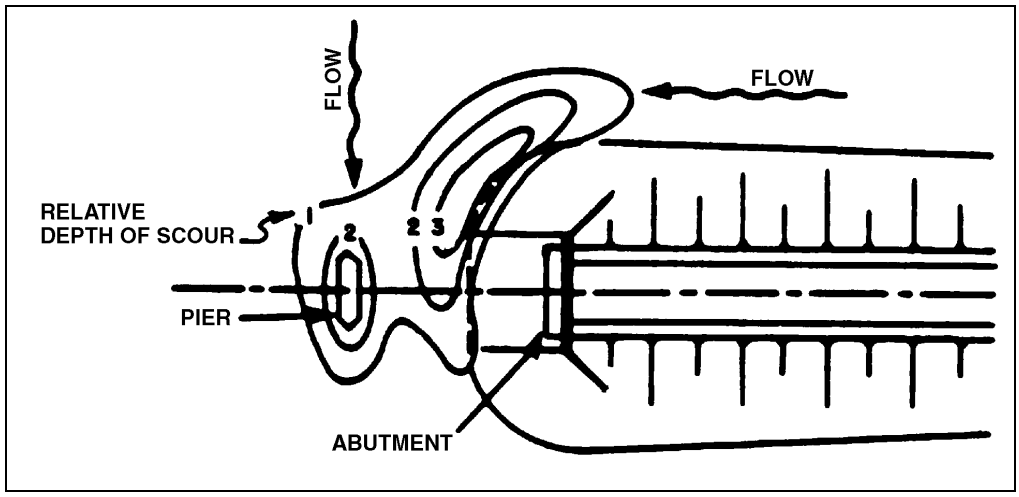
is acknowledged and planned for, documented justification of money committed to construction and flood damage repair can be provided.

Preventive measures used to protect approach fills from erosion and scour from overflow are revetments usually of rock, wire-enclosed rock, or concrete. Rock riprap should be of adequate size to prevent displacement, dumped rock is preferred over hand-placed riprap, and a filter of granular material or filter cloth should be provided. Economic justification can be established by a comparison of the capital cost of construction with the expected annual traffic costs and repair costs without revetment.

Preventive measures also are needed at some crossings to protect the embankment against wave action, especially at reservoirs. Riprap of durable, hard rock usually is used at these locations. The top elevation of the rock required is dependent upon storage and flood elevations in the reservoir, and wave height, which can be computed using wind velocities and the reservoir fetch. The *Model Drainage Manual* (AASHTO, 2005) is recommended for design guidance.

(b) Protective and Preventive Measures at Abutments. Scour at abutments usually is caused by turbulence from diversion of floodplain flow into the waterway opening by a fill embankment. For abutments on a fill slope, the scour usually occurs at the upstream corner of the embankment and, dependent upon the degree of contraction, flow depths and volume flow rate in the floodplain may extend to the first or second pier of the bridge (Figure 10.41). Protective measures used at abutments consist of vertical abutment walls, sheet pile toe walls, and deep foundations of piles or drilled shafts. Vertical abutment walls founded below anticipated scour depths will protect bridge ends and the embankment if the walls are extended around the fill slopes to the depth of anticipated scour.

Figure 10.41 Scour Abutments and Adjacent Pier (after Highway Research Board, 1970)



Sheet pile toe walls sometimes are installed to repair scour damage after a flood. They commonly are used where rock is not available or access for placing rock is difficult. Figure 10.42 illustrates the use of deadmen for anchoring to provide an effective structural system. Other measures such as a structural cap or steel H-piles also may be necessary.

Preventive measures commonly used are revetments of rigid or articulated concrete, sacked concrete, or spur dikes or guide banks to align flow with the waterway opening and move scour away from the bridge end, and toe dikes to prevent lateral flow from eroding the corner of the downstream side of the embankment.

Revetment at the abutment usually is placed on the slopes under the bridge end and around the corners of the embankment to guard against progressive erosion of the embankment. Revetment on the fill slope does not inhibit scour from the flow contraction and, therefore, is susceptible to failure from undermining. The revetment should be continued down below the level of expected scour to protect it and the embankment from failure (Figure 10.43). An alternative that can be used on cohesionless soils is

a flexible apron extended to the limits of the expected scour (Figure 10.44). The apron tends to be self-healing since it will settle into any area that scours and inhibit further scour. Flexible aprons may not work as well on cohesive materials because the steep faces of failures are not protected by the material of the apron (Neill, 1973). Materials commonly used for flexible aprons are rock riprap, articulated concrete and wire-enclosed rock.

Figure 10.42 Sheet Pile Toe Wall (after Highway Research Board, 1970)

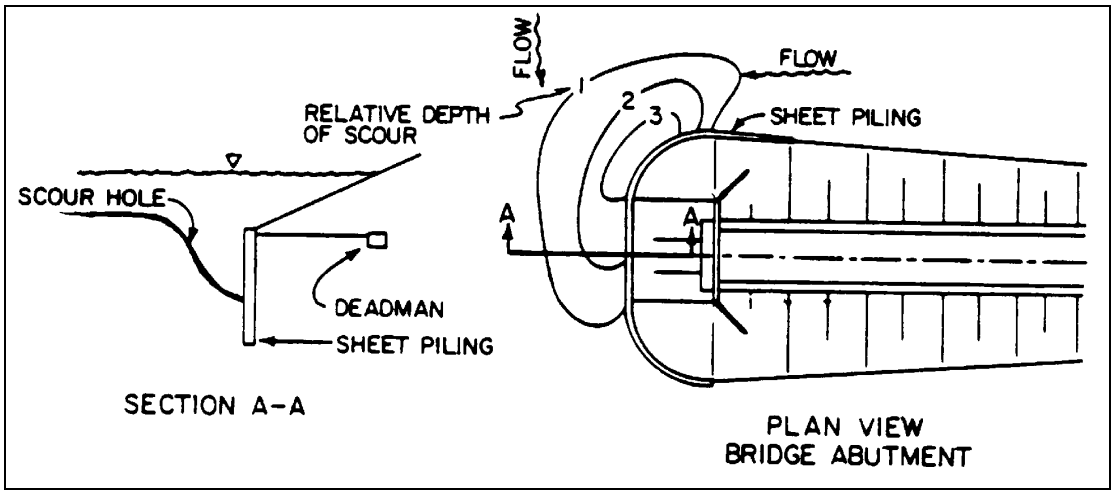
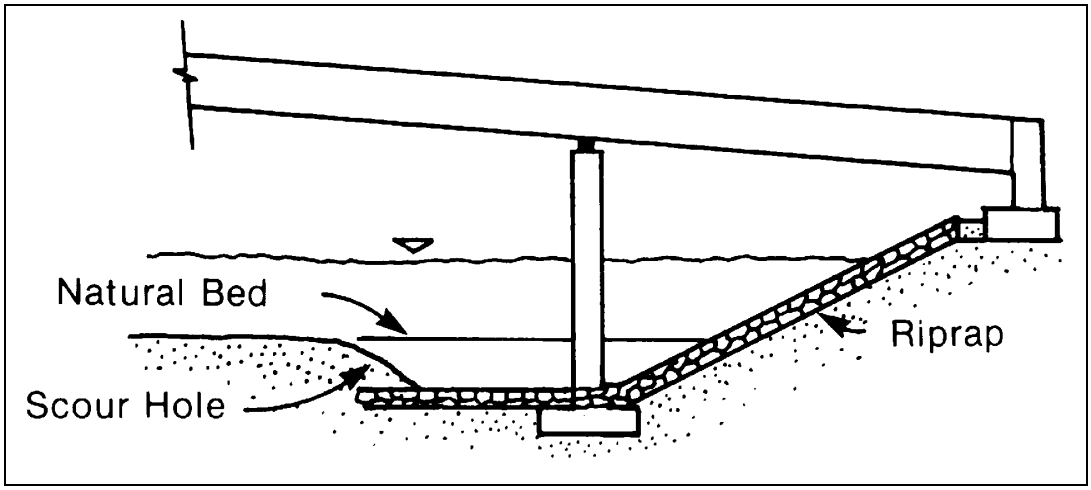


Figure 10.43 Cellular Concrete Revetment on Filter Cloth Revetment Toed in to Prevent Undermining



Figure 10.44 Rock Riprap at Abutment (after Highway Research Board, 1970)



Sheet piling also is used at the revetment toe in lieu of flexible aprons or extending the revetment below the expected scour. This would appear to have merit where rigid revetment is used, severe scour is expected, and materials are not available for a flexible apron. Measures are necessary to ensure the structural integrity of the sheet-piling structure after scour.

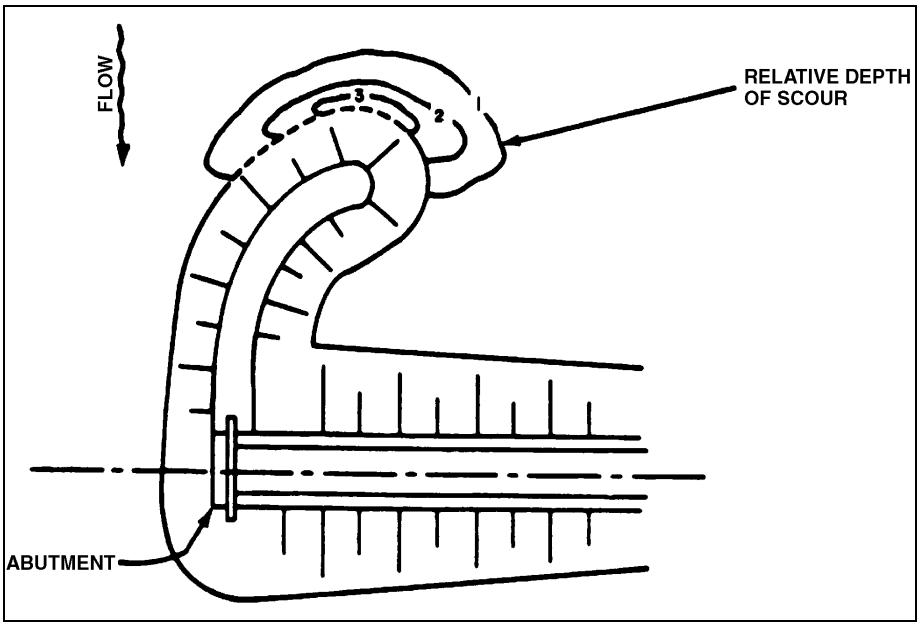
Guide banks are appendages to the highway embankment at the bridge abutment. They usually are smooth extensions of the fill slope on the upstream side. The twofold purpose of guide banks it to align flow from the floodplain with the waterway opening and to minimize scour at the abutment by moving the scour-causing turbulence upstream to the end of the guide bank (Figure 10.45). Guide banks are usually of earthen embankment but are sometimes of rock where an excess of this material is available. Revetment is advisable for protection of the dike where scour is expected to occur, although a failure at the upstream end of a guide bank usually does not immediately threaten the bridge end. Clearing around the end of the dike should be kept to a minimum in wooded floodplains in order to enhance the effectiveness of a guide bank in reducing turbulence. A small culvert through the dike in lieu of a drainage channel around the end to provide for local drainage also serves to minimize the turbulence of mixing flows from different directions.

The suggested shape of Guide banks is elliptical with a major to minor axis ratio of 2.5:1 (Bradley, 1978). The length suggested varies with the ratio of flow diverted from the floodplain to flow in the first 30 meters (100 ft) of waterway under the bridge. The shape suggested is based on laboratory experiments and the length on model and field data (Bradley, 1978). Optimum shape and length undoubtedly will differ for each site and possibly for each flood at a site. Field experience has shown that the recommended elliptical shape is usually quite effective in reducing turbulence. If practical reasons require the use of another shape such as a straight dike, more scour should be expected at the upstream end of the dike.

Toe dikes sometimes also are needed downstream of the bridge end to guide flow away from the structure so that redistribution in the floodplain will not cause erosion damage to the embankment downstream at the ridge end. This need usually occurs in wooded floodplains where the right-of-way is cleared of trees, especially if some portion of the floodplain at some distance removed from the waterway opening has been cleared of trees (Figure 10.46). Toe dikes similar to those described for use upstream of the waterway opening are appropriate, but the shape is of less importance. The length should be sufficient to force flow into the forested area downstream, if present, or long enough to move scour away from the bridge end if the floodplain is not wooded. In some instances, downstream clearing or borrow excavation within the right-of-way has caused erosion downstream at the bridge end and changed the distribution of flow downstream of the highway facility. Imagine a natural ridge in the wooded area in Figure 10.46 between the stream and the cleared field. Prior to construction of the highway facility, the cleared field would not experience floods very often. Use of the right-of-way for borrow could divert flow to the field

during any flood event that is out-of-banks. Restoration of the divide within the right-of-way would alleviate flooding caused by the borrow.

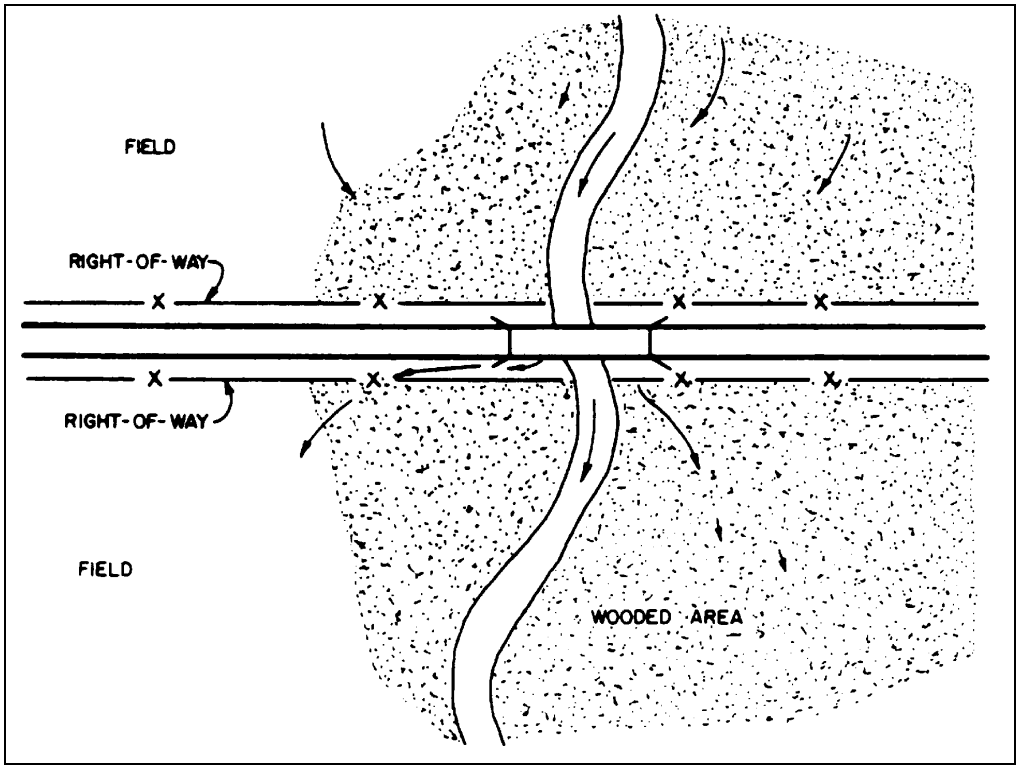
Figure 10.45 Guide Bank (after Highway Research Board, 1970)



3. Bank Stabilization and River Training. Bank stabilization and river training devices are preventive measures installed to inhibit erosion and movement of stream banks. The measures may be necessary as defense against actions of the stream which threaten the highway crossing or to protect the stream banks and the highway from an anticipated response to construction of the highway. Table 1 in *Highway Drainage Guidelines: Hydraulic Analysis and Design of Open Channels* (AASHTO, 2000d) is convenient for making qualitative assessments of stream responses to highway crossings.

Various materials and devices are used for bank stabilization and river training, including rock riprap, concrete lining, wood, steel or rock jetties, steel or concrete jack fields, wire fences, timber bulkheads, articulated concrete mattresses, and guide banks, dikes and spurs usually of earth and rock. The choice of the appropriate device or devices for use depends on the total action of the river for a distance upstream and downstream. Study of long reaches will help to avoid futile attempts at localized control where the river is in the midst of changes that will bypass the control measure or make it unnecessary. Regardless of the size of the stream and the control measure used, stream response to the installation of the measure must be considered. For instance, bank stabilization at a crossing can cause scour in the bed of the channel or redirect the current of the stream toward an otherwise stable bank downstream of the stabilized reach.

Figure 10.46 Partial Downstream Scour from Flood Redistribution



Bank stabilization and river training is a very specialized field which requires intimate knowledge of the stream and its propensity to change, knowledge of the bed load and debris-carrying characteristics of the stream, and experience and experimentation at similar sites on the same or similar streams. Design is, to a large extent, an art, and many questions concerning the relative merits of various measures have not been answered definitively (AASHTO, 2000d; ASCE, 1965; FHWA, 1991; California Division of Highways 1960; Neill, 1973; USACE, 1969, HEC-18, HEC-20, HEC-23, and HDS-6 are recommended reading).

The following general principles for the design and construction of bank protection and training works are adapted from *Guide to Bridge Hydraulics* (Neill, 1973):

a. The cost of the protective measures should not exceed the cost of the consequences of the anticipated action of the stream.

b. Designs should be based on studies of channel trends and processes and on experience with comparable situations; the ultimate effects of the works on the natural channel both upstream and downstream should be considered.

c. Site reconnaissance is imperative; reconnaissance may be by on-site inspection, aerial reconnaissance, and/or aerial photographs taken over a period of years.

d. The possibility of using physical model studies should be considered at an early stage.

e. The works should be inspected periodically after construction with the aid of surveys to check results and modify the design, if necessary.

f. In lieu of maintaining an existing bridge in trouble, consideration should be given to an alternate location away from the river hazard.

The "principle of expendability," paraphrased from *Bank and Shore Protection in California Highway Practice* (California Division of Highways, 1960), can be added to the above list.

g. Don't lose sight of the fact that the objective in installing bank stabilization and river training measures is to protect the highway. The protective measures themselves are expendable.

The effectiveness of protective and training measures in many alluvial streams, and indeed, the need for the measures, may be short-term because the stream will move to attack at another location or even outflank the installation. Extensive works which would be required for long-term effectiveness would violate the first principle above; therefore, they usually are not considered. The alternative to extensive training works is a continuing effort to protect the highway by successive installations intended to counter the most recent actions of the stream. Each successive installation usually is tested against the first principle considering the ability of the highway at that point in time to provide the desired traffic service. In effect, this is an application of the watchful waiting suggested in Section 10.5.C., but the implications are greater than the anticipated future need of a "one-time" installation. In some cases, if the need, the time of need, and the cost of future installations could be anticipated, a cost analysis might show that a bridge long enough to allow the stream to follow natural processes would be in the public interest. Planned-for extension of the bridge after (and if) the stream destroys an approach roadway might be a viable alternative to a long bridge initially.

Where measures are needed to protect a highway facility from anticipated actions by a stream, the possibility of a cooperative project with another governmental agency, particularly the USACE, should be investigated. Other agencies such as NRCS, U.S. Fish and Wildlife, and PA DEP, have responsibilities and authority to undertake stream stabilization efforts, and mutually beneficial projects may be possible.

4. Buoyant, Drag, Debris and Ice Forces on Bridge Superstructures. The first line of defense against hydraulic forces imposed on a submerged or partially submerged bridge superstructure is to locate the bridge at an elevation where the probability of submergence is small. Obviously, this is not economically or physically practical at many locations.

The second line of defense is to make the superstructure as shallow as possible. Box girders which would displace great volumes of water and have a relatively small weight compared to the weight of the water displaced are not a good design alternative unless the probability of submergence is very small. Solid parapets and curbs which increase the effective depth of the superstructure can give increased buoyancy over that of open rail designs. If submerged, the increased effective depth of the superstructure will cause increased general scour, and drag forces on the substructure will be much greater than with open rails.

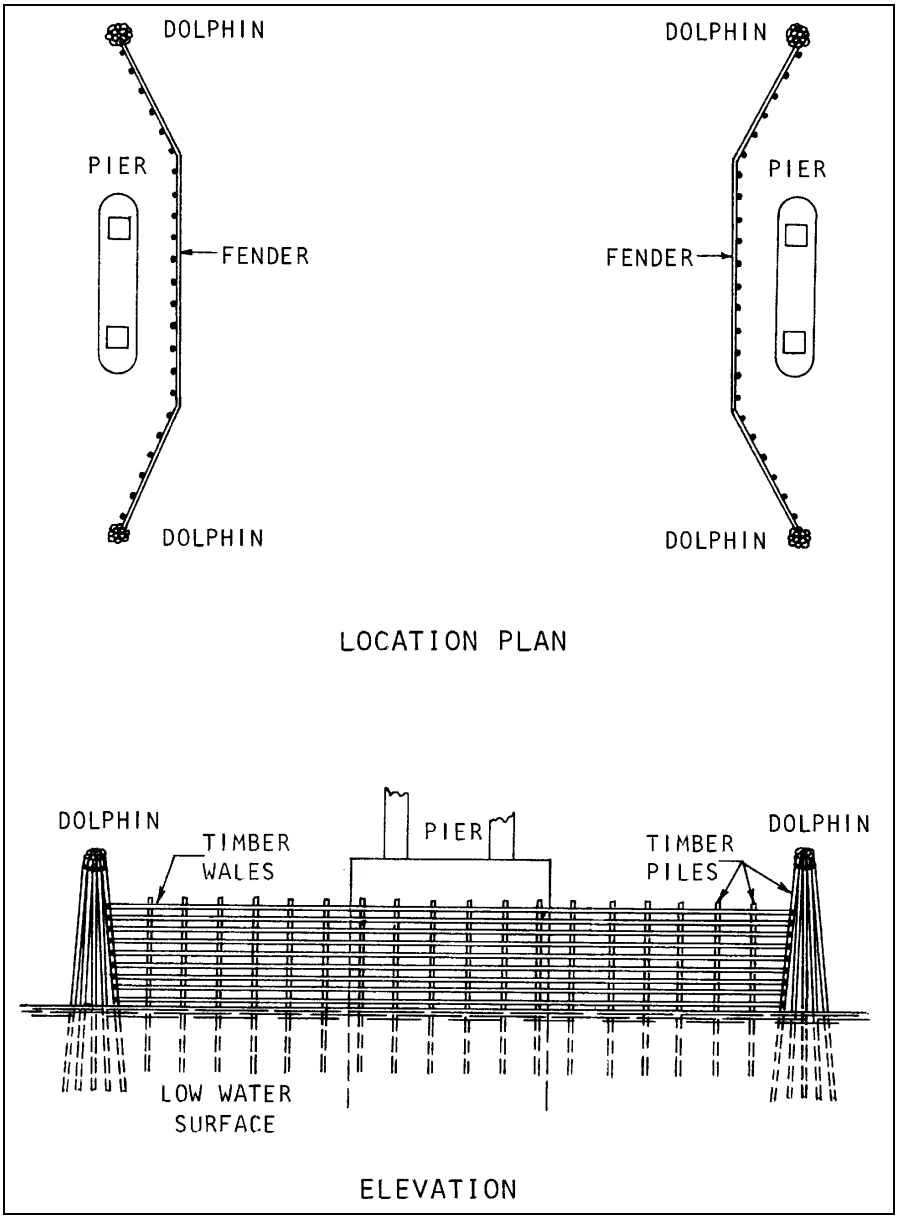
The third line of defense is to provide a roadway approach profile that will be overtopped prior to bridge superstructure submerging. This measure will reduce the probability of submergence for a bridge at a given elevation. It also will afford relief against bridge end scour, general scour and local scour which are accentuated by superstructure submergence, and debris clogging the waterway openings.

Where there is even a small probability of total or partial submergence, superstructures should be securely anchored to the substructure to resist buoyant, drag, debris and ice forces. Air holes also should be provided through each span and between each girder to reduce the volume of displaced water by release of air that would otherwise be trapped under the superstructure.

D. Dolphins and Fender Systems. Dolphins and fender systems are two slightly different structural systems with the same purpose. In reference to bridges, this purpose is to protect piers, bents and other bridge structural members from damage due to collision by marine traffic. Dolphin types range from simple pile clusters to massive concrete structures. Fender system types are less variable, consisting usually of pile-supported stringers, as shown in Figure 10.47.

The need for dolphins and fender systems often can be eliminated by spanning smaller rivers or by judicious pier placement. Construction costs of long spans may be economically unattractive when compared with shorter spans; however, when all costs are considered, both construction- and maintenance-related, the long span solution may be the most attractive. The bridge designer should receive guidance from the engineer in the form of estimated depths of scour for the dolphin and fender systems. This information influences fender and dolphin pile lengths, diameters and spacing, thereby affecting cost comparisons. Also, estimated debris removal costs may be a factor. The bridge designer should consider these factors, as well as maintenance costs due to collisions, when making decisions on span lengths.

Figure 10.47 Dolphin and Fender System



Dolphin and pier scour may combine to cause deeper scour than either would cause separately. In some cases, fender systems may "shield" bridge piers, reducing velocities and scour at the pier; however, this shielding effect can vanish, or be modified, if the fender system is lost due to collision or unforeseen scour problems. Pier and fender systems introduced into relatively narrow rivers may cause general scour between the fender systems. This scour is usually greatest near the downstream end of the system.

10.6 HYDRAULICS OF BRIDGE OPENINGS

A. 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. Alternative methods of analyzing bridge hydraulics are discussed in this section, but emphasis is placed on the use of HEC-RAS.

The hydraulic variables and flow types as defined by FHWA are defined in Figures 10.48 and 10.49:

- Backwater (h_1) is measured relative to the normal water surface (NORMAL W.S. or N.W.S.) elevation without the effect of the bridge at the approach cross section (Section 4). It is the result of contraction and re-expansion head losses and head losses due to drag from bridge piers. Backwater also can 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 10.49.
- Type I flow consists of subcritical flow throughout the approach, bridge and exit cross sections and is the most common condition encountered in practice.
- Types 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 Type IIB, the critical water surface elevation in the bridge span 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.

Figure 10.48 Bridge Hydraulics Definition Sketch

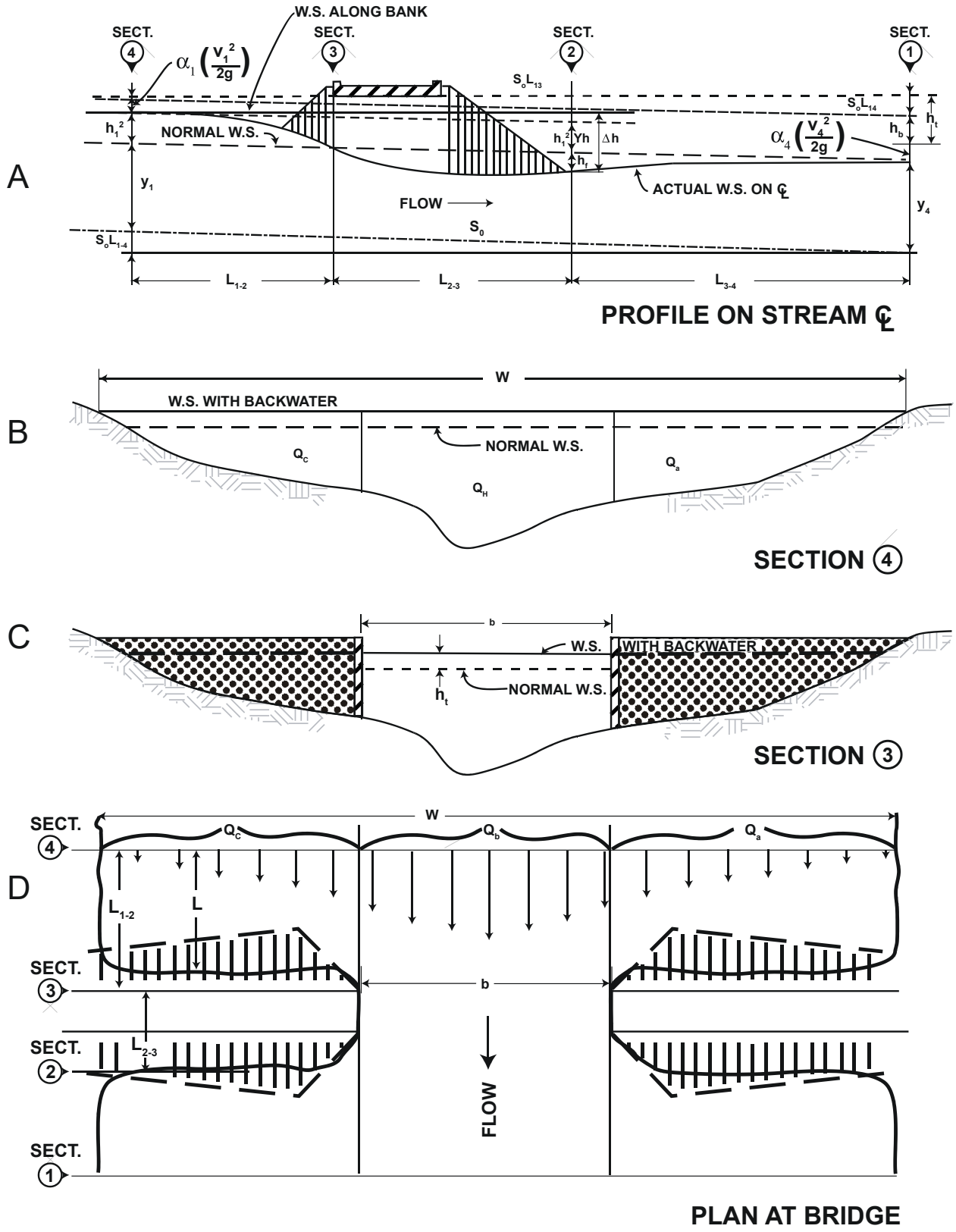
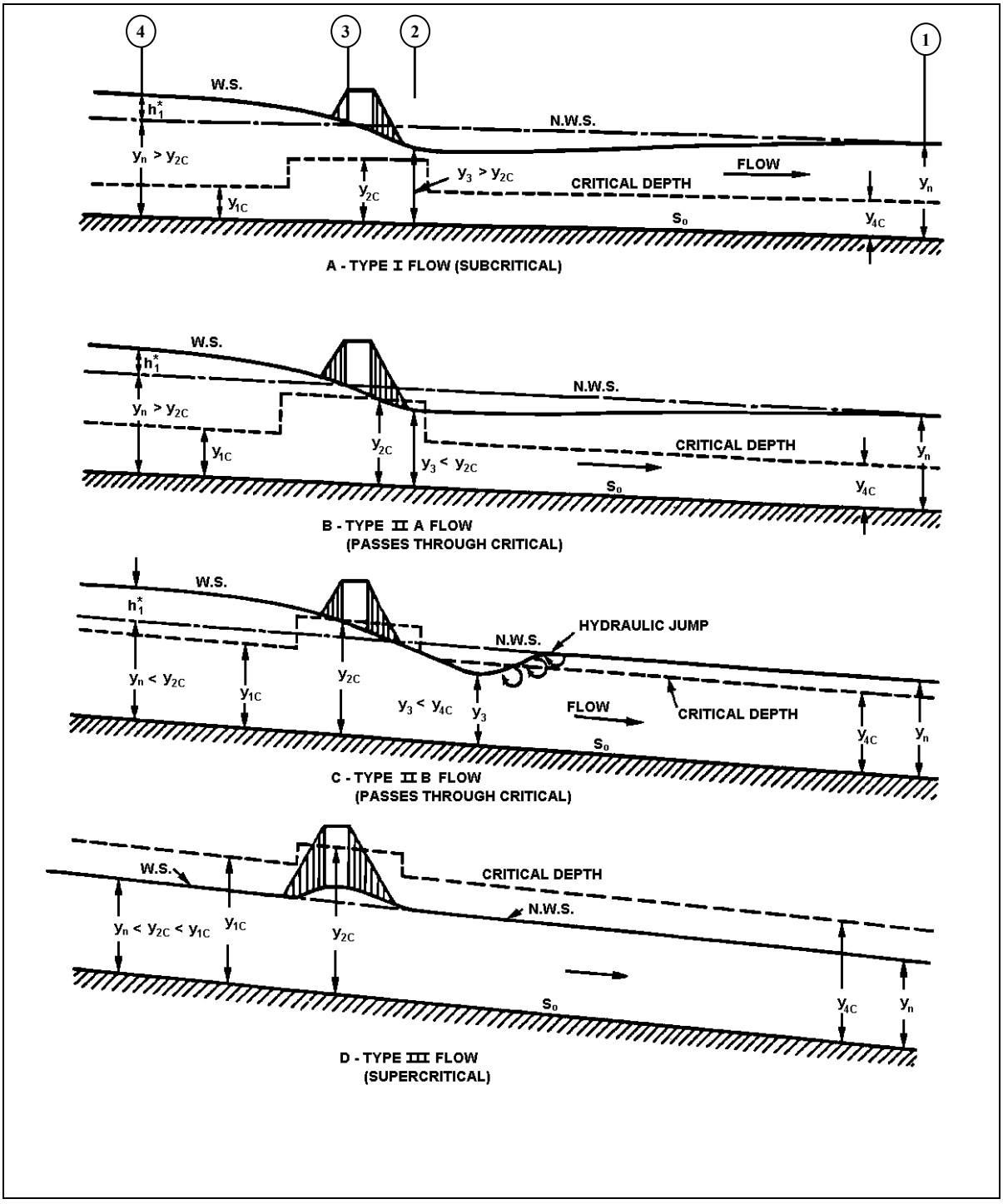


Figure 10.49 Bridge Flow Types



B. 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 usually can be achieved using both momentum and energy methods.

For the purpose of presentation, the computational methodologies used in the Hydrologic Engineering Center's HEC-RAS computer program will be utilized to describe the bridge hydraulic computations.

1. Two-Dimensional Modeling. The water surface profile and velocities in a section of river are often predicted using a computer model. In practice, most analyses are performed using one-dimensional methods such as the standard step method found in HEC-RAS, HEC-2, or WSPRO. While one-dimensional methods are adequate for many applications, these methods cannot provide a detailed determination of the cross-stream water surface elevations, flow velocities or flow distribution.

Two-dimensional models are required when the basic assumption that the water surface elevation and energy grade line elevation are constant across the cross sections is no longer valid, or when it is anticipated that flow velocities in the y and z directions may influence the hydraulic analysis. In one-dimensional modeling, flow velocities are assumed to be parallel in the x direction at any given cross section. Two-dimensional models are more complex and require more time to set up and calibrate. They may require more field data then a one-dimensional model and, depending on complexity, may require a little more computer time.

"Bri-Stars" is a quasi-two-dimensional model capable of computing alluvial scour/deposition through subcritical, supercritical and a combination of both flow conditions involving hydraulic jumps. It is capable of simulating channel widening/narrowing as well as local scour due to highway encroachments. "Bri-Stars" has a bridge component which allows the computation of hydraulic flow variables and the resulting scour. It also includes a companion expert system program which allows the classification of streams by their morphological properties.

The USGS has developed a two-dimensional finite element model for the FHWA that is designated FESWMS. This model has been developed to analyze flow at bridge crossings where complicated hydraulic conditions exist. This two-dimensional modeling system is flexible and may be applied to many types of steady and unsteady flow problems including multiple opening bridge crossings, guide banks, floodplain encroachments, multiple channels, flow around islands and flow in estuaries. Where the flow is essentially two-dimensional in the horizontal plane, a one-dimensional analysis may lead to costly over-design or possibly improper design of hydraulic structures and improvements.

SMS is a surface water modeling system that provides a windows interface to use FESWMS, RMA2 and other two-dimensional models.

2. Physical Modeling. Complex hydrodynamic situations may require physical models 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.

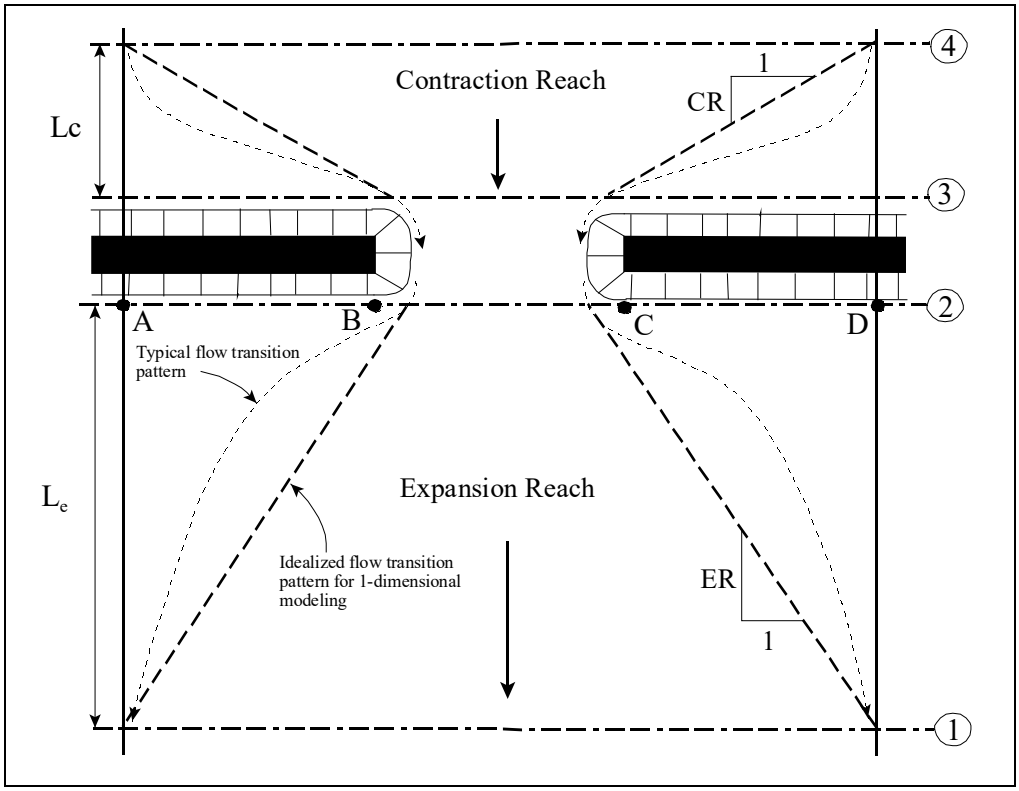
The constraints on physical modeling are:

- Size (scale).
- Cost.
- Time.

C. Bridge Modeling Methods. There are numerous methods for estimating the hydraulic impact of bridge openings on water surface profiles. PennDOT recommends and anticipates that computer programs will be employed to perform such estimates. Generally, the designer should refer to the documentation of the specific computer program for the theory employed and operating instructions.

1. HEC-RAS Modeling. The following sections provide a general discussion of recommended modeling approaches for the USACE Hydrologic Engineering Center's River Analysis System (HEC-RAS) computer program. The HEC-RAS computer model is capable of performing one-dimensional water surface profile calculations for steady and unsteady, gradually varied flow in natural or constructed channels. For illustration purposes, the minimum number of cross sections needed for HEC-RAS to run are shown in Figure 10.50. Additional requirements for cross section locations are described in Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10. The following sections have been excerpted from the HEC-RAS Hydraulic Reference Manual, Version 3.1. For additional information and guidance, the HEC-RAS help manuals should be consulted.

Figure 10.50 Cross Section Locations at a Bridge



Cross section 1 is located sufficiently downstream from the structure so that the flow is not affected by the structure (i.e., the flow has fully expanded). This distance (the expansion reach length, L_e) generally should be determined by field investigation during high flows. If field investigation is not possible, then alternative criteria for locating the downstream section must be used. The USGS has established criteria for locating cross section 1 a distance downstream from the bridge equal to one times the bridge opening width (the distance between points B and C on Figure 10.50, (WSPRO User Manual)). The USACE locates the downstream cross section four times the average length of the side constriction caused by the structure abutments (the average of the distance from A to B and C to D on Figure 10.50 (see HEC-2 User Manual, USACE, 1983). The expansion distance will vary depending upon the degree of constriction, the shape of the constriction, the magnitude of the flow, and the velocity of the flow.

A detailed study was completed by the Hydrologic Engineering Center entitled *RE-42, Flow Transitions in Bridge Backwater Analysis* (USACE, 1995a). The purpose of this study was to provide better guidance to hydraulic engineers performing water surface profile computations through bridges. Specifically, the study focused on determining the expansion reach length, L_e; the contraction reach length, L_c; the expansion energy loss coefficient, C_e; and the contraction energy loss coefficient, C_c. A summary of this research, and the final recommendations, can be found in *RE-45, Hydraulic Reference Manual for HEC-RAS (*USACE, 1995b).

The user should not allow the distance between cross sections 1 and 2 to become so great that friction losses will not be adequately modeled. If warning messages indicate that additional cross sections are needed between cross sections 1 and 2, then intermediate cross sections may be placed within the expansion reach in order to adequately model friction losses. The ineffective flow option can be used to limit the effective flow area of the intermediate cross sections in the expansion reach.

Cross section 2 is located a short distance downstream from the bridge (i.e., commonly placed at the downstream toe of the road embankment). This cross section should represent the effective flow area just outside the bridge.

Cross section 3 should be located a short distance upstream from the bridge (commonly placed at the upstream toe of the road embankment). The distance between cross section 3 and the bridge should only reflect the length required for the abrupt acceleration and contraction of the flow that occurs in the immediate area of the opening. Cross section 3 represents the effective flow area just upstream of the bridge. Note that both cross sections 2 and 3 will have ineffective flow areas to either side of the bridge opening during low flow and pressure flow profiles. In order to model only the effective flow areas at these two sections, the modeler should use the ineffective flow area option at both of these cross sections.

Cross section 4 is an upstream cross section where the flow lines are approximately parallel and the cross section is fully effective. In general, flow contractions occur over a shorter distance than flow expansions. The distance between cross section 3 and 4 (the contraction reach length, L_c) generally should be determined by field investigation during high flows. The USGS has established a criterion for locating cross section 4 a distance upstream from the bridge equal to one times the bridge opening width (the distance between points B and C on Figure 10.50). The USACE used a criterion to locate the upstream cross section one times the average length of the side constriction caused by the structure abutments (the average of the distance from A to B and C to D on Figure 10.50). The USACE has performed a detailed study, *RE-42, Flow Transitions in Bridge Backwater Analysis* (USACE, 1995a). From that publication it is recommended that cross section 4 be located at one times the average length of the side constriction upstream from cross section 3.

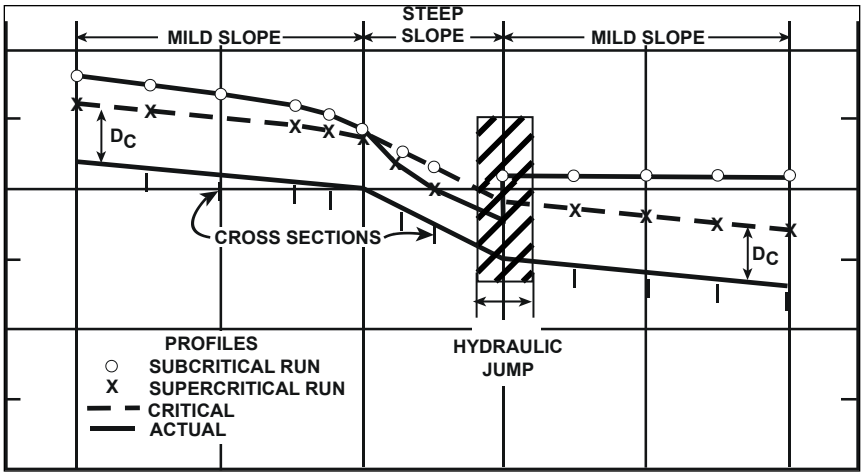
During the hydraulic computations, the program automatically formulates two additional cross sections inside of the bridge structure. The geometry inside of the bridge is a combination of the bounding cross sections (sections 2 and 3) and the bridge geometry. The bridge geometry consists of the bridge deck and roadway, abutments, and piers, if any. The user can specify different bridge geometry for the upstream and downstream sides of the structure if necessary. Cross section 2 and the structure information on the downstream side of the bridge are used as the geometry just inside the structure at the downstream end. Cross section 3 and the upstream structure information are used as the bridge geometry just inside the structure at the upstream end.

2. HEC-2 Modeling. The water surface profile used in the hydraulic analysis of a bridge should extend from a point downstream of the bridge that is beyond the influence of the constriction to a point upstream that is beyond the extent of the caused by the constriction. The cross sections that are necessary for the energy analysis through the bridge opening for a single opening bridge using the special bridge option are shown in Figure 10.51.

Energy losses caused by structures such as bridges and culverts are computed in two parts. First, the losses due to expansion and contraction of the cross section on the upstream and downstream sides of the structure are computed in the standard step calculations. Secondly, the loss through the structure itself is computed by either the normal bridge or the special bridge methods.

The user's instructional manual for HEC-2 should serve as a source for more detailed information for using the computer model. Numerous input/output examples are provided.

Figure 10.51 Cross Section Locations at a Bridge (HEC-2 User's Manual – 1991)



HEC-2 has its own data creation package -- COED -- that assists the user with preparing/editing input data and includes powerful online help features. A separate, stand-alone data editing program which checks for input/modeling errors, EDIT2, also is provided. Cross section, water surface profile and rating curve viewing/plotting/printing are provided using the PLOT2 program.

The normal bridge method handles the cross section at the bridge just as it would any river cross section with the exception that the area of the bridge below the water surface is subtracted from the total area and the wetted perimeter is increased where the water surface elevation exceeds the low chord. The normal bridge method is particularly applicable for bridges without piers, bridges under high submergence and for low flow through circular and arch culverts. Whenever flow crosses critical depth in a structure, the special bridge method should be used. The normal bridge method is automatically used by the computer for bridges without piers and under low flow control, even if data were provided for the special bridge method.

The special bridge method can be used for any bridge, but should be used for bridges with piers where low flow controls, for pressure flow and whenever flow passes through critical depth when going through the structure. The special bridge method computes losses through the structure for low flow, weir flow and pressure flow or for any combination of these.

A series of program capabilities is available to restrict flow to the effective flow areas of cross sections. Among these capabilities are options to simulate sediment deposition, to confine flows to leveed channels, to block out road fills and bridge decks, and to analyze floodplain encroachments.

Cross sections with low overbank areas or levees require special consideration in computing water surface profiles because of possible overflow into areas outside the main channel. Consult the User's Instruction Manual for HEC-2 for more detailed information on using this computer model. HEC-2 is considered legacy software, has been replaced by HEC-RAS since HEC-RAS has more robust algorithms in it than HEC-2. The USACE no longer supports it.

3. WSPRO Modeling. As for HEC-2, the water surface profile should extend upstream and downstream to points beyond the influence of the bridge. The cross sections that are necessary for the energy analysis through the bridge opening for a single opening bridge without guide banks are shown in Figure 10.52. The additional cross sections that are necessary for computing the entire profile are not shown in this figure. Cross sections 1, 3 and 4 are required for a Type I flow analysis and are referred to as the approach section, bridge section and exit section, respectively. In addition, cross section 3F, which is called the full-valley section, is needed for the water surface profile computation without the presence of the bridge contraction. Cross section 2 is used as a control point in Type II flow but requires no input data. Two more cross sections must be defined if a guide bank and roadway profile are specified, one at the upstream end of the guide bank, and one immediately before the defining approach cross section.

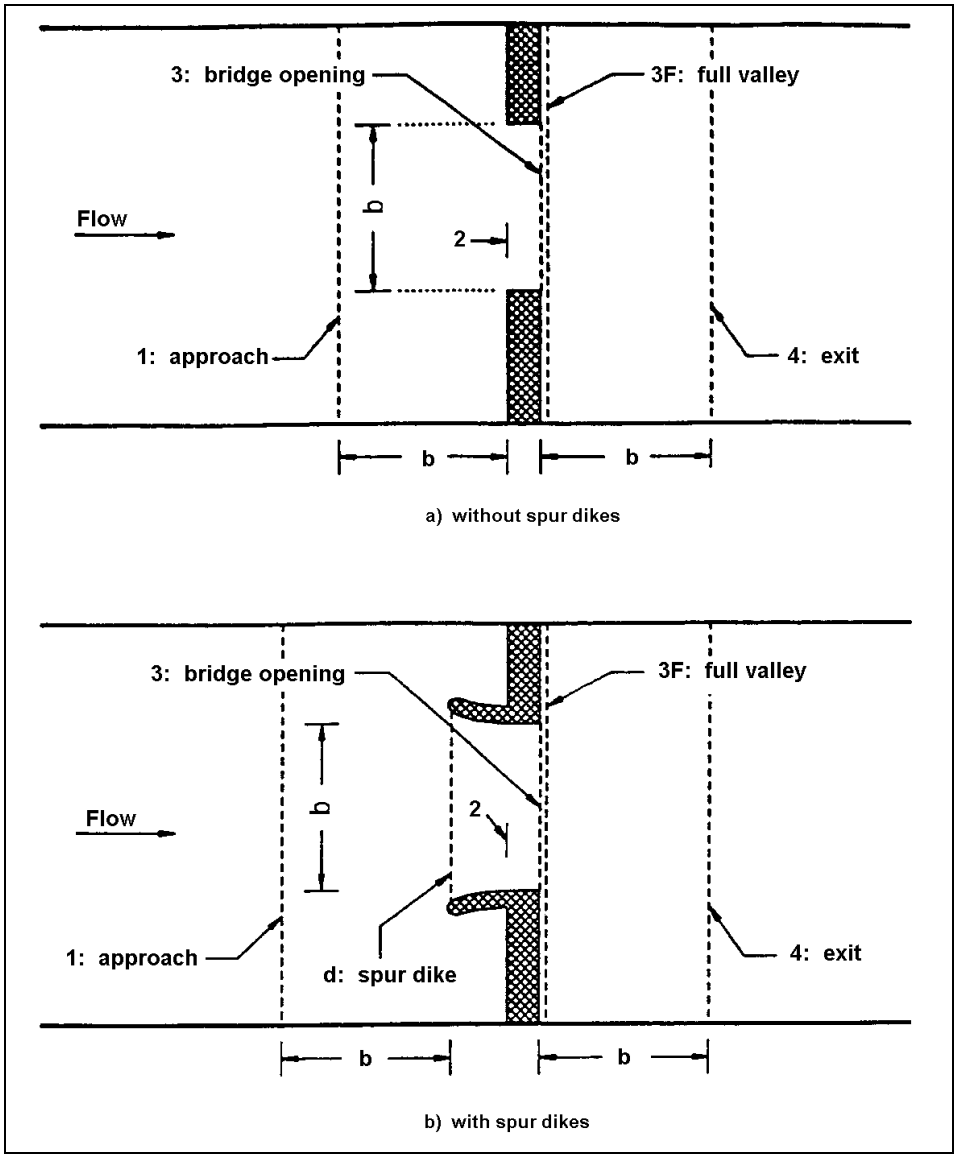
Pressure flow through the bridge opening is assumed to occur when the depth just upstream of the bridge opening exceeds 1.1 times the hydraulic depth of the opening. The flow is then calculated as orifice flow with the discharge proportional to the square root of the effective head. Submerged orifice flow is treated similarly with the head redefined. WSPRO can also simultaneously consider embankment overflow as a weir discharge. This leads to flow Classes 1 through 6 as given in Table 10.2.

In free-surface flow, there is no contact between the water surface and the low-girder elevation of the bridge. In orifice flow, only the upstream girder is submerged; while in submerged orifice flow, both the upstream and downstream girders are submerged. A total of four different bridge types can be treated. WSPRO is no longer supported by the USGS. The user's instruction manual for WSPRO should be used for more detailed information on using the WSPRO computer model.

Table 10.2 Flow Classification According to Submergence Conditions
(WSPRO User's Instruction Manual, 1990)

Flow Through Bridge Opening Only	Flow Through and Over Road Grade
Class 1 - Free surface flow	Class 4 - Free surface flow
Class 2 - Orifice flow	Class 5 - Orifice flow
Class 3 - Submerged orifice flow	Class 6 - Submerged orifice flow

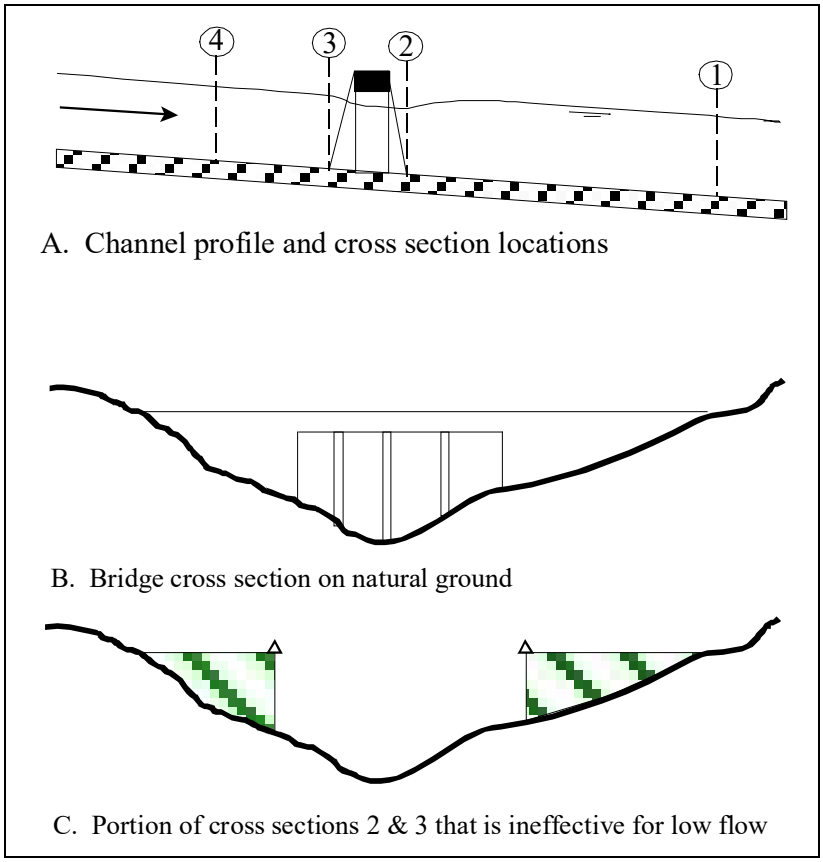
Figure 10.52 WSPRO Cross Section Location for Stream Crossing with a Single Waterway Opening



D. Defining Ineffective Flow Areas. A basic problem in defining the bridge data is the definition of ineffective flow areas near the bridge structure. Referring to Figure 10.50, the dashed lines represent the boundary of effective flow for low flow and pressure flow conditions. Therefore, for cross sections 2 and 3, ineffective flow areas to either side of the bridge opening (along distance AB and CD) should not be included as part of the active flow area for low flow or pressure flow.

The bridge example shown in Figure 10.53 is a typical situation where the bridge spans the entire floodway and its abutments obstruct the natural floodplain. This situation is similar to the one shown in plan view in Figure 10.50.

Figure 10.53 Cross Sections near Bridges



The elimination of the ineffective overbank areas can be accomplished by redefining the geometry at cross sections 2 and 3 or by using the natural ground profile and requesting the program's ineffective area option to eliminate the use of the overbank area (as shown in part C of Figure 10.53). Also, for high flows (flows overtopping the bridge deck), the area outside of the main bridge opening may no longer be ineffective, and will need to be included as active flow area. If the modeler chooses to redefine the cross section, a fixed boundary is used at the sides of the cross section to contain the flow, when in fact a solid boundary is not physically present. The use of the ineffective area option is more appropriate and it does not add wetted perimeter to the active flow boundary above the given ground profile.

The ineffective area option is used at cross sections 2 and 3 to keep all the active flow in the area of the bridge opening until the elevations associated with the left and/or right ineffective flow areas are exceeded by the computed water surface elevation. The program allows the stations and controlling elevations of the left and right ineffective flow areas to be specified by the user. Also, the stations of the ineffective flow areas do not have to coincide with stations of the ground profile; the program will interpolate the ground station. The ineffective flow areas should be set at stations that will adequately describe the active flow area at cross sections 2 and 3. In general, these stations should be placed outside the edges of the bridge opening to allow for the contraction and expansion of flow that occurs in the immediate vicinity of the bridge. On the upstream side of the bridge (cross section 3) the flow is contracting rapidly. A practical method for placing the stations of the ineffective flow areas is to assume a 1:1 contraction rate in the immediate vicinity of the bridge. In other words, if cross section 3 is 3 meters (~10 feet) from the upstream bridge face, the ineffective flow areas should be placed 3 meters (~10 feet) away from each side of the bridge opening. On the downstream side of the bridge (cross section 2), a similar assumption can be applied. The active flow area on the downstream side of the bridge may be less than, equal to, or greater than the width of the bridge opening. As flow converges into the bridge opening, depending on the abruptness of the abutments, the active flow area may constrict to be less than the bridge opening. As the flow passes through and out of the bridge, it begins to expand. Because of this phenomenon, estimating the stationing of the ineffective flow areas at cross section 2 can be very difficult. In general, the user should make the active flow area equal to the width of the bridge

opening or wider (to account for flow expanding), unless the bridge abutments are very abrupt (vertical wall abutments).

The elevations specified for ineffective flow should correspond to elevations where significant weir flow passes over the bridge. For the downstream cross sections, the threshold water surface elevation for weir flow usually is not known on the initial run, so an estimate must be made. An elevation below the minimum top-of-road, such as an average between the low chord and minimum top-of-road, can be used as a first estimate.

Using the ineffective area option to define the ineffective flow areas allows the overbank areas to become effective as soon as the ineffective area elevations are exceeded. The assumption is that under weir flow conditions, the water can generally flow across the whole bridge length and the entire overbank in the vicinity of the bridge would be effectively carrying flow up to and over the bridge. If it is more reasonable to assume only part of the overbank is effective for carrying flow when the bridge is under weir flow, then the overbank n values can be increased to reduce the amount of conveyance in the overbank areas under weir flow conditions.

Cross section 3, just upstream from the bridge, is usually defined in the same manner as cross section 2. In many cases the cross sections are identical. Generally, the only differences are the stations and elevations used for the ineffective area option. For the upstream cross section, the elevation initially should be set to the low point of the top of road. When this is done, the user possibly could obtain a solution where the bridge hydraulics are computing weir flow, but the upstream water surface elevation is lower than the top of road. Both the weir flow and pressure flow equations are based on the energy grade line in the upstream cross section. Once an upstream energy is computed from the bridge hydraulics, the program tries to compute a water surface elevation in the upstream cross section that corresponds to that energy. Occasionally the program may get a water surface that is confined by the ineffective flow areas and lower than the minimum top of road. When this happens, the user should decrease the elevations of the upstream ineffective flow areas in order to get them to turn off. Once they turn off, the computed water surface elevation will be closer to the computed energy gradeline (which is higher than the minimum high chord of the bridge).

Using the ineffective area option in the manner just described for the two cross sections on either side of the bridge provides for a constricted section when all of the flow is going under the bridge. When the water surface is higher than the control elevations used, the entire cross section is used. The program user should check the computed solutions on either side of the bridge section to ensure they are consistent with the type of flow. That is, for low flow or pressure flow solutions, the output should show the effective area restricted to the bridge opening. When the bridge output indicates weir flow, the solution should show that the entire cross section is effective. During overflow situations, the modeler should ensure that the overbank flow around the bridge is consistent with the weir flow.

The maximum effect of the bridge on WSE is expected to occur at cross section 4; this section is sometimes referred to as the approach section. However, in order to determine the extent of the impact, it is recommended that profile computations continue upstream until the water surface does not differ significantly from the estimated pre-construction conditions.

E. Contraction and Expansion Losses. Losses due to contraction and expansion of flow between cross sections are determined during the standard step profile calculations. Manning's equation is used to calculate friction losses, and all other losses are described in terms of a coefficient times the absolute value of the change in velocity head between adjacent cross sections. When the velocity head increases in the downstream direction, a contraction coefficient is used; when the velocity head decreases, an expansion coefficient is used.

As shown in Figure 10.50 the flow contraction occurs between cross sections 4 and 3, while the flow expansion occurs between cross sections 2 and 1. The contraction and expansion coefficients are used to compute energy losses associated with changes in the shape of river cross sections (or effective flow areas). The loss due to expansion of flow usually is larger than the contraction loss, and losses from short abrupt transitions are larger than losses from gradual transitions. Typical values for contraction and expansion coefficients under subcritical flow conditions are shown in Table 10.3.

Table 10.3 Subcritical Flow Contraction and Expansion Coefficients

	Contraction	Expansion
No transition loss computed	0.0	0.0
Graduate transitions	0.1	0.3
Typical bridge sections	0.3	0.5
Abrupt transitions	0.6	0.8

In general, contraction and expansion coefficients for supercritical flow should be lower than subcritical flow. For typical bridges that are under Class C flow conditions (totally supercritical flow), the contraction and expansion coefficients should be around 0.05 and 0.1, respectively. For abrupt bridge transitions under Class C flow, values of 0.1 and 0.2 may be more appropriate.

F. Hydraulic Computations through the Bridge. Flow through a bridge can be classified as low flow (Class A, B, C), low flow and weir flow (with adjustments for submergence on the weir), pressure flow (orifice and sluice gate equations), pressure and weir flow, and highly submerged flows. The determination of which flow type(s) exist(s) can be complicated. For this reason, it is recommended that the bridge modeling be used to analyze flow through a bridge. This section describes in detail how HEC-RAS models each flow type.

1. Low Flow Computations. Low flow exists when the flow going through the bridge opening is open channel flow (water surface below the highest point on the low chord of the bridge opening). For low flow computations, the program first uses the momentum equation to identify the class of flow. This is accomplished by first calculating the momentum at critical depth inside the bridge at the upstream and downstream ends. The end with the higher momentum (and therefore the most constricted section) will be the controlling section in the bridge. If the two sections are identical, the program selects the upstream bridge section as the controlling section. The momentum at critical depth in the controlling section is then compared to the momentum of the flow downstream of the bridge when performing a subcritical profile (upstream of the bridge for a supercritical profile). If the momentum downstream is greater than the critical depth momentum inside the bridge, the class of flow is considered to be completely subcritical (i.e., Class A low flow). If the momentum downstream is less than the momentum at critical depth in the controlling bridge section, then it is assumed that the constriction will cause the flow to pass through critical depth and a hydraulic jump will occur at some distance downstream (i.e., Class B low flow). If the profile is completely supercritical through the bridge, then this is considered Class C low flow.

2. Class A Low Flow. Class A low flow exists when the water surface through the bridge is completely subcritical (i.e., above critical depth). Energy losses through the expansion (cross sections 2 to 1) are calculated as friction losses and expansion losses. Friction losses are based on a weighted friction slope times a weighted reach length between cross sections 1 and 2. The weighted friction slope is based on one of the four available alternatives in the HEC-RAS, with the average-conveyance method being the default. This option is user selectable. The average length used in the calculation is based on a discharge-weighted reach length. Energy losses through the contraction (cross sections 3 to 4) are calculated as friction losses and contraction losses. Friction and contraction losses between cross sections 3 and 4 are calculated in the same way as friction and expansion losses between cross sections 1 and 2.

Four methods are available for computing losses through the bridge (cross sections 2 to 3):

- Energy Equation (Standard Step Method).
- Momentum Balance.
- Yarnell Equation.
- FHWA WSPRO method.

The user can select any or all of these methods to be computed. This allows the modeler to compare the answers from several techniques all in a single execution of the program. If more than one method is selected, the user must choose either a single method as the final solution or direct the program to use the method that computes the greatest energy loss through the bridge at cross section 3 as the final solution. Minimal results

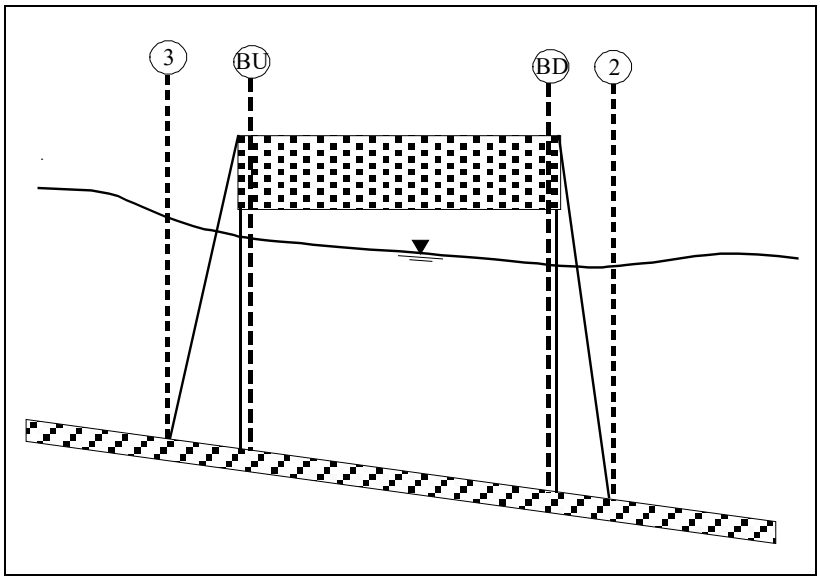
are available for all the methods computed, but detailed results are available for the method that is selected as the final answer. A detailed discussion of each method follows.

(a) Energy Equation (Standard Step Method). The energy based method treats a bridge in the same manner as a natural river cross section, except the area of the bridge structure (i.e., piers and abutments) below the water surface is subtracted from the total area, and the wetted perimeter is increased where the water is in contact with the bridge structure. As described previously, the program formulates two cross sections inside the bridge by combining the ground information of sections 2 and 3 with the bridge geometry. As shown in Figure 10.54, for the purposes of discussion, these cross sections will be referred to as sections BD (Bridge Downstream) and BU (Bridge Upstream).

The sequence of calculations starts with a standard step calculation from just downstream of the bridge (cross section 2) to just inside of the bridge (cross section BD) at the downstream end. The program then performs a standard step through the bridge (from cross section BD to BU). The last calculation is to step out of the bridge (from cross section BU to cross section 3).

The energy based method requires Manning's n values for friction losses and contraction and expansion coefficients for transition losses. The estimate of Manning's n values is well documented in many hydraulics text books, as well as several research studies. Basic guidance for estimating roughness coefficients is provided in Chapter 2 of *RE-45, Hydraulic Reference Manual for HEC-RAS* (USACE, 1995b). Contraction and expansion coefficients are also provided in Chapter 3, as well as parts of Chapter 5 of *RE-45, Hydraulic Reference Manual for HEC-RAS* (USACE, 1995b). Detailed output is available for cross sections inside the bridge (cross sections BD and BU) as well as the user-entered cross sections (2 and 3).

Figure 10.54 Cross Sections Near and Inside the Bridge



(b) Momentum Balance Method. The momentum method is based on performing a momentum balance from cross section 2 to cross section 3. This method can be difficult and time consuming to use by hand, so its use should be restricted to the aid of the computer modeling system HEC-RAS, and its application will not be discussed here. For a detailed discussion of the momentum method, refer to *RE-45, Hydraulic Reference Manual for HEC-RAS* (USACE, 1995b).

(c) Yarnell Equation. The Yarnell equation is an empirical equation that is used to predict the change in water surface from just downstream of the bridge (cross section 2 of Figure 10.50) to just upstream of the bridge (cross section 3). The equation is based on approximately 2600 lab experiments in which the researchers varied the shape of the piers, the width, the length, the angle, and the flow rate. The Yarnell equation is as follows (Yarnell, 1934):

(Equation 10.7)

$$H_{3-2} = 2K\left(K + 10\omega - 0.6\right)\left(\alpha + 15\alpha^4\right)\frac{V_2^2}{2g}$$

where: H_{3-2}= drop in water surface elevation from section 3 to 2
K = Yarnell's pier shape coefficient
ω = ratio of velocity head to depth at section 2
α = obstructed area of the piers divided by the total unobstructed area at cross section 2
V_2 = velocity downstream at cross section 2

The computed upstream water surface elevation (cross section 3) is simply the downstream water surface elevation plus H_{3-2}. With the upstream water surface known, the program computes the corresponding velocity head and energy elevation for the upstream cross section (cross section 3). When the Yarnell method is used, hydraulic information is only provided at cross sections 2 and 3 (no information is provided for sections BU and BD).

The Yarnell equation is sensitive to the pier shape (K coefficient), the pier obstructed area, and the velocity of the water. The method is not sensitive to the shape of the bridge opening, the shape of the abutments, or the width of the bridge. Because of these limitations, the Yarnell method should be used only at bridges where the majority of the energy losses are associated with the piers. When Yarnell's equation is used for computing the change in water surface through the bridge, the user must supply the Yarnell pier shape coefficient, K. Table 10.4 gives values for Yarnell's pier coefficient, K, for various pier shapes.

Table 10.4 Yarnell's Pier Coefficient, K, for Various Pier Shapes

Pier Shape	**Yarnell K Coefficient**
Semi-circular nose and tail	0.90
Twin-cylinder piers with connecting diaphragm	0.95
Twin-cylinder piers without diaphragm	1.05
90 degree triangular nose and tail	1.05
Square nose and tail	1.25
Ten pile trestle bent	2.50

(d) FHWA WSPRO Contracted Opening Method. The low flow hydraulic computations of FHWA's WSPRO computer program has been adapted as an option for low flow hydraulics in HEC-RAS. The WSPRO methodology had to be modified slightly in order to fit into the HEC-RAS concept of cross section locations around and through a bridge.

The WSPRO method computes the water surface profile through a bridge by solving the energy equation. The method is an iterative solution performed from cross section 1 to cross section 4 of Figure 10.50. The energy balance is performed in steps from cross section (1) to the cross section just downstream of the bridge (2), from just downstream of the bridge (2) to inside of the bridge at the downstream end (BD), from inside of the bridge at the downstream end (BD) to inside of the bridge at the upstream end (BU), from inside of the bridge at the upstream end (BU) to just upstream of the bridge (3), and from just upstream of the bridge (3) to the approach cross section (4). A general energy balance equation from the exit section to the approach section can be written as follows:

(Equation 10.8)

$$h_4 + \frac{\alpha_4 V_4^2}{2g} = h_1 + \frac{\alpha_1 V_1^2}{2g} + h_{L(4-1)}$$

where: h_1 = water surface elevation at cross section 1.
V_1 = velocity at cross section 1.

h_4 = water surface elevation at cross section 4.
V_4 = velocity at cross section 4.
$h_{L(4-1)}$ = energy losses from cross section 4 to 1.

The incremental energy losses from cross section 4 to cross section 1 are calculated as follows:

From Section 1 to 2

Losses from cross section 1 to cross section 2 are based on friction losses and an expansion loss. Friction losses are calculated using the geometric mean friction slope times the flow weighted distance between cross sections 1 and 2. The following equation is used for friction losses from cross sections 1 to 2:

(Equation 10.9)

$$h_{f(1-2)} = \frac{BQ^2}{K_2 K_1}$$

where B is the flow weighted distance between cross sections 1 and 2, and K_1 and K_2 are the total conveyance at cross sections 1 and 2, respectively. The expansion loss from cross section 2 to cross section 1 is computed by the following equation:

(Equation 10.10)

$$h_e = \frac{Q^2}{2gA_1^2}\left[2\beta_1 - \alpha_1 - 2\beta_2\left(\frac{A_1}{A_2}\right) + \alpha_2\left(\frac{A_1}{A_2}\right)^2\right]$$

where α_1 and β_1 are energy and momentum correction factors for nonuniform flow. α_1 and β_1 are computed as follows:

(Equation 10.11)

$$\alpha_1 = \frac{\sum\left(K_i^3/A_i^2\right)}{K_T^3/A_T^2}$$

α_1 and β_1 are related to the bridge geometry and are defined as follows:

(Equation 10.12)

$$\beta_1 = \frac{\sum\left(K_i^2/A_i\right)}{K_T^2/A_T}$$

(Equation 10.13)

$$\alpha_1 = \frac{1}{C^2}$$

where C is an empirical discharge coefficient for the bridge, which was originally developed as part of the Contracted Opening method by Kindswater, Carter, and Tracy (USGS, 1953), and subsequently modified by Matthai (USGS, 1968).

(Equation 10.14)

$$\beta_1 = \frac{1}{C}$$

From Section 2 to 3

Losses from cross section 2 to cross section 3 are based on friction losses only. The energy balance is performed in three steps: from cross section 2 to BD, BD to BU, and BU to cross section 3. Friction losses are calculated using the geometric mean friction slope times the flow weighted distance between cross sections. The following equation is used for friction losses from BD to BU:

(Equation 10.15)

$$h_{f(BU-BD)} = \frac{L_B Q^2}{K_{BU} K_{BD}}$$

where K_{BU} and K_{BD} are the total conveyance at cross sections BU and BD, respectively, and L_B is the length through the bridge. Similar equations are used for the friction losses from cross section 2 to BD and BU to cross section 3.

From Section 3 to 4

Energy losses from cross sections 3 to 4 are based on friction losses only. The equation for computing the friction loss is as follows:

(Equation 10.16)

$$h_{f(3-4)} = \frac{L_{av} Q^2}{K_3 K_4}$$

where L_{av} is the effective flow length in the approach reach, and K_3 and K_4 are the total conveyances at cross sections 3 and 4. The effective flow length is computed as the average length of 20 equal conveyance stream tubes (WSPRO, 1986).

3. Class B Low Flow. Class B low flow can exist for either subcritical or supercritical profiles. For either profile, Class B flow occurs when the profile passes through critical depth in the bridge constriction. For a subcritical profile, the momentum equation is used to compute an upstream water surface (cross section 3 of Figure 10.50) above critical depth and a downstream water surface (cross section 2) below critical depth. For a supercritical profile, the bridge is acting as a control and is causing the upstream water surface elevation to be above critical depth. Momentum is used to calculate an upstream water surface above critical depth and a downstream water surface below critical depth. If for some reason the momentum equation fails to converge on an answer during the Class B flow computations, the program will automatically switch to an energy-based method for calculating the Class B profile through the bridge.

Whenever Class B flow is found to exist, the user should run the program in a mixed flow regime mode. If the user is running a mixed flow regime profile, the program will proceed with backwater calculations upstream, and later with forewater calculations downstream from the bridge. Also, any hydraulic jumps that may occur upstream and downstream of the bridge can be located if they exist.

4. Class C Low Flow. Class C low flow exists when the water surface through the bridge is completely supercritical. The program can use either the energy equation or the momentum equation to compute the water surface through the bridge for this class of flow. A summary of the low flow classes is presented in Table 10.5.

Table 10.5 Low Flow Classes

Type Designation	Description
I or Class A	Subcritical flow through Zones 1, 2 and 3
IIA or Class B	Subcritical flow Zones 1 and 3, flow through critical depth Zone 2
IIB or Class B	Subcritical Zone 3, flow through critical Zone 2, hydraulic jump Zone 1
III or Class C	Supercritical flow through Zones 1, 2 and 3

5. High Flow. High flow refers to conditions in which the water surface impinges on the bridge superstructure. When the tailwater does not submerge the low chord of the bridge, the flow condition is comparable to a pressure flow sluice gate. At tailwaters which submerge the low chord but do not exceed the elevation of critical depth over the road, the flow condition is comparable to orifice flow. If the tailwater

exceeds critical depth over the road, neither sluice gate flow nor orifice flow are reasonable. Head losses under such high tailwater conditions usually are estimated as friction and turbulence losses using the energy balance method.

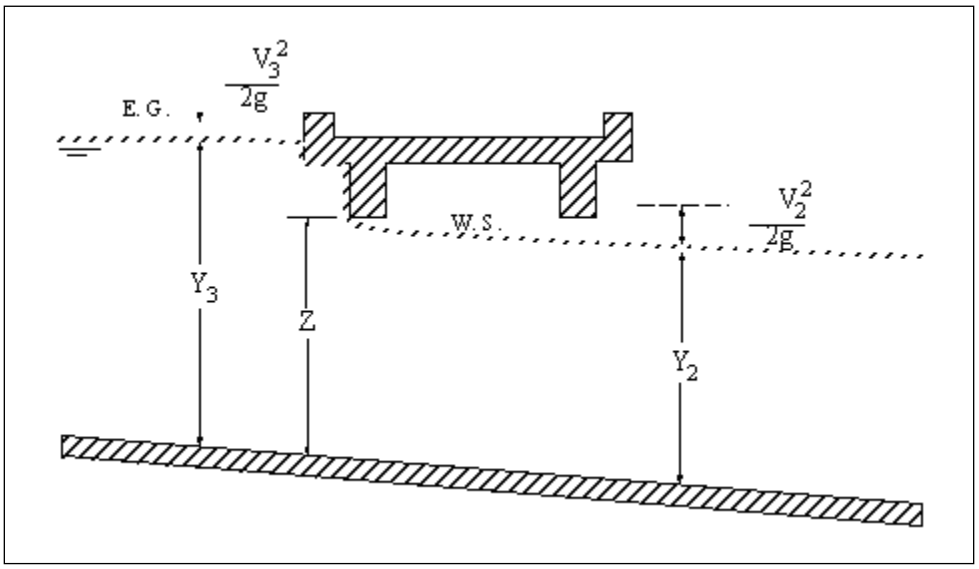
The HEC-RAS program has the ability to compute high flows (flows that come into contact with the maximum low chord of the bridge deck) by either the Energy equation (standard step method) or by using separate hydraulic equations for pressure and/or weir flow. The two methodologies are explained below.

(a) Energy Equation (Standard Step Method). The energy-based method is applied to high flows in the same manner as it is applied to low flows. Computations are based on balancing the energy equation in three steps through the bridge. Energy losses are based on friction, contraction, and expansion losses. Output from this method is available at the cross sections inside the bridge as well as outside.

As mentioned previously, friction losses are based on the use of Manning's equation. Guidance for selecting Manning's n values is provided in Chapter 8, *Open Channels*. Contraction and expansion losses are based on a coefficient multiplied by the difference in velocity head between adjacent cross sections.

The energy-based method performs all computations as though they are open channel flow. At the cross sections inside the bridge, the area obstructed by the bridge piers, abutments, and deck is subtracted from the total flow area and additional wetted perimeter is added. Occasionally the Hydraulic Grade Line (HGL) inside the bridge (between sections BU and BD) are elevations that would be inside of the bridge deck. The HGL shown as water surface by HEC-RAS inside of the bridge reflects the hydraulic grade line elevations, not necessarily the actual water surface elevations. Additionally, the active flow area is limited to the open bridge area.

Figure 10.55 Example of a Bridge under a Sluice Gate Type of Pressure Flow



(b) Pressure and Weir Flow Method. A second approach for the computation of high flows is to utilize separate hydraulic equations to compute the flow as pressure and/or weir flow. These two types of flow are presented below.

(1) Pressure Flow Computations. Pressure flow occurs when the flow comes into contact with the entire low chord of the bridge. Once the flow contacts the full length of the low chord on the upstream side of the bridge, a backwater occurs and orifice flow is established. HEC-RAS will handle two cases of orifice flow; the first is when only the upstream side of the bridge is in contact with the water, and the second is when the bridge opening is flowing completely full. The HEC-RAS program will automatically select the appropriate equation, depending upon the flow situation. For the first case (see Figure 10.55), a sluice gate type of equation is used (FHWA, 1978):

(Equation 10.17)

$$Q = C_d A_{BU} \left[2g \left(Y_3 - \frac{Z}{2} + \frac{\alpha^3 V_3^2}{2g} \right) \right]^{\frac{1}{2}}$$

where: Q = total discharge through the bridge opening
C_d = coefficient of discharge for pressure flow
A_{BU} = net area of the bridge opening at section BU
Y_3 = hydraulic depth at section 3
Z = vertical distance from maximum bridge low chord to the mean river bed elevation at section BU

The discharge coefficient C_d, can vary depending upon the depth of water upstream. Values for C_d range from 0.27 to 0.5, with a typical value of 0.5 commonly used in practice. The user can enter a fixed value for this coefficient or the program will compute one based on the amount that the inlet is submerged. A diagram relating C_d to Y_3/Z is shown in Figure 10.56. As shown in Figure 10.56, the limiting value of Y_3/Z is 1.1. There is a transition zone somewhere between Y_3/Z = 1.0 and 1.1 where free surface flow changes to orifice flow. The type of flow in this range is unpredictable, and Equation 10.17 is not applicable.

In the second case, when both the upstream and downstream side of the bridge are submerged, the standard full flowing orifice equation is used (see Figure 10.57). This equation is as follows:

(Equation 10.18)

$$Q = CA\sqrt{2gH}$$

where: C = coefficient of discharge for fully submerged pressure flow; typical value of C is 0.8.
H = the difference between the energy grade elevation upstream and the water surface elevation downstream
A = net area of the bridge opening

Typical values for the full-flow discharge coefficient C range from 0.7 to 0.9, with a value of 0.8 commonly used for most bridges. The user must enter a value for C whenever the pressure flow method is selected.

Figure 10.56 Coefficient of Discharge for Sluice Gate Type Flow

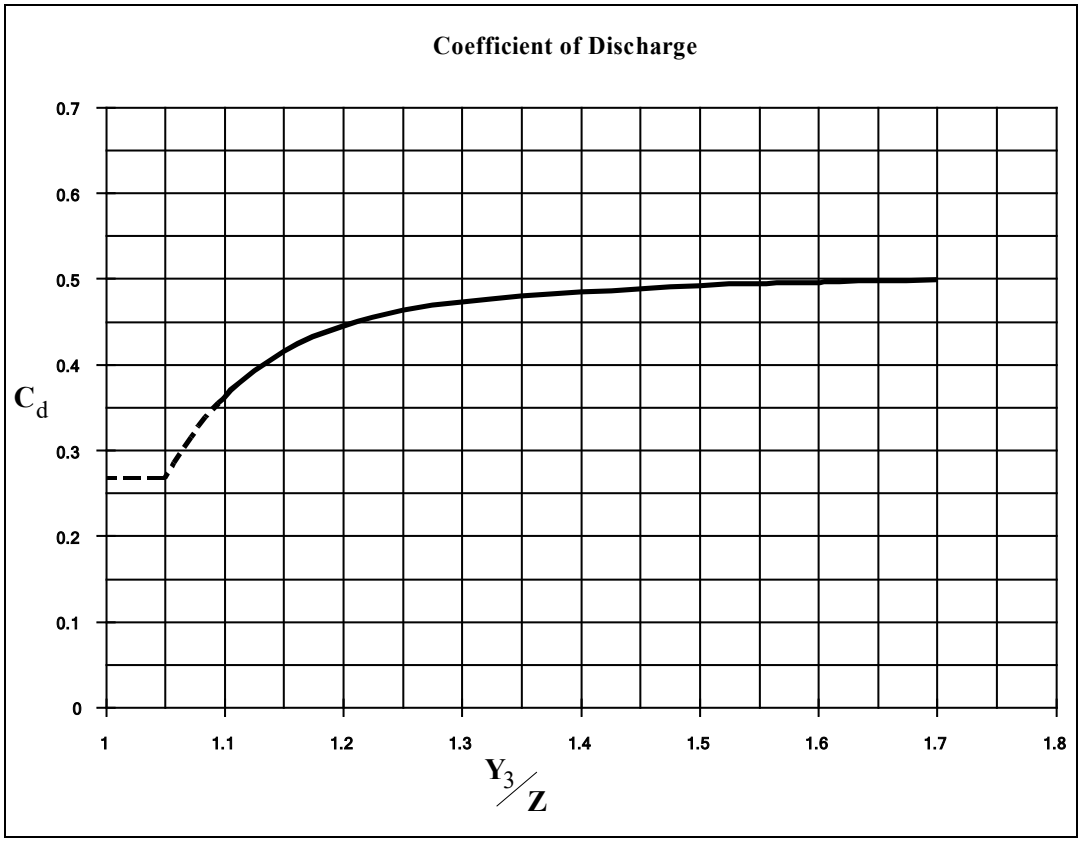
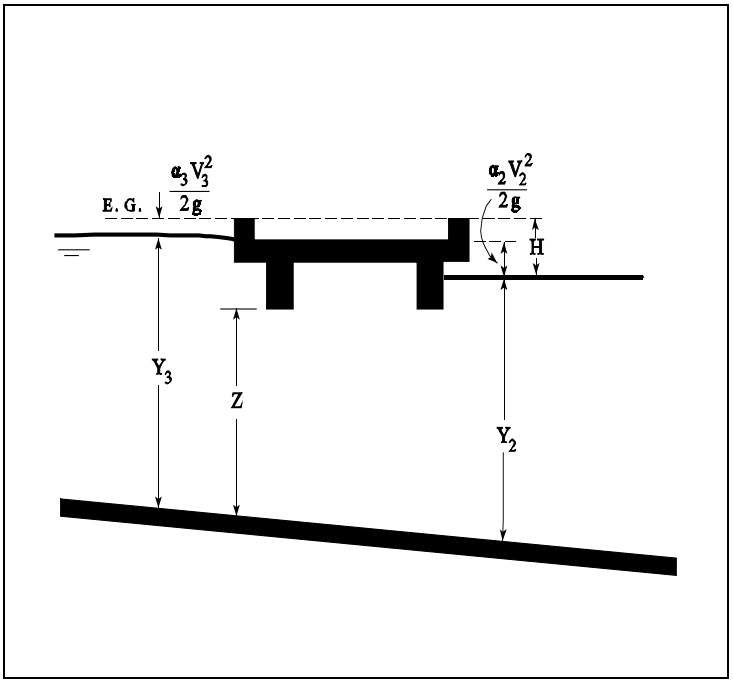


Figure 10.57 Example of a Bridge under Fully Submerged Pressure Flow



The discharge coefficient C can be related to the total loss coefficient, which comes from the form of the orifice equation that is used in the HEC-2 computer program:

(Equation 10.19)

$$Q = A\sqrt{\frac{2gH}{K}}$$

where K = total loss coefficient. The conversion from K to C is as follows:

(Equation 10.20)

$$C = \sqrt{\frac{1}{K}}$$

The program will begin checking for the possibility of pressure flow when the computed low flow energy grade line is above the maximum low chord elevation at the upstream side of the bridge. Once pressure flow is computed, the pressure flow answer is compared to the low flow answer, and the higher of the two is used. The user has the option to tell the program to use the water surface, instead of energy, to trigger the pressure flow calculation.

(2) Weir Flow Computations. Flow over the bridge, and the roadway approaching the bridge, is calculated using the standard weir equation (see Figure 10.20):

(Equation 10.21)

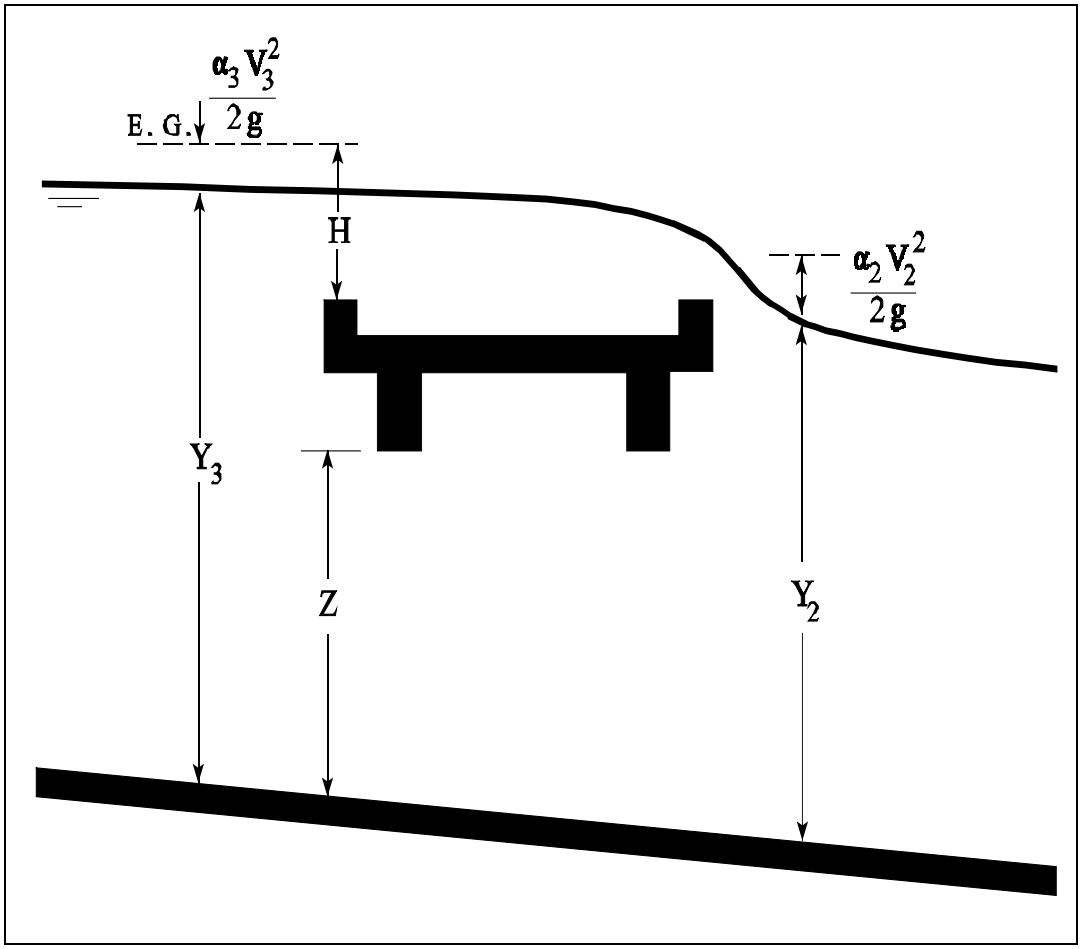
$$Q = CLH^{3/2}$$

where: Q = total flow over the weir
C = coefficient of discharge for weir flow
L = effective length of the weir
H = difference between energy upstream and road crest

The approach velocity is included by using the energy grade line elevation in lieu of the upstream water surface elevation for computing the head, H.

Under free flow conditions (discharge independent of tailwater) the coefficient of discharge C, ranges from 2.5 to 3.1 (1.4 - 1.7 for metric units) for broad crested weirs depending primarily upon the gross head on the crest (C increases with head). Increased resistance to flow caused by obstructions such as trash on bridge railings, curbs, and other barriers would decrease the value of C.

Figure 10.58 Example Bridge with Pressure Flow and Weir Flow



Tables of weir coefficients, C, are given for broad-crested weirs in King's Handbook (King, 1963), with the value of C varying with measured head H and breadth of weir. For rectangular weirs with a breadth of 4.6 meters (~15 feet) and an H of 0.3 meters (~1 foot) or more, the given value is 1.45 (2.63 for U.S. Customary Units). Trapezoidal weirs generally have a larger coefficient with typical values ranging from 1.5 to 1.7 (2.7 to 3.08 for U.S. Customary Units).

Hydraulics of Bridge Waterways (Bradley, 1978) provides a curve of C versus the head on the roadway. The roadway section is shown as a trapezoid and the coefficient rapidly changes from 1.5 for a very small H to 1.07 for H = 0.2 meters (2.9 for a very small H to 3.03 for H ≈ 0.6 ft). From there, the curve levels off near a value of 1.69 (3.05 for U.S. Customary).

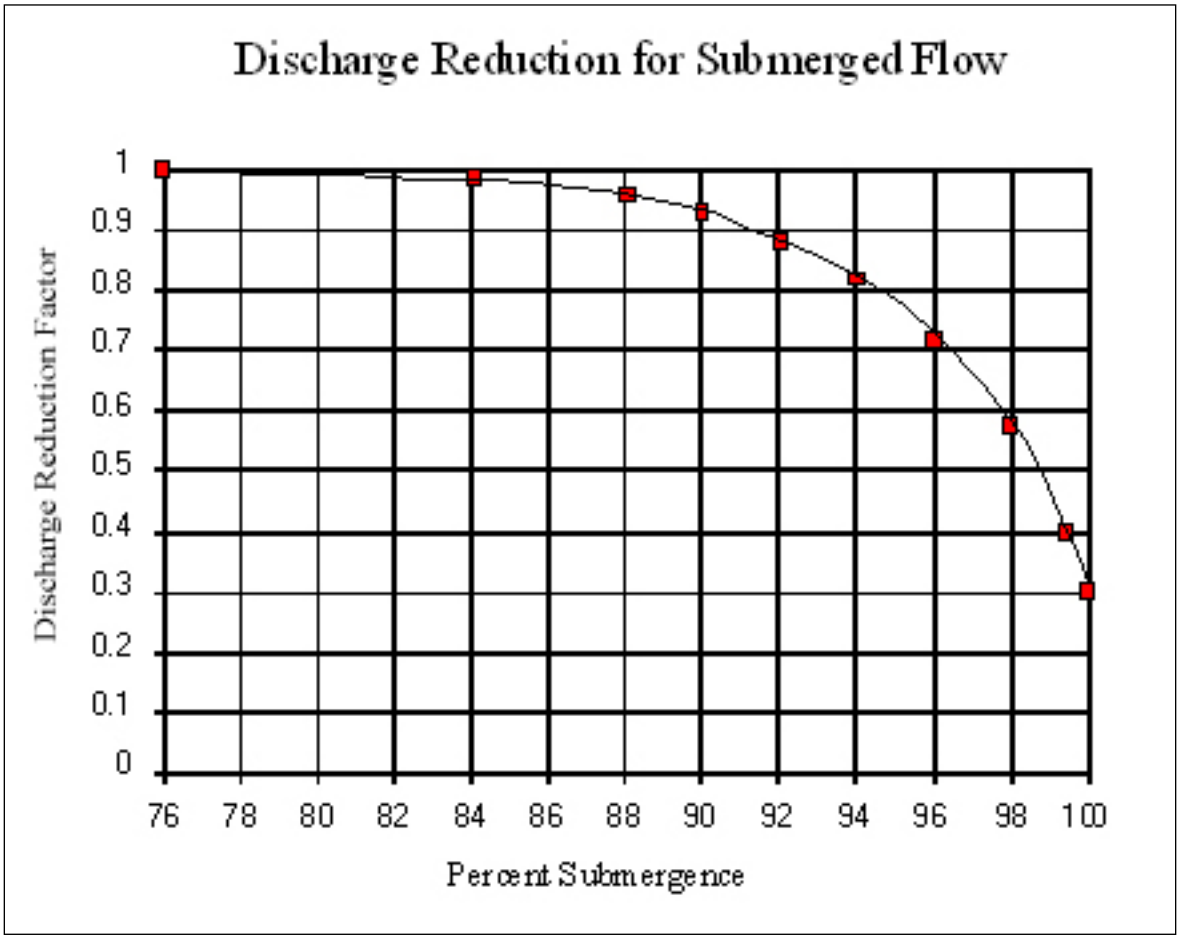
According to the data available, for application of weir flow computations the assumption of a rectangular weir for flow over the bridge deck (assuming the bridge can withstand the forces) and a coefficient of 1.44 (2.6 for U.S. Customary Units) is acceptable. If the weir flow is over the roadway approaches to the bridge, a value of 1.66 (3.0 for U.S. Customary) should be used. If weir flow occurs as a combination of bridge and roadway overflow, then an average coefficient (weighted by weir length) should be used.

For high tailwater elevations, the HEC-RAS program automatically reduces the amount of weir flow to account for submergence on the weir. Submergence is defined as the depth of water above the minimum weir elevation on the downstream side (cross section 2) divided by the height of the energy gradeline above the minimum weir elevation on the upstream side (cross section 3). The reduction of weir flow is accomplished by reducing the weir coefficient based on the amount of submergence. Submergence corrections are based on a trapezoidal weir shape or optionally an ogee

spillway shape. The total weir flow is computed by subdividing the weir crest into segments; computing L, H, a submergence correction, and a Q for each section; then summing the incremental discharges. The submergence correction for a trapezoidal weir shape is from *Hydraulics of Bridge Waterways* (Bradley, 1978). Figure 10.59 shows the relationship between the percentage of submergence and the flow reduction factor.

When the weir becomes highly submerged, the HEC-RAS program automatically switches to calculating the upstream water surface by the energy equation (standard step backwater) instead of using the pressure and weir flow equations. The criteria for when the program switches to energy based calculations is user controllable. A default maximum submergence is set to 0.95 (95%).

Figure 10.59 Factor for Reducing Weir Flow for Submergence



6. Combination Flow. Sometimes combinations of low flow or pressure flow occur with weir flow. In these cases, an iterative procedure is used to determine the amount of each type of flow. The HEC-RAS program continues to iterate until both the low flow method (or pressure flow) and the weir flow method have the same energy (within a specified tolerance) upstream of the bridge (cross section 3, Figure 10.50). The combination of low flow and weir flow can only be computed with the energy and Yarnell low flow methods.

The HEC-RAS program will begin checking for the possibility of pressure flow when the computed low flow energy grade line is above the maximum low chord elevation at the upstream side of the bridge. Once pressure flow is computed, the pressure flow answer is compared to the low flow answer and the higher Q value of the two is used.

The user has the option to direct the program to use the water surface, instead of energy, to trigger the pressure flow calculation.

G. Selecting a Bridge Modeling Approach. Several choices are available to the user when selecting methods for computing the water surface profile through a bridge. For low flow (water surface is below the maximum low chord of the bridge deck), the user can select any or all of the four available methods. For high flows, the user must choose between either the energy based method or the pressure and weir flow approach. The choice of methods should be considered carefully. The following discussion provides some basic guidelines on selecting the appropriate methods for various situations.

1. Low Flow Methods. For low flow conditions (water surface below the highest point on the low chord of the bridge opening), the energy and momentum methods are the most physically based, and in general are applicable to the widest range of bridges and flow situations. Both methods account for friction losses and changes in geometry through the bridge. The energy method accounts for additional losses due to flow transitions and turbulence through the use of contraction and expansion losses. The momentum method can account for additional losses due to pier drag. The FHWA WSPRO method originally was developed for bridge crossings that constrict wide flood plains with heavily vegetated overbank areas. The method is an energy-based solution with some empirical attributes (the expansion loss equation in the WSPRO method utilizes an empirical discharge coefficient). The Yarnell equation is an empirical formula. The following examples are some typical cases where the various low flow methods might be used:

- In cases where the bridge piers are a small obstruction to the flow, and friction losses are the predominate consideration, the energy-based method, the momentum method, and the WSPRO method should give the best answers.
- In cases where pier losses and friction losses are both predominant, the momentum method should be the most applicable. But any of the methods can be used.
- Whenever the flow passes through critical depth within the vicinity of the bridge, both the momentum and energy methods are capable of modeling this type of flow transition. The Yarnell and WSPRO methods are for subcritical flow only.
- For supercritical flow, both the energy and the momentum method can be used. The momentum based method may be better at locations that have a substantial amount of pier impact and drag losses. The Yarnell equation and the WSPRO method are applicable only to subcritical flow situations.
- For bridges in which the piers are the dominant contributor to energy losses and the change in water surface, either the momentum method or the Yarnell equation would be most applicable. However, the Yarnell equation is only applicable to Class A low flow.
- For long culverts under low flow conditions, the energy based standard step method is the most suitable approach. Several sections can be taken through the culvert to model changes in grade or shape or to model a very long culvert. This approach also has the benefit of providing detailed answers at several locations within the culvert, which is not possible with the culvert routines in HEC-RAS. However, if the culvert flows full, or if it is controlled by inlet conditions, the culvert routines would be the best approach. For a detailed discussion of the culvert routines within HEC-RAS, see Chapter 6 of the *Hydraulic Reference Manual*.

2. High Flow Methods. For high flows (flows that come into contact with the maximum low chord of the bridge deck), the energy-based method is applicable to the widest range of problems. The following examples are some typical cases where the various high flow methods might be used:

- When the bridge deck is a small obstruction to the flow, and the bridge opening is not acting like a pressurized orifice, the energy-based method should be used.
- When the bridge deck and road embankment are a large obstruction to the flow, and a backwater is created due to the constriction of the flow, the pressure and weir method should be used.
- When the bridge and/or road embankment is overtopped, and the water going over top of the bridge is <u>not</u> highly submerged by the downstream tailwater, the pressure and weir method should be used. The pressure and weir method will automatically switch to the energy method if the bridge becomes 95% submerged. The user can change the percent submergence at which the program switches from the pressure and weir method to the energy method. This is accomplished from the Deck/Roadway editor in the Bridge/Culvert Data editor.
- When the bridge is highly submerged, and flow over the road is not acting like weir flow, the energy-based method should be used.

10.7 SINGLE OPENING DESIGN

A. Single Opening Design Approach. The term "single opening" refers to a stream crossing requiring only one opening (of one or more spans) in the highway embankment.

The steps discussed in this section are described to provide the designer with a means of establishing an initial size of opening. The effect of the trial opening must then be analyzed using the designer's selection of methods outlined in Section 10.6. If the resulting backwater or the through bridge velocities are unacceptable, the designer must modify the opening until the estimated conditions are satisfactory for both the design and check flood conditions. The Department recommends the use of automated procedures for such analyses.

B. Recommended Procedure for Single Opening Design. The recommended procedure for establishing a single structure length and elevation of the low chord begins with estimating the design flood, obtaining accurate controlling cross sections, and determining the design and check flood water surface profiles (Chapter 7, *Hydrology*, and Chapter 8, *Open Channels*). Also useful, and sometimes necessary for complete documentation, is a compilation of past flood history, existing structures, and other characteristics of the highway crossing of the stream.

1. As a trial, assume an average through-bridge velocity (v_t) that is less than the maximum allowable velocity but which is not lower than the unconstricted average velocity.

2. Apply the unconstricted design water surface elevation to the section and find the cross sectional area (A_t) subtended by this water surface that will satisfy the Continuity Equation for trial velocity and design discharge.

(Equation 10.22)

$$A_t = \frac{Q}{v_t}$$

where: A_t = submerged cross sectional area, m^2 (ft^2)
Q = design discharge, m^3/s (cfs)
v_t = trial velocity, m/s (ft/s)

3. By inspection of the section, estimate an average depth of water (D_t) in the cross section where the bridge is to be located.

4. Find the trial length (L_t) of the bridge using Equation 10.23.

(Equation 10.23)

$$L_t = \frac{A_t}{D_t}$$

5. Position the abutments in the stream cross section (same cross section as in Step 3) so that they are approximately L_t apart and at locations which appear to maximize the through-bridge area.

6. Find the waterway area (A_w) below the design high-water within the structure limits.

7. Use the Continuity Equation to find the average through-bridge velocity (v_b) for the actual waterway area (A_w).

(Equation 10.24)

$$v_b = \frac{Q}{A_w}$$

8. If v_b is close to the target average velocity, the initial bridge length may be reasonable. (The allowable maximum velocity should be evaluated and established based on individual site characteristics.) This length usually must be adjusted slightly to fit standard span length requirements.

If v_b is much lower or greater than the allowable maximum velocity, the length should be adjusted as necessary, repeating Steps 6 and 7. This routine should be repeated until the average through-bridge velocity is close to the target velocity. To minimize the cost of the structure, it usually is desirable to adjust the bridge length so that the design velocity is at or very near the maximum allowable velocity.

9. Establish a low chord (as discussed in Section 10.4.G. on Freeboard).

10. Estimate the backwater caused by the constriction of the bridge opening using the procedures outlined in Section 10.6. The bridge length may need to be adjusted to ensure that the backwater effects are not excessive.

11. Determine the maximum scour potential envelope as discussed in Section 10.9.

10.8 MULTIPLE OPENING DESIGN

A. Multiple Opening Design Approach. In situations where a bridge crosses a relatively wide flood plain with multiple discharge concentrations, it may be necessary to design multiple openings. A multiple opening configuration usually consists of a main channel bridge with relief openings. This type of crossing provides openings at or near the flow concentrations. The result is a reduction in along-embankment flow and backwater effects.

This type of problem can be modeled in two ways within HEC-RAS. The preferred method is to use the multiple opening capability in HEC-RAS, which is discussed in detail in the HEC-RAS Reference Manual. A second method is to model the two openings as divided flow. This method would require the user to define the flow path for each opening as a separate reach.

Refer to *RE-45, Hydraulic Reference Manual for HEC-RAS* (USACE, 1995b) for more information on analysis of multiple openings.

10.9 BRIDGE SCOUR

A. Scour Components. Scour is the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams. Potential scour can be a significant factor in the analysis of a stream crossing system. The design of a crossing system involves an acceptable balance between a waterway opening that will not create undue damage by backwater or suffer undue damage from scour and a crossing profile sufficiently high to provide the required traffic service.

For simplicity, scour is considered to consist of three components:

- Long-term aggradation and degradation.
- Contraction scour.
- Local scour.

Long-term aggradation and degradation is also referred to as natural scour as discussed in Section 10.5.B.5.

For a comprehensive evaluation of scour which includes defining the various types of scour and equations and computational methodologies used to quantify it, refer to HEC-18, *Evaluating Scour at Bridges* (FHWA, 2001). The remainder of this section provides an introduction to the concepts associated with scour.

B. Rates of Scour. The rate of scour in different materials and under different flow conditions depends on the following factors:

- Erosive power in the flow.
- Erosion resistance of the material.
- A balance between sediment transported into and out of a section.

In erosion resistant materials, equilibrium may not be reached in any one flood event but may develop over a long series of events.

Generally, the methods currently available do not specifically accommodate cohesive bed materials, nor is time dependency considered. Therefore, the results of any scour calculations should be considered only as an indication of the maximum potential scour. Judgment based on experience must be exercised to decide whether or not calculated depths are likely for the given site conditions and life expectancy of the bridge.

C. Requirements for Scour Analysis. Design projects involving new, rehabilitated, widened, and existing bridges over waterways should include estimates of the potential scour envelope using velocities and flow depths resulting from Q_{100}, Q_{500}, flood of record, and Q-overtopping. In some cases, other events could produce worse scour. Such events with return periods not exceeding 500 years should be considered. In all cases, subsequent foundation design is based on the worst case scour. The designer should refer to HEC-18, *Evaluating Scour at Bridges* (FHWA, 2001) for a detailed discussion of analytical procedures.

D. Aggradation and Degradation. The depth and area of general scour at a waterway opening may be affected by any or all of the following factors:

- Slope, natural alignment, and shifting of the channel.
- Type and amount of bed material in transport.
- Nature and occurrence of flood events.
- Accumulations of debris.
- Constriction or realignment of flow due to the stream crossing.
- Layout and geometry of hydraulic structure works.
- Geometry and orientation of piers.
- Classification, stratification, and consolidation of bed and sub-bed materials.
- Placement or loss of riprap and other protective materials.
- Natural or man-made changes in flow or sediment regimes.
- Failures such as collapse of a nearby structure.

It is important to consider the potential for long-term aggradation and degradation. Generally, projections based on evaluation of the history of the site, or ones similar to the site, may suffice. Sometimes, estimation of long-term aggradation and degradation may require a qualitative determination based on general geomorphic and river mechanics relationships, an engineering geomorphic analysis based on qualitative and quantitative relationships to estimate the probable behavior of the stream system to various scenarios or future conditions, and mathematical models such as BRI-STARS and the USACE HEC-6 to make predictions of quantitative changes in streambed elevations due to changes in the stream and watershed.

E. Contraction Scour. Contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction or by a bridge. From continuity, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction and more bed material is removed from the contracted reach than is transported into the contraction. This increase in transport of bed material from the reach lowers the natural bed elevation. As the bed elevation is lowered, the flow area increases, and the velocity and shear stress decrease until relative equilibrium is reached, i.e., the quantity of bed material that is transported into the reach is equal to that removed from the reach.

Contraction scour is typically cyclic. That is, the bed scours during the rising stage of the runoff event, and fills on the falling stage. The contraction of flow due to a bridge can be caused either by a natural decrease in flow area of the stream channel, by abutments projecting into the channel, or by the piers blocking a large portion of the flow area. Contraction also can be caused by the approaches to a bridge cutting off the flood plain flow. This can cause clear-water scour on the setback portion of a bridge section (the portion of the bridge located in the floodplain) or within relief bridges because the flood plain flow normally does not transport significant concentrations of bed material sediments.

Depending on the stream flow, contraction scour can be either live-bed or clear-water. Live-bed scour occurs when the bed material upstream of the constriction is in motion. The scour that results at the constriction is reflective of an equilibrium condition between the sediment transported into the section and that transported away from the

section. Under live-bed conditions, scour holes created during the rising stages of a flood often are refilled during the recession stages.

Clear-water scour occurs when the bed material is not in motion outside the contracted section. The sediment transported into the contracted section is essentially zero. Clear-water scour occurs when the shear stress induced by the water flow exceeds the critical shear stress of the bed material. Generally, when clear-water scour occurs, there is no refilling during the recession of the flood due to the lack of sediment supply. During the initial stages of a flood, clearwater scour could occur then be followed by live-bed scour at higher flood stages.

Typical clear-water scour situations include the following:

- Streams with coarse bed material.
- Flat gradient streams during low flow.
- Local deposits of bed materials that are larger than the biggest fraction being transported by the flow (rock riprap is a special case of this situation).
- Armored stream beds in which tractive forces are large enough to penetrate the armor only at the piers and abutments.
- Vegetated channels in which the tractive forces are large enough to penetrate the cover only at piers and abutments.

F. Pier Scour. Either live-bed or clear-water scour may occur at pier locations. The upstream part of a local scour hole tends to have the shape of a truncated cone with the cone angle approximating the angle of repose of the sediment in water. Downstream slopes are flatter where the flow mixes with other flow, and a bar is formed downstream of the hole. The lateral extent of the scour hole can be determined from the wetted angle of repose of the material and the depth of scour.

G. Abutment Scour. Several abutment scour equations currently exist and are presented in HEC-18, *Evaluating Scour at Bridges, Fourth Edition* (FHWA, 2001b). However, none of the equations presented to date gives consistently acceptable results. Generally, they give conservative estimates even for low Froude numbers. Abutments should be protected to reduce the potential for scour failure by the use of properly designed mitigative measures according to the guidance in Publication 15M, Design Manual, Part 4, *Structures*.

H. Total Scour Envelope. In reality, a total scour envelope at any given section is the result of a complex interaction of flow, sediment transport, bed material, and time. Currently, the procedures available assume that components of scour (long-term degradation, contraction, and local scour) act independently of each other and are ultimate depths for non-cohesive bed materials. The total scour envelope, then, is considered to be the summation of the individual components at the appropriate locations. All components of scour are considered to be equal to or greater than zero. Any negative scour depths are set to zero. Without better methods, the assumption is that the natural degradation and contraction scour depths occur evenly across the portion of the cross section for which they were estimated. Where local scour is considered to occur (at piers and abutments), the total scour is assumed to be the sum of natural degradation, contraction scour, and local scour.

I. Pressure Flow. Refer to HEC-18, *Evaluating Scour at Bridges, Fourth Edition* (FHWA, 2001b) for the computation of scour in pressure flow situations.

J. Other Scour Considerations.

1. Borrow. Borrow pits, either upstream or downstream of a highway-stream crossing, can cause scour at the bridge. Scour occurs upstream of the borrow because of the increased gradient of the stream bed. The bed load of the stream will be deposited in the borrow area and scour occurs downstream as the stream regains its bed load.

If there is any concern about the effects of borrow from a stream, sediment transport models such as BRISTARS or HEC-6 should be used.

2. In-Stream Mining and Dredging. In-stream mining for aggregates and dredging for navigation and flood control can be extremely damaging in cases where so much material is removed from the stream bed that all of the incoming sediment supply is trapped and degradation of long reaches occurs.

At some locations where dredging may be necessary, measures to stabilize the stream bed elevation and the stream bank may be required. Otherwise, the pier and abutment foundations must be set below the expected future elevation due to dredging of the stream bed.

3. Armoring. Armoring occurs because a stream or river is unable, during a particular flood, to move the more coarse material comprising either the bed or, if some bed scour occurs, its underlying material. Scour may occur initially but later become checked 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 formulas developed for sand or other fine material channels for a particular flood magnitude. When a larger flood occurs than the flood used to define the probable scour hole depths, scour will probably penetrate deeper until armoring again occurs at some lower threshold.

Armoring also may cause bank widening. Bank widening encourages rivers or streams to seek a more unstable, braided regime. Such instabilities may pose serious problems for bridges since they encourage 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.

4. 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 eroded 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.

5. Scour Analysis Methods. Before the various scour forecasting methods for contraction and local scour can be applied, it first is necessary to (l) obtain the fixed bed channel hydraulics, (2) estimate the profile and plan form scour or aggradation, (3) adjust the fixed bed hydraulics to reflect these changes and (4) compute the bridge hydraulics. Two methods are provided in this chapter for combining the contraction and local scour components to obtain total scour. The first method, identified as Method 1, has application when armoring is not a concern or insufficient information is available to permit its evaluation, or where more precise scour estimates are not deemed necessary. The second method, Method 2, can be used when stream bed armoring is of concern, more precise contraction scour estimates are deemed necessary, or deposition is expected and is a primary concern.

(a) Method 1. This is considered to be a conservative practice since it assumes that the scour components develop independently. The potential local scour to be calculated using this method would be added to the contraction scour without considering the effects of contraction scour on the channel and bridge hydraulics. The general approach with this method is as follows:

- Estimate the natural channel's hydraulics for a fixed bed condition based on existing conditions.
- Assess the expected profile and plan form changes.
- Adjust the fixed bed hydraulics to reflect any expected profile or plan form changes.
- Estimate contraction scour using the empirical contraction formula and the adjusted fixed bed hydraulics assuming no bed armoring. If the reach is expanding, contraction scour is not expected.
- Estimate local scour using the adjusted fixed bed channel and bridge hydraulics assuming no bed armoring.
- Add the local scour to the contraction scour to obtain the total scour.

(b) Method 2. This analysis method is based on the premise that the contraction and local scour components do not develop independently. As such, the local scour estimated with this method is determined based on the expected changes in the hydraulic variables and parameters due to contraction

scour or deposition; i.e., through what may prove to be an iterative process, the contraction scour and channel hydraulics are brought into balance before these hydraulics are used to compute local scour. Additionally, with this method the effects of any armoring may be considered. The general approach for this method is as follows:

- Estimate the natural channel's hydraulics for a fixed bed condition based on existing site conditions.
- Estimate the expected profile and plan form changes based on the procedures in the *Model Drainage Manual* and any historic data.
- Adjust the natural channel's hydraulics based on the expected profile and plan form changes.
- Select a trial bridge opening and compute the bridge hydraulics.
- Estimate contraction scour.
- Revise the natural channel's geometry to reflect these contraction scour or deposition changes and then again revise the channel's hydraulics (repeat this iteration until there is no significant change in either the revised channel hydraulics or bed elevation changes -- a significant change would be a 5% or greater variation in velocity, flow depth or bed elevation).
- Using the foregoing revised bridge and channel hydraulic variables and parameters obtained considering the contraction scour the local scour.
- Extend the local scour assessment below the predicted contractions scour depths in order to obtain the total scour.

10.10 DECK DRAINAGE

Effective bridge deck drainage is important for several reasons. These include the susceptibility of the deck structural and reinforcing steel to corrosion from deicing salts, ice forming on bridge decks while other roadway surfaces are still ice-free, and the possibility of hydroplaning on decks with little surface texture. Bridge decks often are less effectively drained than approach roadways because of a lower cross slope; uniform cross slopes for traffic lanes and shoulders; parapets or curbs which contain the water and debris within the roadway section; grates, scuppers and curb openings which usually are not depressed and are smaller than on roadway sections; and, where used for downspouts, clogging in sharp bends of small diameter pipes.

Deck drainage can be improved by providing sufficient gradient to cause the water to flow to inlets or off the ends of the bridge, avoiding zero gradients and sag-vertical curves on bridges, intercepting all flow from curbed roadways before it reaches the bridge. Currently, there is a trend toward the use of watertight joints and carrying all deck drainage to the bridge ends for disposal because deck drains may be difficult to maintain.

A. Deck Inlets. Inlets used on bridge decks include grates, scuppers, curb inlets and slotted curbs. Inlets should be spaced to avoid spread on the roadway which may cause interference with traffic or the creation of traffic hazards. Collection systems and downspouts should be avoided, where possible. Collection systems should be designed with cleanouts at all bends, sufficient gradient to minimize problems with debris, and runs as short as practical. Collection systems with excessive gradients often have clogging problems because of insufficient flow depths to carry debris introduced through the inlets.

Figure 10.60 Ineffective Bridge Deck Drainage Inlet



Deck drains over traveled ways should be spaced so that water does not drain directly onto the roadway or railroad below. Where downspouts are used, splash basins should be considered to minimize erosion. Drainage should not be allowed to discharge against any part of the structure, Chapter 13, *Storm Drainage Systems* and *Bridge Drainage Systems* (Transportation Research Board, 1979a).

B. Bridge End Drains. Because of the vulnerability of approach roadway shoulders and foreslopes to erosion from concentrated flow, sufficient inlet capacity should be provided off the bridge ends to intercept flow from the bridge. A closed conduit often is preferable to an open chute down the foreslope because it controls the water in a more positive manner, is more aesthetically pleasing, and is less susceptible to damage by maintenance equipment.

When bridge end drains are not provided with the bridge construction, temporary provisions for protecting the approach fill from erosion should be utilized until permanent measures are installed and functional.

10.11 HYDRAULIC-RELATED CONSTRUCTION CONSIDERATIONS

Numerous considerations that are within the purview of the construction engineer and/or the contractor can affect the integrity of the hydraulic design and the highway-stream crossing system. Also, features of the design may be logical insofar as the engineer is concerned, but ill-conceived from the viewpoint of the construction engineer. Lines of communication between the designer and construction personnel should be established to ensure that designs are not unnecessarily complex or difficult to construct and that construction methods and measures do not invalidate design assumptions or create conditions which will adversely affect the stream crossing.

The responsibility for construction-related hydraulic considerations of stream crossings ordinarily rests with the contractor, but in some cases the highway agency may include construction-related details in the plans in order to mitigate potential environmental effects or reduce the risk of failure during construction. Whether the highway agency or the contractor assumes the risk and responsibility, hydraulic considerations during construction usually differ from the design considerations for the completed facility.

A. Verification of Plans. Plans should be checked to verify that site conditions at the stream crossing have not changed from those that existed at the time design plans were completed. Meander migration, bank caving, aggradation, headcutting or other natural or construction-induced changes in the channel may have occurred which would require that the designer reconsider decisions made on the basis of conditions which were different from those which existed at the beginning of construction. The changed conditions may require river control works,

revisions to pier locations and orientation, rearrangement of spans, or other modifications of the design to accommodate the changes that have occurred.

Dependent upon the time that has elapsed between completion of the design plans and the beginning of construction, changes in land use could significantly affect the validity of design considerations. Commercial mining of materials for construction is a rather common practice that can change flow velocities, volume and character of bed load, and flow direction and distribution at the crossing site. Land clearing for agricultural purposes may create a need to reconsider the location and size of waterway openings and the need for spur dikes. Land development near the site could change damage risk considerations for the crossing.

Hydraulic and structural designers should be consulted regarding the need to modify the design at any stream crossing which has changed significantly from the conditions which existed during design.

B. Plan Changes. Plan changes that become necessary during construction that affect pier or abutment locations, configuration or orientation, pile tip or foundation elevation should be approved by the engineer prior to proceeding with structural redesign and construction. Scour, debris and backwater considerations for the changed conditions may indicate that the proposed plan change is not advisable or that an alternative to the proposed change would serve the construction need without adversely affecting the hydraulics.

C. Borrow Areas. Borrow areas should be located so that they will not contribute to the hazards of the stream crossing. Stream bed and bank borrow can induce changes in the stream that may cause active meandering, a new channel to be formed, or result in deposition of the bed load which will cause clear water scour in the waterway. Borrow areas established along the embankment can cause concentrated flow along the embankment and contribute to serious scour at the bridge abutment.

Borrow area drainage and dewatering should be accomplished in a manner that will not contribute to sedimentation in the stream. It may be desirable to provide settling basins to remove contaminants from such drainage prior to release into the stream.

Full details on considerations in borrow area location are discussed in previous sections of this manual.

D. Detours, Contractor Crossings and Work Areas. Stream crossings for detours are built to much lesser standards than the crossing designed for the highway. Temporary stream crossing structures are usually substantially smaller than the permanent crossing structure because of the lower probability of occurrence of an extreme flood event during the relatively short construction phase. This is good practice from both hydraulics engineering and economic points of view. During a one-year construction period, the odds are 4-to-1 against a flood as large as a 5-year flood, and there is an even chance that the mean annual event will not be exceeded. The odds that a 10-year flood will occur during a one-year construction period are 9-to-1 against and 27-to-1 against occurrence in a three-year construction period. Similarly, the probability of exceedance during other construction periods can be read from the curve or computed with the equation:

(Equation 10.30)

$$Risk = R = \left(1 - \left(1 - \frac{1}{T_R}\right)^n\right)$$

where R is equal to the risk, n equals the number of years under construction, and T_R represents the return period.

It follows that the criteria used for the hydraulic design of detour stream crossings should be based on risk factors which should be evaluated considering the probability of flood exceedance during the anticipated service life of the detour (the construction period for the highway crossing), the risk to life and property, and traffic service requirements.

The smaller temporary structure will create excessive backwater and scour even during minor floods unless the gradeline is kept low enough to be overtopped before upstream damage occurs. The structure should be anchored to avoid loss during minor floods.

As in the case of the design for highway-stream crossings, detour designs should accommodate floods larger than the event for which they are designed in order to avoid undue liability for damages from excessive backwater and to reduce the probability of losing the detour stream crossing structure during a larger flood. In most instances, the conveyance of floods larger than the detour design flood is provided for by a low roadway profile which allows overflow without creating excessive velocities or backwater.

PennDOT prefers a hydraulic design of detour stream crossings by the highway agency because of environmental considerations and risks associated with contractor designs. Designs furnished to the contractor vary from a required waterway opening to a complete design of the temporary stream crossing. Considering that both the approach roadway profiles and the waterway opening are integral parts of a stream crossing system, detour designs should not be limited to specification of a required waterway area.

Temporary stream crossings necessary for the construction of highways usually are the responsibility of the contractor. It may be desirable in some instances, however, for the highway agency to design such crossings in order to minimize or mitigate the adverse effects on the stream environment, to facilitate securing permits, or to reduce the risk assumed by the contractor and thereby reduce construction costs.

E. Environmental and Ecological Aspects. Minimum disturbance of the banks and bed of a stream during the construction period will reduce erosion damage to the banks, sedimentation, and harm to fish and wildlife. Embankments in or along streams should be constructed of erosion-resistant material and/or protected against erosion to avoid adverse sediment concentrations which contribute to the turbidity of the stream.

Consideration should be given to precluding instream operations that would cause turbidity during the spawning season of certain types of fish. This information is available from the Pennsylvania Fish and Boat Commission. Detours and construction roads are other sources of turbidity and either should be constructed at a time that fishery activities will not be disturbed, or provisions should be made to control any harmful effects of erosion. Silts and clays will generally flush out of the substrate over a period of time, but sands tend to become embedded. Gravel and rock similar to the gradations found in the existing substrate will do the least damage to the aquatic habitat.

Pumping of cofferdams and other dewatering operations may have a discharge of unacceptable quality to the receiving stream. Mitigation measures such as settling basins may be necessary if the ecosystem of the stream would be upset by the temporary degradation of water quality.

F. Hydrologic Information. The hydrology for the construction site is the same as for the design of a bridge. However, temporary structures will have a different design frequency storm than the bridge and it will depend upon the how long the temporary structure is in place. The hydrology for the construction site is the same as for the design of a bridge; however, the design storm will depend upon how long the temporary structure is in place. The design storm for the temporary structure will usually be different from the one used for the bridge.

G. Cofferdams, Caissons, Barges, and Falsework. Cofferdams, causeways, falsework and occasionally contractor's equipment, such as barges, constrict the stream channel more than the completed substructure and consequently have greater potential for causing scour and bank caving, and for collecting debris. Scheduling of work to avoid flood seasons is especially important if these types of operations will be involved.

H. Feedback. Most highway designers do not have an opportunity to participate in the construction of the works that they have created. For this reason, designs that could be improved upon for construction purposes tend to be perpetuated simply because the designer is not informed of the deficiencies.

Construction engineering personnel are encouraged to invite designers to visit construction sites to discuss problems with designs and possible improvements in future designs. Upon completion of a project, a design critique conducted jointly by designers and field personnel can be a very useful learning experience for both.

10.12 HYDRAULIC-RELATED MAINTENANCE CONSIDERATIONS

A sizeable proportion of highway maintenance resources are expended on protecting and preserving the capital investment in stream crossings. Many expenditures are made on an emergency basis, while many others are made to maintain the hydraulic and structural integrity of the crossing and to guard against future damage by flood flow. As

is true of all aspects of highway and bridge maintenance, it is important that when maintenance work is undertaken, it is a high-priority need and cost-effective.

Engineers and other highway personnel with expertise in specialized areas can provide important expert assistance in assessing priority needs and in recommending cost-effective repairs and protective measures. Similarly, maintenance personnel can provide important information to engineers which will enable the engineers to provide solutions for problems at existing crossings and possibly avoid similar problems in future designs.

Discussion in the following sections is intended to advocate the merits of utilizing the services of engineers in the maintenance of highway-stream crossing systems. Streams are dynamic systems, and, in general, it is necessary to provide countermeasures at each problem site commensurate with the hazard and the resources available. Therefore, it is not the intention here to provide a detailed discussion of conditions which warrant corrective measures, or to prescribe the measures which should be undertaken.

A. Maintenance Inspections. The National Bridge Safety Inspection Program has been expanded to require detailed consideration of scour conditions that could threaten the structural stability of the bridge. The FHWA recommends that a team consisting of structural, hydraulic and geotechnical personnel be employed in the evaluation of bridge scour. In addition to regularly scheduled maintenance inspections, stream crossings may be inspected following flood events to discover conditions which may threaten the integrity of the crossing. Items that should be noted and commented on include bend migration, aggradation, degradation, unusual scour depths, bank erosion, approach fill erosion, bank protection and river training device damage or destruction, and changes in channel alignment and flow conditions. Conditions which appear to threaten the bridge or approaches should be referred to hydraulic and structural engineers for recommendations on countermeasures to be undertaken. PennDOT's inspection manual, Publication 238, *Bridge Safety Inspection Manual* provides detailed discussion of the requirements for maintenance inspection teams and processes.

B. Flood Damages. Although maintenance personnel often are occupied with emergency repair during times of floods, data which can be collected only during and immediately following a flood, such as highwater elevations and scour depths, should be given attention for scour critical bridges. These data are important to any decision regarding the necessity for providing remedial or corrective measures and the measures that should be used.

Some of the serious problems that develop during floods have been anticipated by the designers. For example, scour in alluvial streams is a well-known phenomenon, and although predictive techniques are not precise, bridges are designed to withstand the anticipated scour; therefore, it is important to determine the extent and depth of scour that develops during a flood, but corrective measures are not necessary unless the expectations of the designer were exceeded. Similarly, the "as-built" channel at a bridge may not need to be restored if piers were designed in anticipation of degradation or bend migration through the bridge opening.

Stream bank protection and river training measures are generally "temporary" since it is not cost-effective to provide protection against the worst possible contingency. Most such measures are intended to provide protection against most of the floods that occur but eventually will be damaged or destroyed. Thus, the designer is "buying time" when the measures are specified with the expectation that other measures will be necessary after the stream defeats the original installation. At that time, it is probable that restoration of the original installation would be unlikely to stabilize the banks or train the stream in its changed condition, and a decision must be made on the need for new or additional measures.

The above discussion is to illustrate that all "damage" need not be repaired, and cost-effective repair or reconstruction may not restore measures as originally constructed. Maintenance files should contain data on "as-built" construction to aid in making decisions regarding the need for corrective measures. It may be expedient to consult structural and hydraulic engineers to review the design and advise on the necessity for emergency measures. Some protective measures cannot be provided or existing measures cannot or need not be repaired during the flood in which damage occurs. After the flood subsides, there is an opportunity to evaluate the changed conditions and the damage caused by the flood and to make new decisions regarding protective measures to be employed. The expertise of hydraulic and structural engineers should be utilized to ensure that cost-effective measures are provided.

1. Remedial Construction and Repair. Additional protective measures often are required at stream crossings to protect the crossing from destruction by the stream. Bend migration may be endangering the approach embankment and/or the structure, or an unanticipated headcut or scour hole may have exposed pier footings or

pilings to an unacceptable depth. These conditions should be corrected as soon as practical, and hydraulic and structural engineers should be consulted regarding the remedial measures or countermeasures to be employed. It should be kept in mind that the security of the crossing rather than that of any existing protective measures is the objective of the maintenance effort. Relatively inexpensive, expendable measures may be more cost-effective than more expensive, relatively permanent measures.

2. Recurring Damage. At some stream crossings, similar damage occurs during each flood event, requiring relatively frequent, recurring maintenance. Some of the problems can be eliminated by remedial construction, as by the construction of spur dikes, finger dikes or downstream dikes to avoid damage to approach embankments. The logical and feasible solution to other recurring damage problems may be to discontinue repairing the damage if the crossing and adjacent property are not endangered. An example of this type of problem might be a scour hole that develops at midspan, or downstream, of a structure on the floodplain.

Many highway-stream crossing systems were designed with relatively low profiles where the roadway encroaches on the floodplain. Overflow over the roadway may cause traffic interruption and embankment damage even though the bridge over the stream is above highwater. The seemingly obvious solution is to raise the profile of the roadway, but this may have implications that are not immediately apparent. A higher profile in the floodplain will cause higher backwater, force larger floods through the bridge waterway, and change flow distribution downstream of the highway facility. These changes may significantly increase the risk of flood damage to property upstream and downstream of the highway, and severe scour may occur at the bridge because larger flows are forced through the opening. Also, the probability that the bridge superstructure will become submerged will be greater and the hazard of failure from drag, debris, ice or buoyant forces will be increased.

Recurring maintenance requirements at some stream crossings are such that the only remedy is reconstruction. This often occurs with aggrading and degrading streams where continual maintenance and countermeasures are expensive and often ineffective. The need to reconstruct may also arise from changes in the watershed which have changed the hydrology of the stream. Maintenance personnel should take steps to ensure that design personnel are fully cognizant of the history of the crossing and the reasons for its inadequacy.

10.13 APPURTENANCES

A. Bridge Rail. The type of bridge rail used can be an important hydraulic consideration. This is particularly true in instances where overtopping of the bridge is a factor.

A solid bridge rail used where the bridge superstructure overtops will constitute a significant impediment to flood flow.

A more desirable type of rail for accommodation of flood flow is a style which will offer the floodwater an opening. An open slender type of bridge railing will have a lower backwater and smaller lateral forces than a more impervious type. However, parapets must always be designed for traffic loads appropriate to the traffic on the anticipated bridge.

10.14 DESIGN DOCUMENTATION

Design data should be assembled in an orderly fashion and retained for future reference. The amount and detail of documentation for each highway-stream crossing system should be commensurate with the risk and the importance of the crossing. For example, a small stream in a rural area would not ordinarily require the same degree of documentation as a small stream in a developed area.

Design data and documentation are important in the post-construction period for the following reasons:

- The performance of structures over a period of time, as compared with information developed for the design, is very helpful in evaluating design policies and procedures and the validity of design assumptions.

- In the event of failure, contributing causes can be identified, compared with design assumptions and computations, and considered in the design of replacement structure(s).
- Documentation of data for existing structures is a valuable source of information when structures are replaced, repaired or rehabilitated, and for the design of other structures in the vicinity.
- Information collected and analyzed for purposes of highway design can be of value to others considering plans for the vicinity.
- Records of data, analyses, and decisions are essential for responding to subsequent complaints and litigation.

Project files are the most permanent of all highway agency records and are a convenient and appropriate place for recording the results of analyses and decisions. Chapter 4, *Documentation and Document Retention,* should be referenced for required documentation for bridge projects.

10.15 CHAPTER 10 NOMENCLATURE

Symbol	Definition	Units
A	Net area of the bridge opening in Orifice Equation	m^2 or ft^2
A	Cross sectional area of flow	m^2 or ft^2
B	Flow weighted distance in FHWA WSPRO Method Equations	m or ft
C	Coefficient of discharge for fully submerged pressure flow in Orifice Equation	dimensionless
C_d	Coefficient of discharge for pressure flow in Pressure-Weir Flow Methods	dimensionless
C_d	Coefficient of drag in Drag Force Equation	dimensionless
F	Force of impact imparted by debris in Debris Force Equation	N or lb
F_d	Drag force per unit of bridge length in Drag Force Equation	N/m or lb/ft
g	Gravitational acceleration constant	m/s^2 or ft/s^2
H	The difference between the energy grade elevation upstream and The water surface elevation downstream in Orifice Equation	m or ft
H	Depth of submergence of superstructure in Drag Force Equation	m or ft
H_{3-2}	Drop in water surface elevation from section 3 to 2 in Yarnell Equation	m or ft
h, h_1, h_2, h_3, h_4	Water surface elevation, water surface elevation at respective cross sections in FHWA WSPRO Method Equations	m or ft
h_L, $h_{L(4-1)}$	Energy losses, energy losses from cross section 4 to 1 in FHWA WSPRO Method Equations	m or ft
h_f	Energy losses due to friction	m or ft
$h_{f(1-2)}$	Energy losses due to friction from section 1 to section 2 in Yarnell Method Equations	m or ft
h_e	Energy losses due to expansion	m or ft
K	Yarnell's pier shape coefficient	dimensionless
K	Total loss coefficient in modified form of Orifice Equation	dimensionless
K, K_1, K_2	Conveyance, conveyance at respective cross sections in FHWA WSPRO Method Equations	m^3/s or cfs
L	Effective length of the weir in Weir Flow Equation	m or ft
L_{av}	Effective flow length in approach reach in FHWA WSPRO Method Equations	m or ft
M	Mass of debris in Debris Force Equation	kg or slugs
P_1, P_2	Sinuosity in Channel Modification Equations, (ratio of stream length/valley length or valley slope/channel slope)	m/m or ft/ft
Q	Total discharge, design discharge	m^3/s or cfs
S	Stopping distance in Debris Force Equation	m or ft
S_1, S_2	Channel slope in Channel Modification Equations	m/m or ft/ft
V, V_1, V_2, V_3, V_4	Velocity, velocity at respective cross sections in Yarnell Equation and FHWA WSPRO Method Equations	m/s or ft/s
v	Velocity of flow	m/s or ft/s

W_1, W_2	Stream width in Channel Modification Equations	m or ft
Y, Y_1, Y_2, Y_3	Hydraulic depth, hydraulic depth at respective cross sections	m or ft
Z	Vertical distance from maximum bridge low chord to the mean river bed elevation in Pressure-Weir Flow Method	m or ft
α	Obstructed area of the piers divided by the total unobstructed flow area in Yarnell Method Equations	dimensionless
α_1	Energy correction factor in FHWA WSPRO Method Equations	dimensionless
β_1	Momentum correction factor in FHWA WSPRO Method Equations	dimensionless
ρ	Density of water	kg/m^3 or $slug/ft^3$
ω	Ratio of velocity head to depth in Yarnell Equations	dimensionless

10.16 REFERENCES

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CHAPTER 10, APPENDIX A

FIELD CHECKLIST FOR THE PRELIMINARY DESIGN PERMIT COORDINATION

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DATE: June 17, 2008

SUBJECT: Field Checklist for Preliminary Design Permit Coordination

TO: PennDOT District Executives and PADEP Regional Chiefs

FROM: Brian Thompson, PE
Director
PennDOT Bureau of Design

John Hines
Executive Director
PA DEP

The Pennsylvania Department of Transportation (PennDOT), and The Department of Environmental Protection (PA DEP) have worked jointly to develop *Field Checklist For Preliminary Design Permit Coordination*. The guidance document has been developed by a joint taskforce of PennDOT, PA DEP, and Pennsylvania Fish and Boat Commission (PFBC) representatives from PennDOT Bureau of Design and District Offices, PA DEP Central Office and Regional Offices, and PFBC.

The use of the checklist is highly encouraged for PennDOT projects. The checklist was developed to facilitate early coordination and documentation for projects to help streamline and expedite the permitting process. The checklist is intended to document potential impacts and the proposed designs early in the preliminary design phase. The checklist can be used by the agencies as a record of discussions in the field and anticipated permitting requirements based on the conditions at the time of the meeting.

Attached is a copy of the Field Checklist. For PennDOT the checklist will become part of PennDOT Drainage Manual when it is issued this year. For PA DEP this document will be available online in the Chapter 105 Online Guidance Manual and should be part of PA DEP's desk manual for review of permit applications.

If you have any questions please contact Harold Rogers at PennDOT at 717-787-3767 or Jeff Means at PADEP at 717-772-5643.

Attachment
cc: Richard H. Hogg, PE Reading File
Highway Administration Bureau Directors
PennDOT District ADE's Design
PennDOT District Bridge Engineers
PennDOT District H&H and Permit Coordinators
Kelly Heffner, PA DEP
Jeffrey Means, PA DEP
PA DEP T-21 Staff

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FIELD CHECKLIST FOR PRELIMINARY DESIGN PERMIT COORDINATION

INSTRUCTIONS

This checklist was developed by PADEP and PennDOT to facilitate early coordination and documentation for projects. The checklist is intended to document potential impacts and the proposed designs early in the preliminary design phase. The checklist can be used by both agencies as a record of discussions in the field and anticipated permitting requirements based on the conditions at the time of the meeting. It should be noted that design constraints and/or changes in the impacts may require different permits than originally anticipated when this checklist was completed.

Scan the signed checklist and distribute to all attendees, the PennDOT District ADE-Design, and the PADEP Region Chief Waterways Section within 7 days of the field meeting.

Distribution CC: All Attendees
PennDOT District ADE - Design
PADEP Region - Chief of Waterways Section
PFBC T-21 Represenative
USACE T-21 Representative

PROJECT DETAILS

Project: _____

_____

District: _____

Municipality: _____ County: _____

Attendees	Organization	Email Address

Existing Structure

Type of Structure: _____

No. of Spans: _____ Span Lengths: _____

Piers:

Type: _____ Width: _____

Orientation: _____

Structure:

Underclearance: _____ Skew: _____

Scour: _____

FIELD CHECKLIST FOR PRELIMINARY DESIGN PERMIT COORDINATION

Proposed Structure

Type of Structure: _____

No. of Spans: _____ Span Lengths: _____

Piers:

Type: _____ Width: _____

Orientation: _____

Structure:

Underclearance: _____ Skew: _____

Will Project Result in Change in Roadway Profile ☐ Yes ☐ No

If Yes, Explain: _____

Environmental

USGS Quad Name: _____ Latitude: _____ Longitude: _____

Stream Name: _____

Stream Flow: ☐ Perennial ☐ Intermittent ☐ Ephemeral

Fish Habitat: ☐ Stocked ☐ Wild Trout Chapter 93 Classification: _____

Other Environmental Concerns:

Threatened/Endangered Species ☐ Yes ☐ No PNDI Search Date: _____

If Yes, Describe: _____

_____

Wetlands Present ☐ Yes ☐ No Chap 105 Class: ☐ EV ☐ Other

If Yes, Impacts Anticipated ☐ Yes ☐ No Cowardin Class: _____

Amount Impacted: _____ acres Total Acreage Wetland Onsite: _____ acres

Reason for Impact: _____

_____

Historical, Cultural, Archeological Site ☐ Yes ☐ No

If Yes, Describe: _____

_____

National Registry/Nation landmark ☐ Yes ☐ No

If Yes, Describe: _____

_____

Anticipated Mitigation Requirement: ☐ Yes ☐ No

If Yes, Describe: _____

_____

FIELD CHECKLIST FOR PRELIMINARY DESIGN PERMIT COORDINATION

Hydrologic and Hydraulic Information

Drainage Area: _____

FEMA Study Area ☐ Yes ☐ No

If Yes, Detailed ☐ Yes ☐ No

Fill in Floodplain (Regulated by Chapter 106) ☐ Yes ☐ No

Existing Structures within floodway/floodplain immediate vicinity of project ☐ Yes ☐ No

- ☐ Residential, How Many: _____
- ☐ Commercial, How Many: _____
- ☐ Other, Describe, How Many: _____

Distance from the Stream: _____

_____

Is it anticipated that the project will increase the 100-yr WSE ☐ Yes ☐ No

If Yes, Explain: _____

_____

Consistency Letter Required from Municipality ☐ Yes ☐ No

Flood Easement(s) Required from Affected Property Owners ☐ Yes ☐ No

Will Traffic be Detoured as a Part of this Project ☐ Yes ☐ No

Will Construction Involve a Cofferdam or Causeway ☐ Yes ☐ No

Is Project in an Approved (after July 2000) Act 167 Area ☐ Yes ☐ No

Total Area of Disturbance _____ acres

Permitting - To Be Completed in the Field

Anticipated Permit Based on Scoping:

- ☐ GP- _____
- ☐ JPA _____

Anticipated NPDES Permit

- ☐ General ☐ N/A
- ☐ Individual

Special Considerations: _____

_____

_____

_____

_____

Signatures of Authorized Representatives - To Be Completed in the Field

PennDOT/PennDOT Representative		PADEP	
Name:	_____	Name:	_____
Title:	_____	Title:	_____
Signature:	_____	Signature:	_____
Date:	_____	Date:	_____

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FIELD CHECKLIST FOR PRELIMINARY DESIGN PERMIT COORDINATION

Additional Notes

Instructions

This checklist was developed by PA DEP and PennDOT to facilitate early coordination and documentation for projects.
The checklist is intended to be used to provide documentation of the potential impacts and the proposed designs for the site early in the preliminary design phase. The checklist can be used by both agencies as a record of discussions in the field and anticipated permitting requirements based on the conditions at the time of the meeting.
It should be noted that design constraints and/or changes in the impacts may require different permits then originally anticipated.

Instructions:

1. All sections except the attendees, permitting and signature sections should be completed prior to the field visit

2. Forward a copy of the partially completed checklist including the following to all invited participants one week before the meeting:
    * 8.5X11 Location Map
    * Directions to the site
    * Preliminary Plan (if available)
    * Wetland delineation plan (if available)

3. At the field meeting the permitting section should be completed based on the anticipated permit - note any alternatives that were discussed, etc.

4. The PennDOT and PA DEP representatives should sign the checklist under the signatures section

5. Scan the signed checklist and distribute to all attendees, the PennDOT District ADE-Design and the PA DEP Region Chief Waterwasy Section within 7 days of the field meeting.

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CHAPTER 11

SURFACE WATER ENVIRONMENT

11.0 INTRODUCTION

A. Introduction. This chapter provides the designer with an overview of the surface waters and how these resources may be affected by roadway development. Publication 325, *Wetland Resources Handbook* provides design and environmental professionals with additional details for the identification, regulation, and design of wetland resources in relation to roadway development. This chapter also encourages a team approach in evaluating the effects of highway construction on surface waters. In general, the designer's role is to determine the hydrology and hydraulic related highway effects and significance on sensitive surface waters.

B. Purpose. The purpose of this Chapter is to provide the background and regulatory framework for what is required when highway projects may affect surface waters, wetlands and/or groundwater. Due to the complex and controversial nature of surface water environmental engineering, this chapter will address only the simpler, more reliable and accepted engineering practices. Other, more complex practices may be briefly noted, but only to provide background and suggest possible analytical courses of action. In general, PennDOT believes that the subjects and practices provided in this chapter will address 90% or more of the surface water environmental problems encountered by the designer. Other subjects and practices may be added when the need arises.

It is imperative that the designer recognize and impress on others that all the practices presented or alluded to in this chapter are not routinely employed on all highway projects or sites. In reality, rarely would more than one or two practices be needed, and then only on selected projects or at unique sites. The methods and practices employed must be approved by the District Project Manager and Environmental Manager in response to specific concerns of the appropriate resource and regulatory agency(ies).

C. Surface Waters. Natural surface waters addressed by this chapter are considered "Waters of the Commonwealth" and include:

- Streams or rivers.
- Ponds, lakes and reservoirs.
- Wetlands.
- Springs.

Not addressed in this chapter are groundwaters such as:

- Underground streams or rivers.
- Underground ponds or lakes and aquifers.

D. Sensitive Surface Waters. In general, the primary factors that may make surface waters sensitive are:

- Chemical
- Physical
- Biological
- Regulatory

The identification of sensitive waters will be made by the resource agencies with concurrence by PennDOT. Sensitive waters may require additional assessment and/or protection and will generally include waters classified as Exceptional Value (EV), High Quality (HQ), Class A Wild Trout Streams, Stream Sections that Support Natural Reproduction of Trout, and Wilderness Trout Streams. The classification of waters in Pennsylvania is under the jurisdiction of the Pennsylvania Department of Environmental Protection (PA DEP) and Pennsylvania Fish and Boat Commission (PFBC). The following links provide relevant information on the classification of surface waters in the Commonwealth:

- www.pacode.com/secure/data/025/chapter93/chap93toc.html
- http://www.fish.state.pa.us/waters_trout.htm

E. Functional Values. The functional value of a surface water includes such elements as:

- Flood control.
- Terrestrial wildlife habitat.
- Aquatic habitat.
- Groundwater recharge.
- Aesthetics.
- Shore and bank line geometry.
- Water temperature.
- Scenic and wild rivers designation.
- Endangered species habitat.
- Contaminant abatement.
- Recreation.

A surface water's functional value depends on the interrelationship of key factors including:

- Geographic location.
- Hydrology.
- Climatology.
- Geometric setting.
- Size.
- Quality.
- Classification (see Section 11.0.D).
- Biotic region.

F. Effect and Significance. Determining a surface water effect can determine if a threshold value has been exceeded and if there is a reversible effect.

1. Effect. This is what occurs to the surface waters as a direct or indirect result of the highway improvement.

2. Significance. Significance should be determined by relating an effect to such elements as risk (probability or frequency of occurrence) and cost. Included in such determinations should be any past or future cumulative effects that were either caused by, or expected to be caused by, PennDOT or others. Where practicable and useful, a simple comparison should be made of three conditions:

- Existing (pre-construction).
- During construction.
- Expected post-construction.

This comparison is used to identify net changes expected as a result of the project.

G. Reversible Effects. It may be necessary to assess whether an effect is reversible or irreversible. Given enough time and resources, surface water effects caused by highway improvements can usually be reversed. Whether the reversal of an effect would be considered as practicable is the context within which PennDOT should address this issue.

H. Practicable. In coordination with the applicable resource and regulatory agency(ies), the choices and alternatives may be contingent upon what is practicable. For the purpose of this chapter, practicable shall mean "available and capable of being done after taking into consideration cost, existing technology and logistics in light of overall project purposes."

This is the definition used in the Memorandum of Agreement (MOA) between the U.S. Environmental Protection Agency (USEPA) and the U.S. Army Corps of Engineers (USACE) (USEPA and USACE, 1990).

I. Design Alternatives. When it is determined that highway construction and/or operation activities will have an adverse effect on surface water resources, design alternatives should be considered.

There are three general design alternatives that should be considered as applicable:

- Avoidance
- Minimization.
- Compensatory mitigation.

1. Avoidance. Wherever practicable, avoiding the surface water impact is the best and thus the preferred alternative. It should be demonstrated to the applicable resource and regulatory agency(ies) that this alternative is not practicable before any other alternative can be considered.

2. Minimization. Where surface water disturbances cannot be avoided, such disturbances may be minimized through adjustments in the highway's:

- Alignment.
- Profile.
- Template.
- Other geometry.

The intent is to reduce the impacts to surface water resources when avoidance is not practicable. This may require project down-scoping or reduction of the highway design standards from "desirable" to "acceptable" levels if applicable.

3. Compensatory Mitigation. When required through coordination with the applicable resource and regulatory agency(ies) and approved by the District Project Manager and Environmental Manager, PennDOT will undertake compensatory mitigation measures to offset the adverse effects of highway development on surface water resources. Compensatory mitigation may include on-site mitigation, off-site mitigation, in-kind mitigation, out-of-kind mitigation, or monetary compensation.

11.1 POLICY

A. Introduction. Set forth in this section are PennDOT's guidelines for highway construction affecting sensitive surface waters. These address:

- Rules and regulations.
- Cost effectiveness.
- Analysis/assessment of complexity.
- Upgrading functional values.
- Environmental studies.
- Surface water assessment.
- Surface water analysis.
- Permits and certifications.

B. Rules and Regulations. All surface water design, construction and maintenance must be in compliance with Federal, State and Municipal laws, regulations, executive orders and Memorandums of Agreement (MOAs) and Understanding (MOUs). The principal laws, regulations and executive orders applicable to this chapter are as follows:

1. Federal. These are as follows:

- Clean Water Act (33 USC 1251 -1376, DOT Order 5660.1A, FHWA Notices N5000.3 and 5000.4, 23 CFR 650, Subpart B, E, 33 CFR 209, 40 CFR 120).
- National Environmental Policy Act (NEPA) (FHWA 23 CFR 771)
- Protection of Wetlands (Executive Order 11990, 23 CFR 777).
- Floodplain Management (Executive Order 11988 (23 CFR 650, Subpart A, 23 CFR 771), amended by Executive Order 12148, DOT Order 5650.2).

2. State. These are as follows:

- Dam Safety and Encroachment Act (32 P.S. §§ 693.1-693.27).
- The Clean Streams Law (35 P.S. §§ 691.1-691.1001).
- Floodplain Management Act (32 P.S. §§ 679.101-679.601).

3. Municipal and County. These are as follows:

- Section 1905-A, Commonwealth Administrative Code, amended Act 14.
- Municipal Planning Code, Acts 67 and 68.

C. Cost Effectiveness. It is recognized that costs associated with surface water environmental engineering may not be reliably quantified - commonly those associated with the functional values of a resource. Nevertheless, it is important that any mitigation be both:

- Cost effective.
- Demonstrably beneficial.

D. Enhancing Functional Values. In general, it is the intent of PennDOT to maintain the existing functional values of surface waters. Where cost-effective benefits can be demonstrated, existing functional values will be enhanced consistent with PennDOT and FHWA environmental stewardship policies.

Examples of some benefits are:

- Watershed enhancement.
- Wetland and stream banking.
- Acceptable substitution of a different, cost-effective, easier to construct and/or maintain functional value.
- Receiving concessions on other issues.

E. Evaluation Complexity. The detail of any environmental evaluation involving an assessment or analysis of a surface water system should be commensurate with the:

- Surface water sensitivity.
- Importance of the resource's functional values.
- Applicable resource and regulatory agency(ies) mandates.

Close and ongoing coordination with the applicable resource and regulatory agency(ies) should be maintained throughout the plan development process by PennDOT's environmental team to ensure an acceptable level of assessment or analysis detail. Only that detail essential to securing permit approval of the highway project from the applicable resource and regulatory agency(ies) should be developed. In general, surface water assessments will suffice for most sites involving sensitive surface waters. Where such activities as a major alteration of a surface water feature is required, or when a detailed analysis is mandated by the applicable resource and regulatory agency(ies), a surface water analysis should be prepared.

F. Surface Water Assessment. An assessment is a judgmental or subjective form of surface water analysis and forgoes the need for a complex, objective type of study and/or a study requiring large amounts of costly data.

Assessments should determine constraints, if any, for construction within sensitive surface water areas. These findings will be used to assess the effect of the proposed project on these waters. Wherever practicable, PennDOT's application of Best Management Practices (BMPs) should be consistent with surface water concerns provided in

writing by the applicable resource and regulatory agency(ies). If an assessment is acceptable to the applicable resource and regulatory agency(ies), a surface water analysis will not be required.

If the assessment findings or proposed BMPs are unacceptable to the applicable resource and regulatory agency(ies), or should other surface water related issues be raised that do not lend themselves to resolution through a surface water assessment, then a surface water analysis may be considered.

G. Surface Water Analysis. An analysis is more quantitative than an assessment. The issues to be addressed in a surface water analysis should consider those that PennDOT's environmental team determines to be significant based on their coordination with the applicable resource and regulatory agency(ies). All other issues should continue to be addressed at the assessment level of investigation.

H. Permit Criteria. The disturbance of sensitive surface waters may require a permit. The conditions of such permits may mandate hydraulic design criteria that must be included in the design. These criteria should be identified early in the design process to minimize delays in developing the contract plans. For this reason, the designer should maintain close contact with the PA DEP and USACE. These permits include:

- USACE Section 404 permits.
- USACE Section 10 Permit.
- State Section 401 Water Quality Certifications.
- USCG Section 9 Permit.
- PA DEP Chapter 102 Erosion and Sedimentation Control Plan approval
- PA DEP Chapter 105 Water Obstruction and Encroachment Permit.
- PA DEP Chapter 106 Floodplain Encroachment Permit.
- PA DEP General Permit BDWM-GP-7, Minor Road Crossings
- PA DEP General Permit BDWM-GP-8, Temporary Road Crossings
- PA DEP General Permit BWM-GP-11, Maintenance, Testing, Repair, Rehabilitation, or Replacement of Water Obstructions and Encroachments.
- Clean Water Act Section 402 National Pollutant Discharge Elimination System (NPDES) Permit.

11.2 DESIGN CRITERIA

A. General Criteria. General criteria that could be considered for all surface water locations include the following:

- Best Management Practices.
- Mitigation alternatives.

1. Best Management Practices. Best Management Practices (BMPs) will be routinely used to mitigate expected adverse surface water impacts. The BMPs are described in Chapter 12, *Erosion and Sediment Pollution Control*, and in Chapter 14, *Post-Construction Stormwater Management*. The BMPs in Chapter 12, *Erosion and Sediment Pollution Control*, are generally used during construction, while the BMPs in Chapter 14, *Post-Construction Stormwater Management* are designed for post-construction purposes only. Non cost-effective practices should be avoided unless they are:

- Necessary to minimize maintenance.
- Determined by PennDOT to be essential to the best public interest or safety.
- Mandated by the applicable regulatory agency(ies).

2. Mitigation Alternatives. The following six mitigation alternatives, listed in order of priority, should be considered as applicable for adverse surface water impacts as determined by PennDOT, when identified by a resource agency or when mandated by the applicable regulatory agency(ies):

- No mitigation required because minimal impact when agreed to by the applicable regulatory agency(ies).
- Avoiding the impact altogether.

- Minimizing the impact by limiting its degree, magnitude and implementation.
- Rectifying the impact by repair, rehabilitation and/or restoration.
- Reducing/eliminating the impact through preservation and/or maintenance.
- Compensating for the impact with substitute resources.

Findings for rejecting any alternative will be documented to the satisfaction of the applicable regulatory agency(ies) before selecting a lower priority alternative. Required wetland mitigation shall be accomplished in accordance with the PA DEP's *Design Criteria for Wetland Replacement* (PA DEP, 1992) and other available guidance including *Compensating for Wetland Losses Under the CWA* (National Academy of Sciences, 2001), *Wetlands Restoration, Enhancement or Creation* (NRCS, 1997), and *Mitigation and Monitoring Guidelines* (USACE, 2004). Guidance for the required mitigation for other surface waters should be identified by PennDOT and coordinated with the applicable resource agency(ies) as appropriate.

B. Design Criteria. Below are design criteria for the following six hydraulic-related surface water features:

- Water quality.
- Channels.
- Lakes or ponds.
- Wetlands.
- Fish passage.
- Stream geometry and cover.

1. Water Quality. At a minimum, three design criteria are considered when selecting the base-line water quality for a project:

- Seasonal pre-construction quality.
- Expected future quality where improvements will be made by others.
- Mandates of the applicable regulatory agency(ies).

There are two design criteria for surface water quality:

- Short Term. Water quality occurring during and immediately following construction may be temporarily degraded within the limits approved by the applicable regulatory agency(ies).
- Long Term. Water quality will, at a minimum, equal the seasonal pre-construction quality as determined by PennDOT or as mandated by the applicable regulatory agency(ies).

Water quality will be monitored when it is:

- In the best public interest or for safety reasons.
- Requested by a resource agency as agreed to by PennDOT.
- Mandated by the applicable regulatory agency(ies).
- Jointly approved in Monitoring Plans by PennDOT and the applicable regulatory agency(ies).

2. Channels. Functional value design criteria for channels address:

- Bankfull stage.
- Stability.
- Mitigation.

a. Bankfull Stage. The Bankfull stage line should be determined based on any of the following characteristics: (Dunne and Leopold, 1978; Harrelson et al, 1994; USDA Forest Service, 2003; USDA Forest Service, 2005)

- Clear or natural line impressed on the bank or shore.
- Shelving.
- Changes in bank materials.

- Changes in or scouring of terrestrial vegetation.
- Presence of litter and debris.
- Ordinary High Water marks.
- Inundation line of the Normal Operating Pool Elevation (NOPE) for reservoirs.

Should none of the foregoing indicators be present, use the inundation line of the water surface corresponding between the 1 and 2 year recurrence interval as a conservative estimate of the bankfull stage; when available however, the foregoing indicators should be used.

b. Stability. The stability of a channel reach should be based on an evaluation of the following criteria over a discharge range of normal flow to Q_{100}(the flow rate corresponding to the 100-year design storm). A channel will be considered relatively stable for a particular discharge when displaying some or all of the following characteristics except as noted for braiding and headcutting:

- Tractive Shear Stress. The proposed bed and bank shears should approximate that allowable in a stable channel given the bed and bank material available.
- Regime Slope. Present or expected channel slope approximates the channel's regime slope for a stable channel.
- Bank Caving. Nominal bank caving or no caving is in evidence.
- Braiding. Braiding must not be in evidence.
- Headcuts. Headcutting must not be in evidence.
- Sinuosity. The degree of sinuosity is consistent with the regime slope. Sinuosity is also related to sediment load, bed and bank material and channel dimensions.

The following terminology and related design criteria for classifying channel stability can be used:

- Relatively Stable. Meandering alluvial channels or straight channels incised into rock.
- Transitionally Stable. Straight alluvial channels.
- Marginally Stable. Alluvial channels in a transitional range between the foregoing two types and the unstable type (below).
- Unstable. Braiding or headcutting channels.

PennDOT should avoid taking any action that would further destabilize a channel at normal flows.

c. Mitigation. There are five criteria to consider before electing to mitigate a transitionally stable, marginally stable or unstable channel with channel control facilities:

- Cost effectiveness.
- Necessary to protect the road.
- Necessary to protect property.
- In the best public interest.
- Mandated by the applicable regulatory agency(ies).

Other channel criteria for stabilizing a channel can be applied with the approval and consent of the District Environmental Manager and the appropriate regulatory agency(ies) personnel.

3. Lakes or Ponds. Criteria for lakes or ponds should be established on a case-by-case basis due to their unique needs and the current lack of reliable or proven technical guidance for practicable criteria.

4. Wetlands. Wetlands should be classified by using the USACE's current regulatory definition (USACE's Wetlands Delineation Manual). More details on wetlands classification, hydrology, creation and restoration can be found in Section 11.6, Publication 325, *Wetlands Resource Handbook*, and Chapter 10 of AASHTO's Highway Drainage Guidelines (AASHTO, 2003).

5. Fish Passage. The designer shall refer to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.10 for regulations and requirements concerning fishable streams within the

Commonwealth. Additional coordination with the Pennsylvania Fish and Boat Commission is recommended for guidance and regulations concerning fishable streams.

6. Stream Geometry and Cover. Channel disturbances may require design criteria for the following:

- Riparian cover.
- In-stream cover.
- Riffles.
- Pools.
- Substrate.
- Bank geometry.
- Conveyance.

a. Riparian Cover. Riparian vegetation that provides temperature control for the surface waters, surface water habitat, or terrestrial habitat that is disturbed by construction should be replaced in kind or as needed.

b. In-stream cover. Vegetation that provides in-stream cover such as overhanging tree limbs, should be maintained to provide shade for aquatic habitat.

c. Riffles. Natural riffles should be restored (typically by placed rock) in locations of the stream where water appears to stagnate, in order to maintain an oxygenated state in the stream.

d. Pools. Any disturbed pools should be restored to provide natural habitat for fish species and other aquatic species.

e. Substrate. As practicable, the natural, pre-construction substrate should be replicated in all disturbed areas except where it is necessary to provide some type of erosion protection or armor for stability.

f. Bank Geometry. In rural areas, banks should be left in their natural form except where it is necessary to provide for such functions as ingress and egress of such items as the following:

- Vehicles.
- Boats.
- Livestock.
- Swimmers.
- Amphibians.
- Water fowl.

In certain urban areas, aesthetics may dictate the need for uniform channel alignments and sloped banks.

Unless a critical facility is being protected and as practicable, banks should generally be protected with vegetation, rip-rap, or other stabilizing elements.

g. Conveyance. As practicable, the design should replicate the conveyance associated with the natural stage-discharge relationship for the low-flow channel (dominate flow channel) and the attendant overbank floodway. These two replications should be considered separately.

11.3 DESIGN CONSIDERATIONS

A. Introduction. Surface water design considerations include:

- Water quality.
- Channels.
- Lakes and ponds.

- Wetlands.
- Fish passage.
- Stream geometry.
- Riparian cover.

The following information is general guidelines that the designer could use in design where it is demonstrably appropriate. For most highway projects, the surface water design considerations are as agreed to by PennDOT and their environmental team through coordination with the applicable resource and regulatory agencies.

B. Water Quality. Water quality is considered as the interrelated combination of such factors as:

- Soil erosion.
- Pollutant loading.
- Aesthetic appearance.
- Odor.
- Taste.
- Chemical composition.

Note: Only the first two factors will be addressed in this chapter.

1. Soil Erosion. A surface water assessment or surface water analysis may be useful where there is concern that soil erosion will cause an adverse effect on sensitive surface waters. The scope of the assessment shall be determined by the environmental team and agreed upon by PennDOT through coordination with the applicable resource and regulatory agency(ies). Where highway projects are planned in areas of sensitive surface waters, the subjective BMPs of Chapter 12, *Erosion and Sediment Pollution Control*, and Chapter 14, *Post-Construction Stormwater Management*, will normally be sufficient to minimize any adverse impacts prior to resorting to more unconventional management measures.

Management measures requiring an analysis are addressed below in Subsection 11.B.3. The designer should initially try to identify whether the erosion source is caused by PennDOT's facility. Studies to identify sediment quantities expected to be delivered to any receiving surface waters should use the guidance in Chapter 12, *Erosion and Sediment Pollution Control*. A wide range of recurrence intervals are necessary in applying these procedures so that the significance and any thresholds may be identified.

2. Pollutant Loading. Pollutants, other than sediment, may originate from:

- Highway construction sites.
- In-situ hazardous material exposed by construction.
- Highway maintenance activities.
- Adjacent land use.
- Highway surfaces and storm drains.

a. Highway Construction Sites. Normally, BMPs will control pollutants that might originate at highway construction sites. Other management measures are discussed in Subsection 11.B.3.

b. Hazardous Materials Exposed By Construction. As a guide, many pollutants attach (adsorb) themselves to sediment particles. If the quantities of eroded sediment and the percent by weight of the exposed, hazardous pollutants subject to erosion at a site are known, the designer may be able to assemble a creditable estimate of the pollutant loading to any receiving waters. *Pollutant Loadings and Impacts from Highway Stormwater Runoff*, *Vol. I-VI*, (Driscoll, 1990) may be useful if the practices can be calibrated to the condition found at a particular site.

c. Highway Maintenance Activities. Pollutant sources from maintenance activities shall be controlled by similar BMPs as described in Chapter 12, *Erosion and Sediment Pollution Control*, for temporary maintenance conditions and include BMPs as described in Chapter 14, *Post-Construction Stormwater Management*, for longer duration maintenance procedures.

d. Adjacent Land Use. Highway rights-of-way and storm drains, because they often serve as the "delivery system," can result in the highway agency assuming responsibility for pollution caused by others with lands draining onto highway rights-of-way. When such issues arise, it may be necessary to perform a site investigation for pollutant source and if necessary to perform sample testing and employ computer models that are acceptable by the permitting agencies.

e. Highway Surfaces and Storm Drains. The technology provided in *Pollutant Loadings and Impacts from Highway Stormwater Runoff, Vol. I-VI*, (Driscoll, 1990) coupled with estimates of runoff from highway surfaces is useful for assessing potential pollutant loadings from highway surfaces and storm drains. For estimates of runoff from storm drains, see Chapter 7, *Hydrology*, for approved computer modeling methods.

3. Management Measures. The findings of *Managing Pollution from Highway Stormwater Runoff* (Maestri, et al., 1988) is the primary source for the majority of the material in this subsection.

a. General. Highway runoff pollution may affect the water quality of any receiving waters through:

- Shock and immediate (acute) loadings.
- Chronic effects due to prolonged accumulation.

Management techniques must recognize these effects and the characteristics of the commonly encountered pollutants.

Pollutants such as solids, heavy metals and organics (found in fuel and motor fuels) correlate directly with traffic volume. Other pollutants such as herbicides and nutrients are not a function of Annual Average Daily Traffic (AADT) because they commonly derive from such factors as highway maintenance activities and adjacent land use contributions. Therefore, management techniques for traffic or maintenance-related pollutants are different.

The ability of a pollutant to migrate to receiving waters from the right-of-way is a function of its:

- Chemical nature.
- Physical-chemical properties (e.g., water solubility, vapor pressure).
- Tendency to adsorb to organic matter or sediment.
- Interception by any mitigation or control measures.

Of the major migration processes, the key mechanism for transporting a pollutant originating on a highway right-of-way is a combination of adsorption and settling. This is due to many of the runoff constituents being in a settled, particulate form. This is further enhanced because soluble, organic chemicals and heavy metals tend to adsorb (see Table 11.1) to suspended sediments, which in turn will settle given sufficient time and a suitable environment. Once settled, these pollutants will be most commonly transformed through the biological action of:

- Degradation.
- Assimilation by microbes.
- Assimilation by rooted vegetation.

Settling management measures are complicated. This is due to most suspended loadings in highway runoff being associated with very fine material that has a relatively low settling velocity. If acceptable through coordination with the applicable resource and regulatory agency(ies), required mitigation should be designed as practicable to take advantage of the following highway practices and runoff characteristics:

- Conventional PennDOT Best Management Practices (BMPs) will commonly suffice for most highway activities.
- Nonpoint pollution discharges from frequent minor storms are more critical than discharges during infrequent major storms.

- First-flush conditions result in relatively high pollutant concentrations (analogous to raw sewage) during the initial, relatively low discharge stages of stormwater runoff that induces shock-loading and a short-term degradation of the water quality of receiving waters.
- Loadings of heavy metals and other toxic materials tend to be of greater concern than loadings of nutrients that have a high biological oxygen demand (BOD). Arsenic, cadmium, and chromates are examples of heavy metals that can cause death due to their inherent high levels of toxicity. Reference the Occupational Safety & Health Administration website (www.osha.gov) for more information on heavy metals.
- Critical pollutants such as heavy metals tend to appear primarily in suspended form.
- Measures designed for storms producing less than one inch of rainfall will control nonpoint pollution discharges for approximately 90% of the storms each year.
- Runoff from large, uncontrolled storm events tends to produce flows from non-urban areas that can dilute discharges from the paved, urban areas.

Table 11.1 Fate of Pollutants by Management Measures

Pollutant	Management Measures			
	Vegetative Controls	**Detention Basin**	**Infiltration Systems**	**Wetlands**
Heavy metals	Filtering	Adsorption Settling	Adsorption Filtration	Adsorption Settling
Toxic organics	Adsorption	Adsorption Settling Biodegradation Volatilization	Adsorption Biodegradation	Adsorption Settling Biodegradation Volatilization
Nutrients Solids	Bioassimilation Filtering	Bioassimilation Settling	Adsorption	Bioassimilation Adsorption Settling
Oil and Grease	Adsorption	Adsorption Settling	Adsorption	Adsorption Settling
BOD	Biodegradation	Biodegradation	Biodegradation	Biodegradation
Pathogens	Not applicable	Settling	Filtration	Not applicable

Source: *Managing Pollution from Highway Stormwater Runoff* (Maestri, et al., 1988).

Commonly, pollution management relies on controls for minor storms having a recurrence interval of one year or less. As such, management techniques that isolate first-flush discharges can take advantage of the smaller required storage capacities for these discharges. The larger storage capacities required when these facilities are perceived as being needed to treat all runoff flows need not be considered unless required for a specific application by the applicable resource and regulatory agency(ies).

First-flush effects are attributed primarily to the "washoff" of particles from paved areas. This means that, compared to more rural areas, the first-flush effects from urban areas tend to exhibit:

- Relatively high loadings of suspended sediments.
- A higher per-acre concentration of heavy metals (contributions from vehicular traffic).
- Less significant sources of nutrient loadings than unpaved areas.

b. Effectiveness of Management Measures. At this time, only qualitative ratings of management techniques can be offered. This is due to the variance in the design and management of these measures and the intangible site-specific factors that determine the runoff characteristics and pollutant loads.

Notably, these measures may require additional right-of-way for construction and maintenance. Because in many cases there is a correlation between mitigation and high ADT (i.e. pollutant loadings generally increase as ADT increases) which, in turn, commonly occurs near urban areas, any additional right-of-way may be very costly. As such, cooperative stormwater management agreements with local governments to share the benefits and cost are encouraged. The applicability and cost of pollution management measures are functions of the highway configuration.

These subjective ratings and the applicability of various pollution management measures are further discussed in the BMP design shown in Chapter 12, *Erosion and Sediment Pollution Control*, and Chapter 14, *Post-Construction Stormwater Management*, and are quantified in Chapter 12, *Erosion and Sediment Pollution Control*, Table 12.5

Effective management measures, in order of priority, are discussed below. This prioritization is based on:

- Relative effectiveness.
- Adaptability to highway design.
- Right-of-way requirements.
- Ease of operation.
- Maintenance requirements.

(1) Best Management Practices. These are measures commonly used by PennDOT that are proven to be effective at reducing the effects of roadway construction on surface waters. These measures are discussed in detail in Chapter 12, *Erosion and Sediment Pollution Control*, and in Chapter 14, *Post-Construction Stormwater Management*. The BMPs shown and described in Chapter 12, *Erosion and Sediment Pollution Control*, generally cover both temporary and permanent measures that are used for protection during and for post-construction; this section is mostly concerned with the permanent measures.

(2) Vegetative Filters. Of all of the described BMPs, vegetative filters are PennDOT's preferred control measure and are described in depth in Chapter 12, *Erosion and Sediment Pollution Control*.

(3) Extended Dry Detention Basin/Wet Retention Basin. The purpose of these types of basins is to increase runoff residence time sufficient to remove settleable pollutants to acceptable levels of concentration before they can be released to the receiving body of water. General guidelines for the design are described in Chapter 14, *Post-Construction Stormwater Management*.

Wetlands, a subset of wet retention ponds, can provide a highly effective management measure for mitigation pollution from highway runoff because they have the ability to assimilate large quantities of suspended and dissolved materials from inflow. However, at this time, the design of wetland management measures is very complex and poorly defined. The primary differences to be considered in the design are that wetlands:

- Remove pollutants primarily through sedimentation and vegetative uptake.
- Use vegetation as pollutant removal mechanism.
- Require low flow-through velocities.
- Depend upon well spread "sheet flow."

Wetlands are used in combination with vegetative filters and/or detention, not with infiltration. Typically, wetlands should receive inflow from vegetated conveyance facilities and/or a wet detention pond. Any planned discharge from a wetland is routed through a vegetated conveyance facility. Wetlands cannot effectively precede an infiltration system as accumulated sediment and/or decaying matter may clog the infiltration mechanism. It must be recognized that conditions favorable to wetlands (high water table, impervious soils) are unfavorable for infiltration measures. It should also be noted that natural wetlands are not recommended as a control measure. Constructed wetlands tend to provide a higher level of pollutant removal and stormwater control function.

(4) Infiltration System. These are permanent facilities where runoff is temporarily stored until it can infiltrate the ground. Although infiltration systems are typically designed to control stormwater runoff or recharge groundwater resources, they do have the ability to reduce pollutant loads in runoff. General guidelines for the design are described in Chapter 14, *Post-Construction Stormwater Management*.

(5) Combined Systems. When practicable and effective, management costs can be reduced through the use of combinations of measures. Combinations of measures may:

- Increase pollutant removal effectiveness.
- Allow for filtration of suspended solids.
- Overcome any site-limiting factors.

Examples of combination systems include:

- Use of infiltration wells in a detention basin to increase pollutant removal while concurrently decreasing the long-term storage requirements.
- Use of overland flow through vegetative filters (strips and channels) and/or wetlands to filter suspended sediments from upstream runoff before it reaches an infiltration basin or trench.

(6) Water Quality Inlet. These inlets are designed to remove sediment and hydrocarbon loadings before they are conveyed through a storm drain or into an infiltration facility. These facilities have a high initial cost and must be discussed with PennDOT prior to design.

(7) Miscellaneous Measures. Seven relatively effective, low-cost management measures applicable to most sites are suggested at this time:

- Curb Elimination. Omitting curbs or providing discontinuous curbs (periodic gaps) encourages the transport of pollutants off the roadway and/or the migration of pollutants into vegetated roadside areas. Gaps must be consistent with essential traffic control and highway safety requirements.
- Litter Control. Litter control programs will, as a secondary purpose, achieve pollutant reduction benefits through the elimination of pollution sources.
- Pesticide/Herbicide Management. Such factors as limited application, strict controls, employee training and close supervision will minimize pollution.
- Reduction of Direct Discharges. Avoid the direct discharge of highway runoff into receiving waters or ground waters by using effective management measure(s).
- Reduction of Runoff Velocity. Encourage bed-load deposition by lowering velocities through gradient reduction using drop structures and/or baffles and by providing heavily vegetated waterways.
- Establish and Maintain Vegetation. Dense, vegetal cover and limited mowing with no grass removal provides pollutant reduction through filtration, sediment deposition, infiltration and, in some cases, biological assimilation of pollutants by the vegetation.

C. Channels. Chapter 8, *Open Channels*, addresses the hydraulic design of channels. Desirable environmental functions and values of channels depend on a number of factors, including:

- Terrestrial habitat.
- Aquatic habitat.
- Riparian habitat.
- Flood conveyance.
- Flood storage.
- Recreational uses.
- Agricultural and silvicultural uses.
- Municipal uses.

Channel stability mitigation measures, where cost effective, can be employed as set forth in Chapter 8, *Open Channels*, and *Model Drainage Manual*, Chapter 17 (AASHTO, 2005).

D. Lakes or Ponds. Guidance for lakes and ponds is limited. For now, such designs are to be established on a case basis due to the unique needs of these surface waters and the relatively infrequent disturbance by highways.

E. Wetlands. Wetlands are an important natural resource. Current Federal and State rules and regulations mandate that there be no net loss of wetlands area or functional values. The regulatory wetland classification is as set forth in Section 11.0.D. More details about the classification of wetlands can be found in Chapter 10 of *Highway Drainage Guidelines* (AASHTO, 2003).

1. Wetland Assessment. Quantitative wetland functional values can be estimated using the guidelines as shown in Publication 325, *Wetlands Resource Handbook*. Functional values related to hydrology or hydraulics of wetlands can be estimated using the practices described in Section 11.6.

2. Wetland Mitigation. Mitigation measures are those actions that reduce or eliminate the adverse effects of a proposed action to acceptable levels. At the same time, they may compensate for the affected area for project-related losses. Five ways to mitigate losses are listed below in order of priority for regulatory purposes:

- Avoid the effect altogether by not taking a certain action or part of an action.
- Minimize the effect by limiting the degree or magnitude of the action or its implementation.
- Rectify the effect by repairing, rehabilitating or restoring the affected environment.
- Reduce or eliminate the effect over time by preservation and maintenance operations during the life of the action.
- Compensate for the effect by replacing or providing substitute resources or environments.

The 1990 Memorandum of Agreement (MOA) between USACE and USEPA and *A Guide to Functional Wetland Design* (Marble, 1990) should be reviewed regarding mitigation alternatives before proceeding with a highway action. When it is determined that the mitigation measures will include either restoration of existing damaged wetlands or the creation of new wetlands, the procedures outlined in Publication 325, *Wetlands Resource Handbook* should be followed.

3. Wetland Banking. Wetland banking is a variant of compensatory mitigation and requires some type of agreement between PennDOT and the applicable resource and regulatory agencies. There are generally five essential considerations in a wetlands banking program:

- The highway agency provides replacement wetlands on a project even though none are required.
- These nonessential replacement wetlands are considered "bank credits" available for use on future highway projects.
- Bank credits are used to mitigate wetlands disturbances on future highway projects.
- Credit value is generally finalized at the time it is used. In many cases, however, credits are established or finalized once the wetland is established.
- Compensation wetlands bank credits, when used, are generally in the same geographic or hydrologic region of the wetlands to be disturbed by a future project.

The designer is cautioned that any wetland construction or enhancement for wetland banking purposes must be carefully considered in light of future reconstruction of a proposed project. Proposed wetland bank construction or enhancement must not be such as to significantly curtail or complicate future highway improvements and/or maintenance. The designer should also understand that wetland banking is independent of individual roadway projects. For example, two separate and concurrent roadway projects may disturb wetlands. These disturbances may be accounted for through one wetland banking program.

F. Fish Passage Criteria. There are three primary highway drainage facilities where fish passage is a consideration:

- Channels.

- Bridges.
- Culverts.

1. Channels. Environmental channel design will be only briefly addressed in this chapter. The primary considerations are aquatic habitat and channel stability. Channel stability is addressed in detail in Chapter 8, *Open Channels*. Mitigation for stability is addressed in *Model Drainage Manual,* Chapter 17 (AASHTO, 2005). The designer should refer to the Publication 13M, Design Manual, Part 2, *Highway Design,* Section 10.10.

2. Bridges. Bridges are considered as part of the channel and only given brief mention in this chapter. Where practicable, they should span the ordinary high water channel. However, piers within the ordinary high water channel of larger river channels are acceptable, if not desirable, because they commonly provide resting areas.

3. Culverts. Fish passage is historically the primary concern with the culvert type drainage facilities. Failure to consider fish passage may block or impede upstream fish movements in many ways:

- Outlet of the culvert is installed above the streambed elevation to where fish may not be able to enter.
- Scour lowers the streambed downstream of the culvert outfall and the resulting dropoff or perch creates a potential vertical barrier.
- High outlet velocity may provide a barrier.
- Higher velocities occurring within the culvert than in the natural channel may prevent fish from transiting the culvert.
- Abrupt drawdown, turbulence and accelerating flow at the culvert inlet may prevent fish from exiting the culvert.
- Natural channel is replaced by an artificial channel that may have fewer zones of quiescent water in which fish can rest.
- Debris barriers (including ice) upstream or within the culvert may stop fish movement.
- Shallow depths within the culvert during minimum flow periods may preclude fish passage.

Generally, maintaining subcritical flow throughout the culvert facility will result in successful fish passage.

G. Fish Passage Culverts. The most common fish passage concern with a highway action is usually associated with culverts. The designer shall refer to the Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Section 10.11 for the acceptable design procedures concerning fishable streams within the Commonwealth.

H. Stream Geometry and Cover. Channel disturbances are mitigated by employing those geometries necessary to restore and, where demonstratively appropriate and practicable, upgrade the functional values of a channel. Where mitigation is required, channels are designed and constructed to provide such functions as:

- Riparian cover.
- Instream cover.
- Pools.
- Riffles.
- Water quality.
- Substrate.
- Bank geometry.
- Conveyance for the selected design flood.

With these types of channels, the selected design flood is that used to size the channel and any appurtenant culverts or bridges to avoid a flood hazard to the highway user and any property owners.

Practicable construction practices used to obtain the foregoing geometries and appurtenant features are shown in Chapter 8, *Open Channels*, and Chapter 12, *Erosion and Sediment Pollution Control*. These chapters show PennDOT's preferred geometries and controls used in mitigating channel disturbances. The use of these or other geometries must be coordinated with the applicable resource and regulatory agency(ies) on a case basis. Aggressive

revegetation and soil stabilization practices associated with these geometries are needed to restore and/or where mandated by the applicable resource and regulatory agency(ies), enhance a disturbed stream's:

- Riparian cover.
- Floodplain vegetation.
- Other aquatic functional values.

Flood hazards must not be overlooked when designing an environmentally acceptable channel modification. To avoid flood hazards, appropriate channel roughness values are used in the hydraulic design of channels where instream habitat and/or riparian cover devices are employed. Based upon the selected instream and riparian geometries, select hydraulic channel design criteria and coefficients for such elements as:

- Friction and bendway losses.
- Sediment transport.
- Stream stability.
- Scour.
- Aggradation.

The design of a stable channel to serve the environmental needs set forth in this chapter is addressed in Chapter 8, *Open Channels*. Essentially, the mitigation geometries of Chapter 8, *Open Channels*, channel design become appurtenant channel features in the stable channel analysis and are accounted for in the:

- Manning's n-values selected for the channel analysis.
- Hydraulic channel and structure analyses.
- Stability investigations.

11.4 DESIGN PROCEDURES

A. Design Steps. Design procedures routinely involve some or all of three general steps:

- Acquisition of the design data*.
- Selection of the design fish*.
- Hydraulic analysis.

*Note: This may be accomplished through coordination with the applicable resource and regulatory agency(ies).

In this section, design procedures are provided to illustrate the foregoing steps.

- Water quality control measures.
- Environmental channels.
- Culvert fish passage.

Other procedures and examples may be included in this chapter at a future date as they become available.

B. Design Data. Design, at a minimum, requires two types of data:

- Site-specific information.
- Surface water information.

Acceptable criteria for this design data should be coordinated with any and all applicable resource and regulatory agency(ies).

On rare occasions, the acquisition of some of the following data may require a controlled monitoring program. Such programs may have sufficient duration to provide unbiased data to identify any changes due to natural events and man-made activities such as:

- High runoff periods.
- Annual climatic changes (seasons).
- Practices relating to agriculture, silviculture and development.

1. Site-Specific Data. Hydraulic design data commonly obtained for each site as set forth in the other chapters of this manual will usually suffice. Additional data may be required for the:

- Design of water quality control measure(s).
- Special conditions for fish passage analysis

Water quality analysis addressed by this manual focuses primarily on surface waters that may receive runoff from the highway right-of-way and thereby incur:

- Significant degradation.
- Temporary degradation.
- Preservation needs.

Site-specific data other than that required in the other manual chapters might be required for some or all of the following:

- Erosion predicting variables for the soil such as allowable velocities, tractive shear and Manning's n-values.
- Hydrology and hydraulic variables associated with any selected vegetative measures such as daily hydrograph (24-hour clock time vs. discharge), annual hydrograph, allowable construction slopes, flow velocities, tractive shear, vegetal growth characteristics, and Manning's n-values.
- Vegetative area requirements to obtain acceptable rates of sedimentation and/or pollutant uptake.
- Surface and subsurface soil and geology.
- Surface and subsurface infiltration rates.
- Rainfall-frequency-duration relationships.
- Gradation curve for sediment subject to erosion, transport and deposition.
- Topography, area and location proposed for any pollutant management measures.
- Climate related erosion factors.
- Source and amount of natural and/or man-made pollutants.
- Hydrology and hydraulic characteristics of any pollutants such as solubility, adherence to sediments and settling velocities.
- Seasonal groundwater elevations and water quantity.
- Required multiple uses for basins.
- Other measures.

With fishery-related analyses, it may be desirable to have data to define the site's:

- Annual (daily peak) hydrographs.
- Daily (24-hour) hydrographs.

2. Surface Water Data. Water quality analyses for receiving surface waters may also require data for some or all of the following:

- Hydraulic characteristics.
- Hydrology.
- Fish design.
- Other.

a. Hydraulic Characteristics. The characteristics include such factors as relationships for:

- Stage-discharge.
- Stage-velocity.

- Stage-recurrence interval.

b. Hydrology. Additional data may consist of such factors as the:

- Annual (daily) hydrographs.
- Daily (hour) hydrographs.

c. Fish Design. It may be necessary for special design conditions, only if required by PFBC under special site conditions, to determine the design fish:

- Run type(s).
- Species that migrate and/or inhabit the stream.
- Minimum swimming depths.
- Swimming speeds.
- Jumping or burst speeds.
- Jump heights.
- Migration delay durations.
- Probable hourly run size.
- Water volume requirements.
- Vertical ascent time.
- Instream, riparian and floodplain habitat needs.
- Approximate date and times of migration.
- Fish size (fork length and total length).

Depending on the run type, this information may be needed for the entire fish population of the stream including both resident and migratory species.

d. Other. In addition to the foregoing data, the designer may need to negotiate design data for the following aquatic and floodplain-related investigations:

- Material requirements for spawning bed restoration or preservation.
- Variances for tolerable but temporary pollutant concentrations for the pre-construction, construction and post-construction periods - primarily for turbidity and sediment, but on rare occasions possibly other pollutants.
- Instream and riparian habitat requirements for water fowl and other surface water inhabitants.
- Habitat requirements for terrestrial (floodplain dwelling) wildlife.
- Domestic animal requirements and their impact on surface waters*.
- Wetland requirements such as cover, water depths and pond geometries.

*Note: In many riparian localities, it is the domestic livestock that are the prime source of bank deterioration and other environmentally damaging actions.

C. Water Quality Measures. *Managing Pollution from Highway Stormwater Runoff* (Maestri, et al., 1988) is the principal source for these procedures.

Procedures for accomplishing mitigation using four measures are provided:

- Vegetative.
- Wet detention basin.
- Wetland.
- Infiltration.

1. Vegetative Measures. Vegetative design procedures to reduce the migration of pollutants off the highway right-of-way are considered as essentially the same for:

- Vegetated channels.

- Overland (sheet) flow.

The essential design criteria are:

- Provide sufficient vegetative cover to remove pollutants.
- Ensure the stability of the selected vegetation.
- Provide for ongoing maintenance.

Considerations in selecting these design criteria are:

- Topography.
- Soils.
- Area required.
- Climate.
- Erosion potential.

The design steps are summarized as follows:

Step 1 Estimate the expected runoff flow rates for a range of the design runoff events; see Chapter 7, *Hydrology*.

Step 2 Determine the most critical slope of a proposed channel or overland flow strip. Note, with tractive shear practices, there are two variables to consider—slope and flow depth. As such, either a very steep or very flat slope should be considered.

Step 3 The designer will:

- Select an acceptable vegetative cover.
- Select an acceptable armor lining if needed.
- Determine the amount of cover needed for pollutant removal.
- Determine acceptable locations and configurations (channel, strips) for the vegetative cover and any armor lining.

Step 4 From the Chapter 8, *Open Channels*, select appropriate allowable values for:

- Manning's n-value.
- Critical velocity threshold.
- Critical tractive shear threshold.

Step 5 For the proposed steepest and flattest slope and a range of recurrence intervals, determine the expected channel and/or overland:

- Flow depths,
- Maximum (not average) velocity.
- Maximum (not average) tractive shear.

Step 6 Compare the expected range of hydraulic variables of Step 5 with the allowable variable of Step 4 for a range of recurrence intervals. If velocity or depth criteria are exceeded, upgrade the protection or modify the design. Repeat the foregoing steps.

2. Detention Basin Measure. This procedure is applicable for only wet detention basins. The designer should refer to Chapter 14, *Post-Construction Stormwater Management*, for design criteria for wet detention basins.

The following is an adaptation of the Driscoll method for estimating pollutant removal rates as recommended in *Managing Pollution from Highway Stormwater Runoff* (Maestri, et al., 1988). The method can be used to estimate either the:

- Long-term efficiency of an existing wet basin.
- Dimensions of a proposed wet basin.

This method assumes that the design will ensure that the following design concepts will be met:

- Permanent pool in the detention basin.
- Infiltration from the retained pool will increase mitigation performance under both dynamic and quiescent conditions.

The design steps are summarized as follows:

Step 1 Determine an acceptable location for a wet detention basin(s). For the proposed highway geometry, determine the hydrology for a range of recurrence intervals to include the base 100-year event; i.e., estimate the:

- Flood-frequency relationship.
- Flood hydrographs.

Step 2 Determine the expected size distribution of sediment particulates and the distribution of pollutants in the runoff for a range of recurrence intervals from either:

- Field discharge tests.
- Field sampling.
- Computer simulations.

Step 3 Estimate the settling velocity versus time and depth relationship of the smallest expected particulates to be removed from either:

- Laboratory tests.
- Selection of the fall velocity.
- A current hydraulic text.

Step 4 For the selected range of recurrence intervals:

- Estimate the dynamic removal efficiency.
- Estimate the quiescent performance.
- Plot the percentage of total suspended solids removed versus basin surface area.

Determine the dimensions of a proposed wet basin necessary to meet the following criteria:

- Achieve the desired removal rates for a selected recurrence interval coordinated by PennDOT's environmental team with all applicable resource and regulatory agency(ies) using the foregoing plot as a guide.
- Minimize the potential for the inadvertent discharge of pollutants (sometimes termed "short circuiting").
- Have basin bank slopes of 1V:3H or flatter to maintain a good vegetative cover where practicable.

Step 5 (Optional) If the basin is to be used for flood peak attenuation in addition to mitigation, use Chapter 14, *Post-Construction Stormwater Management*, to either:

- Verify that the dimensions from Step 4 are adequate.
- Determine new dimensions.

New dimensions must be determined as being compatible with both flood peak attenuation and pollution control. This is done by repeating the foregoing steps.

3. Wetland Measure. The design of a wetland to provide acceptable levels of pollution control is presently beyond the scope of this manual.

4. Infiltration Measure. The design of infiltration measures to provide acceptable levels of pollution control is presently beyond the scope of this manual.

D. Fish Passage Analyses Type. The designer shall refer to Design Manual Part 2, Chapter 10, Section 10.10, and Section 10.11, for the acceptable design procedures concerning fishable streams within the Commonwealth.

11.5 GROUNDWATER

A. Occurrence. Study of groundwater is interdisciplinary in nature. It is relevant to geologist, hydrologists, geotechnical engineers, soil sciences, agricultural engineers, foresters, geographers, ecologists, mining engineers, agriculture engineers, sanitary engineers, and probably others. The term "groundwater" is usually reserved for the subsurface water that occurs beneath the water table in soils and geologic formation that are fully saturated. In here only subsurface flow processes important to surface water environment are described. The broader field of groundwater flow is covered in a number of text books (Freeze and Cherry, 1979; de Marsily, 1986).

Geological environments control the occurrence of groundwater. The natural distribution of aquifer and aquitards in a geologic system are controlled by the lithology, stratigraphy and structure of the geological deposits and formations. The lithology is the physical makeup, including the mineral composition, grain size and grain packing, of the sediments or rocks that make up the geological systems. The stratigraphy describes the geometrical and age relations between the various lenses, beds, and formation in geological systems of sedimentary origin. Structure features, such as faults, fractures, are the geometrical properties of the geological system produced by deformation after deposition and crystallization. In most regions knowledge of the lithology, stratigraphy and structure leads directly to an understanding of the distribution of aquifer and aquitards. Freeze and Cherry (1979) provides detailed discussion of the groundwater geology.

Groundwater is derived from precipitation that falls on the earth's surface. The water that accumulates on the surface in a river, or becomes overland flow on the land is called surface water, and water that leaks into the ground is called groundwater. Groundwater that flows out of the ground to the surface as springs or as drainage into a stream channel becomes surface water. For water to collect underground there must be a recharge area, a unit of land surface, where the precipitation can infiltrate and thus charge or fill the storage space in the rocks and soils.

Infiltration is the process of water penetrating from the ground surface into the soil. Many factors influence the infiltration rate, including the condition of the soil surface and its vegetative cover, the properties of the soil, such as its porosity and hydraulic conductivity, and the current moisture content of the soil. Soil strata with different physical properties may overlay each other, forming horizons; for example a silt layer with relatively high hydraulic conductivity may overlay a clay layer of low hydraulic conductivity. In addition, soils exhibit great spatial variability within relatively small areas. Due to the great spatial variability and the time variation in soil properties, infiltration is a very complex process that can be described only approximately with mathematical equations. Infiltration tests are a field procedure used to determine the infiltration rate of soils.

B. Groundwater Movement. In Pennsylvania, nearly all water entering the groundwater system flow from hilltops to the nearest stream. Because of the large number of streams in Pennsylvania, shallow groundwater is the major groundwater system. Almost all shallow groundwater in Pennsylvania reaches a stream within days, weeks, or months after the water enters the groundwater system.

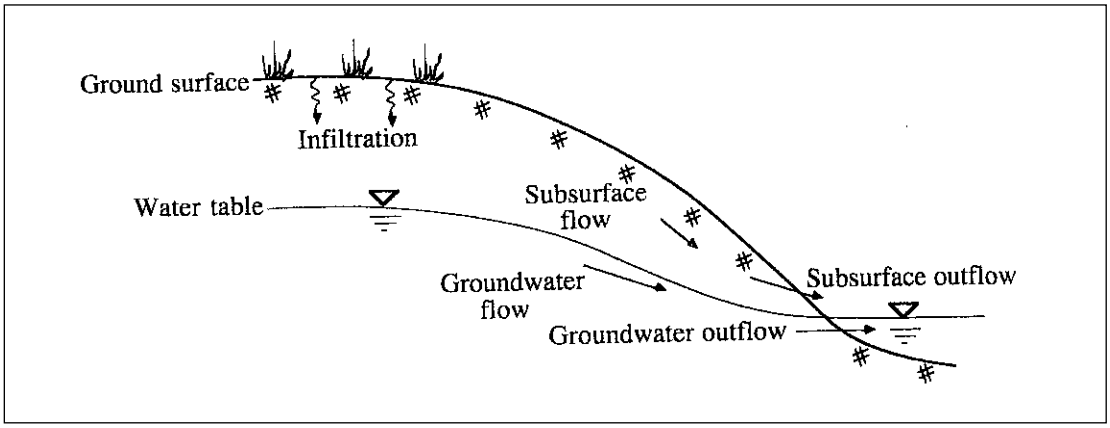
Groundwater flows due to gravity. Groundwater always flows downward from recharge areas on the hills toward discharge areas in the valleys. In addition, groundwater moves in response to difference in head and is retarded by its own viscosity. For highway and bridge projects within water supply and wellhead protection areas, understanding the movement of groundwater is essential to preventing threats to public water supplies.

Subsurface flow processes and the zones in which they occur are shown schematically in Figure 11.1. Three important processes are infiltration of surface water into the soil to become soil moisture, subsurface flow or unsaturated flow through the soil, and groundwater flow or saturated flow through soil or rock strata. Porous media

are layers of soil and rock that permit water flow. The flow is unsaturated when the porous medium still has some air-occupied voids and is saturated when the voids are filled with water. The water table is the surface where the water in a saturated porous medium is at atmospheric pressure. Below the water table, the porous medium is saturated at greater pressure than atmospheric. Above the water table capillary forces can saturate the porous medium for a short distance in the capillary fringe, about which the porous medium is usually unsaturated except after rainfall when infiltration from the land surface can produce temporary saturated conditions. Subsurface and groundwater outflow occur when subsurface water emerges to become surface flow in a stream or spring. Soil moisture is extracted by evapotranspiration as the soil dries out.

The groundwater body receives rainwater or snowmelt that has percolated through the unsaturated zone. This recharge tends to increase the amount of groundwater in storage, and therefore tends to raise the water table. As the water table rises, it causes water to drain out more quickly to streams. Such drainage then acts to lower the elevation of the water table.

Figure 11.1 Subsurface Water Zones and Processes



The difference in elevation and pressure between two points controls the speed at which groundwater flows. The greater the combined difference in elevation and pressure between two points, the faster the groundwater will flow between those points. The rocks and sediments containing the water also control of the rate of flow. As water flows through a deposit of sediments or a rock, friction between the water the sediment grains or rock slows the water's movement. The smaller the pores, openings and fractures between rock layers, the slower the water flows. If the pores are not connected to each other, or the fractures and openings do not intersect each other, the groundwater cannot flow. However, water still can be stored in those void spaces. Permeability is usually used to measure the ease of water flowing through rocks and sedimentary layers. Permeability is dependent on the size of the pores, fractures, and openings between rock layers, and on the degree to which the pores are connected and the fractures and openings intersect. Compared to the surface water, most groundwater flows very slowly (on the order of feet per day).

Sinkholes are a natural part of Pennsylvania's landscape and are considered a geologic hazard in the central and eastern parts of the state. They occur in the limestone and dolomite bedrock areas of the state, and are greatly influenced by the presence and absence of groundwater. Introduction or withdrawal of excess water in a limestone or dolomite bedrock can speed the development of sinkholes. Therefore, proper management of roadway drainage is essential to prevent the formation of sinkholes in the highway environment. From PennDOT's perspective, sinkhole collapse or gradual subsidence can cause structural damage and instability in roads and bridges. Repairing structures after subsidence is difficult and expensive, and requires specialized knowledge from engineers and contractors. Even if a repair appears successful, it may not be permanent, as sinkhole formation can reoccur if the cause of its formation is not properly addressed. Sinkholes affect stream quality, groundwater quality and public health and safe and are affected by the interaction between surface runoff and groundwater.

C. Loss of Groundwater. Loss of groundwater occurs when impervious surfaces are constructed and stormwater is released to a stream or river instead of seeping into the ground. Precipitation soaks into the soil, infiltrates into the subsurface and travels through the cracks and openings of rocks to either be stored as groundwater or emerge as seepage or spring flow in a nearby wetland or stream. Construction of highways and bridges normally involves

covering portions of a recharge area with impervious paving, which, in turn, reduces groundwater recharges and increases surface runoff. As a watershed's imperviousness increases, dry-weather stream flow becomes increasingly depleted. Recharge and inflow depletion alone will severely degrade a headwater stream as well as any affected wetlands.

Detention basins work as a stormwater management tool by storing the stormwater to control the peak flow and timing of stormwater flowing downstream. However, a portion of the stormwater which ordinarily infiltrates into subsurface is retained and routed to a surface water body, resulting in reduced infiltration.

When working in high water table areas, it is typical for a highway project to affect the storage and flow of groundwater by the project directly cutting into the aquifer (typically unconfined). When piles or caissons are driven into the ground, the groundwater must adjust to the new interference and, as a result, will find a new route to the previously existing surface discharge location (stream), or it will discharge at a new location altogether. Storage can be disrupted if the project takes place in a groundwater recharge area. The recharge area is typically defined by Hydrologic Soils Group A and B soils that have a higher permeability and are more susceptible to infiltration, whereas Group C and D soils are less permeable and encourage more surface runoff than infiltration.

Another way in which groundwater is adversely affected is through the loss of forest land. Forested areas slow the movement of surface runoff, which enables more permeable soils associated with these lands to infiltrate runoff. When a project requires the removal of forest land, a decrease in groundwater recharge will occur with a corresponding increase in surface runoff. Other degrading impacts can occur from the removal of forest lands, such as more erosion due to the increased surface flows, and increased sedimentation in streams and lakes.

D. Infiltration. Infiltration of stormwater runoff can often be an appealing alternative to more traditional stormwater control methods such as detention basins and wet ponds in that it promotes groundwater recharge. The concept of groundwater recharge is an important consideration in areas where aquifers have been depleted due to pumping, or in more urbanized areas where impervious cover has greatly reduced the quantity of surface water infiltration into the ground. This reduction has a compound effect on the hydrologic system in that while reducing infiltration of surface water depletes underground aquifers, the base flow from groundwater movement that feeds streams in periods of low flow is reduced.

1. Regulations, Ordinances, and Considerations. When considering the practice of groundwater infiltration techniques for stormwater runoff management, there are always regulations and ordinances that specify design criteria and restrictions based on zoning and land use characteristics, etc. Each level of government has regulations that pertain to ground requirements. The Commonwealth of Pennsylvania through its Act 167 of 1978 has embodied many municipalities with groundwater infiltration criteria based on management districts determined by local flow rates in relation to location within a particular watershed of study. If the site is located within High Quality or Exceptional Value waters, then infiltration requirements are dictated by the Department of Environmental Protection Chapter 93 Anti-degradation Regulations.

For further guidance on regulations, ordinances, and considerations, please refer to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 13.

2. Infiltration Ideas and Site Considerations. The use of infiltration practices depends on careful site investigation. Should a site investigation reveal conditions that are not feasible for the proper function of an infiltration basin or other infiltration measures, then the implementation of infiltration as a method managing of excess runoff volume should not be pursued. Before infiltration is considered as the best option for site stormwater management, general site data should be collected, soils should be sampled and analyzed, and the surrounding land use and land cover should be assessed.

When assessing a site for infiltration purposes, data should be collected in a manner consistent with that for the recommendation and design of any stormwater management facility. Considerations that should be taken into account in general deal with the hydrologic and natural elements (are wetlands present?), the size and shape of the site (terrain and slope?), and existing developed portions and features of the site. Extra investigation should take place for infiltration facility designs concerning the hydrologic characteristics, specifically subsurface features. Three important items that should be addressed are:

- Is the site in a location that was previously mined - is there potential for acid mine drainage?

- Is the site in a region underlain by carbonate or limestone, where slightly acidic (pH < 7) rainfall runoff can lead to the dissolution of the carbonate material and karstification (cumulative effect of dissolution resulting in cavern formation)?
- Is the site underlain by either a naturally high water table or shallow depth bedrock that would inhibit the ability to infiltrate surface water?

The soil parameter that is a direct measure of the infiltration potential of a soil is permeability, or the hydraulic conductivity, K. In conjunction with performing tests such as the single or double ring infiltrometer test or hydraulic conductivity test, a more general understanding of the regional soils should be investigated. Soils are classified into hydrologic soils groups A, B, C, and D. Each group has different hydrologic properties with groups A and B having the highest potential for infiltration. Maps are readily available online and from Soil Surveys performed by the United States Department of Agriculture (USDA), Soil Conservation Service that show the location of groups of soils, from which hydrologic soils groups can be determined. These are broad scale maps that show regional relationships, and the hydrologic soils grouping shown by these maps are suitable for preliminary assessment of infiltration.

Another characteristic of a site that dictates whether infiltration is used as a method for stormwater management is the existing land uses and land cover at and adjacent to the site. For sites that are highly urbanized, sometimes the only option for stormwater management is an infiltration facility assuming hydrologic and soils characteristics permit. For sites that are abandoned mine lands, infiltration is not desirable as pollutants can more easily enter the groundwater flow regimes through increased surface water infiltration.

3. Infiltration Volume, Rate and Time. For further guidance on infiltration volume, rate and time, please refer to Chapter 14, *Post-Construction Stormwater Management*.

4. Work in Water Supply / Wellhead Protection Areas. Subsurface infiltration facilities are generally not recommended in areas that are adjacent to water supply wells or wellhead protection areas as these areas often do not allow for the design of percolation through a soil/compost mixture. One option when considering the design of stormwater management facilities in these areas is to disperse stormwater runoff to scattered facilities throughout the site rather than to concentrate stormwater flows to one or two locations. Other options that should be considered are pretreatment techniques that remove pollutants prior to entering the stormwater facility.

Nearly half of Pennsylvania's twelve million residents get drinking water directly from groundwater sources. Contaminants can present a serious threat to groundwater used as a public source of water, and transportation routes are often responsible for the release of contaminants through both point source pollution (contaminants originate from a specific location) and non-point source pollution (contaminants that cannot be traced back to a single source, rather are carried into the flow regimes from a broad area). Sources of pollution from roadways are often oils, antifreeze, other leaking chemicals from vehicles, accidental spills and releases, and deicing agents, specifically calcium magnesium acetate (CMA). It is critical that the infiltration BMPs be designed to remove pollutants from stormwater that is to be infiltrated in close proximity to public or private water supply wells and be sufficiently isolated from groundwater supply sources. When a transportation project is planned in the vicinity of a wellhead protection area, controls should be placed to minimize the release of contaminants to the source water based on the location of the project in relation to the PA DEP defined wellhead protection zone.

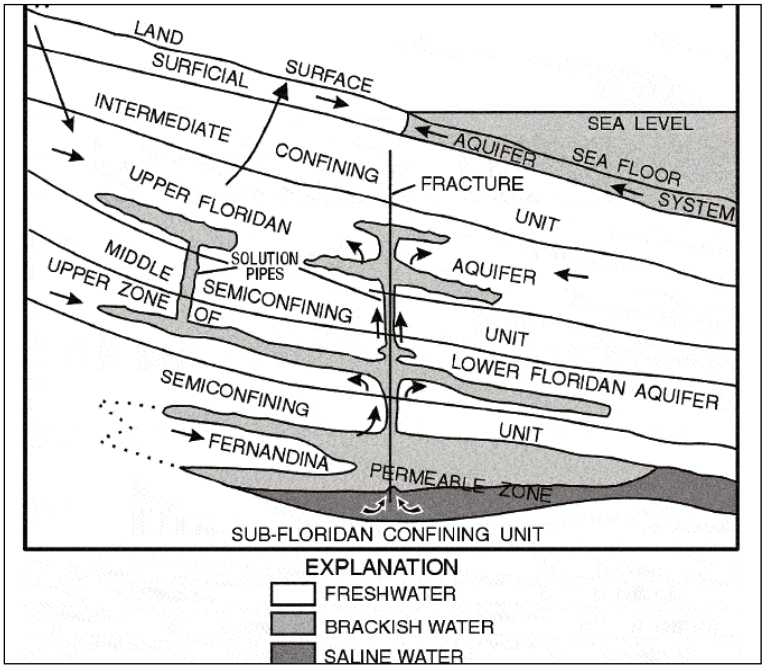
A wellhead protection area is defined by the Pennsylvania's Safe Drinking Water Regulation (25 PA Code § 109) as the surface and subsurface area in the vicinity of a water well or well field that supplies a public water system, through which contaminants are reasonably likely to move toward and reach the water well or well field. In order to more effectively monitor activity in the well vicinity the PA DEP Safe Drinking Water Regulations designate a wellhead protection area into three specific zones with unique regulatory measures; Zone I, Zone II, and Zone III. Zone I is defined as the area immediately surrounding the well which is between 30 and 120 m (100 and 400 ft) depending on site and/or aquifer characteristics, and is the most protected of the three zones. Zone II is characterized by the area in which water flows through the aquifer towards a well or spring, and is by default 800 m (1/2 mile) in radius unless a hydrogeologic delineation is performed. Zone III is the area beyond Zone II that still contributes a significant amount of surface and/or groundwater flow to the interior two zones. For more detailed information about protecting underground drinking water supplies, please refer to PA DEP's Source Water Protection Program.

5. Work Near Surface Water Supplies and Special Protection Waters. Pennsylvania also employs a three-tiered approach for surface water source protection. Zone A is a 400 m (.25 mile) buffer on either side of the river or stream extending from the area 400 m (.25 mile) downstream of the intake and upstream to the five hour time-of-travel (TOT). Zone B is a 3.2 km (two miles) buffer on either side of the water body extending from the area 400 m (.25 mile) downstream of the intake to a point upstream to the 25 hour TOT. Zone C constitutes the remainder of the basin.

Care must be taken when planning stormwater infiltration BMPs for use in areas within 3.2 km (two miles) on either side of special protection water or surface water used for public water supply. Infiltration BMPs in these areas must be designed to encourage maximum pollutant removal before the stormwater is infiltrated into the ground or discharge to a receiving stream. Please refer to PA DEP's Source Water Protection Program for more information.

6. Work In Coastal Areas: Saline Water Intrusion in Aquifiers. Saline water intrusion into aquifers can pose a threat to potable water sources as the salinity of the groundwater increases through time as more fresh water is pumped out allowing more saline water to intrude. Although the geologic gradient exists to route freshwater towards the sea (i.e. the land surface slopes towards the sea), because saline water is heavier than fresh water, saline water can intrude below the freshwater as it is forced landwards underneath the freshwater. With sea levels rising, this concern is exacerbated through an increased saline pressure in coastal areas responsible for forcing more saline water into fresh water aquifers. The concern is that an introduction of saline water will result in the production of chloride, which is toxic in nature and serves as the primary constituent for the contamination of coastal groundwater. There are several mechanisms that explain how saltwater can move into terrestrial aquifers (Figure 11.2). One is through the movement of unflushed pockets of stagnant seawater within the aquifer system. A second is the landward movement of the freshwater-saltwater interface. A third and common method in which saltwater enters the freshwater hydrogeologic system is through the regional upconing of saltwater below pumped wells. Another mechanism is through semiconfining units characterized by fractures or other structural inconsistencies that allow the vertical leakage of saltwater from deeper, saline water-bearing zones in the coastal region.

Figure 11.2 Mechanisms for Saline Water Intrusion into Freshwater Aquifers. (Spechler, 2001).



Special consideration should be taken when working in coastal areas, as subsurface work can have an impact on freshwater by expediting the saline intrusion process through introducing geologic inconsistencies into the ground network. These inconsistencies may arise from a pile or caisson altering the soil and groundwater

environment and can result in the increased intrusion of saline water as capillary forces in soils can increase or fractures develop from increased stresses associated with higher surface loadings.

7. Water Quality. As mentioned before, non-point source pollutants are the primary constituents in surface runoff that are degrading both the surface and groundwater environments. These pollutants are from a variety of sources and range from agricultural processes that produce high nutrient loadings and urbanized areas where human activities are more concentrated, to transportation facilities where vehicular leaks and discharges can accumulate on the road surface and be conveyed via rainwater runoff through the hydrologic system as both surface and subsurface flows. Ineffective site controls in the long term can lead to an accumulation of pollutants, while improper construction measures in the short term can lead to high concentrations of pollutants and sediments. There are a variety of ways to reduce and even eliminate some of the pollutants from entering natural water systems, and these methods should be investigated and employed wherever possible for any new projects or replacement projects.

8. Alternative Design Procedures. For further guidance on alternative design procedures, please refer to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 13.

E. Groundwater Quality. For further guidance on groundwater quality, please refer to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 13.

11.6 WATER BUDGET

The water budget (water balance) is a hydrologic procedure that refers to the balance between the inflow of water from precipitation, snow melt, and groundwater, and the outflow of water by evapotranspiration, groundwater, and streamflow. In a natural state the system or water budget is considered in balance, or in a state of equilibrium, and the most responsible and environmentally sensitive design should strive to maintain the natural balance (DeBarry, 2004). This balance can be evaluated using the water budget equation which assesses the relationship of water flowing into and out of a particular system. The water budget may be used in a variety of scenarios, and is able to predict human impact on a wetland, watershed, and the hydrologic cycle. Although approximate, the water budget may be used as a basis for determining a management approach to sustaining the natural hydrologic conditions of a site.

When a water budget is prepared for a project, the results should be provided to members of the design team and wetlands specialists for use in the design of post-construction stormwater management controls and wetland mitigation projects. Wetland specialists and members of the design team will use the water budget data to determine if there is adequate water supply and sufficient drawdown, at a particular site during selected times of the year to support proposed vegetation and the design concept for proposed stormwater controls.

This section presents background on the water budget, useful equations necessary to calculate the water budget, and potential sources for the various parameters used in the equations. The water budget as discussed in this section is presented in a generic format which may be applied annually or for any period of time as long as the parameters input into the equations are representative for that length of time. Thus, if a monthly water budget is needed only those variables representative of the monthly quantities should be used in the equations. It should be noted that many of the input variables vary depending on the time of year and simply using a fraction of the annual data, is not acceptable approach to determining the water budget for a portion on the year (i.e. using $1/12^{th}$ of the annual precipitation to obtain the monthly precipitation for one particular month of the year).

A. The Water Budget Equation. The water budget equation is a form of the basic routing equation:

(Equation 11.1)

$$I - O = \Delta V/\Delta t$$

where: I = inflow per unit time
O = outflow per unit time
$\Delta V/\Delta t$ = the change in storage per unit time

As demonstrated below, by rearranging Equation 11.1 the formula can be used to determine a volume based upon the rate of inflow and outlflow from a system over some period of time. This relationship can then be further manipulated to determine the depth of water in a wetlands or other surface storage facility, such as a lake or a pond.

(Equation 11.2)

$$\Delta V = \Delta t(I - O)$$

and

(Equation 11.3)

$$\Delta D = \Delta V/A$$

where: V = the volume of water in the wetland
A = the surface area of the water
D = the depth of the water
t = time

Along with a change in volume term ($\Delta V/\Delta t$), the following factors combine to form the water budget equation:

Inflows:

- Direct precipitation, P
- Surface inflows, SWI
- Groundwater inflows, GWU

Outflows:

- Surface water outflows, SWO
- Groundwater outflows, GWO
- Evapotranspiration, ET

The water budget equation with its various components is expressed in equation form as:

(Equation 11.4)

$$P + SWI + GWI = ET + SWO + GWO + \Delta V/\Delta t$$

where: P = precipitation
SWI = surface water inflow
GWI = groundwater inflow
ET = evapotranspirtation
SWO = surface water outflow
GWO = groundwater outflow
$\Delta V/\Delta t$ = change in storage

For a balanced system, the difference between the input and output of the surface water (SWI, SWO) and groundwater (GWI, GWO) will be zero (DeBarry, 2004), and the change in volume term ($\Delta V/\Delta t$), in the water is equal to zero (0). By combining inflow and outflow terms, the water budget equation for a balanced system can be rewritten in a simplified form as:

(Equation 11.5)

$$P = R + WL$$

where: P = precipitation (both rainfall and snow)
R = total runoff (overland surface runoff and base flow)
WL = water loss (evaporation, transpiration, and groundwater losses)

According to this simplified form of the water budget equation all of the precipitation entering a system can be classified as either runoff or a water loss. The runoff term includes both overland surface runoff and base flow. Overland runoff may be conveyed to a feature either as overland flow or through the various components of a drainage system (i.e. storm sewer and drainage channels). Conversely, base flow is comprised of water that infiltrates the soil, percolates into the groundwater and eventually enters the stream through the interface between

the stream and the water table (Dunne and Leopold, 1978). To fully evaluate the water budget at a project site, these two sources of stream flow must be separated, and Equation 11.5 can be rewritten as shown in Equation 11.6:

(Equation 11.6)

$$P = R_{osr} + R_e + WL$$

where: R_{osr} = overland surface runoff
R_e = groundwater recharge/discharge

The water loss component in the simplified form of the water budget equation includes those systems and components that do not contribute flow to a particular feature but remove it from the system. Water losses include both natural processes, such as evaporation, and manmade losses, such as extraction for water supply.

Ordinarily, it is customary to put all terms in the water budget equation, except for time, in units of depth of water.

B. Data Requirements. When developing a water budget, data requirements for Equation 11.6 include precipitation, total runoff (comprised of overland surface runoff and groundwater recharge), and water loss. Data requirements to complete a detailed analysis can be very extensive. Rain gages and stream gages located within or around the study area need to be identified and the data examined for use in the budget. The entire record of these gages should be studied to determine budget implications for the wettest year of record, the driest year of record and the average year of record which are representative of the study area.

Although recorded data at a particular site may be quite extensive, caution is suggested in using an entire period-of-record for rain and stream gages in urbanized areas or any area that has experienced large land-cover changes. Urbanization can change rainfall patterns, quantities of precipitation and generally affects the stage-discharge relationship of streams and other water bodies, particularly impacting the peak discharge and timing of rising and falling limbs of the flood hydrograph.

1. Precipitation. Precipitation is recorded at weather stations, which are typically not located on project sites. Many factors affect the accuracy of the weather station data and the transposing of data from distant recording sites to the study area. Factors affecting the transposition of precipitation data include rain shadows, changes in elevation, lake effects, complex topography and human activities including urbanization, deforestation and any large scale land-use changes. When any of these factors are present, it may be necessary to obtain data precipitation gages from other facilities. If a rain gage is not located near the site, extrapolation may be necessary and a sound basis for extrapolation used. Rainfall extrapolation procedures are generally found in most good hydrology textbooks. Common methods of extrapolation include the arithmetic method, the Isohyetal method, the Horton-Thiessen method, and the Station method.

If local data is not available, National Oceanic and Atmospheric Administration (NOAA) precipitation data should be acquired from a station near the study region. The Thiessen-weighted method of area rainfall determination can then be employed to determine the average rainfall for the specific region.

In the Thiessen-weighted method a watershed or region is subdivided into polygonal subareas with the precipitation stations as centers. Subareas are then used to assign a weight to the rainfall amount measured at the precipitation station by calculating the percentage of the watershed's area that is shared with the area of the precipitation gage polygon. It should be noted that the polygon and weights must be recalculated when a precipitation gage is added, removed, or moved to a new location. More information on the Thiessen-weighted method can be found in *Engineering Hydrology, Principles and Practices* (Ponce, 1989), *Elementary Hydrology* (Singh, 1992), and *Applied Hydrology* (Chow, et. al., 1988).

In the absence of regional rainfall data, distributions of average annual precipitation in Pennsylvania are available from Oregon State University. Precipitation data from 1961-1990 was published by the Precipitation-elevation Regressions on Independent Slopes Model (PRISM) Group at Oregon State University. *A Statistical-Topographic Model for Mapping Climatological Precipitation over Mountainous Terrain* (Daly et al., 1994), *The PRISM Approach to Mapping Precipitation and Temperature* (Daly et al., 1997) and Figure 11.3 shows the annual precipitation of Pennsylvania using the aforementioned data.

Figure 11.3(a) Average Annual Precipitation of Pennsylvania (Metric)

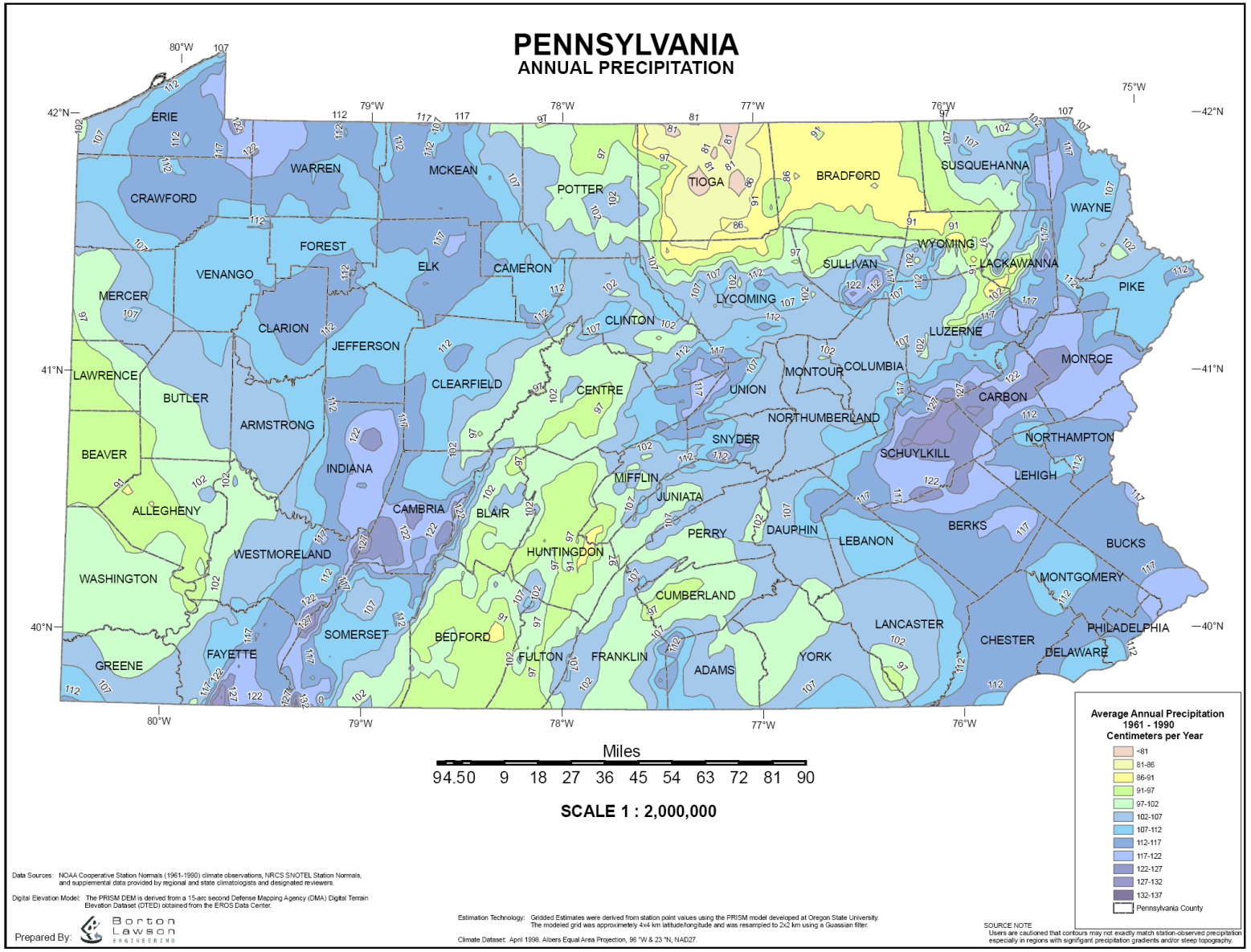
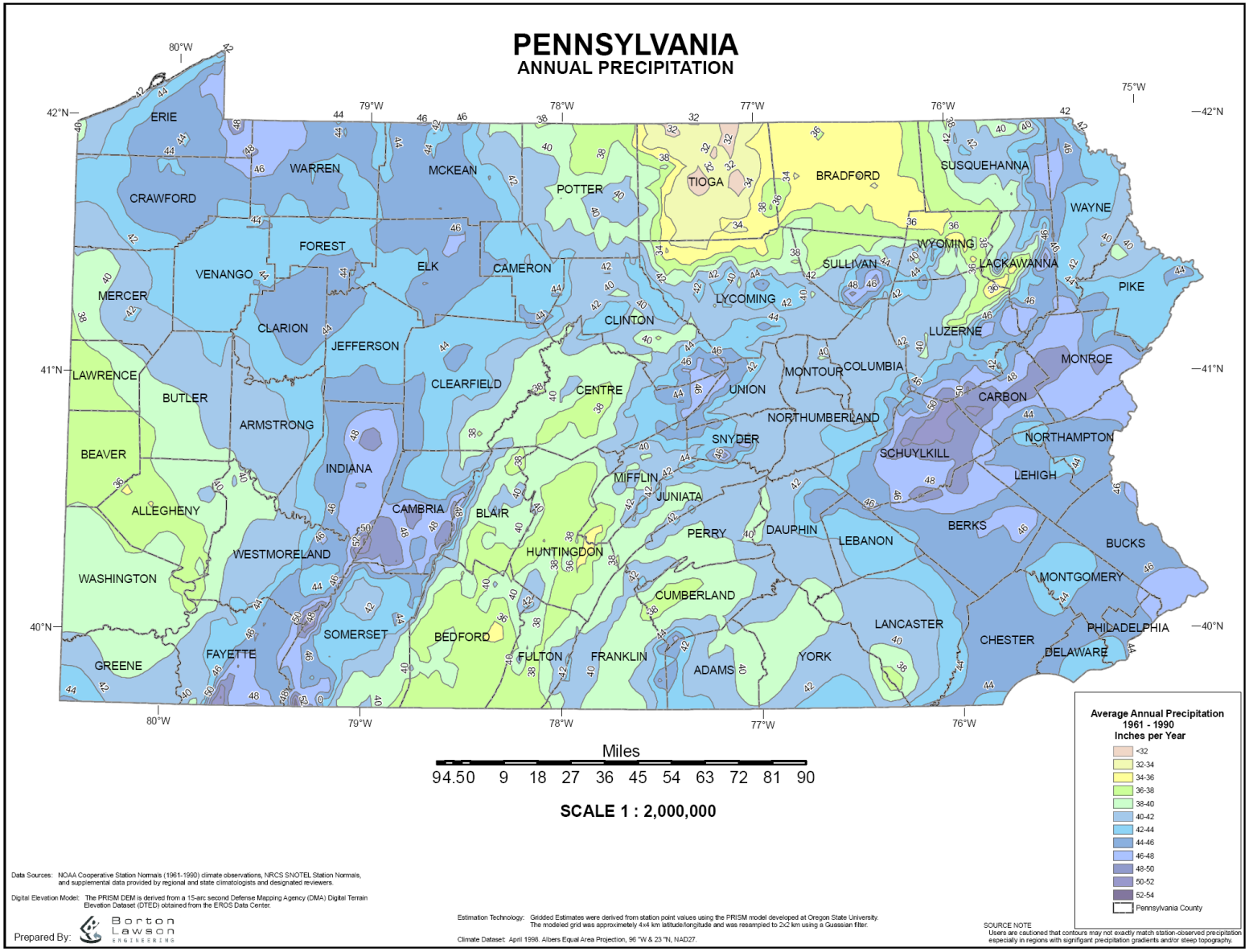


Figure 11.3(b) Average Annual Precipitation of Pennsylvania (U.S. Customary)



Regional rainfall data for Pennsylvania can often be found in publications from the United States Geological Survey (USGS), the Pennsylvania Department of Conservation and Natural Resources (DCNR), the Pennsylvania Geological Survey, the Pennsylvania Department of Environmental Protection (PA DEP), and various Universities and research institutes throughout the state. As numerous data sources are available a search for supplementary publications from the aforementioned agencies and organizations may yield regional precipitation information useful in the development of a water budget for a particular site.

Although precipitation provides water inflow into the system and the water budget equation, not all of the precipitation that enters a watershed results in runoff. Some of the precipitation that falls is intercepted by vegetation and held in small surface depressions and prevented from running off. The portion of the precipitation that is intercepted typically evaporates. As quantifying the amount of interception that occurs can be difficult, good interception estimates are generally not available except for forested areas and certain agricultural crops. To obtain useful interception data, information from regional hydrology and canopy interception studies may be necessary and practical interception data may be able to be obtained from research organizations such as local universities. Additional discussion on water losses to the water budget is presented in subsequent sections of this chapter.

2. Total Runoff. Information pertaining to total runoff may be obtained from quite a few sources. One of the best sources for total runoff is from a USGS stream gage station, which is able to supply recorded streamflow data directly for application in the water budget. In addition to the gage data, external references such as the USGS Annual Hydrologic Data Report of Pennsylvania (Durlin and Schaffstall, 2003) provide detailed analytical analysis of the flows recorded by stream gages. Several different types of flows can be generated for these stations which provide statistical analyses on daily, monthly, and annual flows as well as the lowest, highest and mean values. The flow data measured by the gage can be used in the water budget analysis with the proper flow data selected based upon the appropriate period of time to be evaluated in the water budget. A sample printout from the report using stream gaging records is provided with the example problem. The data provided in this figure is only applicable to those sites within the same watershed or in close proximity to the gage.

In those regions or watersheds without gaging stations, the streamflow data may be estimated from nearby stream-flow gaging stations in an adjacent or similar watershed. Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Section 10.6 provides specific criteria for use of stream gage records for application in hydrologic analyses such as the water budget.

As with the precipitation data, other external or independent references apart from the stream gage data can be used to obtain useful hydrologic information pertaining to total runoff. Therefore, prior to undertaking any analysis of the total runoff it is recommended that government agencies and local research institutions be contacted to determine if regional hydrologic information is available which may be useful in the analysis.

3. Surface Water. Total runoff is comprised of both overland surface flow, which occurs during rainfall events, and the groundwater flow, which occurs below the surface of the ground. To develop the water budget, as shown in Equation 11.6 it is necessary to separate the total runoff, measured from a stream gage or another source into its two main components. Once the total runoff is known the surface runoff may be determined by subtracting the groundwater flow from the surface runoff, or conversely, the surface runoff may be calculated directly and the groundwater flow determined by subtracting the overland flow from the total runoff. Thus, if two of the three variables are known, the unknown value can be determined through simple mathematical manipulation.

Surface water inflows can be calculated directly using several of the hydrologic methods and procedures presented in Chapter 7, *Hydrology*. The runoff calculated by these methods and procedures represent overland flow to streams and rivers, and includes sheet flow, shallow concentrated flow, and stream or channel flow to rivers lakes and other surface features. When using any direct computation of overland runoff it is best to calibrate the flow measurements in order to verify or substantiate the accuracy of the flows. When stream flow is a factor, computer models, such as HEC-RAS or WSPRO, direct measurements and data from Federal Emergency Management Agency (FEMA), can be used to calculate water levels and velocities.

The Natural Resources Conservation Service (NRCS) Curve Number (CN) approach is one method which may be used to determine the overland runoff from a region. A procedure demonstrating how this method may be used to determine the overland runoff of the total runoff is presented below.

a. Map the NRCS soil types. Determine the extent of each soil type in watershed in acres. Determine the Hydrologic Soil Group (HSG) for each soil type.
b. Map the land cover for the watershed.
c. Overlay the land-cover map over the soil type map. This will divide the watershed into sub-areas based on land cover and soil type.
d. Determine NRCS curve numbers (CN) for each sub-area.
e. Determine weighted curve number for watershed using the equation:

(Equation 11.7)

$$CN_{weighted} = \frac{\sum_{i=1}^{n}(CN_i)\ (A_i)}{\sum_{i=1}^{n} A_i}$$

where: CN_i = NRCS curve number for sub-area *i*
A_i = area of sub-area *i*
N = number of sub-areas

f. Determine the wettest year, the driest year and an average year from the rainfall data.
g. Determine the minimum amount of precipitation (P) that will cause runoff. This is done by setting the rainfall-runoff equation from Chapter 7, *Hydrology,* Equation 7.13 equal to zero and solving for P:

(Equation 11.8)

$$\begin{aligned} Q &= (P - 0.2S)^2/(P + 0.8S) \\ 0.0 &= (P - 0.2S)^2/(P + 0.8S) \\ (P - 0.2\mathrm{S})^2 &= 0.0 \\ P &= 0.2S \end{aligned}$$

(Equation 11.9)

where: $S = 25.4\ [(1000.0/CN) - 10.0]$

(Equation 11.10)

Therefore, $P = (5080/CN) - 50.8$

h. Calculate runoff depth, Q, for all precipitation events large enough to produce runoff. This can be done by solving Equation 11.8.
i. Calculate the runoff volume for the watershed by multiplying the runoff depth, Q, by the drainage area in square feet: Volume = $(Q)(A)$
j. Determine either total runoff or baseflow using surface runoff and total runoff or groundwater runoff data.

A search for supplementary publications from USGS, DCNR, PA DEP, Pennsylvania Geological Survey, Universities and research institutes and/or County Conservation Districts may yield additional regional surface water flow information that may be useful in determining surface runoff.

4. Groundwater Recharge/Discharge. Groundwater recharge involves both natural processes and manmade devices which function to convey runoff from overland surface runoff to groundwater flow which occurs below the surface of the ground. One type of common natural recharge area occurs in highly permeable soils, such as those soils of Hydrologic Soil Group A and B. In these areas water flowing over pervious surfaces, percolates through the soil mantle and enters the groundwater table. Conversely, groundwater discharge occurs at points, both natural and manmade, which function to transfer water flowing slowly beneath the ground and return it back to surface flow. Examples of natural groundwater discharge areas include springs, seeps, cut slopes and the interface between river channels and the ground.

Depending upon the hydrogeology of a region, groundwater inflow may or may not be significant to the hydrologic budget. Areas with glacial-landscapes, sloped wetlands, and sites with Karst geology are all examples of features that can facilitate the conveyance of significant quantities of water into the ground.

Alternatively, if land cover and the hydrology of recharge areas is altered significantly, the normal conveyance of surface runoff into the ground is halted. In such areas where groundwater outflow on a site is greater than the potential water inflow, maintaining the water table in these areas is difficult without modification of the site. Groundwater data is typically more difficult and time-consuming to collect than surface water data, therefore, the easiest way to obtain groundwater values for application in the water budget is often to subtract surface flow from the total runoff measure at gaging stations or through the use of existing regional data described later in this section.

Often monitoring wells are used to determine groundwater flow in areas. However, because of the nature and scope of transportation related projects their wide spread use to determine groundwater flow for transportation projects is not practical. Therefore, the subsequent information is provided as background information only to provide a broader understanding of groundwater implications.

To fully comprehend groundwater flow in a region, it is necessary to have a good understanding of the hydrogeology of the aquifers present (confined, unconfined or leaky). Monitoring wells or existing wells, constructed with a well screen and casing, can be used to establish water levels within the project area. Normally, the water level in an unconfined aquifer is referred to as the water table. In unconfined aquifers, the groundwater is not stored under pressure. Conversely, in confined aquifers, groundwater is stored under pressure, and the interface between the upper level of the groundwater, and the confining geology above the water table is referred to as the piezometric surface. Using the water levels in a well, the water table and/or piezometric surface at a site can be determined. Normally, a minimum of three wells in a single aquifer are needed to determine the general direction of groundwater flow, with data collection efforts over an extended period of time required in order to obtain the direction of groundwater flow, which may vary with time. To determine the rate of groundwater flow, the hydraulic conductivity of the geologic materials and the hydraulic gradient must be determined. The hydraulic conductivity is the rate at which water flows through the various layers of the geologic strata and will vary depending on the type and characteristics of the geology present. The hydraulic gradient is determined using the water level data and is usually expressed in terms of horizontal and vertical gradient. The volumetric flow rate is defined by Darcy's Law:

(Equation 11.11)

$$Q = KA(dh/dx)$$

where: Q = the discharge, (volume per time)
K = the hydraulic conductivity or permeability, (length per time)
A = the cross sectional area perpendicular to flow
dh/dx = the hydraulic gradient in the direction of the flow

Stream-base flow is related to groundwater discharge in that base flow in stream channels is supplied from groundwater flow that exits the groundwater supply. A computer program called HYSEP can be used to separate base flow and surface runoff from a stream hydrograph and detailed information on this procedure can be found in *HYSEP: A Computer Program for Streamflow Hydrograph Separation and Analysis* (Sloto and Crouse, 1996).

Base flow estimates may be determined from other references such as *Base-Flow-Frequency Characteristics of Selected Pennsylvania Streams* (White and Sloto, 1990). This document reports streamflow hydrographs of 309 streamflow stations in Pennsylvania that were analyzed to separate groundwater and surface runoff from various recurrence intervals. Also, *Estimates of Ground-Water Recharge Based on Streamflow-Hydrograph Method Pennsylvania* (Risser et. al., 2005) reports estimated recharge using streamflow records collected during 1885-2001 for 197 streamflow gaging stations in Pennsylvania using RORA (Rorabaugh Method) and PART (Streamflow Partitioning Approach) Programs.

Even though areas outside of the watershed or region may contribute groundwater flow to a region, the groundwater inflow, is normally considered zero as a conservative approach.

As with the other components of the water budget detailed information may be obtained from a number of resources. Therefore, it is necessary that the design professional select the data and models which are most appropriate for a particular project. Other potential sources of groundwater information include USGS, DCNR, PA DEP, Pennsylvania Geological Survey, Universities and research institutes and/or County Conservation Districts.

5. Evapotranspiration (ET). The most common mechanism by which water losses occur in a region is evapotranspiration. Evapotranspiration includes both the surface evaporation of water and transpiration of water through plants. The evaporation from the water surface is usually affected by land cover (i.e. grass or forest), and evaporation alone rarely adequately estimates total losses. Evapotranspiration is difficult to measure, however, potential evapotranspiration (PET), defined as the maximum possible water loss, can be used as an estimation of ET losses as it has been more rigorously studied. Pan evaporation rates (evaporation from a shallow pan) are used to determine the ratio of total precipitation to total evaporation (P/E) for any specific region. Factors affecting evapotranspiration are exposed water surface area, solar radiation, temperature of the air and the water, wind speed, and relative humidity. Plants can control transpiration rates to some degree by closing leaf stomata, thus slowing the loss of water through the plant and its return to the atmosphere. Basically, in dry areas, plants can activate water conservation measures when they experience dry conditions.

There are many factors which impact the amount of evapotranspiration that occurs. For instance, in wetlands, vegetation reduces evaporation rates by reducing the exposed water surface area. In these areas wind velocities at the water surface are reduced by the shielding effects of vegetation and microclimates are created by the vegetation. These microclimates can have higher humidity than the surrounding air which ultimately results in reduced evaporation rates. In a pond, vegetation may reduce evaporation rates to about three-fourths of pan evaporation.

The evaporation component of evapotranspiration can be reasonably estimated using regional data, but the transpiration component depends on specific and accurate knowledge of the plants involved in the process. Typically, transpiration rates are much higher than evaporation, with transpiration rates estimated to be from 0.53 to 5.40 times higher than evaporation alone.

While pan evaporation can be applied to gain an estimate of the evapotranspiration of a region, there are several other methods available to more accurately predict evapotranspiration. They vary in difficulty of application and accuracy. Some of these methods are physically based while others are climatologically based models. Physical methods require information on solar radiation and detailed information on transpiration specifically for the types of plants in the region. One such physically based method is the Penman-Monteith equation (Penman, 1948) which utilizes the energy balance equation to compute evapotranspiration. Due to the complexity of this procedure, it is not included in this manual. If you wish to apply it, *Evapotranspiration from Isolated Stands of Hydrophytes: Cattail and Bulrush* (Allen et. al., 1992); *Surface Geometry and Stomatal Conductance Effects on Evaporation from Aquatic Macrophytes* (Anderson and Idso, 1987); *Transpiration and Stomatal Conductance of Two Wetland Macrophytes (Cladium jamaicense and Typha domingensis) in the Subtropical Everglades* (Koch and Rawlik, 1993); *Evapotranspiration from Sedge-Dominated Wetland Surfaces* (Lafleur, 1990); and *Treatment of Dairy Wastewater in a Constructed Wetland System: Evapotranspiration, Hydrology, Hydraulics, Treatment Performance, and Nitrogen Cycling Processes* (Niswander, 1997), are suggested. Although evapotranspiration rates can be calculated, calculated values often overestimate actual evapotranspiration rates that occur.

The U.S. Department of Agriculture (USDA) and the Natural Resource Conservation Service (NRCS) prepared and published a map of potential evapotranspiration of Pennsylvania landscapes using data collected from 1961 to 1990. Figure 11.4 is a reproduction of the mean annual potential evapotranspiration using the USDA data and can be used in water budget computations in the absence of regional or site specific data. Use of this data is a conservative approach to water budget analysis, as potential evapotranspiration represents the maximum water loss that can occur through this natural process, with actual evapotranspiration being less, and the available water supply being somewhat higher than calculated. Regional evapotranspiration data may also be available from the state climatological center. Prior to preparing water budget computations, those completing the analysis are encouraged to contact this organization for any regional information applicable to evapotranspiration rates and the water budget.

Figure 11.4(a) Mean Annual Potential Evapotranspiration of Pennsylvania Landscapes (Metric) (adopted from US DOA/NRCS)

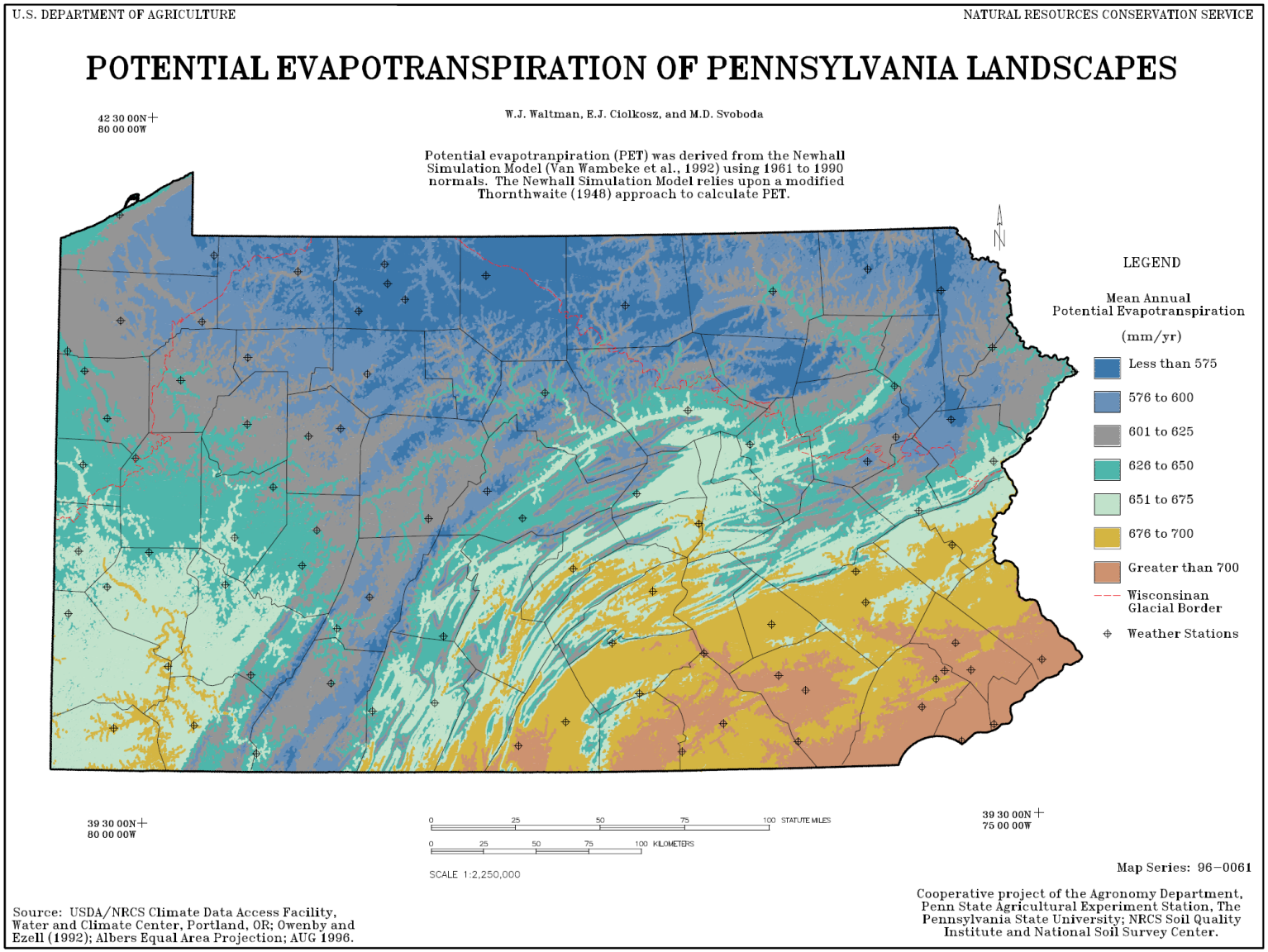
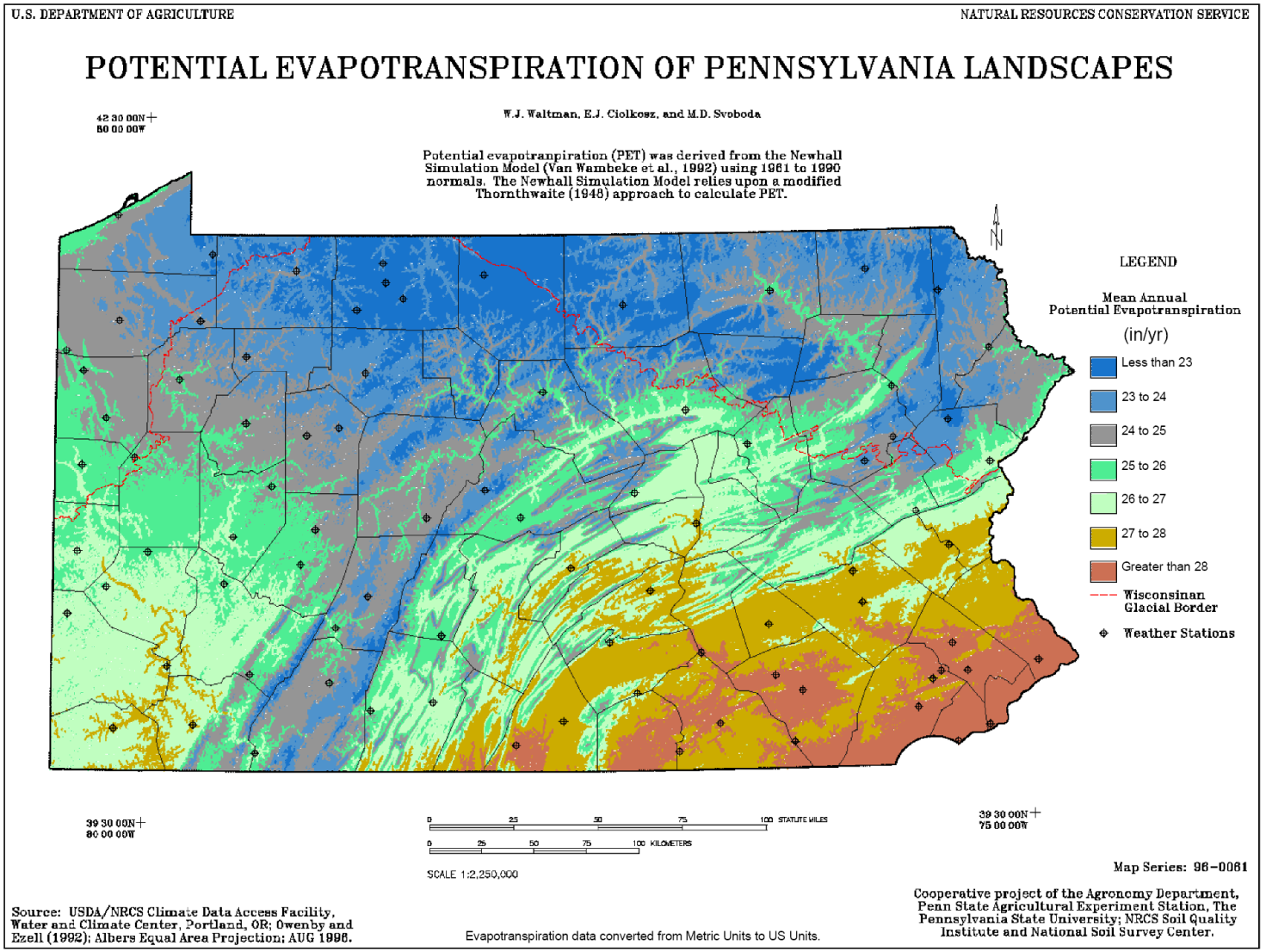


Figure 11.4(b) Mean Annual Potential Evapotranspiration of Pennsylvania Landscapes (U.S. Customary) (adopted from US DOA/NRCS)



C. Tidal Considerations. If the site is located in a tidal region, locate all the tide gages that have been installed in the region. If one or more of the gages has long-term gage data, obtain the daily high- and low-tide elevations for a complete 19-year tide cycle. The 19-year tide cycle is caused by the cyclic variations in the moon's orbit around the earth. NOAA has tide gage sites, called reference stations, for which tide predictions are published in Annual Tide Tables. If there are no long-term tide gages near the site, the data must be synthesized for the site. To do this, a continuous recording gage should be installed at the site and measurements obtained for at least 30 days. This data must then be correlated to the data for the same time period from the nearest reference station. From the data of the reference station, tide data should be synthesized for the site. This data should cover the highest tides, the lowest tides and average tides over the 19-year tidal cycle.

The second alternative is to compute the cycle using a tidal hydraulic model. The nearest tide gage should be identified and used as the boundary condition for the tidal hydraulic model. There are three hydraulic models recommended for use in analyzing tidal flow. The choice of which program is used depends on the complexity of the flow path. A one-dimensional flow program, UNET, developed by USACE, is recommended for flow paths dominated by linear flow. The two-dimensional flow programs Finite-Element Surface-Water Modeling System (FESWMS) or RMA2 are recommended where the flow is more complex. A more complete description of tidal hydraulics procedures is given in *Tidal Hydraulic Modeling for Bridges Users Manual* (Zevenbergen, et. al., 1997).

D. Water Budget Computation Procedures.

Step 1 Determine Precipitation (P), cm (in)
Step 2 Determine Total Runoff (R), cm (in)
Step 3 Determine Groundwater flow (R_e), cm (in)
Step 4 Determine Overland Surface Runoff (R_{osr}), cm (in)
Step 5 Determine Water Loss (WL), cm (in)
Step 6 Determine Project Implications on Water Budget

All inflows and outflows should be expressed as depth (volumes can be divided by site area to get depth).

E. Example Pennsylvania Water Budget Problem. An example project site located about 3 km (2 mi) southwest of Interstate Highway 81 (Figure 11.4) is used to demonstrate the water budget. The project has a total drainage area of roughly 20.3 ha (50 acres) draining through the site in the Wapwallopen Creek watershed near Wapwallopen, Luzerne County, Pennsylvania.

Evaluate the impact of a new single lane roadway upon the water budget of the drainage area in the vicinity of the proposed transportation project. The hypothetical roadway is approximately 11 m (36 ft) wide, including travel lanes and paved shoulder, and has a total length of about 244 m (800 ft).

Step 1 Determine Precipitation (P)

Average annual precipitation (P) for the site can be determined using the site location in Luzerne County and Figure 11.4. For this site the average annual precipitation is estimated to be 106.7 cm (42 in).

Step 2 Determine Total Runoff (R)

Total Runoff (R) for the site can be found from a stream gage located on the Wapwallopen Creek near Wapwallopen, PA (Station I.D. 01538000). The stream gage is located in the same watershed as the project about 10 miles (16.1 kilometers) west (downstream), of the example study area.

Using the stream gage records for this gage, the USGS report (Durlin, 2003) as shown in Table 11.2 indicates the mean annual flow or total runoff (R) is 51.1 cm (20.1 in) per year (see Table 11.2 for detail). Gage records vary by site and the period of record for this gage is from 1920 to 2000. Depending on the application of the water budget data, the depth of water representing the total runoff, can be converted from a depth to a rate per unit area of the watershed, using the drainage area of the watershed. In this case, the drainage area to the gage is 113.4 km^2 (43.8 mi^2) and the conversion can be completed using the following equation:

$$R = (51.1 \text{ cm/yr}) (1 \text{ yr} / 365 \text{ days}) (1 \text{ day} / 24 \text{ hr}) (1 \text{ hr} / 3600 \text{ s}) (10002 \text{ m}^2/\text{km}^2) (1 \text{ m} / 100 \text{ cm})$$
$$R = 0.016 \text{ m}^3\text{s} / \text{km}^2$$

$$R = (20.1 \text{ in/yr}) (1 \text{ yr} / 365 \text{ days}) (1 \text{ day} / 24 \text{ hr}) (1 \text{ hr} / 3600 \text{ s}) (640 \text{ acre} / 1 \text{ sq mi}) (43560 \text{ ft}^2 / 1 \text{ acre}) (1 \text{ ft} / 12 \text{ in})$$
$$R = 1.480 \text{ cfs} / \text{mile}^2$$

Combining the precipitation data with the runoff data indicates that of the 106.7 cm (42 in) of average annual rainfall (P) that falls annually on the watershed 51.1 cm (20.1 in) flows to the streams either as overland surface runoff or groundwater discharge. Therefore, total runoff (R) defined in terms of depth equals 51.1 cm (20.1 in).

Table 11.2 Wapwallopen Creek Near Wapwallopen, PA
Water Resources Data: Pennsylvania Year 2002, Volume 2,
Susquehanna and Potomac River Basins, Water-Data Report PA-02-2.

WAPWALLOPEN CREEK BASIN

01538000 WAPWALLOPEN CREEK NEAR WAPWALLOPEN, PA

LOCATION.--Lat 41°03'33", long 76°05'38", Luzerne County, Hydrologic Unit 02050107, on left bank 100 ft upstream from Harts Bridge on SR 3012, 2.2 mi southeast of Wapwallopen, and 3.7 mi upstream from mouth.

DRAINAGE AREA.--43.8 mi^2.

PERIOD OF RECORD.--October 1919 to current year.

REVISED RECORDS.--WSP 1302: 1926(M), 1929(M), 1938(M). WSP 1432: Drainage area.

GAGE.--Water-stage recorder. Datum of gage is 752.41 ft above National Geodetic Vertical Datum of 1929 (Penn Central Railroad bench mark). Prior to Mar. 15, 1930, nonrecording gage at same site and datum.

REMARKS.--Records good except those for estimated daily discharges, which are poor. Several measurements of water temperature were made during the year. Satellite telemetry at station.

PEAK DISCHARGES FOR CURRENT YEAR.--Peak discharges greater than a base discharge of 580 ft^3/s and maximum (*):

Date	Time	Discharge ft^3/s	Gage Height (ft)
May 18	1200	664	4.28
May 28	2230	*929	*5.04

DISCHARGE, CUBIC FEET PER SECOND, WATER YEAR OCTOBER 2001 TO SEPTEMBER 2002
DAILY MEAN VALUES

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	11	12	31	e19	71	24	90	99	111	24	12	14
2	10	12	21	e19	71	24	72	125	87	23	11	18
3	9.8	12	18	e18	54	49	67	107	71	23	9.9	14
4	9.7	12	16	e18	50	40	61	84	61	21	9.8	12
5	9.6	12	15	e20	e46	31	55	76	58	19	10	11
6	9.6	12	15	e20	e42	28	52	70	145	17	13	10
7	9.9	12	15	e20	39	28	48	66	293	16	11	9.8
8	9.5	12	16	e18	38	27	46	61	133	16	10	9.4
9	9.2	11	34	e20	35	26	45	91	100	15	9.9	9.3
10	9.8	11	29	e20	34	64	45	95	83	15	9.4	9.4
11	9.9	12	22	e22	71	45	40	66	71	14	9.2	8.9
12	11	11	20	e26	50	38	37	119	73	12	9.1	8.8
13	9.8	12	19	e24	43	36	39	255	88	12	9.2	8.9
14	10	11	20	e24	e36	36	69	351	104	11	8.8	8.7
15	17	11	22	e22	e35	33	128	216	114	12	9.0	13
16	15	11	19	e22	35	39	79	163	90	11	18	36
17	20	11	19	e22	35	38	64	135	71	8.7	12	16
18	16	11	64	e20	32	38	59	398	59	8.1	9.9	13
19	12	11	52	e20	31	42	56	248	51	11	9.9	12
20	11	12	37	e30	31	53	56	189	46	12	9.7	11
21	11	13	32	e24	32	82	53	159	42	9.5	9.2	11
22	10	12	28	22	30	60	52	134	37	8.4	9.0	12
23	11	11	26	21	29	52	50	112	33	19	13	21
24	17	11	29	30	27	50	45	96	32	37	15	16
25	16	19	26	55	26	48	46	83	31	17	21	13
26	13	59	e23	42	26	58	52	84	29	14	13	12
27	12	23	e21	37	29	245	43	72	29	14	11	59
28	12	18	e23	40	27	123	130	168	44	14	10	125
29	12	16	e22	45	---	101	149	347	29	15	12	32
30	13	18	e20	67	---	89	97	173	26	13	14	22
31	12	---	e20	67	---	77	---	129	---	12	12	---
TOTAL	368.8	431	774	874	1105	1724	1925	4571	2241	473.7	350.0	576.2
MEAN	11.90	14.37	24.97	28.19	39.46	55.61	64.17	147.5	74.70	15.28	11.29	19.21
MAX	20	59	64	67	71	245	149	398	293	37	21	125
MIN	9.2	11	15	18	26	24	37	61	26	8.1	8.8	8.7
CFSM	0.27	0.33	0.57	0.64	0.90	1.27	1.46	3.37	1.71	0.35	0.26	0.44
IN.	0.31	0.37	0.66	0.74	0.94	1.46	1.63	3.88	1.90	0.40	0.30	0.49

STATISTICS OF MONTHLY MEAN DATA FOR WATER YEARS 1920 - 2002, BY WATER YEAR (WY)

MEAN	38.06	59.90	73.51	70.63	82.87	117.2	114.7	86.33	49.74	34.58	24.46	26.97
MAX	202	203	206	284	284	327	362	243	248	172	149	160
(WY)	1956	1927	1997	1979	1981	1936	1993	1947	1972	1947	1933	1987
MIN	4.95	5.35	5.90	6.39	14.9	48.7	47.0	25.8	10.9	5.21	4.46	3.37
(WY)	1964	1931	1931	1931	1940	1981	1955	1955	1962	1955	1953	1936

e Estimated.

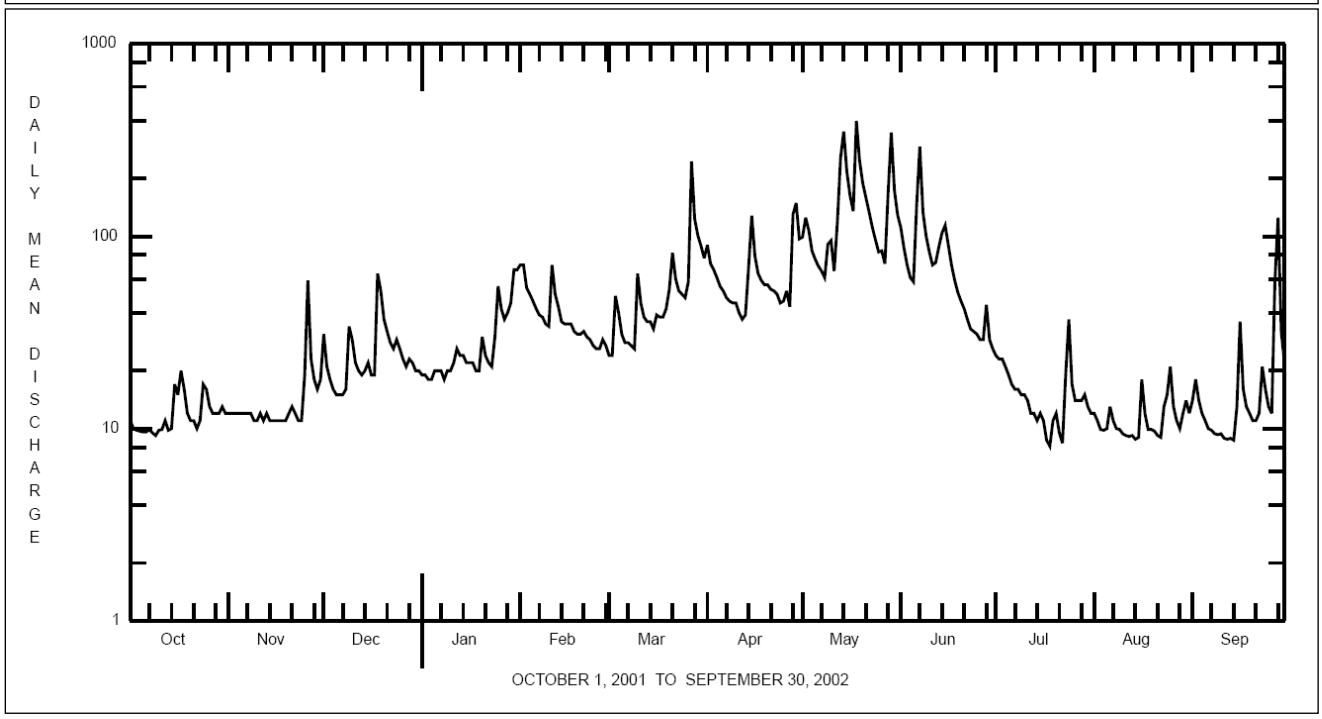
Table 11.2 (continued) Wapwallopen Creek Near Wapwallopen, PA
Water Resources Data: Pennsylvania Year 2002, Volume 2,
Susquehanna and Potomac River Basins, Water-Data Report PA-02-2.

WAPWALLOPEN CREEK BASIN

01538000 WAPWALLOPEN CREEK NEAR WAPWALLOPEN, PA--Continued

SUMMARY STATISTICS	FOR 2001 CALENDAR YEAR		FOR 2002 WATER YEAR		WATER YEARS 1920 - 2002	
ANNUAL TOTAL	13413.1		15413.7			
ANNUAL MEAN	36.8		42.2		64.8	
HIGHEST ANNUAL MEAN					108	1978
LOWEST ANNUAL MEAN					30.9	1965
HIGHEST DAILY MEAN	238	Mar 30	398	May 18	e2200	Jan 19 1996
LOWEST DAILY MEAN	6.5	Sep 9,10	8.1	Jul 18	1.5	Aug 31 1953
ANNUAL SEVEN-DAY MINIMUM	6.8	Sep 7	9.1	Sep 8	1.8	Aug 27 1953
MAXIMUM PEAK FLOW			929	May 28	a5410	Jun 22 1972
MAXIMUM PEAK STAGE			5.04	May 28	11.04	Jun 22 1972
INSTANTANEOUS LOW FLOW					1.1	Aug 4 1955
ANNUAL RUNOFF (CFSM)	0.84		0.96		1.48	
ANNUAL RUNOFF (INCHES)	11.39		13.09		20.10	
10 PERCENT EXCEEDS	96		93		138	
50 PERCENT EXCEEDS	20		23		40	
90 PERCENT EXCEEDS	9.3		10		9.0	

a From rating curve extended above 1,400 ft^3/s on basis of contracted-opening measurement of peak flow.
e Estimated.



Step 3 Determine Groundwater Flow (R_e)

Groundwater flow for this problem is assumed to be equal to the base flow occurring in the stream and is established using USGS Water-Resources Investigations Report 90-4161, Base-Flow Frequency Characteristics of Selected Pennsylvania Streams. This reference tabulates the base flow in selected Pennsylvania streams by recurrence interval. Table 11.3 summarizes the base flows found at the streamflow station for the Wapwallopen Creek near Wapwallopen, PA. (Station I.D. 01538000) by two different methods, the Local Minimum Method and the Fixed Interval Method. The base flows tabulated by this reference are used to determine the mean annual base flow, a flow with a recurrence interval of 2.33-years, through interpolation of a log-log plot of the base flow and recurrence interval, as shown in Figure 11.5.

Table 11.3 Base Flows for Selected Recurrence Intervals
at Example site from USGS WRIR 90-4167,
Base-Flow-Frequency Characteristics of Selected Pennsylvania Streams.

Wapwallopen Creek near Wapwallopen, PA				
Recurrence Interval	**Local-Minimum Method**		**Fixed-Interval Method**	
Year	mld/km^2 (mgd/mi^2)	cm (in)	mld/km^2 (mgd/mi^2)	cm (in)
2	0.835 (0.572)	30.48 (12)	0.931 (0.638)	33.02 (13)
5	0.666 (0.456)	25.40 (10)	0.744 (0.510)	27.94 (11)
10	0.622 (0.426)	22.61 (8.9)	0.644 (0.441)	23.62 (9.3)
25	0.559 (0.383)	20.32 (8.0)	0.609 (0.417)	22.35 (8.8)
50	0.499 (0.342)	18.29 (7.2)	0.562 (0.385)	20.57 (8.1)
100	0.467 (0.320)	17.02 (6.79)	0.515 (0.353)	18.80 (7.4)

*mgd/mi^2: million gallon per day per square mile.
*mld/km^2: million liter per day per square kilometer.

Figure 11.5(a) Mean Base Flow at Example Site Using WRIR 90-4167 (Metric)
Base-Flow-Frequency Characteristics of Selected Pennsylvania Streams (White and Sloto, 1990)

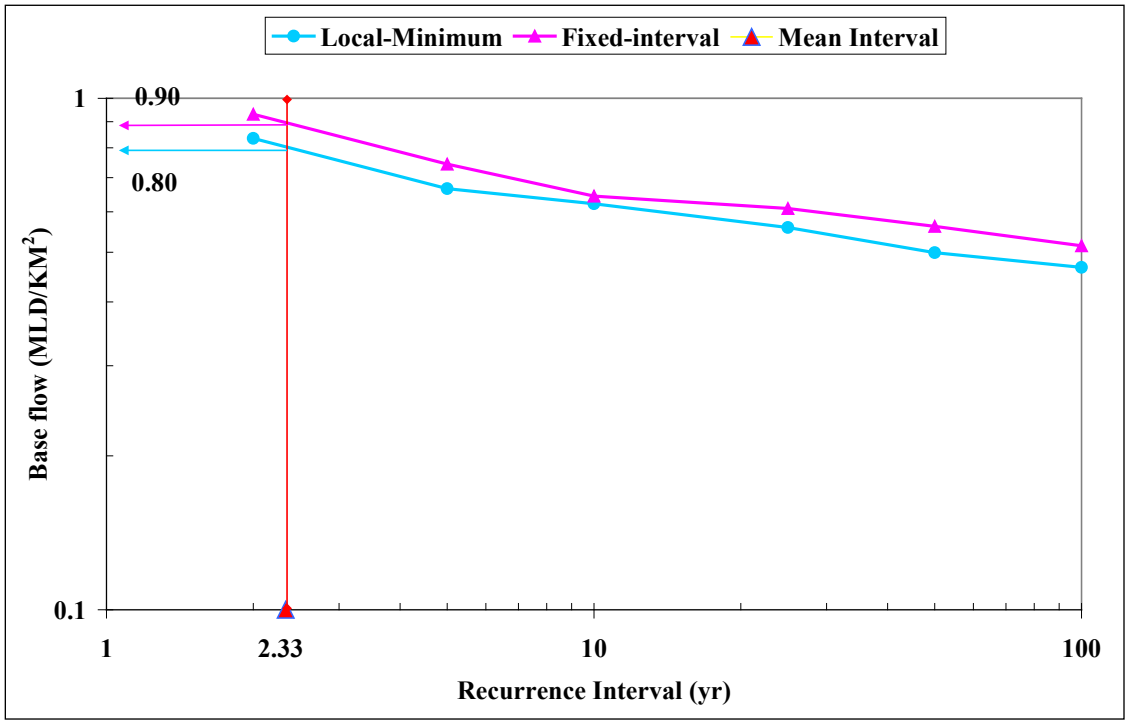
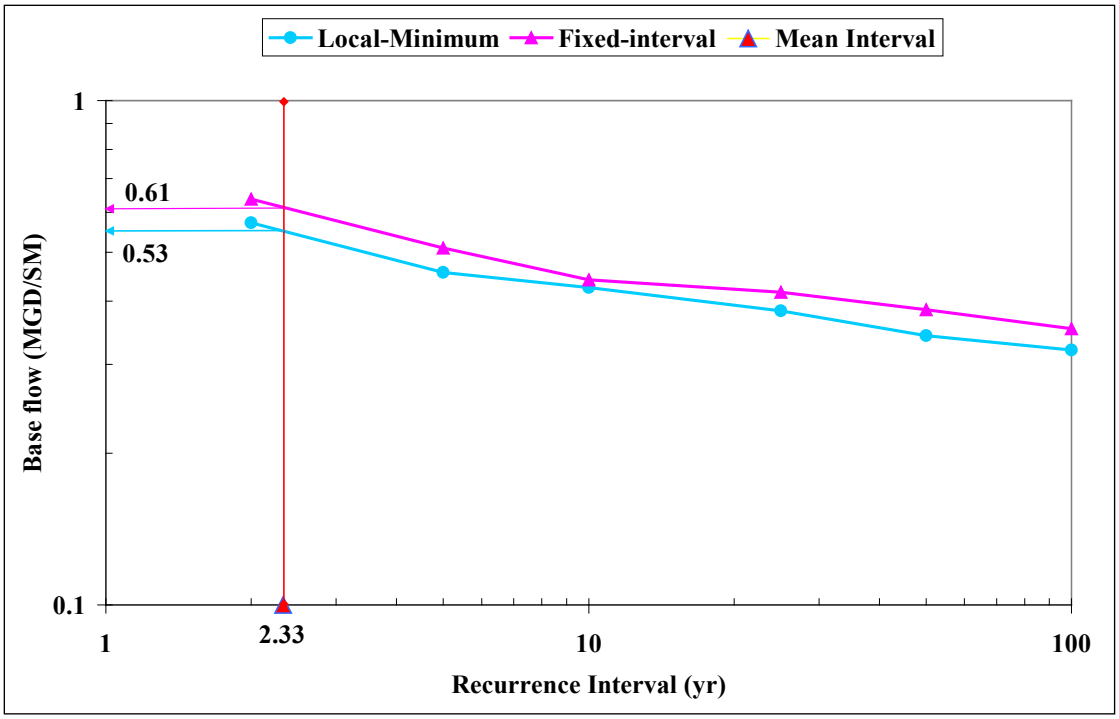


Figure 11.5(b) Mean Base Flow at Example Site Using WRIR 90-4167 (U.S. Customary)
Base-Flow-Frequency Characteristics of Selected Pennsylvania Streams (White and Sloto, 1990)



From Figure 11.5, the mean annual base flow is determined to be 0.84 mld/km^2 (0.57 mgd/mi^2). This can be converted to cm (in) by:

$$R_e = 840{,}000 \text{ lpd/km}^2 \times (365 \text{ days/yr}) \times (1 \text{ km}^2 / (1.0 \times 10^{10} \text{ cm}^2)) \times (1{,}000 \text{ cm}^3/\text{l})$$
$$= 30.5 \text{ cm/yr}$$

$$R_e = 570{,}000 \text{ gpd/mi}^2 \times (1 \text{ ft}^3 / 7.481 \text{ gal}) \times (365 \text{ days/yr}) \times (1 \text{ mi}^2 / 640 \text{ ac}) \times (1 \text{ ac} / 43{,}560 \text{ ft}^2) \times (12 \text{ in/ft})$$
$$= 12.0 \text{ in/yr}$$

Therefore, the mean groundwater flow (R_e) for the project area is 30.5 cm (12.0 in) per year.

Step 4 Determine Overland Runoff

As the total runoff is comprised of both an overland runoff and groundwater flow, the overland or surface runoff can be determined by subtracting the groundwater recharge from total runoff, as shown below.

$$R_{osr} = R - R_e$$
$$9.4 \text{ cm } (8.1 \text{ in}) = 51.1 \text{ cm } (20.1 \text{ in}) - 30.5 \text{ cm } (12.0 \text{ in})$$

Therefore, of the 51.1 cm of total runoff that flows off of the site, 9.4 cm flows off overland to the stream and 30.5 cm flows to the stream through groundwater as base flow to the stream. If groundwater flow is unavailable for a specific site, the overland flow can be calculated directly using one of the hydrologic procedures presented in Chapter 7, *Hydrology*, and groundwater flow separated from the total runoff using the previous relationship.

Step 5 Determine Water Loss

With annual precipitation and total runoff known, the water loss can be determined by subtracting the total runoff from the precipitation. Therefore, the total water loss for this site is:

$$WL = P - R$$
$$55.6 \text{ cm } (21.9 \text{ in}) = 106.7 \text{ cm } (42.0 \text{ in}) - 51.1 \text{ cm } (20.1 \text{ in})$$

Water Resource Report 40, *Summary of Groundwater Resources of Luzerne County, Pennsylvania* (Newport, 1977) reports the total evaporation and transpiration in Luzerne County, Pennsylvania as 55.9 cm (22 in), in other words, 55.9 cm (22 in) of water is returned to the atmosphere annually and does not enter the groundwater system or runoff overland to the stream. The evapotranspiration data from this report concurs with the water loss estimate developed independently by this example. As the water loss portion of the water budget is very close to the evapotranspiration estimate listed in the report, groundwater inflow into the project area from another watershed and extraction of groundwater from below the surface is assumed to be insignificant and are established as zero. If a source for evapotranspiration is not available, an estimate of the evapotranspiration can be attained from Figure 11.4.

Therefore, the average annual water budget for the project site can be written as:

$$P = R + WL$$
$$106.7 \text{ cm } (42 \text{ in}) = 51.1 \text{ cm } (20.1 \text{ in}) + 55.6 \text{ cm } (21.9 \text{ in})$$

Step 6 Project Assessment and Water Budget Implications

The new roadway results in the construction of approximately 0.66 acres of new impervious surface.

$$(244 \text{ m of road}) (11 \text{ m wide}) (1 \text{ ha}/10{,}000 \text{ m}^2) = 0.27 \text{ ha}$$
$$(800 \text{ ft of road}) (36 \text{ ft wide}) (1 \text{ acre}/43{,}560 \text{ ft}^2) = 0.66 \text{ acres}$$

Assuming the pavement is entirely impervious; all of the precipitation hitting this surface will runoff and not infiltrate into the ground. Therefore, using the water budget developed previously, this will cause the groundwater recharge component of the water budget to get smaller and the overland surface component to increase.

1. Calculate annual precipitation falling on new roadway impervious surface by multiplying precipitation by roadway surface area and convert to inches for the project area.

$$(106.7 \text{ cm}) (0.27 \text{ ha}) / 20.3 \text{ ha} = 1.4 \text{ cm}$$
$$(42 \text{ in}) (0.66 \text{ acres}) / 50 \text{ acres} = 0.6 \text{ in}$$

2. Without any post-construction stormwater management controls, the water budget must be adjusted to compensate for changes to the site by subtracting depth of rainfall calculated in the step above from groundwater recharge and adding it to overland surface runoff (ignoring any effect of road on annual evapotranspiration).

$$P = R_{osr} + R_e + WL$$
$$106.7 \text{ cm } (42 \text{ in}) = 22.0 \text{ cm } (8.7 \text{ in}) + 29.1 \text{ cm } (11.4 \text{ in}) + 55.6 \text{ cm } (21.9 \text{ in})$$

3. To maintain the hydrologic balance of the site, post-construction stormwater management controls, such as a stormwater wetland or infiltration device should be installed to return the 1.4 cm (0.6 in) of annual runoff back into the ground. Thus, maintaining groundwater levels and preventing any potential deleterious impacts to the downstream waterway.

11.7 GLOSSARY

Adsorption - The chemical process in which molecules adhere to the surface of a solid.

Assimilation - The chemical transformation of pollutants and nutrients into other compounds.

Bioassimilation - The chemical transformation of pollutants and nutrients by living organisms into other compounds.

Biodegradation - The process by which organic substances are broken down by other living organisms.

Biota - The total collection of organisms of a geographic region or a time period.

Headcut - The downcutting or degradation of a stream channel in an upstream direction.

Microbial - Of or about organisms that are microscopic and too small to be visible to the human eye.

Tractive Shear Stress - The force per unit area acting parallel to the surface.

Volatilization - The process whereby a dissolved substance is vaporised.

11.8 CHAPTER 11 NOMENCLATURE

Symbol	Definition	Units
A	The surface area of the water in the Water Budget Equations	m^2 or ft^2 (acres)
A	Drainage area of sub-area	km^2 or mi^2 (acres)
CN	NRCS curve number	dimensionless
D	The depth of the water in Water Budget Equations	cm or in
dh/dx	The hydraulic gradient in Darcy Equation	m/m or ft/ft
EDV	Extended Detention Volume	cm or in
ET	Evapotranspiration in Water Budget Equations	cm or in
GWI	Groundwater inflow in Water Budget Equations	cm or in
GWO	Groundwater outflow in Water Budget Equations	cm or in
I	Inflow per unit time	m^3/s or cfs
I	Monthly heat index in Thorntwaite Equation	dimensionless
K	The hydraulic conductivity or permeability of a soil	cm/s, m/d or ft/d
O	Outflow per unit time	m^3/s or cfs
P	Total Precipitation in Water Budget Equations	cm or in
PET	Potential evapotranspiration in Thorntwaite Equation	mm/mo or in/mo
PRV	Permanently Removed Volume	cm or in
Q	Groundwater discharge in Darcy Equation	cm^3/s, m^3/d or cfs, gpm
Q	Runoff	cm or in
R	Average annual total runoff in Water Budget Equations	cm or in
R_e	Groundwater recharge/discharge in Water Budget Equations	cm or in
R_{osr}	Overland surface runoff in Water Budget Equations	cm or in
S	Potential max retention for NRCS Rainfall-Runoff Equation	mm or in
SWI	Surface water inflow in Water Budget Equations	cm or in
SWO	Surface water outflow in Water Budget Equations	cm or in
T_a	Mean monthly air temperature	°C or °F
t	Time	s, min, hr, d, mo, yr
V	The volume of water in the wetland in Water Budget Equations	m^3 or ft^3
WL	Average annual water loss in Water Budget Equations	cm or in
$\Delta S/\Delta t$	The change in storage per unit time	m^3/s or cfs
$\Delta V/\Delta t$	Change in storage in Water Budget Equations	cm/time or in/time

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CHAPTER 12

EROSION AND SEDIMENT POLLUTION CONTROL

12.0 INTRODUCTION

Controlling erosion and sedimentation (E&S) during construction of our highways is an integral part of protecting and maintaining our water resources. For this reason, the Federal Clean Water Act, 33 U.S.C. § 1251 et seq., and the Pennsylvania Clean Streams Law, Act of June 22, 1937 (P.L. 1987, No. 394) as amended 35 P.S. § 691.1 et seq. provide regulatory requirements for construction activities and the management of stormwater runoff generated by highways. Best Management Practices (BMPs) serve as the primary means of meeting these requirements. BMPs include activities, facilities, measures, or procedures used to prevent or reduce the discharge of pollutants to waters and to minimize erosion and sedimentation. This Chapter categorizes Erosion and Sediment Pollution Control (E&SPC) into three groups: Stabilization BMPs, General BMPs, and In-Channel BMPs. These BMPs are temporary controls used by construction site operators during the period of earth disturbance to control erosion and sedimentation during construction activities.

This chapter will also: 1) give an overview of the regulatory basis for these requirements; 2) guide the reader through the steps of coordinating with the federal, state, and local regulatory authorities; 3) explain the types of analyses that should be performed; 4) describe a variety of BMPs including where and when they can/should be used and how they should be designed to achieve their optimal functionality.

This guidance is consistent with the following regulations:

- Pennsylvania's Clean Stream Law, Act of June 22, 1937 (P.L. 1987, No. 394) as amended 35 P.S. § 691.1 et seq.
- The Federal Clean Water Act, 33 U.S.C. § 1251 et seq.
- 40 CFR § 122.26 – Implementation of the National Pollutant Discharge Elimination System (NPDES) Program – Stormwater Discharges (USEPA, 2006).
- Commonwealth of Pennsylvania, PA Code, Title 25, Chapter 102: Erosion and Sediment Control.
- Commonwealth of Pennsylvania, PA Code, Title 25, Chapter 92: NPDES Permitting, Monitoring, and Compliance.
- Commonwealth of Pennsylvania, PA Code, Title 25, Chapter 93: Water Quality Standards.

The Pennsylvania Department of Environmental Protection (PA DEP) and the Pennsylvania County Conservation Districts (CCDs) participated in the development of this chapter.

It should be noted that all dimensions shown in figures are in millimeters unless otherwise noted. U.S. Customary units are provided in parentheses.

12.1 REGULATORY REQUIREMENTS

A. Overview of the Regulations. Two primary laws address the regulation and management of the discharge of stormwater from construction sites (during construction and post construction): the Federal Clean Water Act, 33 U.S.C. § 125.1 et seq., and the Pennsylvania Clean Streams Law, Act of June 22, 1937 (P.L. 1987, No. 394) as amended 35 P.S. § 691.1 et seq. The Federal NPDES regulations (40 CFR Part 122); PA Code, Title 25, Chapter 92 regulations; and PA Code, Title 25, Chapter 102 regulations have been promulgated pursuant to these two laws, and it is these regulations that set forth the criteria for the permitting of construction projects that involve the discharge of stormwater from construction sites.

While this chapter focuses on the NPDES, Chapter 92, and Chapter 102 regulations, the requirements they impose on a designer, and the associated permit programs, there are numerous other related laws which have been promulgated pursuant to the Clean Water Act and the Clean Streams Law that become intertwined with these permits during the design process and therefore need to be summarized. Appendix 12A, *E&S Related Regulations*, Section 12A.1

provides an overview of related regulations that have an E&SPC and/or Post-Construction Stormwater Management (PCSM) component, and/or may regulate activities that require submission of an E&SPC and/or PCSM Plan.

1. The Federal National Pollutant Discharge Elimination System (NPDES). The NPDES was authorized by the Federal Clean Water Act, and is intended to control water pollution by regulating stormwater discharges into waters of the United States. While this is a Federal program, it is administered by PA DEP.

40 CFR Part 122 contains provisions to implement the NPDES Program under the Clean Water Act. These regulations serve as the guidelines for what each state must do to develop its own NPDES Program, in lieu of a Federal program. Related permits include, but are not limited to the following:

- NPDES Permit for Industrial Wastewater (Part 1).
- NPDES Permit for Stormwater Discharges Associated with Industrial Activities.
- NPDES Permit for Stormwater Discharges Associated With Construction Activities.
- NPDES Phase II Municipal Separate Storm Sewer System (MS4) Permit.

There are two types of NPDES permits for construction activities: a general permit and an individual permit. PA DEP has designed a single set of forms that serve as either the Notice of Intent (NOI) for the General NPDES Permit, or as the application for the Individual NPDES Permit. Information regarding specific requirements of the General and Individual Permits, as well as the qualification for the General Permit can be found on PA DEP's website, at www.dep.state.pa.us. A summary of the qualifications for the General NPDES Permit can be found in Section 12.1.B.

A NPDES Permit is required for projects that propose earth disturbances (other than agricultural plowing and tilling, and timber harvesting activities) that:

- Disturb from 0.4 hectare (1 acre) to less than 2 hectares (5 acres) with a point source discharge to surface waters of the Commonwealth (Note: Based on State regulations, PA DEP permitting requirements apply to surface waters of the Commonwealth, which is broader coverage than the Federal requirement which extends only to waters of the U.S.); or
- Disturb five or more acres (regardless of the type of discharge).

Another program under NPDES is the Phase II MS4 Program. The use of construction and post construction BMPs and an E&SPC Plan serve to meet certain regulatory requirements within the Phase II NPDES Permit. Additional information regarding the NPDES Phase II MS4 program and permit is provided in Appendix 12A, *E&S Related Regulations*, Section 12A.1.

As stated in the General or Individual NPDES permit instructions, a point source is defined as any discernible, confined and discrete conveyance, including, but not limited to, any pipe, ditch, channel, tunnel, well, discrete fissure, or container from which pollutants are or may be discharged.

2. PA Code, Title 25, Chapter 92: NPDES Permitting, Monitoring and Compliance. Chapter 92 was promulgated pursuant to the Pennsylvania Clean Streams Law, provides state implementation authority for the NPDES Program. The NPDES Permits are governed under Chapter 92 and are administered by PA DEP. The Permits governed under Chapter 92 are the same as those listed in Section 12.1.A.1.

3. Commonwealth of Pennsylvania, PA Code, Title 25, Chapter 102: Erosion and Sediment Control. Chapter 102 provides regulatory guidance in accordance with Sections 5 and 402 of the Clean Streams Law, and the NPDES Permits for Stormwater Discharges Associated with Construction Activities. The primary pollutant of concern under Chapter 102 is sediment. Chapter 102 imposes requirements on earth disturbance activities that create accelerated erosion or a danger thereof, and which require planning and implementation of effective soil conservation measures (Section 102.2). It requires individuals to develop, implement, and maintain an approved E&SPC Plan prior to performing earth disturbance activities. In order to meet the requirements, the E&SPC Plan must include the implementation of BMPs to minimize erosion and discharge of sediment. In addition, any timber harvesting or roadway maintenance activity disturbing 25 or more acres of land must apply for, and receive an E&SPC Permit according to Chapter 102 regulations (roadway maintenance activities that disturb less than 25 acres are exempt from permitting requirements). Permitting for PA Code, Title 25, Chapter 102 is handled by the NPDES Permit for Stormwater Discharges Associated with Construction Activities.

B. Permitting Process - The NPDES Permit Application For Stormwater Discharges Associated With Construction Activities. A General Permit (sometimes referred to as a PAG-2 Permit) can be used unless one or more of the following conditions occur. Specifics on these conditions can be found in the PA DEP Instructions for a General or Individual NPDES Permit for Stormwater Discharges Associated with Construction Activities, which can be found on PA DEP's website at www.dep.state.pa.us. These conditions include, but are not limited to the following:

- The project discharges to waters with a designated or existing use of High Quality (HQ) or Exceptional Value (EV) pursuant to PA Code, Title 25, Chapter 93.
- The project discharges to EV wetlands.
- The project is capable of affecting existing water quality standards.
- The project may affect threatened or endangered species or their critical habitat.
- Discharges would contain hazardous pollutants, toxics, or other substances that may cause or contribute to an increase in mortality or morbidity in an individual or the total population, or would pose a substantial present or future hazard to human health or the environment when discharged into waters of the Commonwealth.
- Discharges which individually or cumulatively have the potential to cause significant adverse environmental impact.
- Discharges to waters for which general permit coverage is prohibited under PA Code, Title 25, Chapter 92.83.

Where these instances occur, an Individual NPDES Permit is required. An E&SPC Plan is required as part of both the General and Individual NPDES Permits (as described under Chapter 102 above), along with a Preparedness, Prevention, and Contingency (PPC) Plan and a PCSM Plan. For guidance on how to prepare these plans, refer to Chapter 4, *Documentation and Document Retention*, Section 4.1.B for the NPDES permit application, Section 4.1.C for E&SPC Plans, Section 4.1.F for PPC plans and Section 4.1.E for PCSM plans. The PCSM Plan identifies BMPs that manage and treat stormwater discharges after construction resulting from additional impervious surfaces. The purpose of the PCSM BMPs is to prevent or minimize any increase in quantity (rate and volume) of runoff while also minimizing the factors affecting the quality. In areas where approved Act 167 Stormwater Management Plans exist, the PCSM Plans should be consistent with standards of the Act 167 Plan. (Act 167 Plans are stormwater management plans adopted by a county and approved by PA DEP in accordance with Sections 5 and 9 of the Pennsylvania Storm Water Management Act, of October 4, 1978, P.L. 864 No. 167 32 P.S. § 680.1 et seq. (as amended by Act 63) as described in Appendix 12A, *E&S Related Regulations*, Section 12A.1). See Chapter 14, *Post-Construction Stormwater Management,* for more details on PCSM BMPs.

As discussed above, the NPDES, Chapter 92, and Chapter 102 regulations require permit application and approval for different earth disturbance activities. Table 12.1 provides an overview of these activities and their associated permits.

It is important to determine what regulations and permits apply to project activities early in the project development process. By doing so, coordination with the appropriate resource agencies, county and local planning offices, CCDs, and others can be conducted to gain consensus on approaches and best management practices to best avoid and/or minimize impacts to streams and other natural resources due to erosion and sedimentation or stormwater runoff. This section serves as a summary of the laws and regulations governing earth disturbance activities. Additional details regarding each in the regulation and its associated permits and requirements can be obtained using the citations and website links provided in the preceding sections.

Table 12.1 Relationship of Earth Disturbance Activities, Regulations, and Associated Permits

Activity	Regulations	Requirements/Permits
Earth disturbance of any amount within a special protection watershed	PA Code, Title 25, Chapter 102: Erosion and Sediment Control PA Code, Title 25, Section 93.4c: Implementation of Antidegradation Requirements (if point source discharge).	Written E&SPC Plan
Earth disturbance from 5,000 square feet to less than 1 acre	PA Code, Title 25, Chapter 102: Erosion and Sediment Control	Written E&SPC Plan
Earth disturbance of 1 to less than 5 acres with no point source stormwater discharge	PA Code, Title 25, Chapter 102: Erosion Control	Written E&SPC Plan
Earth disturbance of 1 to less than 5 acres with a point source stormwater discharge to surface waters of the Commonwealth*	PA Code, Title 25, Chapter 102: Erosion and Sediment Control PA Code, Title 25, Chapter 92: NPDES Permitting, Monitoring, and Compliance PA Clean Streams Law Federal Clean Water Act, Section 402 Federal NPDES Regulations at 40 CFR Part 122 PA Stormwater Management Act PA Code, Title 25, Chapter 93: Antidegradation regulations apply to all waters not just special protection waters.	Written and Approved E&SPC Plan NPDES Permit for Stormwater Discharges Associated With Construction Activities. Note: both a General Permit (PAG-2) and an Individual Permit are available. See the summary above on NPDES for details on the qualifications for the General Permit.
All earth disturbances of 5 or more acres *	PA Code, Title 25, Chapter 102: Erosion Control PA Code, Title 25, Chapter 92: NPDES Permitting, Monitoring, and Compliance PA Clean Streams Law Federal Clean Water Act, Section 402 Federal NPDES Regulations at 40 CFR Part 122 PA Stormwater Management Act PA Code, Title 25, Chapter 93: Antidegradation regulations apply to all waters not just special protection waters	Written and Approved E&SPC Plan NPDES Permit for Stormwater Discharges Associated With Construction Activities. Note: both a General Permit (PAG-2) and an Individual Permit are available. See the summary above on NPDES for details on the qualifications for the General Permit.
* Activities that occur within EV or HQ Watersheds, as per PA Code, Title 25, Chapter 93, require an Individual NPDES Permit for Stormwater Discharges Associated With Construction Activities.		

12.2 OVERALL PROJECT COORDINATION

Planning for E&SPC BMPs and PCSM BMPs, when required, should be initiated as early in the design process as possible. Integral to this planning is the initiation of a proactive coordination effort with the agencies involved in reviewing and approving plans and permits related to E&SPC and PCSM efforts/controls. The magnitude and complexity of a project will dictate the approvals required and the type and amount of coordination necessary. The approach for a project needs to be tailored to fit that specific project. This section provides a framework to lead the designer through the process of developing a planned and coordinated approach, and identifies key issues to be considered during project development.

A. Early Planning. Early planning is a key element to the successful development and approval of an E&SPC Plan. By planning early, the designer can accelerate project delivery, reduce construction change orders, and reduce overall project costs. This is accomplished by identifying critical design issues early, so that design solutions can be considered at the earliest point possible. These solutions can be incorporated into the initial project design, rather than having to perform design modifications later in the design process to address E&SPC and PCSM issues.

Another important aspect of the planning process is to identify any specific administrative requirements of the CCDs. Although similar, there are differences in the county requirements, including but not limited to, plan presentation and content, the number of draft and final E&SPC Plans needed, whether review fees are required and their amounts, and whether the CCDs have their own E&SPC application forms. It should be noted that review fees are waived for state agencies. By knowing the requirements of the CCD ahead of time, the proper information can be collected and assembled, and the proper contacts can be made early on to help expedite the process.

B. Agency Coordination. Agency coordination can be important to the time-efficient preparation and approval of E&SPC Plans. There are two primary agencies involved in the E&SPC and PCSM process: PA DEP and the CCDs.

1. Pennsylvania Department of Environmental Protection. PA DEP has oversight into the review and approval of E&SPC Plans as required by Title 25, Chapter 102, and review and approval authority for permits under the NPDES Program and Title 25, Chapter 92. Although PA DEP often delegates their approval authority for E&SPC Plans and NPDES Permits to the CCDs, they are still the primary regulatory authority. Coordination with PA DEP is therefore essential in the process of obtaining permits. They can provide useful input into implementation of E&SPC and PCSM Plans. PA DEP should be given the opportunity to be involved in agency field views and agency coordination meetings for the project. This will help to ensure that they have adequate information about the project, and have an opportunity to help identify project concerns, and offer potential solutions early in the design process.

2. County Conservation Districts (CCDs). In many counties, PA DEP has delegated review and approval authority for E&SPC Plans to the CCD. Some CCDs also have authority for the review and approval of NPDES Permits. This is based on their Delegation Level. Coordination with the CCDs is important for the timely approval of E&SPC and Stormwater Management Plans.

Starting coordination early in the project development process ensures that development of a strategy for control of erosion, sediment, and stormwater occurs at the most appropriate stages of project development. Table 12.2 provides a template for coordination between PennDOT and the CCDs. Columns 2 through 4 of Table 12.2 separate projects by size and complexity using the level of the National Environmental Policy Act (NEPA) documentation and the project type as indicators. The left column (Column 1) lists some of the major project activities in the project development process. The suggested level and frequency of coordination is related to the magnitude and complexity of the project.

Table 12.2 Suggested Points for Project Coordination with County Conservation Districts

	Environmental Documentation Level	CEE[1]	EA[1]	EIS[1]
	Typical Project Type(s)[2]	**NA, BR, 3R/4R Betterment**	**NA, BR**	**NA**
Project Activities	Scoping Field View	__	__	__
	Initial Agency Coordination	X[4]	X	X
	ACM Coordination/Agency Field Views[3]	__	X	X
	Public Meetings	__	__	X
	Draft NEPA Document	__	__	X
	Final NEPA Document	__	X	X
	Design Field View	X[4]	X	X
	Pre-Application Meeting	X[4]	X[4]	X
	Pre-Construction Meeting	X[4]	X[4]	X

[1] An "X" indicates a suggested coordination point with the CCD.
[2] Project types listed are intended to reflect the general magnitude of the projects.
[3] CCDs should be invited to attend ACM-type meetings and field views – if a meeting or field view is being held with one specific agency or to address one specific issue, it may not be necessary to invite the Conservation District.
[4] Coordination later in project development may not be necessary for certain types of projects.

KEY: CEE = Categorical Exclusion Evaluation
EA = Environmental Assessment
EIS = Environmental Impact Statement
NEPA = National Environmental Policy Act
ACM = Agency Coordination Meeting
NA = New Alignment
BR = Bridge Replacement
3R/4R = Resurfacing, Restoration, Rehabilitation, & Reconstruction
Betterment = Betterment Projects

Holding joint meetings between PA DEP and the CCDs can be very helpful in accelerating a project by ensuring that communication among the agencies involved occurs "early and often", thereby fostering a coordinated effort to find solutions and make decisions regarding unique situations or areas of concern. Appendix 12D, *E&S Initiative Coordination Information***,** provides the correlation between the various PennDOT Districts, the PA DEP Regional Offices, and the CCDs; it also provides a quick reference of the contact information for the CCD staff and PA DEP Regional Offices.

C. PennDOT District E&S Coordinator Role. The PennDOT District E&S Coordinator serves as the primary point of contact for E&SPC issues, activities, and permits. By providing this point of contact for PA DEP, the CCDs, the Inspector-in-Charge, and others, consistency is provided for all groups involved when dealing with E&SPC concerns. This ensures that concerns will be handled in a timely and efficient manner.

In addition to serving as a primary point of contact, the E&S Coordinator is responsible for coordination of PennDOT's representation at the PA DEP Regional Round Table Meetings. This involves suggesting a PennDOT representative to attend the Round Table Meetings, being responsible for gathering information from the design, construction, and maintenance units, and supplying that information to the selected meeting attendee prior to the meeting, as appropriate.

The role of the E&S Coordinator is very important to maintaining an efficient E&SPC process. By attending the Round Table Meetings, the E&S Coordinator helps to maintain an effective working relationship with PA DEP and the CCDs, and helps to keep PennDOT up to date on new initiatives and concerns regarding E&SPC issues. In

addition, by providing a single point of contact, the E&S Coordinator assists the resource agencies, local officials, and others in providing valuable comments and insight earlier in the process, and provides a conduit for two-way communication and discussions, resulting in an overall project that is more sensitive to water quality issues.

12.3 DESIGN

A. Introduction. Regulatory approval of E&SPC and PCSM Plans can have a significant influence on the design process and the date construction starts. It is important to begin E&SPC and PCSM design early, follow the procedures set forth herein, and keep communication open between the E&SPC and PCSM designers, regulatory agencies (refer to Section 12.2), and other members of the project team. Communication with PennDOT maintenance personnel may also benefit the design process; drawing upon experience and past projects can shed light on those E&SPC and PCSM measures that work best in certain areas.

The purpose of E&SPC Plans is to provide methods for controlling erosion and sediment during the construction of a project. The purpose of PCSM Plans is to provide methods for controlling post-construction stormwater runoff so it will not degrade the physical, biological, and chemical qualities of the receiving surface waters. E&SPC and PCSM should be considered in the preliminary design phase. Guidelines for the presentation of E&SPC Plans are found in Publication 14M, Design Manual, Part 3, *Plans Presentation*.

The following sections provide a framework for gathering information and applying it to E&SPC designs. Since any right-of-way needs must be included in the final Right-of-Way Plan, it is important to evaluate the need for and location of E&SPCs in the preliminary engineering phase in order to provide accurate and complete information for final design.

Publications from the Federal Highway Administration (FHWA) and PA DEP provide commonly recognized methods to design erosion protection and are referenced throughout this guide. The FHWA's publications for hydraulic engineering, including Hydraulic Engineering Circulars (HEC), are available through its website (www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm). The information is subject to frequent changes resulting from updated research findings and design practices. These publications include:

1. FHWA.

a. HEC-11, *Design of Riprap Revetment*. Provides procedures for the design of riprap 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 m^3/s (50 cfs)). (FHWA, 1989)

b. HEC-14, *Hydraulic Design of Energy Dissipators for Culverts & Channels*. Provides design information for analyzing energy dissipation problems at culvert outlets and in open channels. (FHWA, 2006)

c. HEC-15, *Design of Roadside Channels with Flexible Linings*. Provides guidance for the design of stable conveyance channels using flexible linings. The procedures of HEC-15 are applicable for channels carrying discharges less than 1.4 m^3/s (50 cfs). (FHWA, 2005a)

d. HEC-22, *Urban Drainage Design Manual*. Provides guidance for the design of storm drainage systems which collect, convey, and discharge stormwater flowing within and along the highway right-of-way. (FHWA, 2001b)

e. HEC-23, *Bridge Scour and Stream Instability Countermeasures*. Provides guidance for designs implemented by various state departments of transportation in the United States. (FHWA, 2001c)

2. PA DEP.

a. *Erosion and Sediment Pollution Control Program Manual.* Provides an overview and specific design criteria for a vast variety of E&SPC BMPs. (PA DEP, 2000)

b. *Pennsylvania Stormwater Best Management Practices Manual.* Provides an overview and specific design criteria for PCSM BMPs. (PA DEP, 2006)

c. *Water Quality Antidegradation Implementation Guidance*. Provides overview of antidegradation regulations and guidance, including specific requirements for HQ and EV waters. (PA DEP, 2003)

B. Factors Influencing Erosion

1. Principal Factors. The inherent erosion potential of any area is determined by four principal factors: soil characteristics, vegetative cover, topography and climate. Although each is discussed separately herein, they are interrelated in determining erosion potential.

2. Soil Characteristics. The properties of soil that influence erosion by rainfall and runoff are those that affect the infiltration capacity of a soil and those that affect the resistance of a soil to detachment and being carried away by falling or flowing water. Soils containing high percentages of fine sands and silt are normally the most erodible. As the clay and organic matter content of soils increases, the erodibility decreases. Clays act as a binder to soil particles, thus reducing erodibility. However, although clays have a tendency to resist erosion, once eroded they are easily transported by water. Soils high in organic matter have a more stable structure that improves their permeability. Such soils resist raindrop detachment and infiltrate more rainwater. However, soils with high organic content are less desirable for structural fill due to issues related to decomposition. Clean, well-drained and well-graded gravels and gravel-sand mixtures are usually the least erodible soils. Soils with high infiltration rates and permeabilities reduce the amount of runoff and, as a result, the erosion potential.

3. Vegetative Cover. Vegetative cover plays an important role in controlling erosion in the following ways:

- Shields the soil surface from the impact of falling rain.
- Holds soil particles in place.
- Maintains the soil's capacity to absorb water.
- Slows the velocity of runoff.
- Removes subsurface water improving infiltration and permeability, between rainfalls through the process of evapotranspiration.

By limiting and staging the removal of existing vegetation and by decreasing the area and duration of exposure, soil erosion and sedimentation can be significantly reduced. Special consideration should be given to the maintenance of existing vegetative cover on areas of high-erosion potential such as erodible soils, steep slopes, drainage ways and the banks of streams.

4. Topography. The size, shape and slope characteristics of a watershed influence the amount and rate of runoff. As both slope length and gradient increase, the rate of runoff increases, and the potential for erosion is magnified. Slope orientation can also be a factor in determining erosion potential.

5. Climate. The frequency, intensity and duration of rainfall are fundamental factors in determining the amounts of runoff produced in a given area. As both the volume and velocity of runoff increase, the capacity of runoff to detach and transport soil particles also increases. Where storms are frequent, intense, or of long duration, erosion risks are high. Seasonal changes in temperature and variations in rainfall, help to define the high erosion risk period of the year. When precipitation falls as snow, no erosion will take place. However, in the spring the melting snow adds to the runoff and erosion hazards are high. Because the ground is still partially frozen, its absorptive capacity is reduced. Frozen soils are relatively erosion-resistant. However, soils with high moisture content are subject to uplift by freezing action and are usually very easily eroded upon thawing.

C. Erosion and Sediment Pollution Control (E&SPC) Plan. E&SPC Plan development begins in the Preliminary Engineering Phase of a project. E&SPC BMPs can be classified as either temporary or permanent, depending on whether they will remain in use after construction is complete.

The designer should approach the E&SPC design by evaluating the E&SPC principles listed below. These principles usually are integrated into a system of vegetative measures, structural measures, and management techniques to develop a plan to prevent erosion and control sediment. In most cases, a combination of limiting time of exposure,

judicious selection of erosion control practices, and use of sediment trapping facilities will be the most practical strategy.

E&SPC Principles

1. Plan the highway project to fit the particular topography, soil types, drainage patterns and natural vegetation in the most practical way. Try to avoid locations with steep slopes and erodible soils where possible.
2. Plan the phases or stages of construction to minimize the extent and duration of soil exposure. Disturbed areas, not currently under construction, should have temporary or permanent stabilization. Grading should be completed as soon as possible after its initiation. As soon as the grade is finalized, permanent surface stabilization cover should be established in the area. As cut slopes are made and as fill slopes are brought up to grade, these slopes should be stabilized as the work progresses.
3. Apply erosion control BMPs onsite to prevent accelerated erosion. Keep soil covered as much as possible with temporary or permanent vegetation or with various mulch materials. Special grading methods, such as roughening a slope along the contours or tracking with a cleated dozer, may be used. Other practices include diversion structures to divert surface runoff from exposed soils, and grade stabilization structures, such as Geocell Slope Confinement System, Polyacrylamides, and Articulated Concrete Block Revetment System, to prevent erosion caused by surface runoff.
4. Apply perimeter control practices to protect the construction site from upslope off-site runoff and to prevent sedimentation damage to areas downslope of the construction site. This principle relates to using effective means to prevent sediment-laden runoff from exiting the site without first being treated by an E&SPC BMP. BMPs are practices that effectively isolate the construction site from surrounding properties and prevent sediment that is produced from being transported off of the site. Diversions, berms, sediment traps, silt barrier fences, vegetative and structural sediment control measures are used as perimeter controls. Generally, sediment can be retained by two methods: (a) filtering runoff as it flows through an area, and (b) impounding the sediment-laden runoff for a period of time so that the soil particles settle out.
5. Keep runoff velocities low and retain sediment on the site as much as possible. Removal of existing ground cover and loss of permeable surface area during construction may increase both the volume and velocity of runoff. These changes must be considered when designing erosion controls. Keeping slope lengths short, gradients low, and preserving natural vegetative cover can keep stormwater velocities low and limit erosion hazards. Runoff from the construction site should be treated by appropriate sediment removal BMPs and then conveyed to a stable outlet using storm drains, diversions, stable waterways or similar measures.
6. Use appropriate methods to manage groundwater and stream base flows from work area. In-channel work may require regulatory permits and implementation of BMPs for diverting stream base flows around the work area. Effective diversion moves the stream base flows around the work area and introduces it back into the natural drainage course, minimizing the impact to the stream environment. If the diversion involves a channel, appropriate lining will prevent additional sediment from polluting the stream. In-channel work may require dewatering in combination with a diversion.
7. Construction Scheduling (Staging of Earthmoving Activities). Work should be planned to limit the time soils are exposed. E&SPC facilities should be made operational prior to each earth disturbance and prior to use. NOTE: It may be helpful to coordinate this aspect of E&SPC Plans with the MPT plan.

D. Post-Construction Stormwater Management (PCSM) Plan. A PCSM Plan includes a written narrative, identification and location of permanent BMPs, plan drawings of permanent BMPs, operation and maintenance procedures and supporting calculations and measurements. Refer to Chapter 14, *Post Construction Stormwater Management* for information on PCSM design and policy. PCSM discharge data and supporting calculations are required on the Notice of Intent (NOI) form for the general and individual NPDES permits.

E. Ongoing Coordination. In addition to the coordination discussed in Section 12.2 it is important to coordinate the E&SPC design with other areas of the overall project to avoid conflicts and incorporate additional design considerations.

The designer needs to ensure that BMPs will be placed to avoid potential disturbance to their integrity or function. It is also important to place BMPs as to avoid potential construction and traffic disturbances. The following areas should be considered:

1. Clear Zone. The designer should consider Clear Zone issues when designing and placing E&SPC and PCSM BMPs. Refer to Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 12.

2. Maintenance and Protection of Traffic (MPT). The designer should coordinate E&SPCs with the MPT Plan to ensure that good traffic patterns are maintained during construction and that effective E&SPC BMPs are implemented at the same time. BMPs that are incompatible with traffic should not be placed in areas where traffic will traverse them.

3. Construction Phasing/Sequencing. The designer should be aware of the planned project phasing to provide adequate protection of BMPs during the project construction cycle and to ensure that E&SPC BMPs have an adequate useful life. For example, when placing a Sediment Basin consider if a Sediment Basin is needed for just one phase or multiple phases of construction. If one is needed for multiple phases, the designer should attempt to place it in a location where it can be used for the multiple phases.

4. Right-of-Way. The appropriate selection of effective BMPs will require earlier effective right-of-way impact planning. Inappropriate selection of ineffective BMPs is not an acceptable alternative to early effective right-of-way impact planning. Where practical, E&SPC devices should be located within the normal right of way. After the E&SPC and PCSM BMPs are determined, and a preliminary design is complete, determine the right-of-way needs for each of the following categories:

- Temporary Construction Easements.
- Occasional Flowage Easements.
- Drainage Easements.
- Channel Easements.
- Slope Easements.
- Fee Simple Acquisitions.

F. Hydrologic Analysis. Hydrologic analyses of the drainage area associated with the project under the following assumed conditions may be necessary to determine the impact on the project:

- Pre-construction - Evaluate stormwater runoff prior to disturbance.
- Construction - Evaluate stormwater runoff during disturbance.
- Post-construction - Evaluate stormwater runoff resulting from additional impervious surfaces after the project is finished.

For most construction E&SPCs the 2-year design storm should be used. If the project is in a Special Protection or High Quality/Exceptional Value Watershed, a greater return period may be warranted. For example, a 5-year design storm is required for temporary channels in special protection waters; if the channels are permanent then a minimum 10-year design storm is required. Also refer to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10 to determine minimum design requirements.

Refer to the specific BMP design details in Section 12.4 through Section 12.6 for additional guidance on calculation of flows. For additional guidance refer to Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10 and Chapter 7, *Hydrology*.

G. BMP Selection. Table 12.3 summarizes E&SPCs and can provide the designer with guidance when selecting BMPs for a project. The classifications of high, moderate, and low are relative to other BMPs listed in the table. "Varies" classification indicates variance based on independent product selection within that BMP category.

Table 12.3 BMP Selection

		Initial Cost	Construction Labor	Maintenance	Functional Longevity
12.4	Stabilization Methods				
12.4.A	Seeding & Mulching	Low	Low	Low	Varies
12.4.B	Rolled Erosion Control Products (RECP)	Varies	Varies	Varies	Varies
12.4.C	Spray on Mulches	Low	Low	Low	Varies
12.4.D	Geocell Slope Confinement	Moderate to High	Moderate	Low	Permanent
12.4.E	Articulated Concrete Block Revetment System (ACBR)	Moderate to High	Moderate	Low	Permanent
12.4.F	Gabions	Moderate to High	High	Low	Permanent
12.5	General E&SPCs BMPs				
12.5.A	Rock Construction Entrance	Moderate	Moderate	Moderate	Varies
12.5.B	Rock Filter Outlet	Low	Low to Moderate	Moderate	Varies
12.5.C	Compost Filter Sock	Low to Moderate	Low	Moderate	Varies
12.5.D	Compost Filter Berm	Low to Moderate	Low	Moderate	Varies
12.5.E	Silt Barrier Fence	Low to Moderate	Low to Moderate	Moderate	Varies
12.5.F	Heavy Duty Silt Barrier Fence	Moderate	Moderate	Moderate	Varies
12.5.G	Vegetative Filter Strips for E&SPC	Low	Low to Moderate	Low	Varies
12.5.H	Pumped Water Filter Bag	Moderate	Moderate	Moderate	Varies
12.5.I	Temporary Slope Pipe	Moderate	Moderate	Low	Varies
12.5.J	Storm Inlet Protection	Low	Low	Moderate	Varies
12.5.K	Outlet Protection: Rock	Moderate	Moderate	Moderate	Varies
12.5.L	Outlet Protection: Stilling Well	High	Moderate to High	Moderate	Permanent
12.5.M	Diversion Ditch	Moderate	Moderate	Moderate	Varies
12.5.N	Channel Lining	Varies	Varies	Varies	Varies

Table 12.3 BMP Selection (continued)

		Initial Cost	Construction Labor	Maintenance	Functional Longevity
12.5.O	Rock Barrier	Moderate	Moderate	Moderate	Varies
12.5.P	Sediment Trap (Embankment)	Moderate	Moderate	Moderate	5 years
12.5.Q	Sediment Trap (Riser)	Moderate to High	Moderate to High	Moderate	5 years
12.5.R	Sediment Basin	High	Moderate to High	Moderate	5 years
12.6	In Channel Erosion Controls				
12.6.A	Bypass Channel With Non-Erosive Lining	Moderate to High	Moderate to High	Moderate	Varies
12.6.B	Temporary Stream Diversion: Flume Through Work Area	Moderate	Moderate	Moderate	Varies
12.6.C	Temporary Stream Diversion: Pump Around In-Channel Work Area	Moderate	Moderate	Moderate	Varies
12.6.D	In-Stream Cofferdam	Moderate	Moderate	Moderate	Varies

H. NPDES Permit Preparation. The NPDES Permit for Stormwater Discharges Associated with Construction Activities can be either a General Permit or an Individual Permit. Document preparation has been designated under PA Code, Title 25, Chapters 92 and 102. The following is a list of documents that are required for the type of permit that is being sought.

- General Permit:
  - Notice of Intent Form.
  - Worksheets (attached to permit).
  - Location Map.
  - Municipal Notification (Including proof and Land Use documentation).
  - PNDI Clearance.
  - E&SPC Plan and Narrative.
  - PCSM Plan and Narrative.
  - Permit Checklist.
  - Permit Fees (these are waived for PennDOT).

- Individual Permit (same as General Permit plus the following):
  - Application for Individual Permit (same form as the NOI).
  - Antidegradation Analysis Module (part of the permit form).
  - General Information Form (GIF).
  - Cultural Resource Notice.

Refer to Chapter 4, *Documentation and Document Retention,* for required documentation and document retention and Chapter 6, *Data Collection,* for required data collection for the NPDES permit.

I. Erosion and Sediment Pollution Control Narrative Preparation. As per requirements set forth under PA Code, Title 25, Chapter 102, when applying for an NPDES construction permit, a written narrative must accompany the permit. The narrative includes all backup documentation needed to substantiate the E&SPC Plans (e.g., design computations). Refer to Chapter 4, *Documentation and Document Retention,* for required documentation and document retention and Chapter 6, *Data Collection,* for required data collection for the erosion and sediment pollution control plan. The suggested narrative outline is as follows:

1. Project Description – A brief but detailed overview of the project, total disturbance and time frame.

2. Location and classification, per PA Code, Title 25, Chapter 93, of Waters of the Commonwealth that may receive runoff from within the project site.

3. Existing topographic features of the project site and immediate surrounding area.

4. Types, depths, slope, locations, and limitation of the soils within the project area.

5. Characteristics of the earth disturbance activities, including past, present and proposed land uses along with the proposed alterations of the project site.

6. Amount of runoff from the project area and upstream watershed area.

7. Description of the location and type of perimeter and onsite BMPs used before, during and after earth disturbance activity.

8. Sequence of BMP installation and removal in relation to the scheduling of earth disturbance activities prior to, during, and after earth disturbance activities.

9. Maintenance program for all temporary and permanent BMPs.

10. Procedures to ensure that the proper measures for the recycling or disposal of materials associated with or from the project site will be undertaken in accordance with PA DEP regulations.

11. Appendices: Location map (USGS) and all supporting calculations organized by BMP type.

12.4 STABILIZATION BMPS

Stabilization BMPs are used to stabilize unvegetated slopes after the earth disturbance is completed.

A. Seeding and Mulching. Seeding and mulching are methods for temporary or permanent stabilization of disturbed earth. Mulch helps to protect the soil from the impact of rain droplets that can dislodge the soil. Mulches are also used to protect the soil and create a better seed germination environment. Mulches will biodegrade, enrich the soil, and improve vegetative growth.

Seeding should be applied after earth moving activities have ceased and when environmental conditions allow proper seed germination and growth. Refer to Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 13 for application guidance. Mulching can be done alone as a temporary erosion protection measure, but seeding must always be accompanied by mulching or a rolled erosion control product.

Refer to Publication 408, *Specifications* for construction guidance. Non-organic mulches used for temporary soil protection may need to be removed in order to establish the finished grade and designated surface treatment. Seed and mulch should be reapplied to bare soil areas if the vegetative cover growth is below uniform 70% coverage, before construction completion.

B. Rolled Erosion Control Products (RECP). Erosion control blankets and mats, collectively known as RECPs, include a wide range of natural and synthetic materials. RECPs reduce soil erosion and, where vegetative cover is established, act as a soil stabilizer. RECPs can be used to provide temporary or permanent stabilization. RECPs are an alternative to mulch. They are effective for stabilizing soils on steep to mild slopes and newly landscaped areas. RECPs hold soil particles in place, reduce the impact force of water droplets on soil particles, and retain soil moisture to promote seed germination. RECPs are also effective in protecting waterways for temporary stabilization until vegetation is established or as permanent stabilization in water channels.

There are many product options available for RECPs. Selecting the right one is based on many factors, such as:

- Duration protection is needed.

- Location parameters, such as slope, soil and hydraulic conditions.
- Relative costs to purchase, install and maintain.
- Aesthetics.
- Rate of degradation.

Most RECPs are considered temporary (i.e., not expected to remain functional for the life of the facility). Turf reinforcement mats are considered permanent. RECPs should be used in lieu of mulching on soil slopes 1V:3H (3H:1V) and steeper.

RECPs should be used in the following situations:

- Slopes 1V:3H (3H:1V) and steeper.
- Earth disturbance areas within 15 m (50 ft) of a non-special protection surface water and within 30 m (100 ft) of a special protection surface water. (Note: E&SPC Plans must address how erosion from disturbed areas will be controlled before the RECP is installed).
- Where soil conditions (e.g., low fertility, low moisture, erodibility, etc.) make revegetation difficult.
- As temporary mulching to protect seeded areas from washouts.
- Areas where it is desirable to accelerate seed germination and growth.
- Areas prone to high winds.

RECPs are not effective in slump prone areas. When slope stability problems are anticipated or encountered, appropriate counter measures such as reducing steepness of slope, diverting upslope runoff, reducing soil moisture, loading the toe, or buttressing the slope should be considered. Areas that contain rocky soil are problematic because proper staking / stapling cannot be achieved, and there tends to be inadequate soil medium to place top soil.

PennDOT's designations for RECPs are described in Publication 13M, Design Manual, Part 2 - Highway Design, Chapter 13, Publication 584, PennDOT Drainage Manual Chapter 8, and Publication 408, Highway Construction Specifications, Section 806.

The design of erosion protection of a slope involves selecting the appropriate product for the situation. Determine the slope of the unstabilized area(s) requiring protection and estimate the duration the protection is needed. Refer to Table 12.4 to select the best product for the project. Refer to Chapter 8, *Open Channels,* when it is desired to use an RECP for channel or ditch protection.

Table 12.4 RECP Types for Slope Application

Type	Functional Longevity	Product Description	Maximum Slope V:H (H:V)
1.A	3 months	Netting/Open Weave Textile	1:5 (5:1)
1.B		Erosion Control Blanket Without a Net	1:3 (3:1)
1.C		Single Net Erosion Control Blanket	1:3 (3:1)
1.D		Double Net Erosion Control Blanket	1:2 (2:1)
2.A	12 months	Netting/Open Weave Textile	1:5 (5:1)
2.B		Erosion Control Blanket Without a Net	1:3 (3:1)
2.C		Single Net Erosion Control Blanket	1:3 (3:1)
2.D		Double Net Erosion Control Blanket	1:2 (2:1)
3.A	24 months	Open Weave Textile	1:2 (2:1)
3.B		Erosion Control Blanket	1:1.5 (1.5:1)
4.A	36 months	Open Weave Textile	1:1 (1:1)
4.B		Erosion Control Blanket	1:1 (1:1)
5.A	Permanent	Turf Reinforcement Mat	1:1 (1:1)
5.B			1:1 (1:1)
5.C			1:0.5 (0.5:1)
5.D			1:0.5 (0.5:1)
5.E			1:0.5 (0.5:1)
5.F		High Performance Turf Reinforcement Mat	1:0.5 (0.5:1)

Cut slopes in soil that are altered or steepened are to be evaluated for overall, long-term stability by the District Geotechnical Engineer, following guidance in Publication 293, Chapter 6. In general, most soil slopes steeper than 1V:2H (2H:1V) require project-specific evaluation. Soil slopes 1V:1H (1H:1V) or steeper are rarely acceptable and would be limited to localized areas no more than a few feet in height.

Refer to Publication 408, *Specifications*, Section 806.3 for construction methods. The performance of any RECP depends on surface preparation to ensure continuous contact of the blanket or mat with the underlying soil as well as following the installation techniques of terminal ends, material joints, overlaps and edges at the crest, toe and sides. The installation instructions provided by the manufacturer of the product should also be followed. If these instructions are not available, a typical anchoring method may be appropriate. Use care that the plans show clear guidance for blanket/mat orientation, check slots, anchoring, and anchoring patterns. If washouts occur, determine the cause of washout and consider replacing RECP with one that has a higher maximum shear stress value or using an alternative BMP.

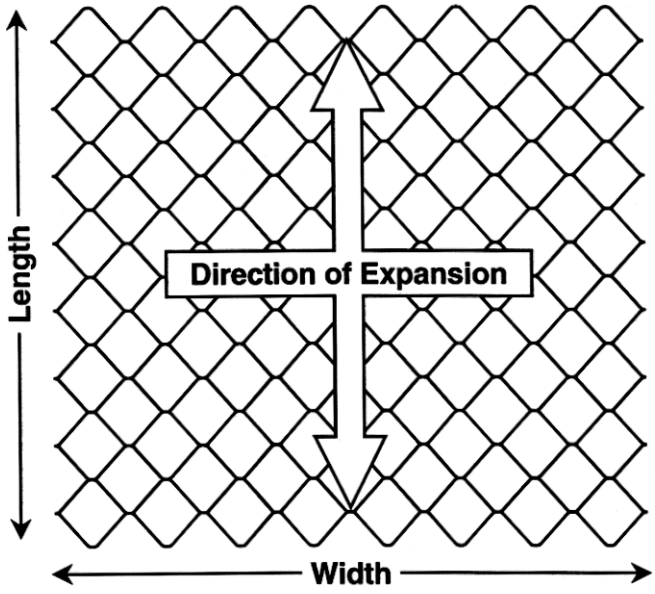
C. Spray on Mulches. Mulching, when applied with seeding, is a method to provide temporary or permanent stabilization for disturbed earth. Spray on mulching is a method of applying mulch, in most cases along with seed and fertilizer, as a controlled spray from a hydraulic tanker truck. Advantages of spray on mulching include ease of applying in a uniform product distribution, ability to apply in remote locations without disturbance of the treated landscape, a faster germination rate of the seed due to a specifically developed micro-environment of the mix, and a higher moisture retention content. Spray on mulches can be used in lieu of RECPs on slopes flatter than 1V:3H (3H:1V).

Vegetated areas are considered to be permanently stabilized when a uniform 70% vegetative cover of erosion resistant perennial species has been achieved. Prior to completion of construction, the spray on mixture should be reapplied on any bare soil areas or locations where the 70% vegetative growth cover is not achieved.

For application guidance, refer to Publication 13M, Design Manual Part 2, *Highway Design*, Chapter 13. For construction consultation, refer to Publication 408, *Specifications* and Publication 35, *Approved Construction Materials* (Bulletin 15).

D. Geocell Slope Confinement. A geosynthetic cellular confinement system made of high density polyethylene (HDPE) used to provide permanent stabilization for steep side slopes and protect against erosion. Geocell slope confinement systems may be used for slope and swale protection to prevent embankment erosion on cut/fill slopes that do not require mowing. "Geocell" is a generic term for a geosynthetic cellular confinement system manufactured from HDPE. In place, geocell has the appearance of a honeycomb structure. The cells are filled with one of a variety of infill materials. The completed confinement system armors, protects, and stabilizes the underlying strata from the erosive effects of wind and water. Geocell has the ability to resist erosive forces by minimizing the downward migration of infill embankment materials.

Figure 12.1 Geocell Slope Confinement System



Geocell is covered by U.S. Patent 4,797,026. Manufacturers wishing to produce geocell must obtain a license from the patent holder during the term of the patent. Publication 35, *Approved Construction Materials* (Bulletin 15) lists suppliers of geocell manufactured by licenses under the U.S. Patent.

Geocell offers the following advantages over traditional rolled erosion control products:

- A wide range of installation temperatures.
- Lightweight, 100 mm (4 in) depth geocell is suitable for most erosion control applications.
- Requires no specialized equipment or labor for installation.
- Flexibility - able to conform to minor inconsistencies in subgrade.
- Utilizes readily available topsoil, Class C concrete, and aggregates (AASHTO No. 8, 67, 57 and PennDOT 2A) for backfill.
- Resistance to corrosion or degradation over time.

Geocell is suggested for use on steep embankment slopes for protection against erosion; however, when used according to this BMP, Geocell does not increase the stability of the underlying slope. Geotechnical engineering

analyses must show that the protected slope is stable prior to consideration of a geocell system using this BMP. This BMP is not suitable for use on natural slopes that are steeper than the angle of repose of the underlying slope material. Do not use geocell to correct erosion problems with a design flow velocity greater than 6 m/s (20 ft/s). When using geocell with a topsoil to topsoil (with plantings) interface, do not use geotextile lining. RECPs can be used to promote seed propagation when topsoil is used as an infill material as an alternative. For design flows greater than 3 m/s (10 ft/s) consider using ACBR or rock.

Geocell applications may include:

- Embankment slope protection.
- Swale surfaces.
- Cut slopes.

The data requirements and calculations for design of a geocell protected surface for cut and fill embankment surfaces include:

- Parameters for stability analysis and dimensional data that define the layout and geometry of the system:
  - Slope/Dimensions: Slope V:H (H:V) and slope length.
  - Soil Properties:
    - Soil Description: angle of internal friction (degrees), cohesion (kN/m^2(lb/ft^2)), unit weight (kN/m^3(lb/ft^3)).
    - Primary Infill Description: angle of internal friction (degrees), cohesion (kN/m^2(lb/ft^2)), unit weight (kN/m^3(lb/ft^3)).
  - Hydraulic Conditions: Surface runoff, concentrated runoff, groundwater seepage.
  - Subgrade/Strata Under Geocell System: soil, aggregate, Class 2 - Type B geotextile.
  - Critical Interface for Sliding and Angle of Shearing Resistance: Geocell infill/foundation soil (degrees), geotextile underlayer/foundation soil (degrees).
  - Select Infill such as: topsoil, coarse aggregate (AASHTO No. 67, 57, 8, PennDOT 2A) and Class C concrete.
  - Ground anchoring options such as: stake anchors, crest anchoring, earth anchors, dead-man anchors, tendons and direct burial.
- Determination of cell depth or minimum allowable angle or repose of the infill.
- Determination of allowable interface friction angle from a given minimum interface friction angle and assumed factor of safety.
- Determination of anchorage requirements, if necessary.
- Cost estimate for geocell system.

The following six-step procedure provides a method for the static analysis of system stability using design figures for a defined slope, soil and infill parameters and an appropriate factor of safety for 100 mm (4 in) deep cells and the standard cell size. The procedure compares the downslope force components and the total resisting forces - interface friction, in-plane tensile anchorage, and in-place resistance of anchor components. It should be noted that this design procedure is only for non-perforate cell walls.

Note: Hydraulic effects are NOT considered in this design procedure. Do not use this procedure to design a geocell channel lining for use under submerged flow conditions.

If the combined resistance of the interface friction and stake anchors is sufficient to resist down slope sliding forces (with a suitable factor of safety), no other anchorage such as a crest anchorage is required. For many applications where additional resistance is required, an anchor trench or minimum length of embedment, with soil cover, can be utilized to develop adequate resistance via a crest anchorage.

When tendons are used in addition to stake anchors to develop the required factor of safety, restraint clip pins are required at specified down slope centers to transfer resisting forces to the tendons which in turn transfer the tensile load to the crest anchor system. Since stake resistance is determined independently of the tendon tensile load, the stakes are required to bear against the cell walls.

Various combinations of crest, stake and restraint pin anchor details are shown in Publication 72M, *Roadway Construction Standards*. Determine system stability through static analysis using the following design figures or the equations accompanying the six-step procedure in Table 12.5.

Table 12.5 Six-Step Geocell Static Design Procedure

Step 1 Determine the Appropriate Cell Depth or Infill Minimum Allowable Angle of Repose
Cell depth. Using Figure 12.2, determine the minimum angle of repose based on the cell infill material. For applications involving Class C concrete as infill material, angle of repose is not applicable; typically 100 mm (4 in) is an appropriate cell depth. On Figure 12.2, find the intersection point of a vertical line up from the design slope and the horizontal line over from the Infill Material Minimum Angle of Repose. Choose the Geocell depth below the intersection point of the two lines.
Infill minimum allowable angle of repose. When the Geocell depth is known, move vertically up from the design Slope to the desired cell depth line and horizontally over to the Infill Material Minimum Angle of Repose. Choose an infill material with a minimum angle of repose no less than that determined. For Class C concrete on a Slope, the steepness may be no greater than the angle of repose of the embankment material.
Step 2 Apply Factor of Safety to Minimum Interface Friction Angle
• Determine the Interface Friction Angle from Figure 12.3 or select from Table 12.6 the appropriate Interface Description and apply the appropriate Factor of Safety (FOS) to the Minimum Interface Friction Angle. The Allowable Interface Friction Angle is then used in Step 3. • The remaining steps of this procedure are illustrated with an assumed FOS of 2.50.
Step 3 Determine Required Resistance of Staking Array for 100 mm (4 in) Cell Depth
• Determine the Required Stake Array Resistance, from Figure 12.4, by moving vertically up from the Slope to the Selected Interface Description (suggested Allowable Interface Friction Angle with applied FOS) and over to the Required Stake Array Resistance. • Refer to Table 12.6 for assumed unit weight and friction angle for each Interface Description. If the actual interface is different than those presented, use the one with the closest values or use the equations provided to perform a site specific analysis.
Step 4 Determine Maximum Downslope Stake Spacing
Using the Required Stake Array Resistance determined in Step 3, the Maximum Downslope Stake Spacing is determined as a function of Slope and Interface Description. Note: The maximum downslope spacing is not to exceed 2.4 m (96 in). This maximum is equal to 12 cells.
Step 5 Determine Individual Stake Resistance for Standard Cell
• Based on the Required Stake Array Resistance determined in Step 3 and considering the Maximum Downslope Stake Spacing as determined in Step 4, determine the Required Individual Stake Resistance by using a downslope spacing which does not exceed the Maximum Downslope Stake Spacing. • Stakes are to be placed in every other cell across the Geocell section. Down slope spacing can vary with infill material. Each cell down is 200 mm (8.0 in). • The solution presented in the Figure 12.6 assumes a 2 cell across by 3 cell down staking pattern for each Interface Description. Use the equations to calculate the Required Individual Stake Resistance for other staking patterns. Note: changing the staking pattern will affect the actual stake length determined in Step 6.
Step 6 Determine Actual Stake Length
• Determine the Total Stake Length (including in-ground length plus 100 mm (4 in) to top of cell) from Figure 12.7 using the staking pattern developed in Step 5 and a typical roadway embankment with the following parameters: o Embankment Soil Unit Weight = 19 kN/m^3 (120 lb/ft^3) o Angle of Internal Friction = 30 degrees • Consideration should be given to the actual Embankment Soil Unit Weight and Angle of Internal Friction, if known, prior to calculation of the Total Stake Length. • J-Pin Stakes, Straight Stakes, and Clip Stakes are three types of stake anchors available. Stakes are #4 rebar. Add additional length for J-Pin type stakes (see detail on Figure 12.7).

Figure 12.2 Relationship Between Cell Depth, Slope Angle and Infill Material Minimum Angle of Repose

STEP 1

$$\phi = \beta - \arctan\left(\frac{d - d_e}{L}\right) \text{ or}$$

d = L tan (β - ϕ) + d_e where:

ϕ = *Minimum angle of repose of the infill material, degrees*

β = Slope angle, degrees

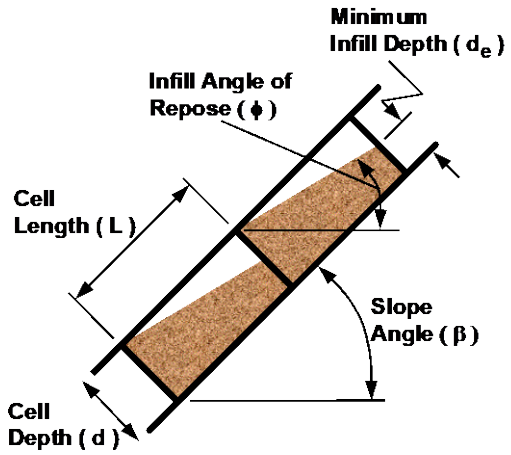
d_e = Minimum acceptable depth of infill against the up-slope wall of the cell, mm (in). This should be 0.5 times the cell depth.

For this case,

$$\phi = \beta - \arctan\left(\frac{0.5\,d}{L}\right)$$

L = Up/down slope-length of the cell, mm (in)
= 8" (assumed cell length)

Note for β: The table below uses slope (H:V) where $\beta = arctan$ (V/H)= $\tan^{-1}$(V/H)



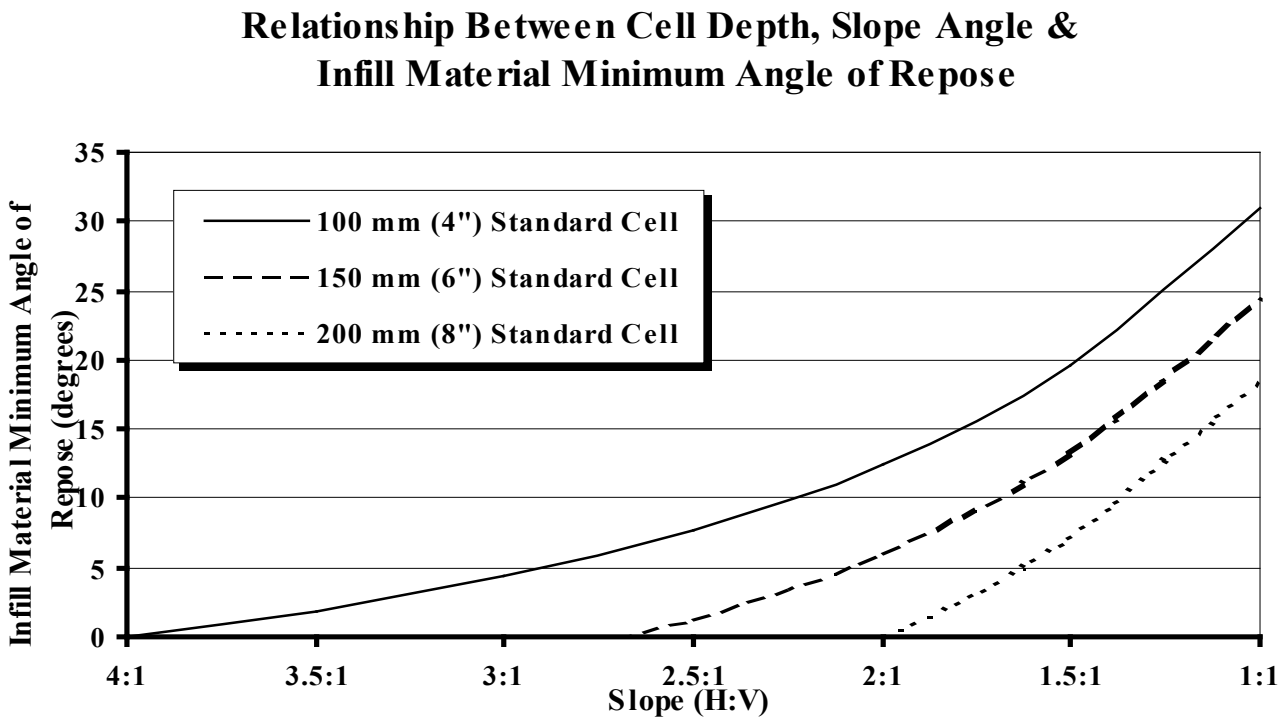


Table 12.6 Typical Soil and Interface Characteristics

STEPS 2 and 3

Values for Cell Infill Material Minimum Angle of Repose are to be used in Step 1 to determine the appropriate cell depth.	**Cell Infill Material**			**Minimum Angle of Repose (degrees) ***
	Topsoil			22
	Gravel			30
	Crushed Stone			34
	Class C Concrete			NA
Values for Interface Friction Angle are to be used in Step 3 after applying the appropriate Factor of Safety.	**Interface Description**	**Unit Weight**		**Minimum Interface Friction Angle**
		kN/m^3	**lb/ft^3**	**degrees ***
	Topsoil to Topsoil (with plantings)	21.2	135	22
	Topsoil over Geotextile	21.2	135	18
	Gravel (2A) over Geotextile	22.0	140	26
	Stone (No. 8) over Geotextile	14.7	94	30
	Concrete over Geotextile	22.7	145	30
	Geotextile over Clay (worst case)	22.7	145	18
Values for Embankment Soil Angle of Internal Friction are to be used in Step 6.	**Embankment Soil**			**Angle of Internal Friction (degrees) ***
	Well/Poorly Graded Gravels (GW/GP)			34
	Silty Gravels (GM)			30
	Clayey Gravels (GC)			28
	Well Graded Sands (SW)			34
	Poorly Graded Sands (SP)			32
	Silty Sands (SM)			28
	Sand – Silt Clay (SM-SC)			26
	Clayey Sands (SC)			26
	Silts/Clayey Silts (ML)			22
	Silts/Clayey Silts (ML-CL)			22
	Clays/Clayey Silts (CL/MH)			20
	Clays (CH)			17

***NOTE**: Friction angle values are maximum presumptive values for preliminary design of geocell infill only. If actual conditions indicate a difference from presumed conditions, laboratory testing shall be conducted to confirm the design.

Figure 12.3 Allowable Interface Friction Angle with Applied FOS

STEP 2

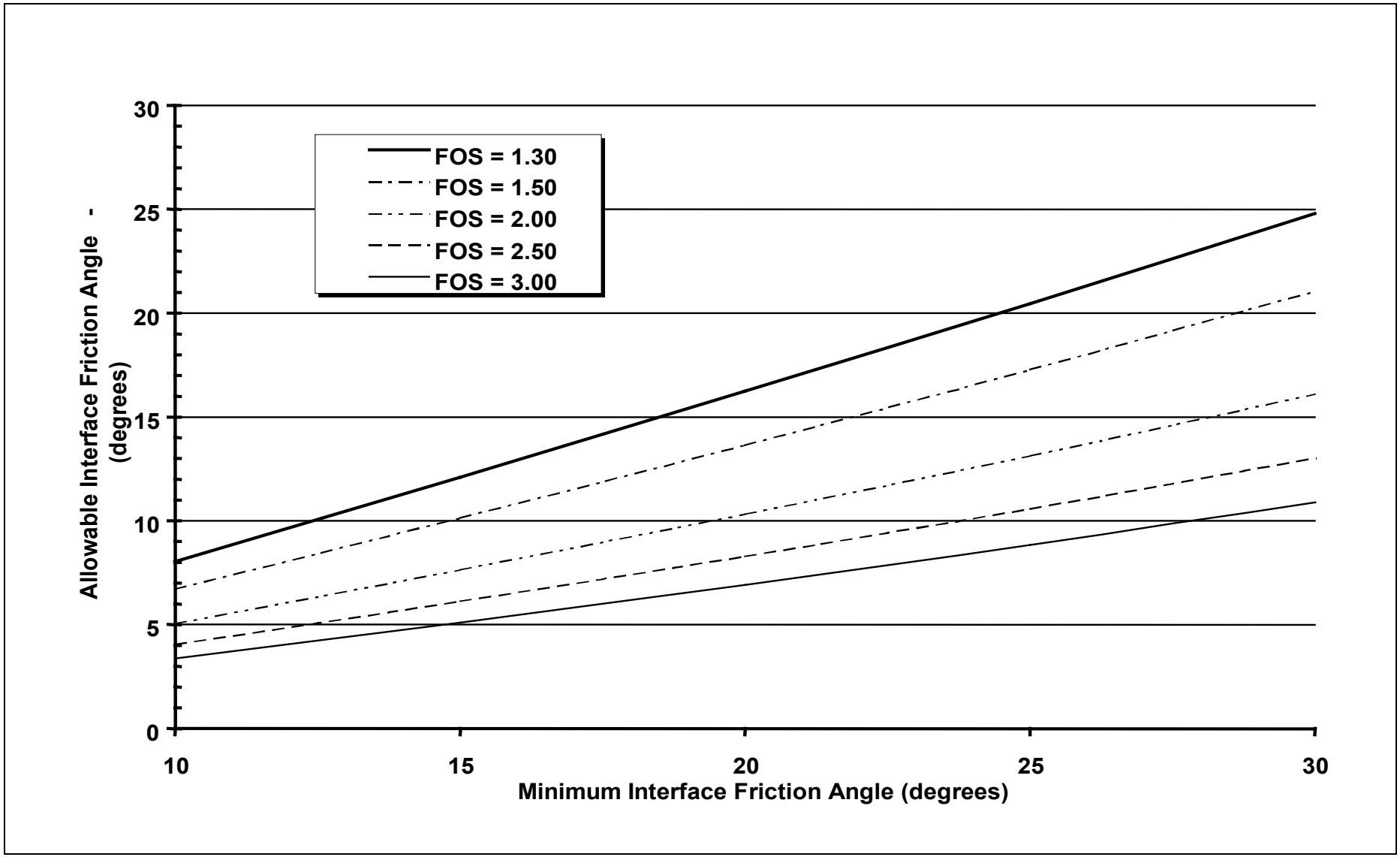


Figure 12.4 Required Stake Array Resistance for Various Slopes and Interface Descriptions with 100 mm (4 in) Cell Depth

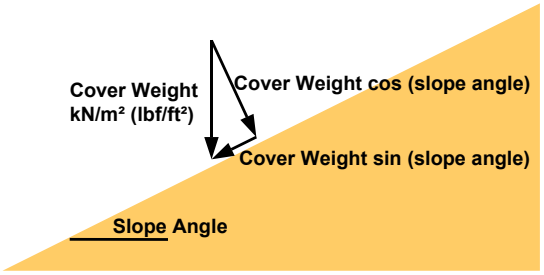
STEP 3

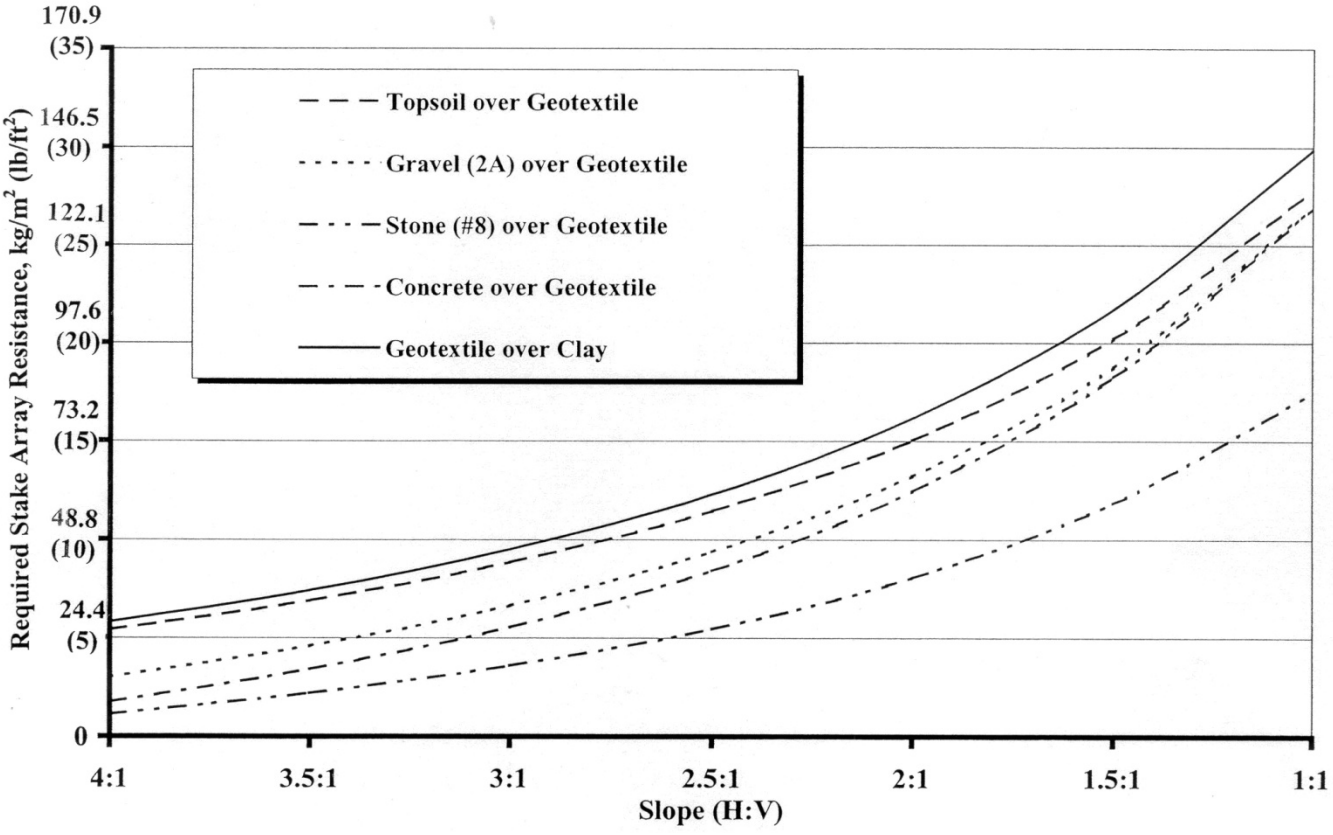
$$RSR_a = W_c \sin\beta - W_c \cos\beta * \tan\left(\frac{\phi}{FOS}\right)$$

Where:

- RSR_a = Required Stake Resistance of the stake Array, kg/m² (lb/ft²)
- W_c = Cover weight, kg/m² (lb/ft²) = $d\gamma$
- d = m (ft) Cell depth
- γ = Unit weight of the infill soil, kN/m³ (lb/ft³)
- β = Slope angle, degrees
- ϕ = Minimum Interface Friction Angle at the bottom of the Geocell, degrees
- FOS = Factor of Safety (use 2.5)

Note for β: Figure in Step 3 uses slope (H:V) where β = arctan (V/H)=tan^{-1}(V/H)





NOTE: The remaining steps of this procedure are illustrated using a FOS value of 2.50.

Figure 12.5 Maximum Downslope Stake Spacing

STEP 4

$$MDS = \frac{SS_u}{FOS\ RSR_a}$$

Where:
MDS = Maximum Downslope Spacing, m (ft)*
SS_u = Ultimate Long-term Seam Strength
100 mm (4 in) deep *Geocell* section, 3.2 kN/m (220 lb/ft)
FOS = Factor of Safety, 2.5 is suggested and used here
RSR_a = Required Stake Resistance, kN/m² (lb/ft²)

NOTE for SS_u: Each 100 mm (4 in) seam must support 0.712 kN (160 lb) for the long-term test. There are 11 seams in a 2.44 m (8 ft) wide panel. Therefore, for a 100 mm (4 in) *Geocell*, SS_u = 0.712*11/2.44 = 3.21 kN/m (160*11/8 = 220 lb/ft).

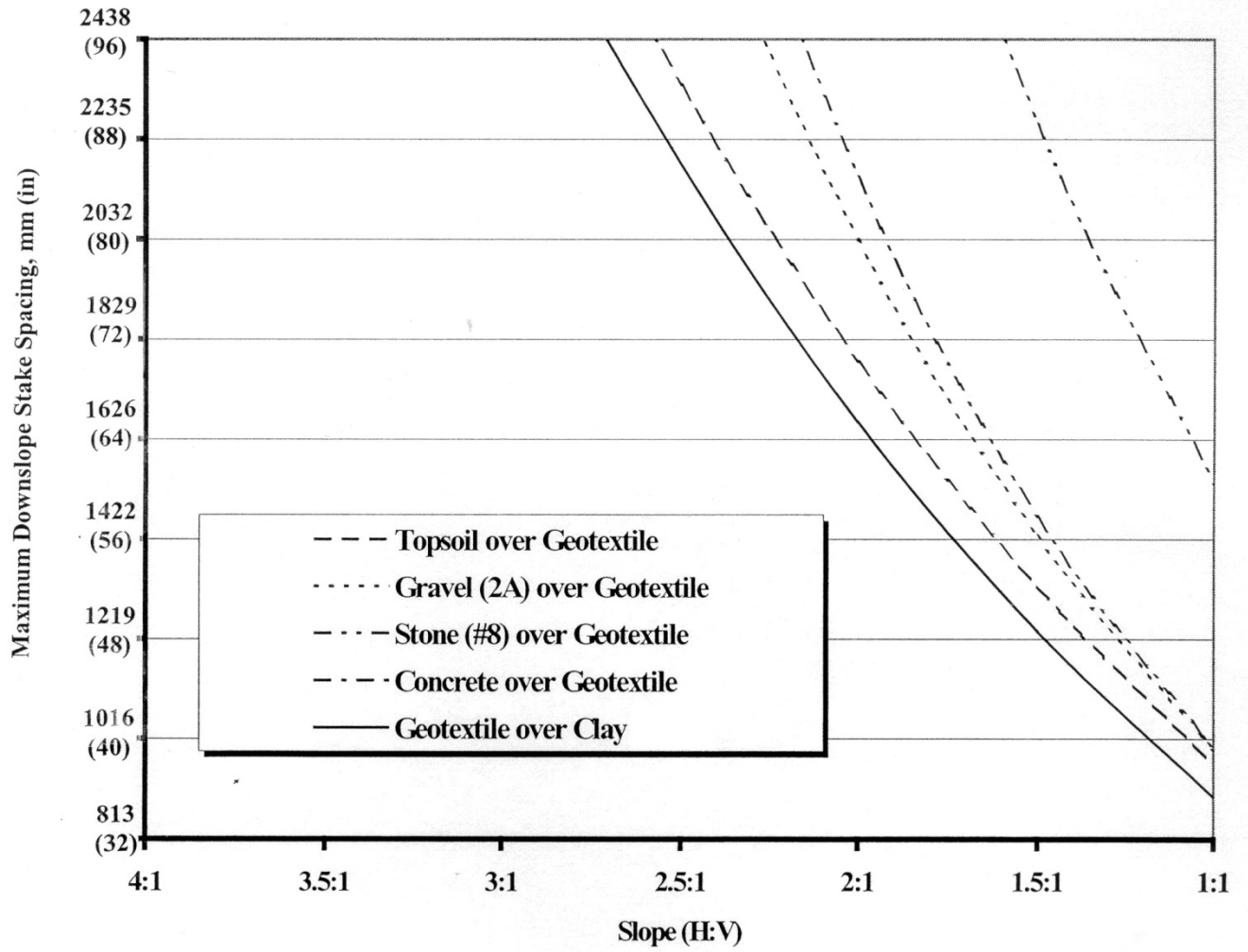


Figure 12.6 Required Stake Resistance for Standard 100 mm (4 in) Cell for 2 Cell Across by 3 Cell Down Staking Pattern

STEP 5

$P_p = RSR_a D_{cs} D_{ds}$	Where: P_p = Required Stake Resistance of the individual stake, N/stake (lb/stake) RSR_a = Required Stake Resistance, kN/m² (lb/ft²) D_{cs} = Cross-slope Spacing number of cells converted to m (ft). D_{ds} = Downslope Spacing number of cells converted to m (ft).

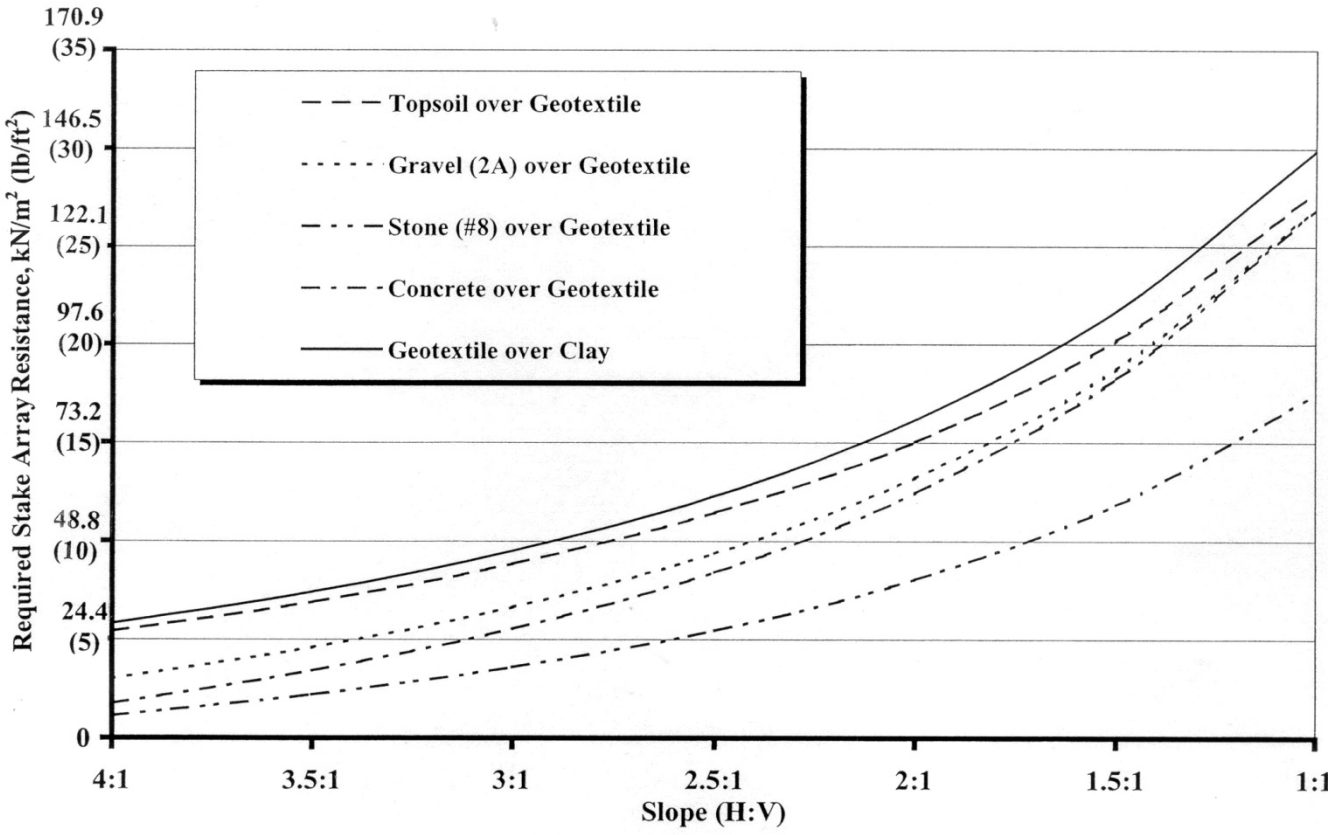


Figure 12.7 Required Stake Length for #4 Rebar Typical Roadway Soil Embankment

STEP 6

Minimum Stake Length l - m (ft) is determined by solving:

$$l = \sqrt{\frac{P_p}{0.5\, d\, \gamma \frac{1+\sin\phi}{1-\sin\phi}}}$$

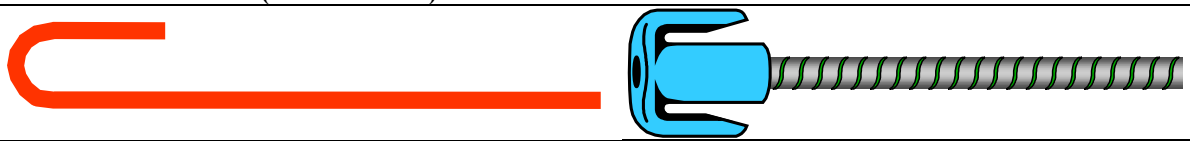
The above is derived from:

$$P_p = 0.5\, \gamma\, d\, K_p\, l^2$$

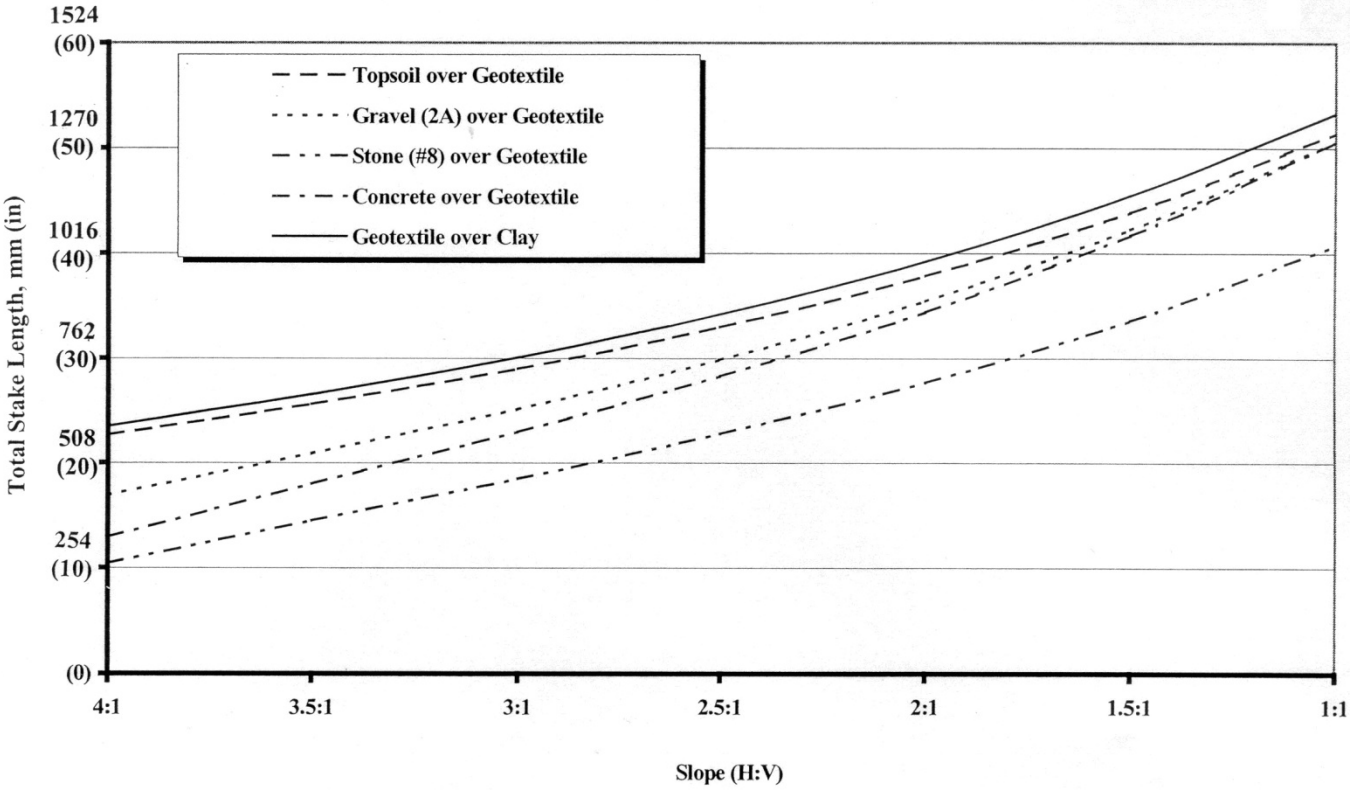
Where:

l = Stake Length, m (ft)
P_p = Required per Stake Resistance, kN (lb)
d = Stake Diameter, m (ft)
γ = Embankment Soil Unit Weight, kN/m³ (lb/ft³)
$Ø$ = Angle of Internal Friction of the Embankment Soil, degrees
Kp = Coefficient of Passive Earth Pressure = $\tan^2(45+\frac{\phi}{2})$

#4 REBAR J-PIN DETAIL (NOT TO SCALE)



For additional stake options, refer to Publication 72M, *Roadway Construction Standards* entitled "Slope Protection, Geocell Cell and Geocell Section Details".

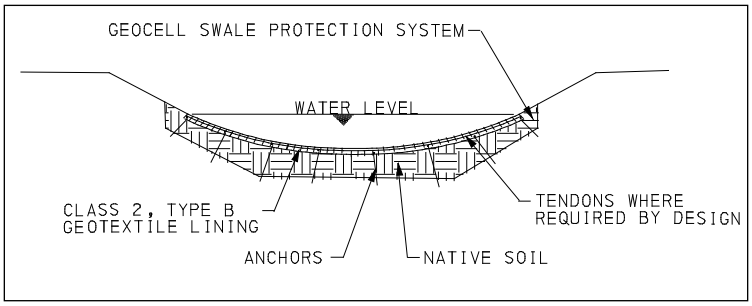


To determine site slope cover stability under hydraulic conditions, consult the appropriate geocell manufacturers or use recognized methods. Suggested geocell infills for swale protection systems are as follows:

Table 12.7 General Guidelines for Selection of Swale Infill Materials

Infill	Peak Flow Velocity		Infill Material
	less than		
	m/s	ft/s	--
Soil with Grass Cover[1]	4.5	15	--
Aggregate	1	3.3	PennDOT 2A
	2	6.6	AASHTO No. 57 or No. 67
Concrete	6	20	--
[1]Swale side slopes above high water level (see Figure 12.8)			

Figure 12.8 Basic Geocell Swale Protection System



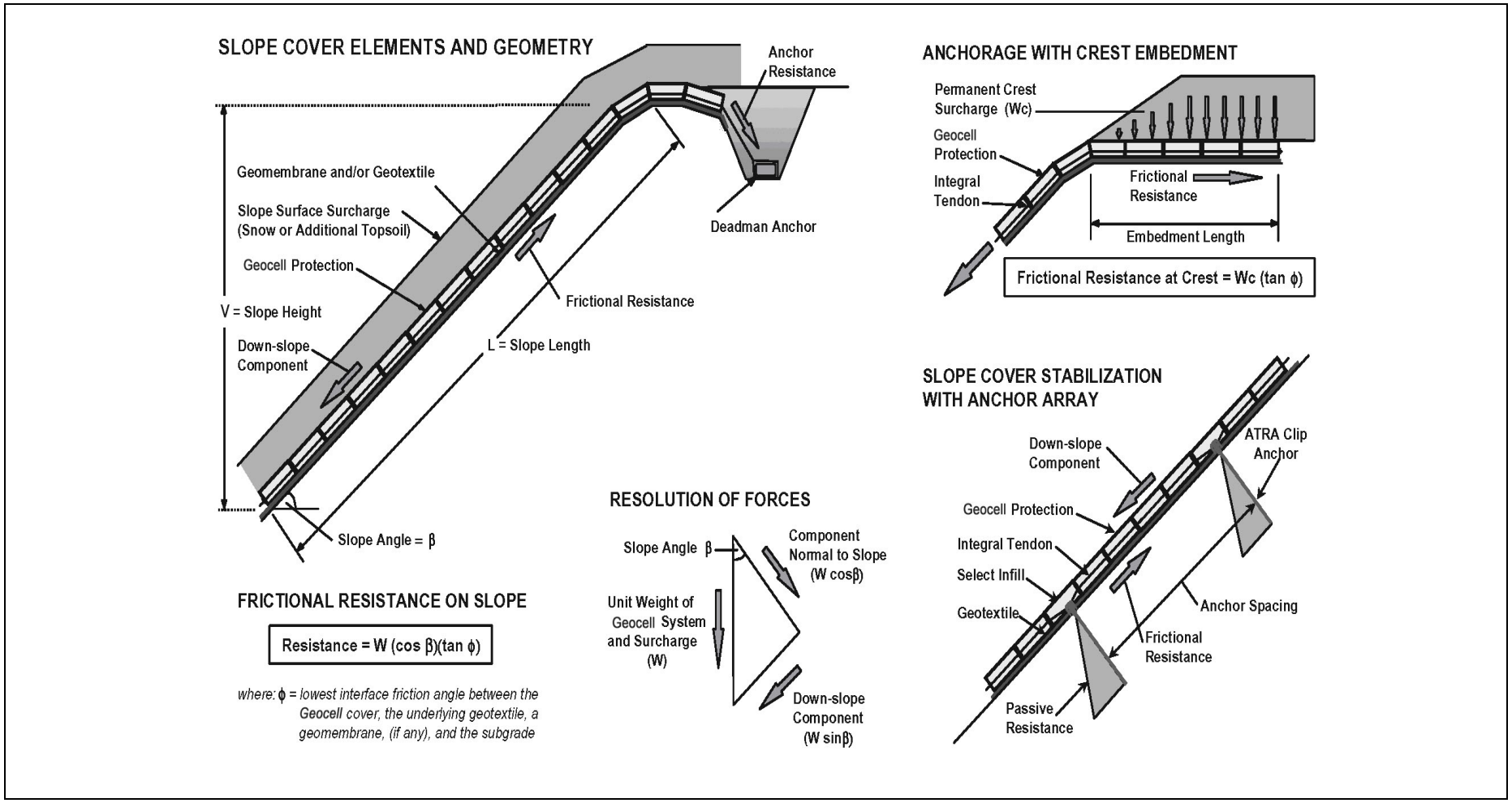
1. General Considerations. A stability analysis of the geocell system in non-hydraulic conditions will determine if the system's weight exceeds the frictional resistance between the system and the subgrade or underlayer (Figure 12.9). If this is the case, supplemental anchorage will be necessary. The integral tendons of the geocell system in conjunction with anchor assemblies then become a critical component to retain the system.

A common method of securing geocell protection on steep slopes involves installation of structural anchors in a uniform grid pattern throughout the cover layer. Slope geometry, subsoil, protection type and possible surcharge loads determine the size, material type and distribution of the anchors. The system requires securing staked anchors or other anchor types to the integral tendons by forming an appropriate knot in the tendon at each anchor location. Driven anchors are in the proper position in the ground when the bottom of the clip or stake end cap is flush with the underside of the geocell section. This ensures that anchors do not project above the surface of the protection after filling the cells.

The tensile resistance of the tendoned geocell system governs the maximum allowable down-slope spacing of individual surface anchors. The size and shear capacity of each anchor dictates the density of the overall anchor array. Use the design procedure in this section or analytical methods provided by the manufacturer to determine complete anchor details. The analysis shall compare downslope forces, both static and dynamic, and resisting forces due to interface friction, in-plane tensile anchorage, and in-plane resistance of anchors. When installation of an anchor array is impractical, a tendoned geocell system with crest anchorage may be one solution.

When additional anchorage is required to resist concentrated surface flow, which imposes a determined flow velocity, depth of flow and hydraulic shear stress, determine the hydraulic shear resistance of the geocell infill materials and the total tractive force on the system in accordance with a system specific design procedure.

Figure 12.9 Stability Analysis of the Geocell Slope Protection System



2. Geocell System with Vegetated Topsoil Infill. Well-established vegetation is an effective and attractive form of protection for slopes that suffer mild or moderate surface erosion. However, the effectiveness of vegetated covers subject to persistent or concentrated surface runoff may not be acceptable. This type of flow can progressively remove soil particles from the root zone, creating rills and gullies that ultimately destroy the protection. For this reason an erosion control blanket should be used to help promote seed propagation.

The geocell walls, which contain the topsoil infill, form a series of check-dams extending throughout the protected slope. By continuously redirecting flow to the surface, the geocell cell walls prevent normal rill development from concentrated flow cutting into the soil. This mechanism also retards flow velocity and reduces the erosive force of runoff. The individual cells contain and protect a predetermined depth of topsoil and the developing vegetative root zone. Roots readily penetrate through the non-woven geotextile underlayer into the subsoil, creating an integrated, blanket reinforcement throughout the slope surface.

Use vegetated topsoil infill in situations where surface flows are intermittent, of moderate intensity, and of relatively short duration (less than 24 hours). Restrict use to peak velocities of 4.5 m/s (15 ft/s) for short durations with established vegetated cover. Prepare existing ground in accordance with manufacturer installation guidelines. Place a Class 2, Type B, non-woven geotextile underlayer. Place degradable erosion control blankets to protect exposed topsoil and seed to promote rapid establishment of vegetation. Select erosion control blankets in accordance with criteria established by respective RECP manufacturers. Determine slope anchorage requirements in accordance with the design procedure in this section or design analysis tools and methods provided by the Geocell manufacturer.

3. Geocell System with Aggregate Infill. If the slope angle is less than the angle of repose of the aggregate infill, it can effectively protect the slope. Aggregate infill systems are dependent on adequate toe support to prevent undermining of the loose aggregate further up the slope. Hydrodynamic forces of concentrated runoff can erode channels within aggregate infills. Confinement of loose aggregate within geocell systems permits use on slopes steeper than otherwise possible. A wide range of aggregate infill/slope geometry combinations can be accommodated by selecting the appropriate cell size and cell depth for the aggregate in question (see Figure 12.10). Aggregate-filled geocell slope protection can stand up to more intense sheet-flow conditions than unconfined aggregate cover layers. The cell walls prevent channeling that could develop within the cover layer by limiting localized flow concentrations and increasing resistance to hydraulic shear stresses.

Loose aggregate infill materials can be effective in geocell systems for surface flows less than 2 m/s (6.6 ft/s). Use a Class 2, Type B, non-woven geotextile underlayer to prevent loss of fine-grained subsoil particles.

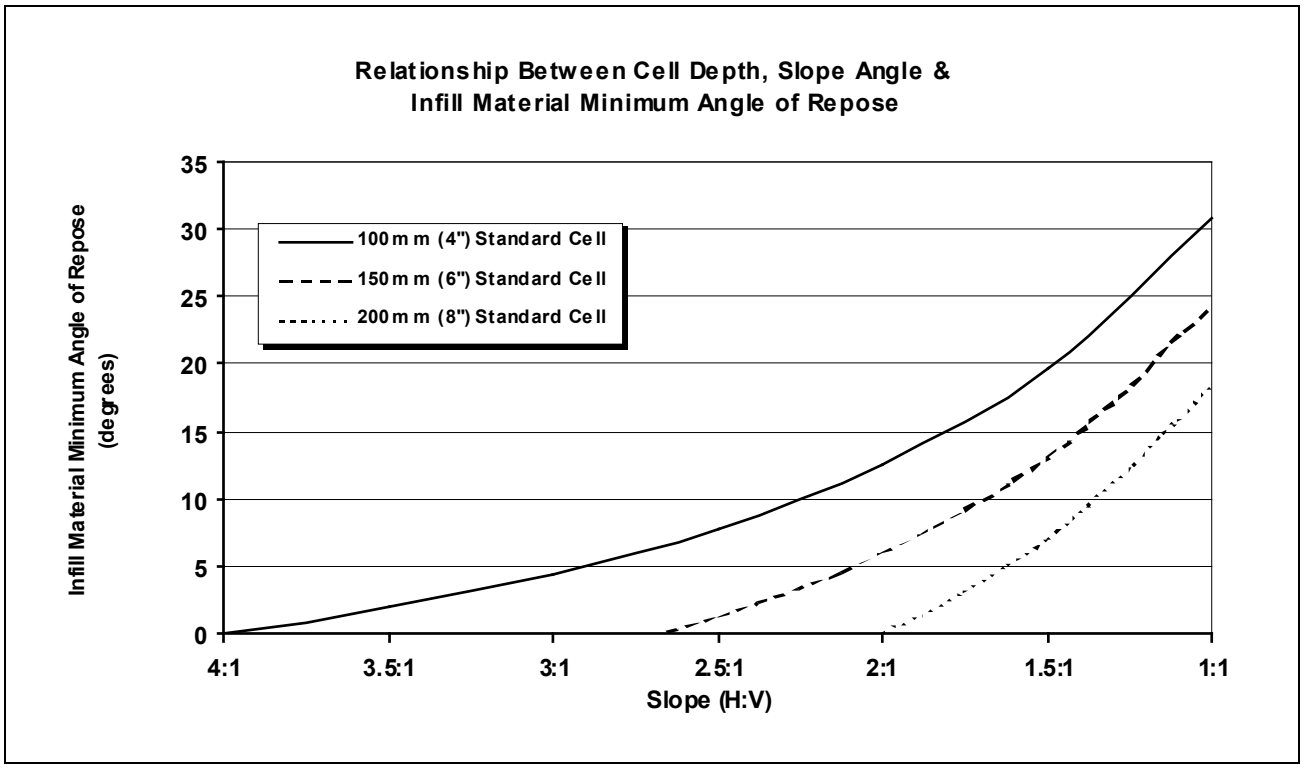
The required cell depth for aggregate infill on steep slopes relates to the natural angle of repose of the aggregate and the slope angle. Figure 12.10 shows minimum suggested cell size and depth for several aggregate types relative to angle of repose and slope angles. If stake anchors cannot meet design requirements, place tendons through every cell to increase the superimposed weight of the aggregate infill bearing directly on the tendon system. Determine slope anchorage requirements in accordance with the design procedure in the section, design analysis tools available from the company, or recognized engineering practice.

4. Geocell System With Concrete Infill. Poured concrete provides a hard, durable protection for slopes. Traditional concrete slab construction requires: steel reinforcing, forms for discrete isolated sections to prevent structural cracking, and construction joints to accommodate shrinkage from drying and thermal expansion/contraction. The potential for damage to traditional construction, as discussed in the previous sections, increases if permanent or seasonal subgrade deformations occur. These factors increase installed costs. Concrete filled geocell reduces these costs and kinds of inherent problems.

Infilling the cells of geocell with ready-mixed concrete produces a durable, erosion-resistant slope cover of uniform thickness which retains its flexibility and maintains an ability to conform to potential subgrade movement. This type of construction avoids compacted granular bedding layers, necessary with conventional poured concrete slabs. Selecting the quality, surface finish and thickness of the concrete allows the designer to meet specific design needs. A non-woven geotextile filter fabric with custom weep holes, if necessary, can assure effective subgrade drainage and subsoil filter protection. Normal drying shrinkage of the concrete infill gives the entire slope surface an ability to drain groundwater from the subgrade. The uniformly distributed shrinkage also imparts a degree of flexibility to the

system. Concrete placement either by pumps, boom-mounted skips or direct discharge from ready-mix trucks is possible.

Figure 12.10 Geocell Selection for Various Slopes and Cell Depth



Flexible concrete forming techniques can accommodate complex slope geometry. Evaluate concrete infill for slopes that are subject to surface flows up to a maximum of 6 m/sec (20 ft/s). Use Class C concrete. Specify appropriate surface finishes (trowel, broom or rake), in order to meet specific aesthetic or surface friction requirements. Embed aggregates or gravel into the surface of wet concrete infill to produce a variety of textures, colors, and surface finishes as desired.

5. Cell Size and Depth Selection. Determine cell depth based on the design procedure. Under hydraulic conditions, consider potential tractive or uplift forces on the slope protection in accordance with the system specific design procedure. Greater cell depth increases the unit weight of the system as well as the flexural stiffness and uplift resistance of the system.

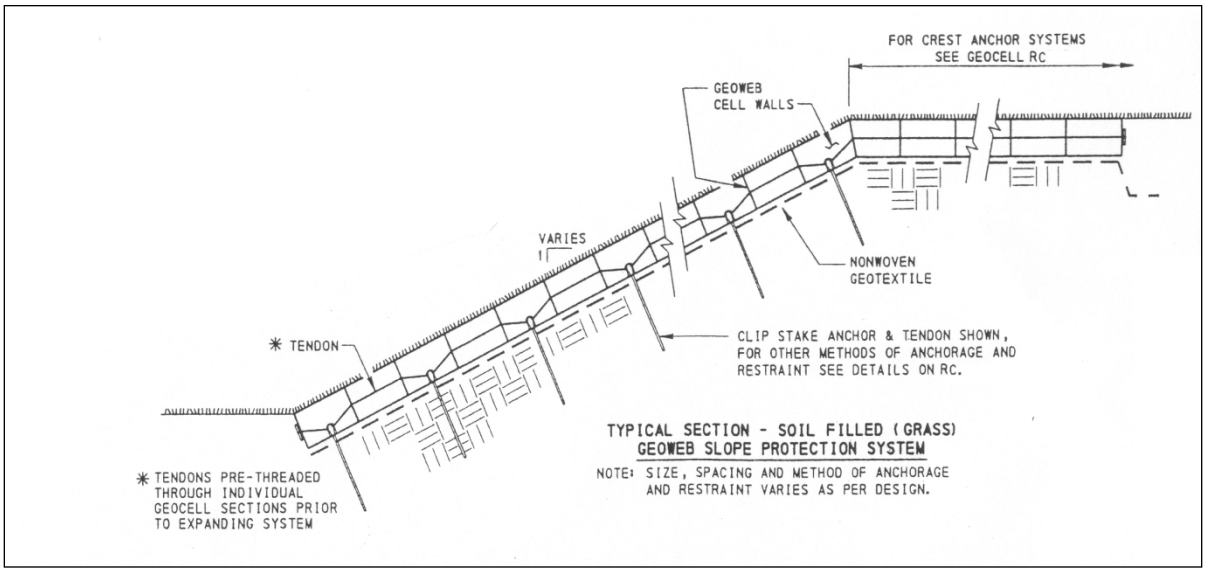
6. Surface Anchorage. Determine what slope anchorage requirements are necessary in accordance with the design procedure in this section or with system-specific design procedures provided by the manufacturer.

The complete geocell system is a multi-component system that includes:

- Geocell panels or mats (plastic cellular panels or mats stapled together).
- Infill material.
- Stakes with a design specific length and pattern of placement.
- Design specific tendons and/or anchor assemblies, when necessary.
- Class 2, Type B geotextile (only required when aggregate is used as a infill material).
- Erosion control blankets (when topsoil is used as a infill to promote seed propagation).

Figure 12.11 depicts a typical geocell slope protection system.

Figure 12.11 Basic Geocell Slope Protection System Components



Additional materials for quantity estimate are:

- Geocell panels or mats:
  - Are available in a variety of panel (mat) lengths and widths, and depths, however, only one standard cell size, 289 cm^2 (44.8 in^2) is currently acceptable. In most cases, a 100 mm (4 in) cell depth is sufficient. Cell depths of 150 mm (6 in) and 200 mm (8 in) are also available for use in special circumstances.
  - The panels collapse to form lightweight, compact bundles for shipping and handling. Panels expand to form, with connected adjacent panels, an expansive honeycomb over the protected area.
  - The number of staples required will vary depending on the ultimate load capacity and site conditions. Refer to Publication 72M, *Roadway Construction Standards* for more information. However, in most cases, staples evenly spaced 25 mm (1 in) part will suffice. Consequently, a 100 mm (4 in) cell depth will require at least 3 staples.
- Infill material:
  - Topsoil with various selected vegetation.
    - On vegetated slopes subject to mild or moderate surface erosion, cellular confinement with the geocell system confines and reinforces the vegetative root mat once it is established. The cells increase the vegetation's natural resistance to erosive forces and protect the root zone from loss of soil particles. The geocell walls, which confine the topsoil infill, form a series of check-dams extending throughout the protected slope. Each cell confines, protects and interlocks a predetermined depth of topsoil and the developing vegetative root mass. This prevents rill development from concentrated flow cutting into the soil. The cells redirect the flow continuously to the surface. Vegetated geocell systems are beneficial when aesthetics are important.
  - Aggregates (AASHTO No. 8, 67, 57 or PennDOT 2A).
    - On non-vegetated slopes, geocell systems with an aggregate infill provide effective slope protection by improving the erosion protection of granular materials such as AASHTO No. 8, 57, 67 and PennDOT 2A subbase material. The confinement of aggregates within cells permits the use of aggregates on steeper slopes than would otherwise be possible. The geocell configuration dissipates low hydraulic energy and minimizes down-slope migration of individual particles caused by gravity and hydraulic action.
  - Class C concrete with in accordance with Publication 408, *Specifications*.
    - Slope protection using concrete infill provides hard, durable protection of slopes by acting as a stay-in-place form on a slope. Geocell systems help prevent uncontrolled cracking of the concrete, reduce the chances of piping or undermining, and retain the flexibility and the ability of the construction to conform to minor subgrade movement. The quality, surface finish,

thickness of the concrete, and cell depth can be selected to meet site-specific design requirements.

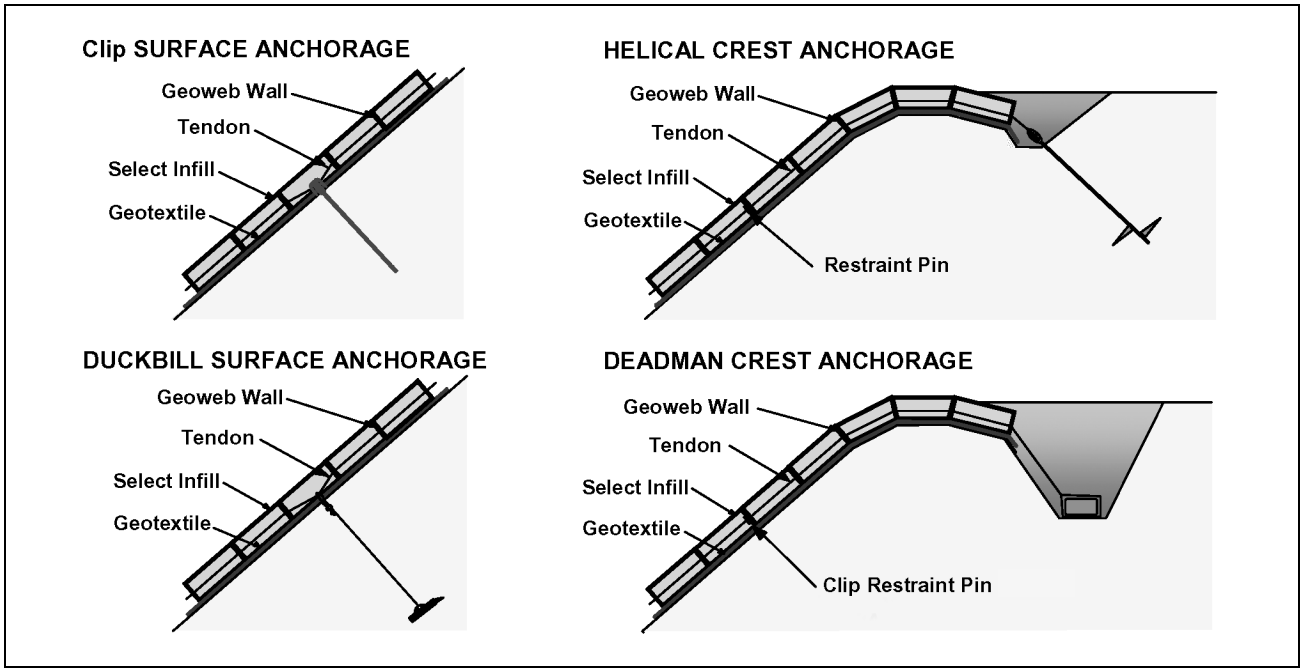
- Stake types and staking patterns:
  - Staking to a slope is the most common anchoring method used if there is no geotextile present and the soil has adequate strength to retain the stakes. Steel reinforcing bars (#4 rebar) bent into a "candy cane" shapes called J-hooks are the preferred type of stake. If the surface of the slope is covered with vegetation that will be mowed, then anchoring methods other than J-hooks, such as plastic clips, should be considered with the use of tendons. The determination of the staking pattern and stake length is explained in the design procedures. For additional stake types see Publication 72M, *Roadway Construction Standards*.
- Tendons and/or anchor assemblies:
  - To ensure the performance of geocell systems and satisfy design requirements, it may be necessary to incorporate tendons and anchor assemblies into the design.
  - Tendons serve to anchor geocell sections to the slope and are integrated into the geocell section through strategically spaced holes drilled through the cell walls running in the direction of expansion (up-down slope) of the sections. In addition to staking to the slope, the design will allow for securing the tendons by an anchoring system at the top (crest) of the slope/swale as indicated in Figure 12.12.
  - Standard tendons are high strength, plastic fibers, polyethylene coated (for corrosive environments or concrete infills) and uncoated, available in various ultimate tensile strengths. The design will determine the required strength, spacing and quantity of individual tendons within each geocell panel. Table 12.8 indicates an array of system specific tendons covering a range of tensile strengths to meet site specific design requirements.

Table 12.8 Typical Tendons

Material	Construction	Tendon Diameter/Width		Minimum Break Strength	
		mm	in	kN	lb
Polyester	Woven Strap (PE coated)	5	0.18	7.12	1600
	Woven Strap (Uncoated)	13	0.50	3.11	700
	Woven Strap (Uncoated)	19	0.75	6.70	1506
	Woven Strap (Uncoated)	19	0.75	9.30	2090
Kevlar	Woven Strap	10	0.375	8.90	2000
	Woven Strap	16	0.625	13.34	3000
Polypropylene	3 Strand Twisted Rope	6	0.25	4.40	990

  - The anchorage assembly for geocell systems consists of several surface anchors or crest anchorages placed with or without tendons (see Figure 12.12). Additional crest anchor systems such as an anchor trench and soil cover/embedment (i.e., direct burial) are detailed in Publication 72M, *Roadway Construction Standards* drawings. Design analysis (see Design Procedure) will determine the necessary anchorage details. Standard rebar in the configuration of a J-Pin or reinforcement bar with a clip-type head or end cap hold the geocell panels to the subgrade in surface anchorages. Special clips or restraints are available from the geocell manufacturers to connect tendons, where necessary, at specific load-transfer points at the cell walls for (crest) type anchorages. Tendons and surface stakes or earth anchors provide system anchoring that will resist sliding.
- Geotextile:
  - When the infill material is granular (aggregate), the geocell system should allow for a Class 2, Type B geotextile under layer to provide.
  - In-plane drainage of groundwater seepage from the slope subgrade.
  - Confinement and filtration of subgrade soil particles.
  - Reinforcement of root-mass with vegetated infills.
  - Tensile reinforcement of slope protection system.
- Slope erosion protection and soil stabilization:
  - When the infill material is topsoil to promote seed propagation, surface treatments such as erosion control blankets provide solutions to particular design requirements.

Figure 12.12 Typical Geocell Anchorage System



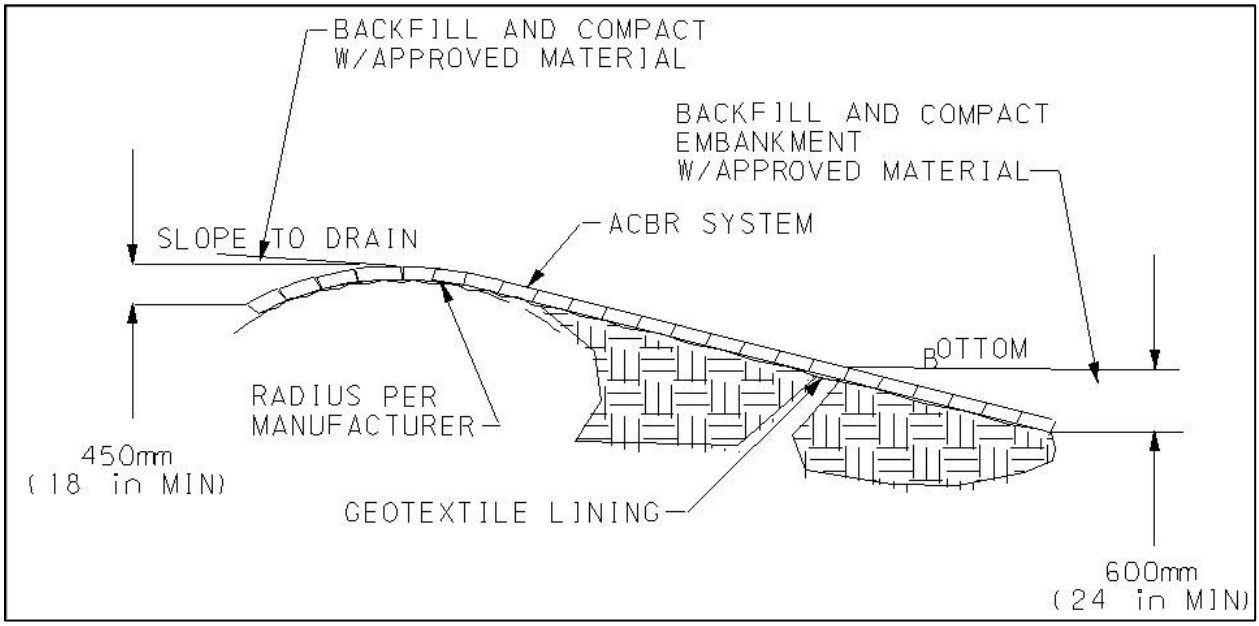
Include appropriate references, drawings, and notes on the plans including the following:

- Geocell cell and panel properties.
- Stapled or hog ring end and side connection details and load capacities.
- Anchor details.
- Crest anchor systems.
- Typical connection to existing structures.
- Drainage details through geocell sections.
- Geocell slope protection systems with restraint clips and tendons only.
- Geocell slope protection systems with tendons and clip anchors only.
- Geocell slope protection systems with restraint clips, tendons, and anchors.
- Curved and tapered section details.

E. Articulated Concrete Block Revetment System (ACBR). An ACBR system is a matrix of individual, permanently installed, concrete blocks placed together to form an erosion resistant overlay that meets static and hydraulic performance characteristics. ACBR systems can be used on subgrade that may expand or contract, and for slope protection to prevent embankment erosion, especially near waterways. ACBR systems consist of three-dimensional wet or dry cast preformed concrete units which are cable connected, interlocked or both. The assembled units form a continuous blanket or mattress over the protected surface. See Figure 12.13.

ACBR systems provide a flexible alternative to riprap, gabions, grouted rock, cast-in-place cement concrete slab slope walls, and pre-cast cement concrete block slope walls. Each revetment block system includes a design specific geotextile filter fabric, which allows infiltration and exfiltration to occur while providing particle retention of the soil subgrade. The blocks within the matrix are dense and durable and the matrix is flexible and porous due to the geometric configuration of each ACBR system.

Figure 12.13 Typical ACBR System



ACBR systems are a relatively recent development in erosion control and slope protection. Applications may include channel and swale lining, slope protection, channel protection at culvert inlets/outlets, temporary and emergency erosion control, etc. ACBR systems do have limitations to their flexibility and are relatively expensive. Their uses should be limited to severe erosion control requirements for permanent control where other alternatives are not as effective.

This section describes several proprietary systems that share common attributes and benefits such as: flexibility, rapid installation, support for vegetation, drainage of slope materials, facilitation of groundwater recharge, reduction of runoff velocities and volumes, enhancement of water quality, resistance to ice damage and freeze-thaw cycles, and mobility (for temporary uses). Unlike dumped rock or riprap, these systems are engineered or designed for a given flow and design shear. These systems are aesthetically pleasing and do not promote a habitat for rodents or a collection area for wind or water born debris.

The following section discusses design of ACBR systems and discusses several manufactured ACBR systems from different companies. The discussion covers the non-cabled ACBR systems first followed by the cabled ACBR systems. All of the systems discussed herein have been tested in accordance with the procedures outlined in *Minimizing Embankment Damage During Overtopping Flow*, (Clopper and Chen, 1988) and *Hydraulic Stability of Articulated Concrete Block Revetment Systems During Overtopping Flow* (Clopper, 1989). Table 12.9 summarizes the systems that are approved for PennDOT use by company, product name, and grouping (non-cabled or cabled).

Table 12.9 ACBR Systems

Company (Licensor)	Product	Non-Cabled	Cabled
Armortec	Armorflex		X
	Armorloc	X	
	A-Jacks	X	
International Erosion Control Systems, LLC	Cable Concrete		X

The design of a revetment system must result in a well-balanced and stable installation that functions as intended. The key design considerations for normal ACBR applications are the static or hydraulic capabilities of the system compared to the static or hydraulic requirements of the site, as well as drainage, separation, and anchorage. Static designs (e.g., slope applications) must then compare downslope force components of the weight of the system with any surcharge to the total resistive forces – interface friction, in-plane tensile anchorage and in-plane resistance of anchors with an appropriate factor of safety. Hydraulic designs (e.g., channels and swales) must compare the systems limiting shear stress and maximum velocity to the actual shear stress and velocity requirements with an appropriate factor of safety for the specific site. Each licensor of ACBR products in this section has conducted full-scale laboratory tests to determine their product's limiting shear stress and maximum velocity under hydraulic conditions.

Revetment design should always provide an engineering fabric or geotextile between the units and foundation soils. This helps to prevent soil loss and potential clogging of an aggregate drainage layer. Revetment designs also should include edge, toe/top entrenchment to prevent scour.

The design of an ACBR system for slope protection must demonstrate, by appropriate static or hydraulic analysis, that the system is stable and will serve as an effective erosion control countermeasure.

For highway slope protection and non-hydraulic conditions, contact the manufacturer to procure the static analysis procedure to determine the slope revetment stability and system requirements with a minimum factor of safety of 1.5.

For channel, swale, and outlet protection, as well as significant slope sheet flow, contact the manufacturer to obtain the system-specific hydraulic design procedure. Select an appropriate design event based on regulatory requirements, PennDOT design guidance, and sound engineering judgment.

The hydraulic design procedures for ACBR systems should follow HEC-23, *Bridge Scour and Stream Instability Countermeasures* (FHWA, 2001c). For use on PennDOT projects, ACBR products must meet testing and protocols requirements of *Minimizing Embankment Damage During Overtopping Flow* (Clopper and Chen, 1988) and *Hydraulic Stability of Articulated Concrete Block Revetment Systems During Overtopping Flow* (Clopper, 1989).

The procedure in HEC-23 determines the factor of safety by use of either design charts or equations. Typically, the revetment design should allow for a minimum factor of safety of 1.5 if the hydraulic conditions are known and variations in the installation can be determined. Assume a higher factor of safety for protection at junctures, outlets and bends due to the more complex shear stress calculations at those points.

HEC-23, *Bridge Scour and Stream Instability Countermeasures* (FHWA, 2001c) allows for consideration of forces due to projecting blocks, side slope correction factor (k), allowable and design shear stress and velocity; therefore, HEC-23 considers the hydraulic forces of lift (buoyant force and differential pressure across the block due to local accelerations), drag (shear stress/tractive force), and impact. This design procedure does not explicitly account for restrictive forces due to cable, anchoring devices or vegetation and is inherently conservative in terms of selection and design of an ACBR system.

Table 12.10 lists the design references for the company specific hydraulic design procedure and software for each ACBR system. The software indicated is not endorsed for use by PennDOT.

The installation of block revetment systems does not increase the stability of the underlying slope. Geotechnical engineering analyses must show that the protected slope is stable prior to consideration of revetment systems.

Design of pre-cast concrete block revetments should consider the following key factors:

- General revetment concept.
- Bank preparation.
- Mattress and block size.
- Slope and articulation.
- Edge treatment.
- Filter design.
- Surface treatment.

1. General Concept. Pre-cast block revetments must form continuous mattresses on the protected slope. The vertical and longitudinal extent of the mattress should be derived from design parameters and analysis. The design approach should emphasize edge treatment (toe of slope, top of slope and edges or flanks of revetment), block type, and filter design to suit site requirements (i.e., slope, soils, sheet flow velocity and duration, etc).

2. Slope/Bank Preparation. Slopes must be graded to a uniform slope. Large boulders, roots, and debris must be removed prior to final grading. Holes, soft areas, and large cavities must be filled, and the graded surface must be true to line and grade within 150 mm (6 in). Compaction of the bank surface must provide for a solid foundation for the mattress.

3. Mattress and Block Size. Identify the overall mattress size in accordance with the required longitudinal and vertical limits of the revetment system. In general, the revetment should be continuous for a distance greater than the required length and width so that erosive forces will not erode the slope material. Similarly, the revetment may form a continuous partial or full height (top of slope) mattress. Articulated block mattresses are assembled in sections. Consult manufacturer's literature when selecting an appropriate block size for given site conditions (i.e., slope, sheet flow, edge treatment, and surface treatment).

4. Slope and Articulation. Full-scale test results for certain systems are available. Table 12.11 indicates critical velocity and critical shear stress for a sampling of block sizes under bare or non-vegetated conditions, without anchors. Consult the manufacturer's design references (Table 12.10) for design values for other slopes and block sizes and threshold stabilities of velocity vs. slope and shear stress vs. slope under hydraulic conditions.

Table 12.10 ACBR System Design Procedures

Company	Product	Design Procedure (Manual)	Software
CONTECH (formerly Armortec Concrete Erosion Control Systems) 9025 Centre Pointe Drive West Chester, OH 45069 www.contech-cpi.com/Products/Erosion-Control.aspx (800) 338-1122	Armorflex	Designing Stable Channels with Armorflex Articulated Concrete Block Revetment Systems	Armorflex Stability Analysis for Open Channel Flow
	Armorloc	Appropriate Engineering Practice or Consult Company	Appropriate Design Software or Consult Company
	A-Jacks	A-Jacks Concrete Erosion Control System Streambank and Scour Applications	Consult Company
Royal Enterprises America (formerly International Erosion Control Systems, L.L.C.) 30622 Forest Boulevard P.O. Box 119 Stacy, MN 55079 www.royalenterprises.net (800) 817-3240	Cable Concrete	Cable Concrete Critical Shear Stress Values	Consult Company

Table 12.11 Actual Test Results

System	Slope (H:V)	Critical Velocity m/s (ft/s)		Critical Shear Stress kPa (psf)	
		m/s	ft/s	kPa	psf
ArmorLoc	Contact Manufacturer for specifications				
Cable Concrete (CC 45)	1.5:1	5.7	18.9	1.67	34.9
Armorflex (Class 305)	1:1	4.5	15	0.72	15

The minimum radius, R, for layout and articulation of the various systems is as shown in Table 12.12.

Table 12.12 Range of Articulation

System	cm	Minimum Radius	ft
ArmorLoc		N/A	
Cable Concrete		N/A	
Armorflex	60		2

5. Edge Treatment. The termination of pre-cast block revetments (the toe of slope, top of slope and possibly leading and trailing edges, also known as lateral edges, flanks or landward end of revetment) may require special treatment to prevent undermining. Toe treatments may include either a toe apron design (Figure 12.14) or a toe trench design to prevent scour in the vicinity of the slope toe.

Two suggested alternatives for termination treatments at the top, leading, and trailing edges are an at-grade termination or a termination trench, see Figure 12.14. Termination trenches are suited to environments subject to significant erosion (silty/sandy soils, and high velocities), or where failure of the revetment would result in significant economic loss. Termination trenches provide greater protection against failure from undermining and lateral erosion or stress than do at-grade terminations. Where upper bank erosion or lateral erosion is not problematic, at-grade terminations may be sufficient.

Specify earth anchors at regular intervals along the top of the revetment based on soil type, mat size, and the size of the anchors. Refer to the manufacturer's design procedures for suggested anchor spacing or determine spacing in accordance with recognized practice.

6. Filter. Specify a geotextile filter fabric on the slope to prevent slope material from leaching through the openings in the mattress structure. Although a fabric filter is desirable, graded filter material may be suitable with proper design and installation to prevent movement of the graded material through the protective mattress. The manufacturer's design procedures and HEC-11, *Design of Riprap Revetment* (FHWA, 1989) provide useful design guidance on filter design. Each of these systems uses a geotextile filter fabric that is unique to that particular system. These filter fabric geosynthetics will work on most highway project applications. Design of a geotextile is required only when the work calls for a concrete block revetment on a project with a large, navigable, tidal estuary that is subjected to wave action. These estuaries would be: Lake Erie, Delaware River, Allegheny River, Ohio River, and the Monongahela River.

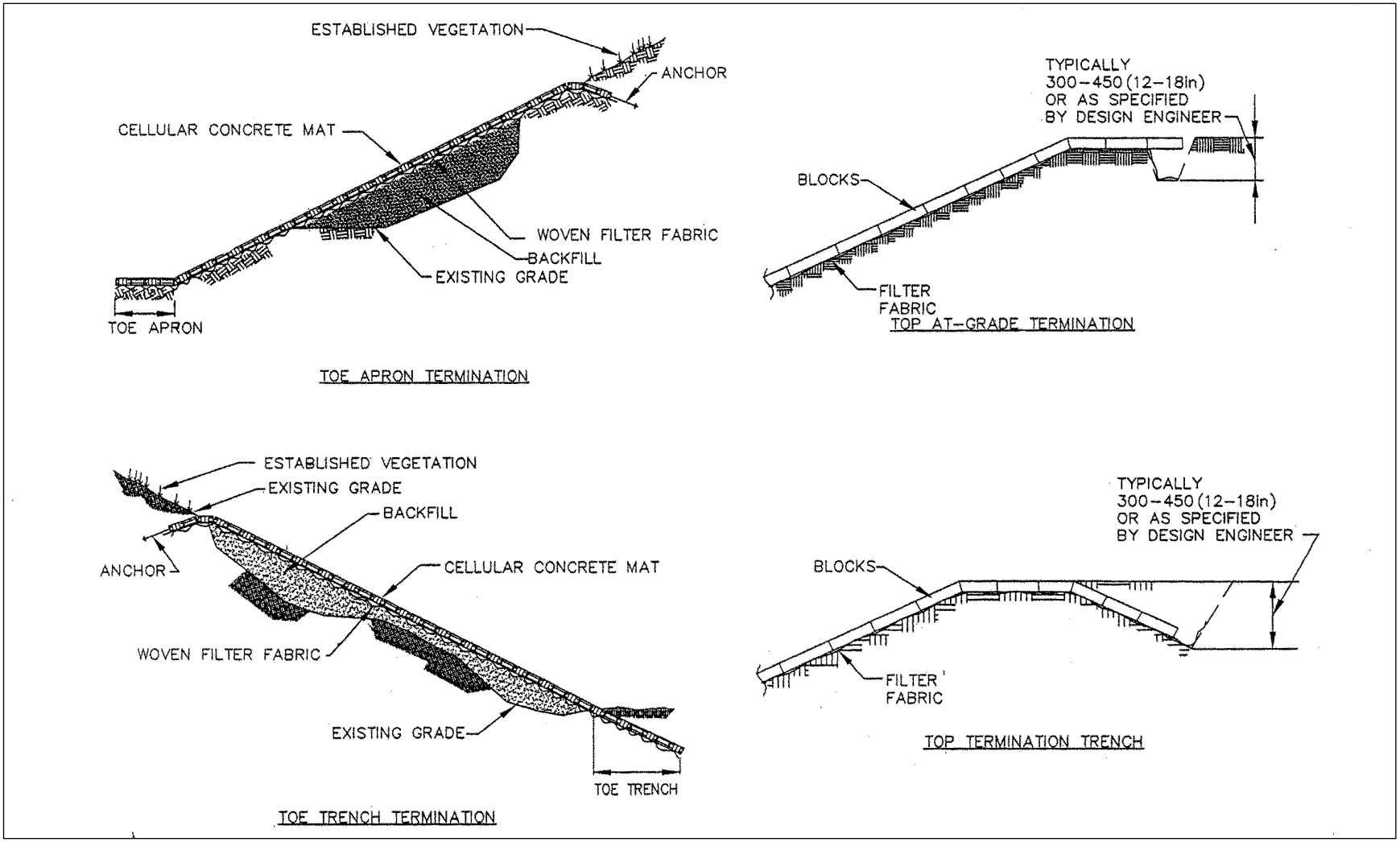
7. Surface Treatment. Specify backfilling of the spaces between and within individual blocks with either earth or aggregate. Soil backfill and seeding promotes the development of natural vegetation on the slope. Vegetation improves the structural stability of the embankment and enhances its appearance. Open-cell blocks are preferable for growth of vegetation. Use closed-cell blocks below the flowline or in areas where no vegetation will grow.

The following information and principles may provide additional guidance in the design and construction of a block revetment installation:

- Costs, aesthetics, and environmental benefits should be the basis for ACBR design.
- History of the ACBR system's performance and experience with comparable construction and sites should support the design process and methods.

Soil, watershed characteristics, and geotechnical characteristics of the site should support the design.

Figure 12.14 Termination Details for ACBR Systems



Proper planning and the appropriate funding for continued maintenance of the revetment should be considered in situations where the revetment may not entirely eliminate the erosion problem.

8. Non-Cabled ACBR Systems.

a. Description. The non-cabled systems consist of non-cabled blocks that are cast into interlocking shapes of various sizes that overlay a filter fabric and provide a positively connected matrix.

b. Attributes. Common attributes for non-cabled systems include:

- No metal to corrode, no fastening devices subject to abrasion or corrosion.
- No cables or additional anchoring.
- Ability to conform to changes in direction.
- Ability to go around structures without affecting system integrity.
- Fabrication as mats for machine installation.

c. Components and Function. Non-cabled systems are multi-component systems that consist of the following:

- Precast concrete blocks.
- Specific backfill or infill material.
- System specific filter fabric.

Non-cabled interlocking blocks are produced on concrete block machines. These blocks are available in various sizes, shapes and thicknesses. This type of ACBR system has excellent resistance to hydraulic shear and overtopping forces.

The blocks rest on a system specific filter fabric. The industry refers to the filter fabric as the carrier filter fabric when fabric is pre-attached to the blocks allowing the block and filter-fabric to be placed as integrated mats. The filter fabric provides:

- In-plane drainage of groundwater seepage from the slope subgrade.
- Confinement and filtration of subgrade soil particles.
- Reinforcement of root-mass with vegetated infills.
- Tensile reinforcement of slope protection system.

Once in place, non-cabled ACBR systems can accommodate several different infills, backfills, or surfaces. Aggregate in joints and voids, lends itself to a clean, unvegetated appearance. Topsoil in joints and voids, with or without Erosion Control Blankets (ECBs), provides an environment for establishing vegetation. Vegetation helps anchor the mat. The backfill material enhances the system's resistance to erosive and hydrostatic pressure, UV radiation and fabric degradation.

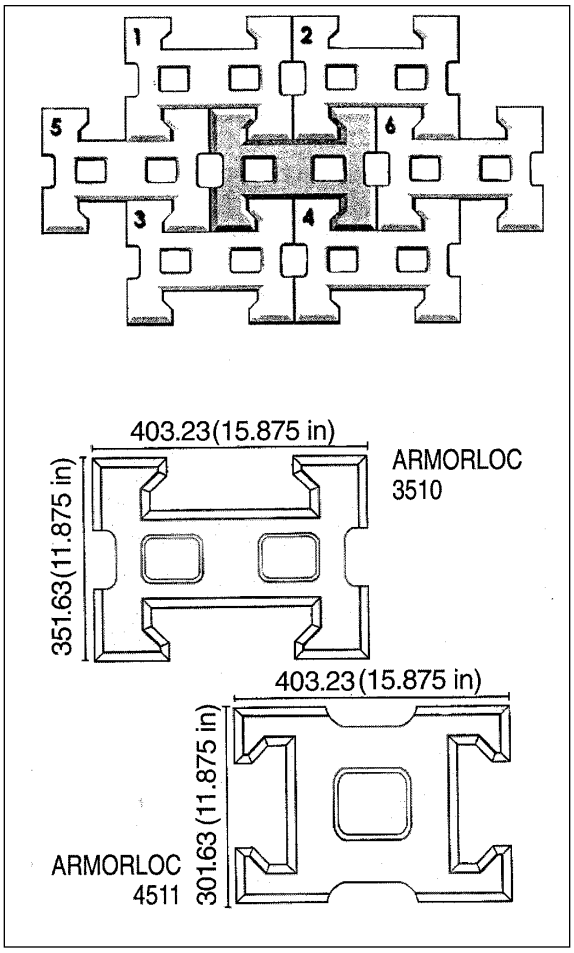
To ensure the integrity of the installation, the standard components must provide for some type of termination at the toe and top of slope, as well as at the edges. Figure 12.14 illustrates the use of the standard components in a variety of terminations.

9. Principal Proprietary Non-Cabled Systems.

a. ArmorLoc. The ArmorLoc system is a proprietary system that achieves an interlocking effect in which each block is keyed with four blocks in adjoining rows as well as abutting blocks on either side so that each individual unit is physically interlocked with six surrounding blocks to resist movement and uplift (see Figure 12.15). Trapped soil particles between block joints act as a dry grout, binding blocks into a continuous mattress.

The ArmorLoc block is available in two size and weight classifications: 40.323 cm (15.875 in) x 35.163 cm (11.875 in) x 10.160 cm (4.0 in) or 12.700 cm (5.25 in) with a weight of 13.6 - 15.8 kg/m^2 (30 - 35 lbs./sq. ft.) or 18.1 - 20.4 kg/m^2 (40 - 45 lbs./sq. ft.) and an open area of 25% or 20% of the grid.

Figure 12.15 Amorloc System Configuration



b. A-Jacks. The A-Jacks three-dimensional revetment system, Figure 12.16, is a proprietary system of interlocking concrete armor units for areas needing protection such as streams or banks. A-Jacks are assembled into a highly permeable, interlocking matrix by sliding one half into another to form a complete unit, refer to Figure 12.16.

Figure 12.16 A-Jacks

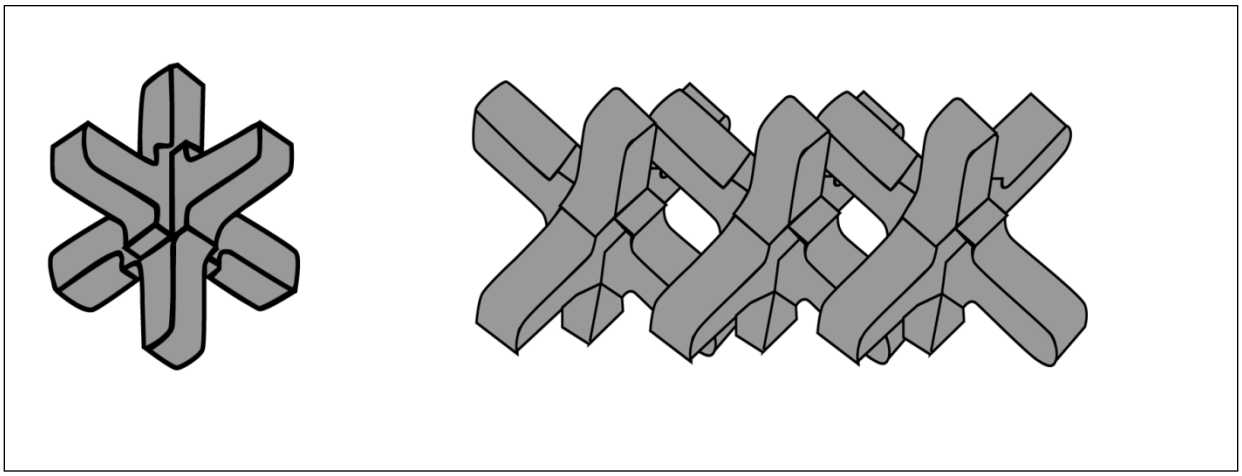
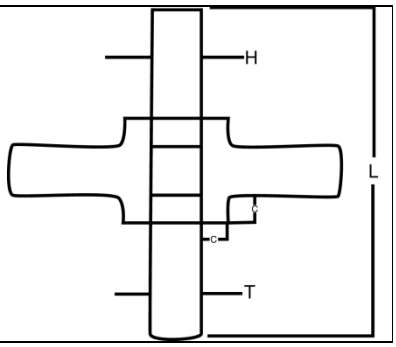


Table 12.13 is a summary of the sizes available. AJ-24 and AJ-36 are best used for streambank restoration. AJ-48, AJ-72, and AJ-96 are intended for energy dissipation. Refer to the manufacturer information for design specifications.

Table 12.13 Summary of A-Jacks Dimensions

A-Jacks	L (in)	T (in)	H (in)	C (in)	V1 (ft^3)	Wt (lbs)	
AJ-24	24	3.68	3.68	1.84	0.56	78	
AJ-36	36	5.52	5.52	2.76	1.89	265	
AJ-48	48	7.36	7.36	3.68	4.49	629	
AJ-72	72	11.04	11.04	5.52	15.14	2120	
AJ-96	96	14.72	14.72	7.36	35.87	5022	



10. Cabled ACBR Systems.

a. Description. These systems consist of interlocking or non-interlocking concrete blocks of various sizes with preformed, cast horizontal holes to permit installation of high-strength stainless steel cables through the matrix of blocks, binding them into a monolithic mat.

b. Attributes. Common attributes for cabled systems include:

- Availability in factory-assembled mats for machine installation.
- Fewer seams possible than with individual blocks.
- Ability to conform to existing ground contours.
- Temporary use and reuse from site to site.
- Permanent anchorage possible by connecting mat cables to anchors e.g. patented Helix, Ducksbill, or Royal anchors.

c. Components and Functions. Cabled systems consist of the following four basic components:

- Precast concrete blocks.
- Specific backfill or infill material.

- System specific filter fabric.
- High-strength stainless steel cable.

The basic components function similarly as in non-cabled systems. The cables facilitate placement and connection of mats, as well as, anchoring (as needed) but offer no hydraulic stability or structural value to the system.

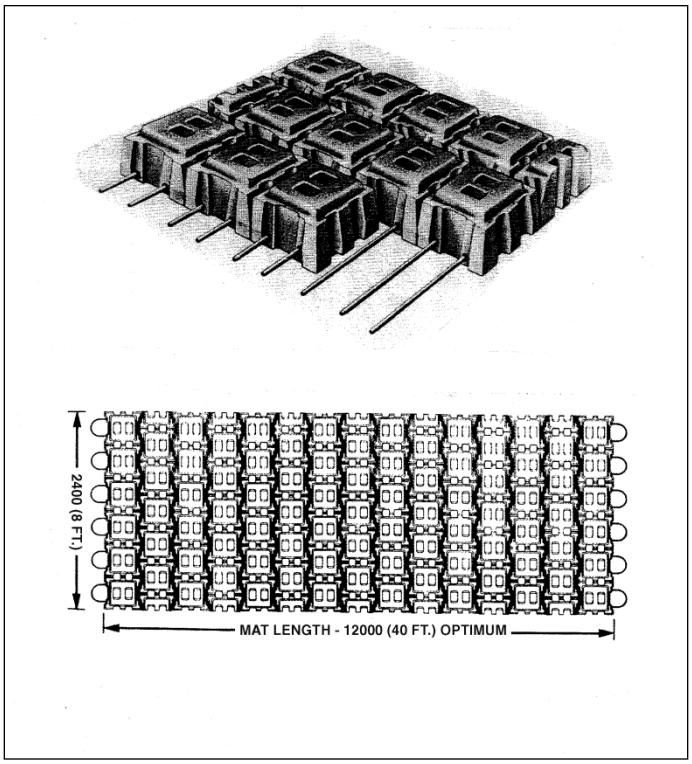
11. Principal Proprietary Cabled Systems.

a. ArmorFlex. The ArmorFlex system is a proprietary system of cable-bound individual blocks or (cellular) concrete block mats with specific hydraulic capabilities for various block classes and types (see Table 12.14). The blocks are open-cell or closed-cell blocks of normal weight aggregates or both, with an open area of not more than 12% or 20%, respectively. Parallel strands of cable extend through two ducts in each block in a manner that provides for longitudinal bending of the blocks within the system. Each row of blocks is laterally offset by one-half block width from the adjacent row so that any given block ties by cable to four other blocks (see Figure 12.17). The blocked interlocking surfaces prevent lateral displacement of the blocks in the event of cable damage or removal and allow for flexibility in all directions. When using concrete block mats, the system provides a minimum of 25° between any given row in the upward direction and a minimum 45° in the downward direction. The cables form lifting loops at either end of the mat. Cables and fittings are stainless steel. The ArmorFlex system may employ a permanent anchoring system by attachment to cables (e.g. in hanging mats on steep slopes without toe construction). The ArmorFlex blocks are useable without cables in which case they function as a non-cabled system.

Table 12.14 ArmorFlex Block System

Concrete Block Class	Open/ Closed Cell	Nominal Dimensions						Gross Area		Block Weight				Open Area %
		Length		Width		Height								
		cm	in	cm	in	cm	in	m^2	ft^2	kg	lbs	kg/in^2	lbs/ft^2	
30s	Open	33.0	13.0	29.5	11.6	12.1	4.75	0.09	0.98	14.0-16.3	31-36	156.2-180.6	32-37	20
50s	Open	33.0	13.0	29.5	11.6	15.2	6.00	0.09	0.98	20.4-23.5	45-52	219.7-258.7	45-53	20
40	Open	44.2	17.4	39.4	15.5	12.1	4.75	0.16	1.77	28.1-32.2	62-71	170.8-195.2	35-40	20
50	Open	44.2	17.4	39.4	15.5	15.2	6.00	0.16	1.77	36.7-42.6	81-94	224.5-258.7	46-53	20
60	Open	44.2	17.4	39.4	15.5	19.1	7.50	0.16	1.77	44.9-51.2	99-113	273.4-312.4	56-64	20
70	Open	44.2	17.4	39.4	15.5	22.9	9.00	0.16	1.77	54.4-62.5	120-138	332.0-380.8	68-78	20
40L	Open	44.2	17.4	59.9	23.6	12.1	4.75	0.23	2.58	40.8-48.0	90-106	170.8-200.1	35-41	20
50L	Open	44.2	17.4	59.9	23.6	15.2	6.00	0.23	2.58	52.6-60.7	116-134	219.7-253.8	45-52	20
60L	Open	44.2	17.4	59.9	23.6	19.1	7.50	0.23	2.58	65.3-76.2	144-168	273.4-317.3	56-65	20
70L	Open	44.2	17.4	59.9	23.6	22.9	9.00	0.23	2.58	78.4-91.1	173-201	327.1-380.8	67-78	20
45s	Closed	33.0	13.0	29.5	11.6	12.1	4.75	0.09	0.98	17.6-23.5	39-45	195.2-219.7	40-45	10
55s	Closed	33.0	13.0	29.5	11.6	15.2	6.00	0.09	0.98	24.0-27.6	53-61	263.6-302.7	54-62	10
45	Closed	44.2	17.4	39.4	15.5	12.1	4.75	0.16	1.77	35.3-40.3	78-89	209.9-244.1	43-50	10
55	Closed	44.2	17.4	39.4	15.5	15.2	6.00	0.16	1.77	42.6-48.9	94-108	258.7-297.8	53-61	10
75	Closed	44.2	17.4	39.4	15.5	19.1	7.50	0.16	1.77	54.4-62.5	120-138	332.0-380.8	68-78	10
85	Closed	44.2	17.4	39.4	15.5	22.9	9.00	0.16	1.77	65.7-75.7	145-167	400.0-478.4	82-98	10
45L	Closed	44.2	17.4	59.9	23.6	12.1	4.75	0.23	2.58	48.9-57.1	108-126	205.0-239.2	42-49	10
55L	Closed	44.2	17.4	59.9	23.6	15.2	6.00	0.23	2.58	63.0-73.9	139-163	263.6-307.5	54-63	10
75L	Closed	44.2	17.4	59.9	23.6	19.1	7.50	0.23	2.58	78.4-91.1	173-201	327.1-380.8	67-78	10
85L	Closed	44.2	17.4	59.9	23.6	22.9	9.00	0.23	2.58	94.8-110.2	209-243	395.4-458.9	81-94	10

Figure 12.17 Armorflex Block Configuration



b. Cable Concrete. The Cable Concrete system is a proprietary system that consists of non-interlocking pyramidal shaped concrete blocks interwoven with stainless steel cable. Figure 12.18 illustrates the original closed-cell block sizes. The units are also available in three other original and three oversized block sizes. The cable runs both lengthwise and widthwise, providing loops on all sides for clamping to adjacent mats and possible anchoring. The manufacturer attaches the underlying geotextile fabric to the mat at the plant. The pyramidal block shape allows for articulation ranging from 20° to 60°, depending upon block size (see Table 12.15).

Figure 12.18 Cable Concrete Configuration

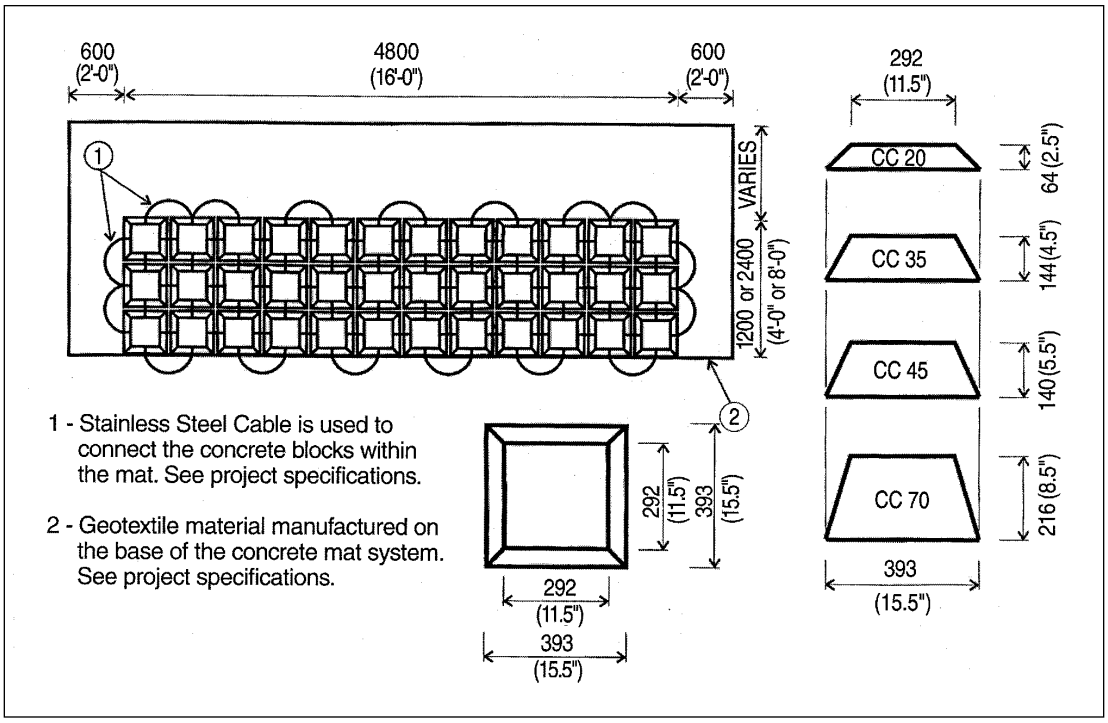


Table 12.15 Cable Concrete Block Specifications Original Block – Closed Cell

	Model	CC 20				CC 35				CC 45				CC 70			
Mat	Unit Weight	kg/m²		lbs/ft²		kg/m²		lbs/ft²		kg/m²		lbs/ft²		kg/m²		lbs/ft²	
		112.2		23		200.1		41		244.1		50		375.9		77	
	Area	m²	ft²	m²	ft²	m²	ft²	m²	ft²	m²	ft²	m²	ft²	m²	ft²	m²	ft²
		5.9	64	11.8	128	5.9	64	11.8	128	5.9	64	11.8	128	5.9	64	11.8	128
	Weight	kg	lb	kg	lb	kg	lb	kg	lb	kg	lb	kg	lb	kg	lb	kg	lb
		615.6	1355	1319.9	2910	1183.8	2620	2458.4	5420	1451.4	3200	2902.9	6400	2245.2	4950	4490.5	9900
	Blocks/Mat	36		72		36		72		36		72		36		72	
Blocks	Spacing @ Base	mm		in		mm		in		mm		in		mm		in	
		13		0.5		13		0.5		13		0.5		13		0.5	
	Spacing @ Top	114		4.5		114		4.5		114		4.5		114		4.5	
	Weight	kg		lb		kg		lb		kg		lb		kg		lb	
		18.1		40		33.1		73		40.3		89		62.1		137	
	Height	mm		in		mm		in		mm		in		mm		in	
		64		2.5		114		4.5		140		5.5		216		8.5	
Cable		Length		Width		Length		Width		Length		Width		Length		Width	
	Diameter	mm	in	mm	in	mm	in	mm	in	mm	in	mm	in	mm	in	mm	in
		3	1/8	3	1/8	4	5/32	4	5/32	4	5/32	4	5/32	5	3/16	4	5/32
	Construction	1 x 19		1 x 19		1 x 19		1 x 19		1 x 19		1 x 19		1 x 19		1 x 19	
		kg	lb	kg	lb	kg	lb	kg	lb	kg	lb	kg	lb	kg	lb	kg	lb
	Breaking Strength	952.5	2100	952.5	2100	1496.8	3300	1496.8	3300	1496.8	3300	1496.8	3300	2131.8	4700	1496.8	3300
Open Area		8.2%															

The following information should be included in the details or General E&SPC Construction Notes on the plans:

- Prepare the area by clearing and grubbing, excavating, removing unstable material, backfilling, placing, and compacting embankment, or other means necessary prior to placing geotextile and ACBR. The graded surface must be true to line and grade within 150 mm (6 in) or per manufacturer's specifications.
- Place geotextile in accordance with specifications in Publication 408, *Specifications*.
- The ACBR will be constructed/installed as specified in the plans. After the ACBR is placed, selected infill or backfill will be placed over the blocks to allow for vegetation. For unvegetated surfaces, the blocks can be left in place with no backfill.
- Refer to seeding specifications in Publication 408, *Specifications*.
- Backfill and compact (with approved material) each end of the ACBR for stabilization. Top fill: 450 mm (18 in) minimum. Bottom fill: 600 mm (24 in) minimum. AASHTO No. 57 crushed stone or equivalent can be used as an approved backfill material to act as a drainage medium.
- Termination of the ACBR will be constructed in accordance with manufacturer's suggestions.

The plans should clearly indicate or note:

- Termination details at toe, edges or flanks and top of slope as required and final grades of slope.
- Details for optional anchors at the top and edges or flanks of the protection as required.
- Filter fabric and/or graded filter on the prepared subgrade.
- Detail of individual block units or individual mats on the slope side by side.
- As applicable, connection details for attachment of adjacent mats to one another and to anchors.
- The backfill details over the mats (and into the open cells or spaces between cells) and into the anchor trenches.
- Seeding and fertilizer limits.

F. Gabions. Gabions are rock-filled, multi-celled, rectangular, open mesh wire baskets used for the construction of erosion control protection structures. Gabions generally are available in two different types based on shape: mattress or blocks. Gabions are used to prevent erosion and scour from high velocity flows in erosion prone areas. Gabions provide both permanent and temporary solutions for erosion problems.

There are several applications for gabions. Their structural versatility allows them to be used as slope walls, channel lining, and channel deflectors to prevent scour and erosion. For additional guidance on the application of gabions refer to HEC-11, *Design of Riprap Revetment* (FHWA, 1989); HEC-15, *Design of Roadside Channels with Flexible Linings* (FHWA, 2005a); HEC-23; *Bridge Scour and Stream Instability Countermeasures* (FHWA, 2001c), and Design Manual Part 4.

In mattress designs, the individual wire mesh units are laid end-to-end and side-to-side to form a mattress layer in a channel or on a slope. This type of gabion generally has a depth dimension that is much smaller than the width or length.

Block designs are typically rectangular in shape where the depth and width are approximately the same. Blocks are installed by stacking the individual blocks on top of each other in a stepped fashion.

Gabions must conform to the details, as shown in Publication 72M, *Roadway Construction Standards*, and may be considered by the designer for the following categories:

- Slope Walls. Slope walls can be constructed using mattresses, i.e., the 225 mm (9 in) thick wire baskets, or Blocks in 300 mm (12 in) or 450 mm (18 in) thickness. The thickness is usually dictated by the size of coarse aggregate available for backfilling the wire baskets. Two overlapping layers of coarse aggregate are the minimum suggested to keep any water from having a direct path to the soil. Maximum permissible velocities for various thicknesses of mattresses and blocks are specified in Table 12.16.

Table 12.16 Maximum Permissible Velocities for Mattress and Block Gabions

Type	Thickness		Permissible Velocity	
	mm	in	m/s	ft/s
Mattress	<152	<6	1.8	6.0
	<254	<10	3.6	12.0
	<305	<12	4.5	15.0
	<457	<18	5.4	18.0
Block	>457	>18	6.7	22.0

Reference: *Erosion and Sediment Pollution Control Program Manual* (PA DEP, 2000).

Slope walls, adjacent to flowing water, must be protected against scour by either an apron or a toe wall of gabions. The apron should be approximately two times as wide (W_G as indicated on Standard Drawing RC-43M) as the anticipated depth of scour. If the channel is sharply curved, toe walls are preferred over aprons and the toe wall height (H_G as indicated on Standard Drawing RC-43M) should be at least equal to the anticipated depth of scour.

- Channel Lining. The lining thickness required for gabions is dictated by the maximum permissible velocities indicated in Table 12.16. On Standard Drawing RC-43M an illustration of gabions as channel linings is shown.

Limitations to the use of gabions include material costs and design requirements. Also the designer should be aware of other considerations which might include impacts to stream dynamics and impacts on stream and wildlife habitat.

Temporary channels are to be designed to convey the peak discharge from the 2-year frequency storm event. In Special Protection Watersheds, the 5-year design storm should be used for temporary channels. All permanent channels are to be designed to convey the 10-year storm peak discharge.

The standard gabion sizes presented in Publication 72M, *Roadway Construction Standards,* represent sizes available in quantity from approved manufacturers. Additional sizes may be available on a special order basis only. When gabions are used for other purposes, refer to the publications listed previously in this section.

Construction and maintenance of gabions should be considered when gabions are placed on a 1V:1.5H (1.5H:1V) side slope or steeper, drive hardwood stakes through the gabions, along the top edge, to anchor the installation. Embed stakes 450 mm (18 in) minimum below gabion bottom. An apron or toe wall is required where the slope wall is installed adjacent to water. Make the apron approximately two times as wide as the anticipated depth of scour and the toe wall height at least equal to the anticipated depth of scour. Channel lining installation can be a sediment producing activity. Maintain E&SPC BMP during channel work in flowing waterways. Temporary diversions or piping should be employed to prevent waterway sedimentation. Check wire of baskets for corrosion and wear. When gabions are used as permanent stabilization, baskets should be inspected periodically for damage or deterioration. In addition, when gabions are used in a submerged environment, consider using corrosion resistant metal baskets to improve durability.

12.5 GENERAL E&SPC BMPS

General BMPs are utilized during earth moving activities to prevent erosion or the transport of sediment off the project site.

A. Rock Construction Entrance. A rock construction entrance is a stabilized pad of aggregate placed at every entrance and exit point of a construction site. Rock construction entrances are used to reduce offsite transport of soil by removing mud and sediment from the wheels of construction vehicles prior to exiting the construction site. A typical rock construction entrance should be designed as shown on Standard Drawing RC-77M. The minimum travel length of the entrance is 15 m (50 ft). This is the length of track over which the truck must drive. The minimum width of the entrance is 6 m (20 ft). In some cases the designer may want to pave the area adjacent to the travelway. Examples of where this may be desirable are in high traffic areas where spilled aggregate on the paved roadway surface may pose a safety hazard, or if pavement integrity where the rock construction entrance is located is an issue. In these cases, the minimum travel length of the entrance is 30 m (100 ft), with the first 15 m (50 ft) being paved.

Maintenance of rock construction entrances should be considered during construction. Remove sediment deposited on paved roadways and return it to the construction site. Do not wash the roadway or sweep the deposits into roadway ditches, sewers, culverts, or other channels. Repair damaged wash racks as necessary to maintain their effectiveness. Rock construction entrances should be inspected daily. Rock construction entrance thickness should be constantly maintained to the specified dimensions.

B. Rock Filter Outlet. Rock filter outlets can be defined as a temporary barrier of coarse aggregate and silt barrier fence used to remove sediment. Rock filter outlets are used to replace areas of silt barrier fence that have been undercut or overtopped. Typical rock filter outlet placement is shown on Standard Drawing RC-70M.

Rock filter outlets should be maintained during construction by removing accumulated sediment it reaches one-third the height of the rock filter. Rock filter outlets should be inspected daily and after each rainfall event. Damaged outlets or loss of aggregate should be repaired immediately to the required dimensions.

C. Compost Filter Sock. A compost filter sock can be defined as a temporary barrier of organic compost in a water permeable mesh used to remove sediment from sheet flow runoff. Compost filter socks are constructed below a disturbed area to protect receiving surface water from runoff from the disturbed area. Compost filter socks range in diameter from 300 mm (12 in) to 600 mm (24 in). Typical compost filter sock placement is shown on Standard Drawing RC-70M.

Compost filter socks should be considered for use in the following areas:

- At the toe of fill slopes and along the downslope perimeter of disturbed areas where runoff continues to sheet flow away from the area.
- Around the perimeter of all temporary soil stockpiles maintained on the project site.
- Where rock or rocky soils inhibit anchoring of silt barrier fence.
- Around the perimeter of wetlands, streams or other sensitive areas.

There is no formal design procedure for compost filter socks beyond placement in the appropriate locations. Place compost filter socks downslope of all disturbances, in stabilized areas, and parallel to contours so that the filter socks are on level grade and so that the sock makes continuous contact with the underlying soil. Filter socks should not be placed across concentrated flow (e.g., channel swales, erosion gullies, across pipe outfalls, or as inlet protection).

Extend both ends of the compost filter sock at least 2.4 m (8 ft) up slope at 45 degrees to the main alignment to allow for pooling of water. Ensure a minimum of 300 mm (12 in) to either side of the area to be protected. The filter sock should also be placed at least 2.4 m (8 ft) from the toe of slope if possible. Limit compost filter sock use to areas where the slope is milder than 1V:2H (2H:1V). The total length of slope behind the filter sock should not exceed the values in Table 12.17. Slope length is the distance from the filter sock to the drainage divide or nearest upslope channel. Filter socks cannot be placed in multiple rows to increase the allowable slope length. Additional compost filter sock specifications and details (e.g., anchor post spacing) are provided in Publication 408, *Specifications*, and in Publication 72M, *Roadway Construction Standards,* respectively.

Maintenance of compost filter socks during construction should be considered. Accumulated sediment should be removed as necessary or when sediment accumulation reaches one-half the height of the exposed compost filter sock. Compost filter socks should be inspected weekly and after each rainfall event. Any section of compost filter sock that has been undermined or washed out should be immediately replaced.

Table 12.17 Maximum Slope Length for Compost Filter Socks

Slope Percent	Maximum Slope Length					
	300 mm (12") Diameter		450 mm (18") Diameter		600 mm (24") Diameter	
	m	ft	m	ft	m	ft
2 (or less)	225	750	300	1000	400	1300
5	150	500	165	550	200	650
10	75	250	90	300	120	400
15	50	170	60	200	100	325
20	38	125	42	140	80	260
25	30	100	33	110	60	200
30	23	75	27	90	40	130
35	23	75	24	80	35	115
40	23	75	24	80	30	100
45	15	50	18	60	24	80
50	15	50	17	55	20	65

Reference: Filtrexx SiltSoxx

D. Compost Filter Berm. A compost filter berm can be defined as a temporary barrier of organic compost used to remove sediment from sheet flow runoff. Compost filter berms are constructed below a disturbed area to protect receiving surface water from runoff from the disturbed area. Compost filter berms are trapezoidal shaped berms that are 1200 mm (48 in) wide and 600 mm (24 in) high. A typical compost filter berm placement is shown on Standard Drawing RC-70M.

Compost filter berms should be considered for use in the following areas:

- At the toe of fill slopes and along the downslope perimeter of disturbed areas where runoff continues to sheet flow away from the area.
- Around the perimeter of all temporary soil stockpiles maintained on the project site.
- Where rock or rocky soils inhibit anchoring of silt barrier fence.
- Around the perimeter of wetlands, streams or other sensitive areas.
- Where rock or uneven ground prevents the use of compost filter socks due to the inability to provide continuous contact with the underlying soil.

There is no formal design procedure for compost filter berms beyond placement in the appropriate locations. Place compost filter berms downslope of all disturbances, in stabilized areas, and parallel to contours so that the filter berms are on level grade. Place compost filter berms a minimum of 2.4 m (8 ft) from toe of fill slopes, where possible. Filter berms should not be placed across concentrated flow (e.g., channel swales, erosion gullies, across pipe outfalls, or as inlet protection).

Extend both ends of the compost filter upslope to the main alignment to allow for pooling of water. Limit compost filter berm use to areas where the slope is milder than 1V:2H (2H:1V). The total length of slope behind the barrier should correspond to the allowable slope lengths for a 750 mm (30 in) silt barrier fence (see Table 12.20). Slope length is the distance from the filter berm to the drainage divide or nearest upslope channel. Filter berms cannot be placed in multiple rows to increase the allowable slope length. Additional compost filter berm specifications are provided in Publication 408, *Specifications*.

Maintenance of compost filter berms during construction should be considered. Accumulated sediment should be removed as necessary or when sediment accumulation reaches one-third the height of the exposed berm. Compost filter berms should be inspected weekly and after each rainfall event. Any section of compost filter berms that has been undermined or overtopped should be immediately replaced.

E. Silt Barrier Fence. Silt barrier fence can be defined as a temporary barrier of entrenched geotextile (filter fabric) stretched across and attached to supporting posts. Silt barrier fence is used to remove sediment from sheet flow runoff.

Silt barrier fences are constructed below a disturbed area to protect receiving surface water from runoff from the disturbed area.

Silt barrier fence should be considered for use in the following areas:

- At the toe of fill slopes and along the downslope perimeter of disturbed areas where runoff continues to sheet flow away from the area.
- Around the perimeter of wetlands, streams or other sensitive areas.
- Around the perimeter of all temporary soil stockpiles maintained on the project site.

Silt barrier fence is NOT appropriate for use in the following areas or manners:

- Concentrated flow (e.g., channel swales, erosion gullies, across pipe outfalls, or as inlet protection, etc.).
- Where rock or rocky soils prevent anchoring of the fence or proper installation of the fence posts.
- In multiple rows to increase slope length.
- On fills or in extremely loose soils (e.g., sandy loam).
- In wooded areas, where tree roots prevent proper anchoring of the fence.

There is no formal design procedure for silt barrier fence beyond placement in the appropriate locations. Place silt barrier fence downslope of all disturbances, in stabilized areas, and parallel to contours so that the fence is on level grade. Extend both ends of each fence section at least 2.4 m (8 ft) up slope at 45 degrees to the main fence alignment to allow for pooling of water. Limit silt barrier fence use to areas where the slope is milder than 1V:2H (2H:1V). The total length of slope behind the barrier should not exceed the values shown in Table 12.18. Slope length is the distance from the fence to the drainage divide or nearest upslope channel. Silt barrier fence post spacing and geotextile classification criteria is provided in Table 12.19 and in Publication 72M, *Roadway Construction Standards*.

Maintenance of silt barrier fences during construction should be performed to ensure continued function. Accumulated sediment should be removed as necessary or when sediment has accumulated to one-half above the ground height of the fence. Silt barrier fence should be inspected weekly and after each rainfall event. Any section of silt barrier fence that has been undermined or overtopped should be immediately replaced with a rock filter outlet.

Additional silt barrier fence specifications and details are provided in Publication 408, *Specifications*.

1. Silt barrier fence, 450 mm (18 in) height. This type of silt barrier fence is the minimum height of silt barrier fence used by PennDOT. On Standard Drawing RC-70M, a typical silt barrier fence, 450 mm (18 in) height detail is shown.

2. Silt barrier fence, 750 mm (30 in) height. This type of silt barrier fence is the maximum height of silt barrier fence used by PennDOT. On Standard Drawing RC-70M, a typical silt barrier fence, 750 mm (30 in) height detail is shown.

Table 12.18 Silt Barrier Fence Slope Length

Slope Percent	Maximum Slope Length			
	Silt Barrier Fence 450 mm (18 in Height)		Silt Barrier Fence 750 mm (30 in Height)	
	m	ft	m	ft
2 (or less)	45	150	150	500
5	30	100	75	250
10	15	50	45	150
15	10	35	30	100
20	8	25	20	70
25	6	20	17	55
30	4.5	15	14	45
35	4.5	15	12	40
40	4.5	15	11	35
45	3	10	9	30
50	3	10	8	25

Reference: *Erosion and Sediment Pollution Control Program Manual* (PA DEP, 2000).

Table 12.19 Silt Barrier Fence Post Spacing and Geotextile Requirements

Silt Barrier Fence, Height	Type of Class 3 Geotextile Material	Nominal Geotextile Height	Post Spacing Without Mesh Support	Max. Post Spacing With Mesh Support
450 mm (18 in)	3A	750 mm (18 in)	2.4 m (8 ft)	NA
750 mm (30 in)	3A	1050 mm (30 in)	NA	2.4 m (8 ft)
450 mm (18 in)	3B	750 mm (18 in)	1.2 m (4 ft)	NA
750 mm (30 in)	3B	1050 mm (30 in)	NA	1.2 m (4 ft)

Reference: Publication 72M, *Roadway Construction Standards*, RC-70M.

F. Heavy Duty Silt Barrier Fence. Heavy duty silt barrier fence is a temporary barrier of entrenched geotextile backed with wire fabric (i.e., chain link) fence. Both the wire fabric and geotextile are stretched across and attached to supporting posts. Heavy duty silt barrier fence may be used to control runoff from small, disturbed areas where the maximum slope lengths for silt barrier fence cannot be met and sufficient room for construction of a sediment trap does not exist.

Heavy duty silt barrier fence should be considered for use in the following areas or manners:

- Where the maximum slope lengths for silt barrier fence are exceeded. The maximum slope length above heavy duty silt barrier fence should not exceed that shown in Table 12.20. Slope length is the distance from the fence to the drainage divide or nearest upslope channel.
- Where access is possible by construction equipment required to install and remove the chain link fencing, including a posthole drill.
- Around the perimeter of all temporary soil stockpiles maintained on the project site.

There is no formal design procedure for heavy duty silt barrier fence beyond placement in the appropriate locations. Place heavy duty silt barrier fence downslope of all disturbances, in stabilized areas, and parallel to contours so that the fence is on level grade. It should not be placed across concentrated flow (e.g., channel swales, erosion gullies, across pipe outfalls, or as inlet protection). Where rock or rocky soils prevent anchoring of the fence or the proper installation of the fence posts, another BMP strategy should be examined.

Extend both ends of each fence section at least 2.4 m (8 ft) up slope at 45 degrees to the main fence alignment to allow for pooling of water. Limit heavy duty silt barrier fence use to areas where the slope is milder than 1V:2H (2H:1V). The total length of slope behind the barrier should not exceed the values shown in Table 12.20. Slope length is the distance from the fence to the drainage divide or nearest upslope channel. Filter berms cannot be placed in multiple rows to increase the allowable slope length. On Standard Drawing RC-70M a typical heavy duty silt barrier fence detail is shown.

Maintenance of heavy duty silt barrier fences during construction should be performed to ensure continued function. Accumulated sediment should be removed as necessary or when sediment has accumulated to one-half above the ground height of the fence. Heavy duty silt barrier fence should be inspected weekly and after each rainfall event. Any section of heavy duty silt barrier fence that has been undermined or overtopped should be immediately replaced with a rock filter outlet. Adhere to the manufacturer's suggestions relative to required geotextile placement during weathering.

Additional heavy duty silt barrier fence specifications and details are provided in Publication 408, *Specifications,* and on Standard Drawing RC-70M.

Table 12.20 Heavy Duty Silt Fence Slope Length

Slope Percent	Maximum Slope Length	
	m	ft
2 (or less)	300	1,000
5	165	550
10	100	325
15	65	215
20	50	175
25	40	135
30	30	100
35	25	85
40	22	75
45	18	60
50	15	50

Reference: *Erosion and Sediment Pollution Control Program Manual* (PA DEP, 2000).

G. Vegetative Filter Strip for E&SPC. A vegetative filter strip is a well-established perennial grassy area located below a disturbed area that can be used to remove sediment from runoff prior to it reaching receiving waters. The purpose of a vegetative filter strip is to provide an area that slows sediment-laden runoff allowing the sediment to filter out, prior to reaching the receiving waters.

Vegetative filter strips are effective if runoff is in the form of sheet flow and if the vegetative cover is established prior to receiving runoff. It is best to have an area of existing, well-established perennial grass. It should be noted that vegetative filter strips take a considerable amount of space. Right-of-way availability should be considered prior to selection and design of this BMP. Use of vegetative filter strips on adjacent properties is not acceptable.

Refer to Table 12.21 for the slope length requirements and maximum length of flow to be treated by the strip. Vegetative filter strips function best at slopes less than 5%. Using Equation 12.1, define the minimum width of a vegetative filter strip. The minimum acceptable width is 15 m (50 ft) as shown in Figure 12.19.

(*Equation 12.1*)

Metric: $W_{(min)} = 0.61S + 7.5$

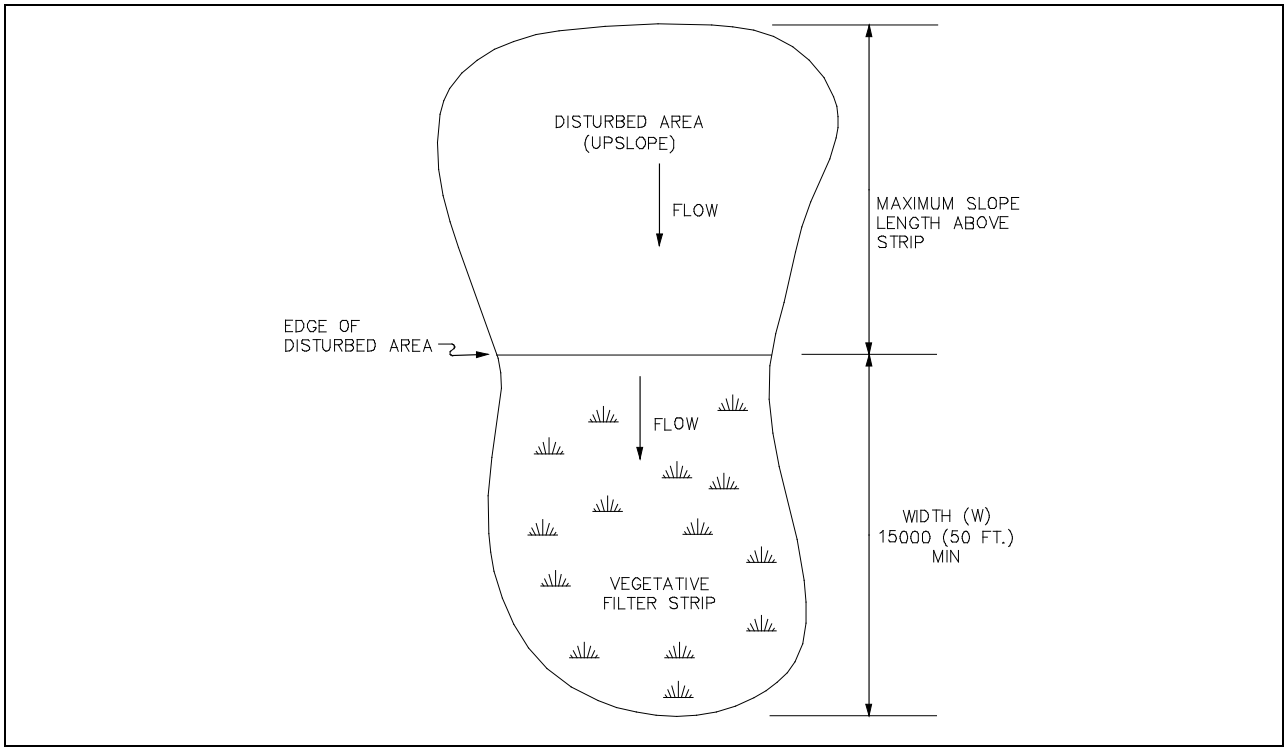
U.S. Customary: $W_{(min)} = 2S + 25$

Where: W_{min} = Minimum Filter Strip Width, m (ft)
S = Average Slope Percent

Table 12.21 Maximum Flow Length For Vegetative Filter Strip Slope

Slope Percent	Maximum Slope Length Above Strip	
	m	ft
2 (or less)	45	150
5	30	100
10	15	50
15	10	35
20	7.5	25
25	6	20
30	4.5	15
35	4.5	15
40	4.5	15
45	3	10
50	3	10

Figure 12.19 Vegetative Filter Strip



Accumulated sediment on vegetated filter strips should be removed as necessary. If the vegetated filter strip width is reduced by more than half of its original width due to accumulated sediment, an alternative, such as a compost filter sock or silt barrier fencing, should be installed immediately.

Refer to Publication 13M, Design Manual, Part 2, *Highway Design*, Section 13.6 and the *Pennsylvania Stormwater Best Management Practices Manual* (PA DEP, 2006), for design guidance on permanent vegetative filter strips. The filter strip design in this section of the manual is for erosion and sediment pollution control only.

H. Pumped Water Filter Bag (also known as a Sediment Filter Bag). Pumped Water Filter Bags (PWFBs) are non-woven, geotextile fabric bags with double stitched "J" type seams and a sewn-in spout. The maximum pore size in a PWFB is 150 microns. Refer to Publication 408, *Specifications,* and on Standard Drawing RC-75M for a detail and general notes associated with this BMP.

PWFBs can be used in a variety of construction areas where sediment-laden water is present and dewatering is needed. In general, they are used when water is pumped from excavation holes associated with bridge piers and abutments. They can also be used to dewater trenches and to filter water pumped from sediment traps or sediment basins when dewatering sediment storage zones of these facilities.

A design for this BMP is not required; however, the contractor must match the PWFB size to the particular pump such that the pumping rate to the bag is no greater than 2,840 L/min (750 gal/min) or ½ the maximum specified by the manufacturer, whichever is less. This requirement is due to the assumption that water will only discharge through to top half of the bag. A general note for this is located on Standard Drawing RC-75M detail and must be included on the plans.

The following construction considerations should be addressed:

- Coarse Aggregate - A coarse aggregate pad should be constructed only when leveling is required.
- Geotextile Channel Linings - These may be necessary to protect the discharge flow path. If it appears that there may be an issue with scouring from PWFB discharge, line the discharge flow path with Geotextile (Class 4, Type A).
- Pump - Required to get water to the PWFB.
- Lifting Straps - Required to lift the bag when pumping has ceased and/or the bag has reached sediment capacity.

The following considerations should be addressed when determining a possible location for a PWFB:

- PWFBs should be located in a well-vegetated (grassy) area, and discharge onto stable, erosion resistant areas. Where this is not possible, a geotextile (Class 4, Type A) lined flow path should be provided.
- PWFBs are to be located in areas that are accessible by construction machinery for maintenance and removal purposes.
- When the slope of the area where the PWFB is to be located exceeds 5%, a coarse aggregate pad should be constructed for leveling purposes.
- Compost berm or compost filter sock should be installed below bags located within 15 m (50 ft) of receiving stream or where grassy area is not available.

The following construction considerations should be addressed:

- A suitable means of accessing the PWFB with machinery required for disposal purposes must be provided at all times.
- Replace PWFBs when they become half filled with sediment.
- Keep spare PWFBs on site for replacement of those that have failed or are filled.
- Upon detection of any problem with a PWFB or hose between the pump and the bag, cease pumping immediately and do not resume until the problem is corrected or another bag or hose is placed into operation. At no time shall there be multiple hoses attached to a single PWFB.
- PWFBs must be removed from the site and disposed of in an appropriate manner. Cutting bags open and seeding the area is not an acceptable practice.

I. Temporary Slope Pipe. A temporary slope pipe is a flexible or rigid pipe, used prior to permanent drainage structures, installed to convey concentrations of runoff from the top to bottom of disturbed slopes. A typical temporary slope pipe plan and section view is shown on Standard Drawing RC-74M. A temporary slope pipe may be installed to transport stormwater runoff down the face of a cut or fill slope to a stabilized area or an interceptor channel that conveys sediment-laden water to a sediment trap/basin. Temporary slope pipes should be used prior to the installation of permanent facilities and/or permanent stabilization of slopes in accordance with the proposed sequence of work for the project.

In general, when temporary pipes are part of a formal E&SPC Plan design, they should be sized appropriately by determining peak flows from drainage areas and cover types. Table 12.22 is also an acceptable method to size slope pipes. The maximum contributing drainage area to a slope pipe is 2 hectares (5 acres).

Table 12.22 Suggested Minimum Sizes

Drainage Area		Corrugated Pipe Diameter		Minimum Berm Height	
hectare	**acre**	**mm**	**in**	**mm**	**in**
0 to 0.8	0 to 2	300	12	600	24
0.8 to 1.6	2 to 4	375	15	675	27
1.6 to 2.0	4 to 5	450	18	750	30

Reference: *Erosion and Sediment Pollution Control Program Manual* (PA DEP, 2000).

Inspection of temporary slope pipes should be done on a weekly basis and after each runoff event. Accumulated sediment should be removed from the entrance of the pipe. Damaged or leaking pipe sections should be repaired immediately.

Adequate outlet protection should be provided as needed at the outlet of the temporary slope pipe. Rock outlet protection design guidance can be found in Section 12.5.K.

J. Storm Inlet Protection. Storm inlet protection provides a method of collecting and/or detaining runoff from disturbed areas to allow sediment to settle before it enters the storm sewer. Storm inlet protection may be constructed around, on, or inserted into an inlet. Storm inlet protection consists of various methods including inlet filters, traps, and berms. Each type differs in application depending on site conditions and type of inlet. Not all designs are appropriate in all cases. The designer must select a design suitable for the project needs and site conditions. Storm inlet protection is not required for inlets tributary to sediment basins and sediment traps.

Storm inlet protection should be considered in the following situations:

- Existing storm sewers in disturbed areas.
- Wherever earth disturbance occurs in an area with a storm sewer system that does not discharge into a functioning sediment trap or sediment basin during construction.
- Wherever the storm sewer system discharges to a detention pond, since detention ponds may not effectively remove sediment prior to discharging.

The following steps should be used to evaluate the project to determine the most suitable method of protection. In addition to the design steps provided below, a downslope berm must be provided for all methods of Storm Inlet Protection for inlets on grade. If the berm would interfere with vehicular traffic, Storm Inlet Protection cannot be used and other sediment control measures should be examined.

Step 1 Determine the contributing drainage area to the inlet.
Step 2 Determine if there will be vehicular traffic in the area.
Step 3 Determine if the inlet will be operational prior to the contributing drainage area being stabilized.

Down gradient berms should be placed downstream of each inlet accepting sediment laden runoff. Do not use berms where they may pose a vehicular hazard or where additional ponding of water may cause flooding and traffic hazards. Refer to Standard Drawing RC-72M for placement of down gradient berms.

Three methods of inlet protection are addressed in this section: Inlet Filter Bags, Concrete Block/Gravel Inlet Protection and Pipe/Gravel Inlet Protection.

1. Inlet Filter Bags. Inlet filter bags (also referred to as inlet sediment sacks) are suitable for drainage areas less than 0.2 hectare (0.5 acre) and areas with limited vehicular traffic, recognizing that ponding may occur creating a hazard. A 150 mm (6 in) sandbag berm should be installed immediately downstream of the inlet to prevent the majority of the runoff from bypassing each inlet. A sandbag berm is not required for inlets in a sump condition as there should be no runoff bypass. Inlet filter bags can be used for inlets with Type C, M or S top units. Refer to Standard Drawing RC-72M for typical use of inlet filter bags.

Inlet filter bags should be inspected weekly and after each rainfall event to ensure that the bag is functioning properly. Any inlet filter bags that become ripped or torn should be removed and properly disposed of and immediately replaced. In addition, any sandbag berms that may become washed out should also be repaired or replaced. Accumulated sediment should be removed when the inlet filter bag reaches one-half maximum capacity. Sediment should be removed and disposed of in accordance with Publication 408, *Specifications*.

2. Concrete Block/Gravel Inlet Protection. Concrete block/gravel inlet protection utilizes a barrier of concrete block overtop of the inlet grate with a gravel and wire mesh filter system to remove sediments from disturbed runoff. Concrete block/gravel inlet protection is suitable for drainage areas less than 0.4 hectare (1 acre) and areas where there is no vehicular traffic. A sandbag or earthern berm should be installed immediately downstream of the inlet to prevent runoff bypass. The berm should be installed 150 mm (6 in) above the height of the proposed block and gravel protection. Additional sandbags may be required upstream of the downstream end of the inlet parallel to the contour to avoid runoff bypass. Berms should not be installed in areas of vehicular traffic. A berm is not required for inlets in a sump condition as there should be no runoff bypass. Concrete block/ gravel inlet protection can be used for inlets with Type C, M or S top units. Refer to Standard Drawing RC-72M for typical use of concrete block/gravel inlet protection.

Concrete block/gravel inlet protection should be inspected after each rainfall event to ensure that the inlet protection is functioning properly. Any concrete blocks or gravel that has been displaced should be immediately repaired. In addition, any sandbag berms that may become washed out should also be repaired or replaced. Accumulated sediment should be removed as necessary. Sediment should be removed and disposed of in accordance with Publication 408, *Specifications*.

3. Pipe/Gravel Inlet Protection. Pipe/Gravel Inlet Protection utilizes corrugated metal pipe (or equivalent), gravel and wire mesh to remove sediment from disturbed runoff prior to entering the inlet. Pipe/gravel inlet protection is suitable for drainage areas less than 0.4 hectare (1 acre) and areas where there is no vehicular traffic. A sandbag or earthern berm should be installed immediately downstream of the inlet to prevent runoff bypass. The berm height should be a minimum of 400 mm (16 in) in height with a minimum of 150 mm (6 in) in freeboard above the top of the filter pipe. Additional sandbags may be required upstream of the downstream end of the inlet parallel to the contour to avoid runoff bypass. Berms should not be installed in areas of vehicular traffic. A berm is not required for inlets in a sump condition as there should be no runoff bypass. Pipe/gravel inlet protection can be used for inlets with Type C, M or S top units. Refer to Standard Drawing RC-72M for typical use of pipe/gravel inlet protection.

Pipe/gravel inlet protection should be inspected after each rainfall event to ensure that the inlet protection is functioning properly. Any pipe or gravel that has been displaced should be immediately repaired. In addition, any sandbag berms that may become washed out should also be repaired or replaced. Accumulated sediment should be removed as necessary. Sediment should be removed and disposed of in accordance with Publication 408, *Specifications*.

K. Outlet Protection: Rock. Rock outlet protection is a temporary or permanent rock lining constructed in the area of an outfall to provide scour protection against discharges from pipes and channels. The purposes are to prevent erosion in areas immediately downstream of pipe and channel outlets and to reduce the velocity of the water.
Several types of outlet protection are described in this section. Outlet protection must be provided for the following locations where outlet velocities exceed that capacity of downstream areas to resist erosion:

- Storm drains.
- Sediment traps.

- Sediment basins.
- Stormwater management basins.
- Temporary slope pipes.
- Ditches or channels (temporary or permanent).

For existing facilities, where scour or erosion is occurring at the outlets, place rock as necessary based on the scour limits. Refer to Standard Drawing RC-72M for minimum protection requirements.

1. Rock Basin or Rock Energy Dissipator. Rock basins and rock energy dissipators are used to dissipate energy and control erosion at pipe outlets where flow will remain concentrated. The rock basin should be used where the outlet pipe diameter is less than 900 mm (36 in) and the rock energy dissipator should be used where the outlet pipe diameter is 900 mm (36 in) or larger. When outlet velocities exceed 4.2 m/s (14 ft/s), the top 150 mm (6 in) of the rock lining can be grouted, or another type of velocity reduction measure should be used (e.g., paved energy dissipator, riprap basin, stilling well, etc.). Another measure, such as a riprap basin, paved energy dissipator, or stilling basin, should be used if the outlet velocity exceed 5.7 m/s (19 ft/s). Rock basins and rock energy dissipators should be constructed as shown to Standard Drawing RC-72M.

It should be noted that Rock Basins and Rock Energy Dissipators are conservatively sized. They are designed to handle severe conditions resulting from common pipe and channel installations. Where the design discharge velocity is low and/or space is limited, consideration should be given to the other outlet protection BMPs in this chapter.

2. Rock Apron. Rock aprons are used to dissipate energy and control erosion at pipe outlets. Rock aprons should be constructed as shown to Standard Drawing RC-72M. Rock aprons can be used at the outfall locations mentioned previously in this section and where velocities are less than 4.3 m/s (14.5 ft/s). Rock aprons can be used is areas where a minimum or maximum tailwater condition exists.

The following design guidelines step through a process of evaluating the flow to determine the appropriate riprap size and dimensions of the rock apron.

Minimum tailwater exists when the normal depth of flow in the receiving watercourse, as calculated by Manning's equation, is less than one-half the diameter of the discharge pipe, or where no channel or swale exists at the point of discharge.

- Locate the design discharge (pipe flowing full) along the bottom of Figure 12.20.
- Follow a vertical line to the point where it intersects the first curve corresponding to the diameter of the discharge pipe. From that point follow a horizontal line to the right to determine the minimum d_{50} stone size of the riprap in feet.
- Check the table in Chapter 8, *Open Channels,* Section 8.7 to ensure that the anticipated discharge velocity does not exceed the maximum permissible velocity for the size of riprap obtained. If the anticipated discharge velocity exceeds the maximum permissible velocity, increase the size of the riprap to a size whose permissible velocity is not exceeded.
- Follow the same vertical line mentioned above to the point where it intersects the second curve corresponding to the diameter of the discharge pipe. From that point, follow a horizontal line to the left and read the Minimum Length of the apron (L_a) in feet.
- After the length (L_a) is determined, use it to determine the apron width using the formula:

(Equation 12.2)

$$W = 3D_o + L_a$$

Where: D_o = the outlet pipe diameter, m (ft)
L_a = the length of the apron, m (ft)

Where the apron design width (W) exceeds the downstream watercourse bottom width, a transition zone must be designed and installed downstream from the apron to the watercourse. When the pipe discharges directly into a well-defined channel, the apron should extend across the channel bottom and up the channel

banks to an elevation 300 mm (1 ft) above the maximum tailwater depth or to the top of the bank, whichever is less. See to Standard Drawing RC-72M.

If the pipe discharges onto a flat area with no defined channel, the width of the upstream end of the apron should be three times the diameter of the outlet pipe. The width of the downstream end of the apron should be equal to the pipe diameter plus the length of the apron. The bottom grade of the apron should be constructed on a flat grade (0%). The invert of the apron should be equal to the invert of the receiving channel. If the pipe discharges into a well-defined channel, the side slopes should be no steeper than 1V:2H (2H:1V).

Maximum tailwater exists when the depth of flow in the receiving water course, as calculated by Manning's equation, is greater than one-half the diameter of the pipe.

- Locate the design discharge (pipe flowing full) along the bottom of Figure 12.21.
- Follow a vertical line to the point where it intersects the first curve corresponding to the diameter of the discharge pipe. From that point follow a horizontal line to the right to determine the minimum d_{50} stone size of the riprap in feet.
- Check the table in Chapter 8, *Open Channels,* Section 8.7 to ensure that the anticipated discharge velocity does not exceed the maximum permissible velocity for the size of riprap obtained. If the anticipated discharge velocity exceeds the maximum permissible velocity, increase the size of the riprap to a size whose permissible velocity is not exceeded.
- Follow the same vertical line mentioned above to the point where it intersects the second curve corresponding to the diameter of the discharge pipe. From that point, follow a horizontal line to the left and read the minimum length of the apron (L_a) in feet. After the length (L_a) is determined, use it to determine the apron width using the formula.

Where the apron design width (W) exceeds the downstream watercourse bottom width, a transition zone must be designed and installed downstream from the apron to the watercourse. When the pipe discharges directly into a well-defined channel, the apron should extend across the channel bottom and up the channel banks to an elevation 300 mm (1 ft) above the maximum tailwater depth or to the top of the bank, whichever is less.

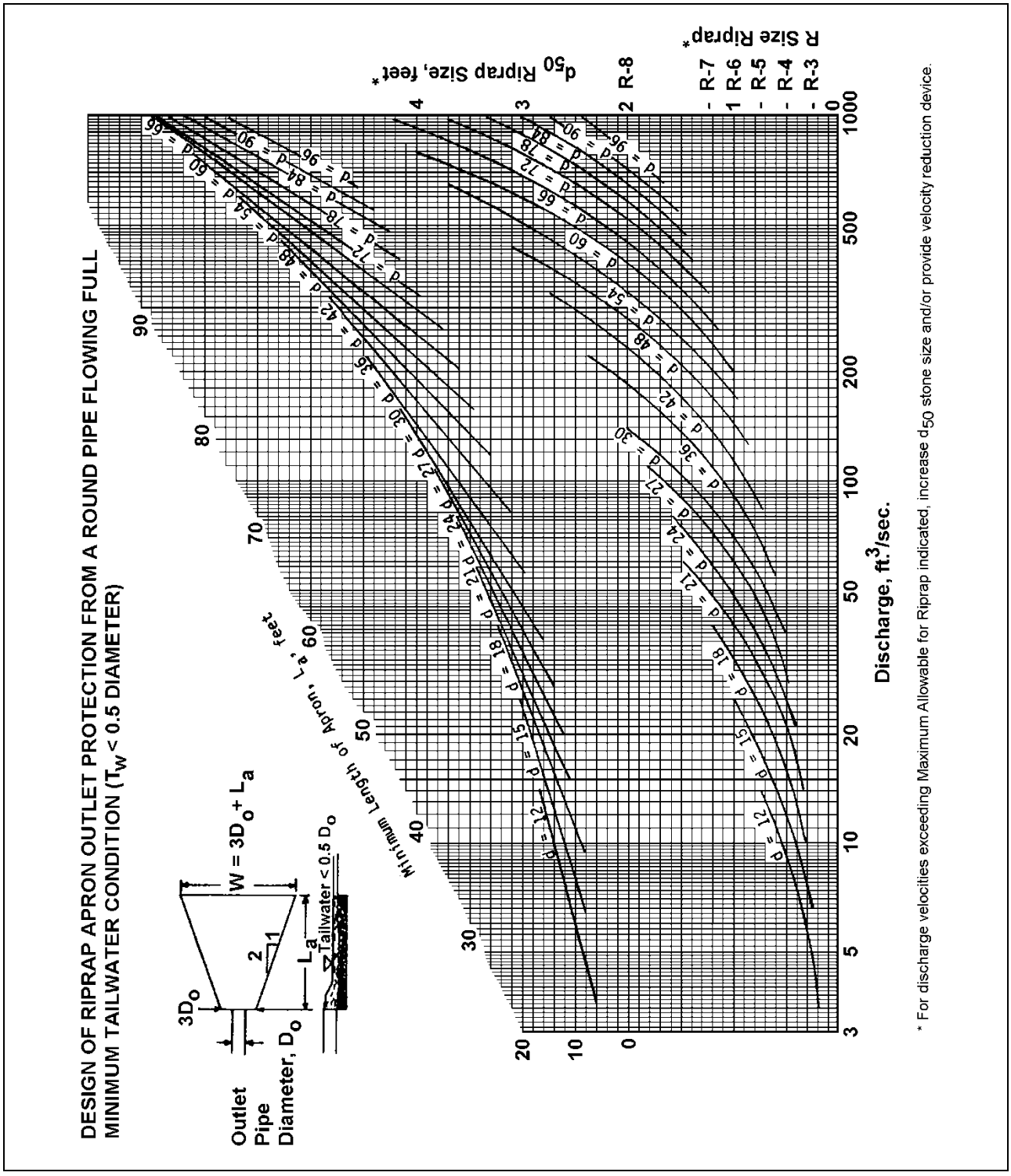
If the pipe discharges onto a flat area with no defined channel, the width of the upstream end of the apron should be three times the diameter of the outlet pipe. The width of the downstream end of the apron should be equal to the pipe diameter plus the length of the apron. The bottom grade of the apron should be constructed on a flat grade (0%). The invert of the apron should be equal to the invert of the receiving channel. If the pipe discharges into a well-defined channel, the side slopes should be no steeper than 1V:2H (2H:1V).

(Equation 12.3)

$$W = 3D_o + 0.4L_a$$

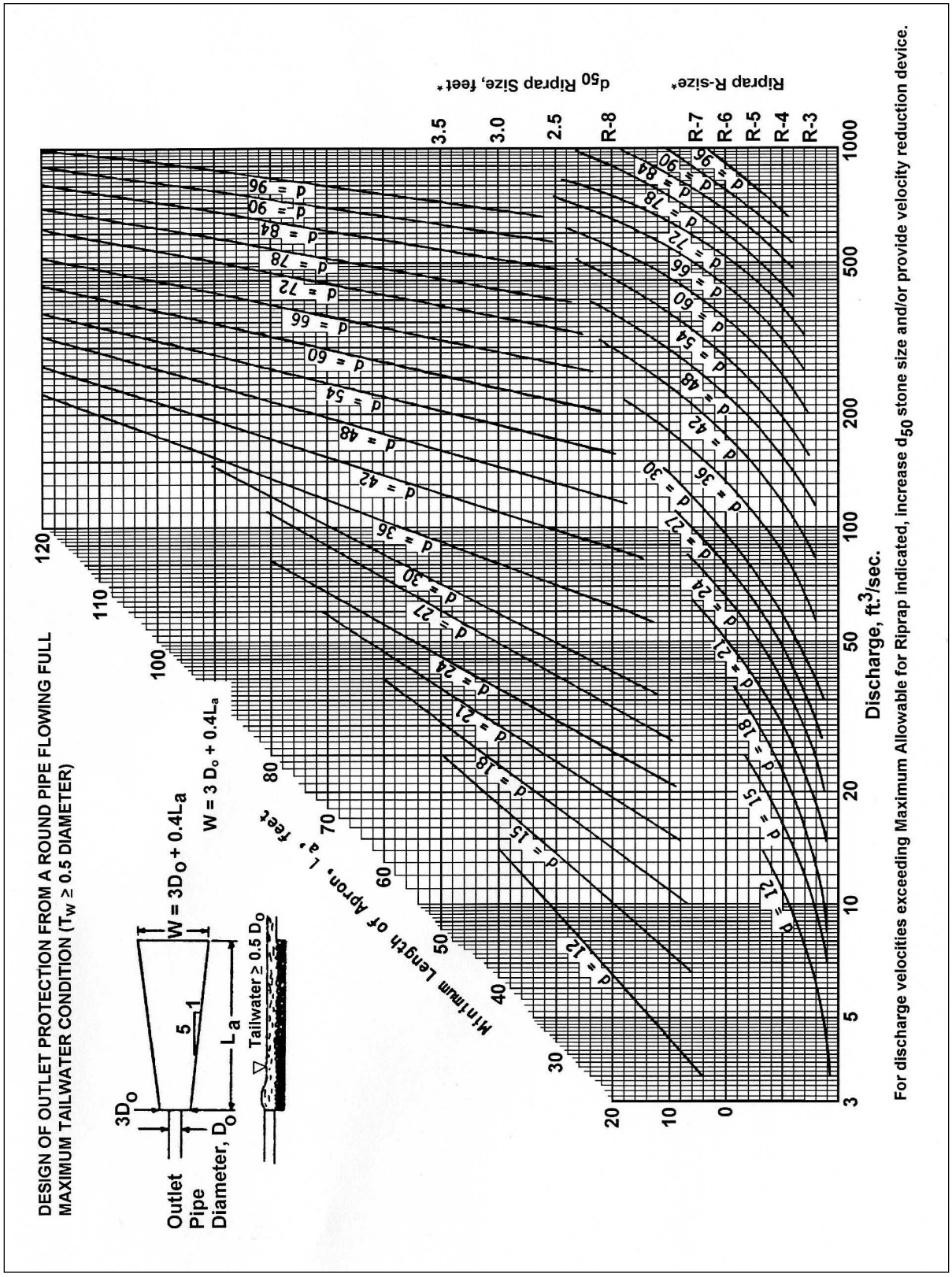
Where: D_o = the outlet pipe diameter, m (ft)
L_a = the length of the apron, m (ft)

Figure 12.20 Riprap Apron Design, Minimum Tailwater Condition



Reference: *Erosion and Sediment Pollution Control Program Manual* (PA DEP, 2000).

Figure 12.21 Riprap Apron Design, Maximum Tailwater Condition



Reference: *Erosion and Sediment Pollution Control Program Manual* (PA DEP, 2000).

3. Paved Energy Dissipators. Paved Energy dissipators basins are used to dissipate energy and control erosion where discharge velocities are greater than 4.3 m/s (14.5 ft/s). Refer to the HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, 2006) for guidance on selecting and designing paved energy dissipators. On Standard Drawing RC-72M a typical paved energy dissipator is shown.

4. Riprap Basins. Riprap basins are used to dissipate energy and control erosion where discharge velocities are greater than 4.3 m/s (14.5 ft/s). Refer to the HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, 2006) for guidance on selecting and designing riprap basins.

L. Outlet Protection: Stilling Well. A stilling well is a type of concrete energy dissipator constructed below grade at the outlet end of culverts and pipes. Stilling wells are used to reduce the velocity and energy of a concentrated discharge of water at an outlet. Stilling wells should be considered when discharge velocities at new culvert or pipe outlets would be sufficient to erode the downstream slope or channel unless an energy dissipation structure is installed. Specific situations include:

- Culvert outlets.
- Outlet pipes.
- Slope pipes
- Conditions where water needs to be slowed prior to entering a receiving channel.

Stilling wells should be considered in areas where little debris is expected. Placing them in non-forested areas proves to be most effective.

For safety reasons, stilling wells should be designed with a removable grate that can be locked into position.

Weep holes can be added to allow for infiltration, if necessary.

The type of stilling well outlined in this section is the Corps of Engineers Stilling Well. Design methods and procedures outlined below are adopted from the *Erosion and Sediment Pollution Control Program Manual* (PA DEP, 2000) and HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, 2006). After the above considerations have been weighed and the decision has been made to use a stilling well, the following step-by-step procedure may be used to guide the designer to complete the design of a stilling well.

1. Determine the maximum design discharge for the outlet in which a stilling well is proposed.

2. Using the Pipe Diameter (D) in m and feet and Design Discharge (Q) in m^3/s or cfs, determine the Well Diameter (DW) from Figure 12.23 and Figure 12.24.

3. Using the culvert slope (V:H) determine h_1/D_W from Figure 12.25. Variables h_1 and D_W are defined in Figure 12.25.

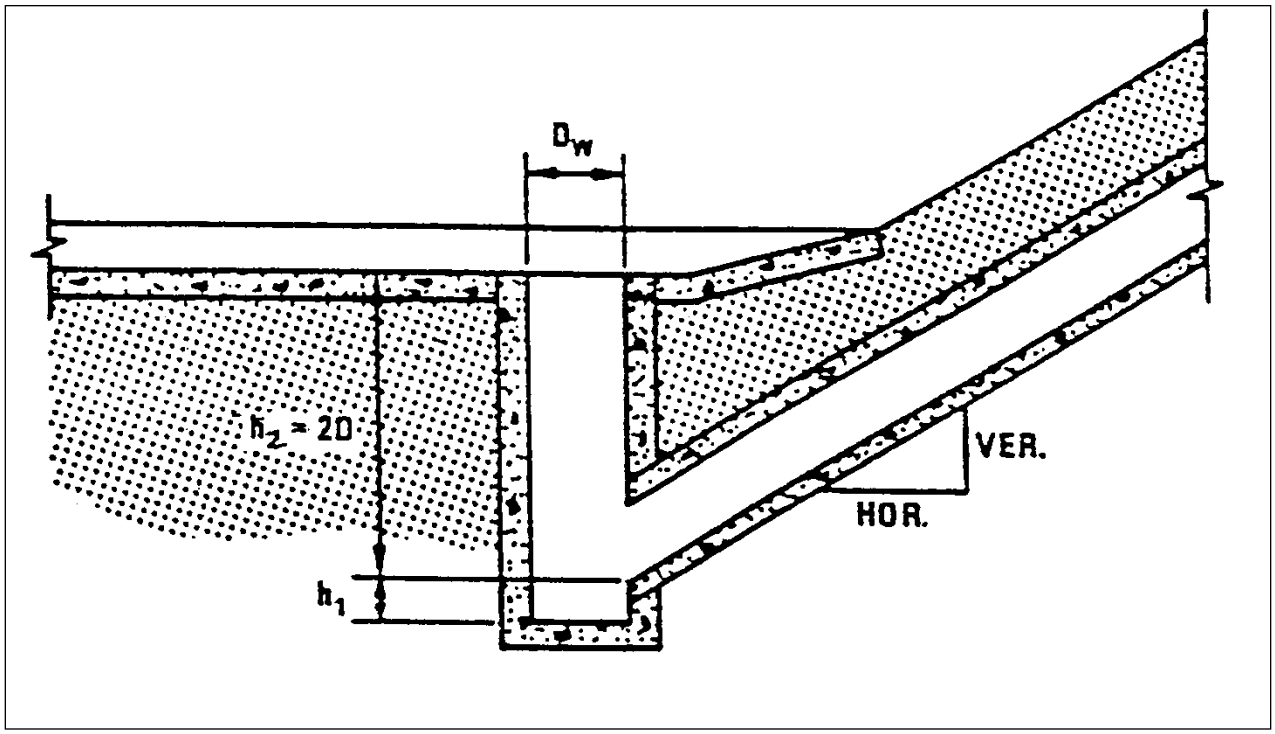
4. Calculate the Depth of Well Below Invert (h_1) in feet by multiplying h_1/D_W by the diameter of the well (DW).

5. Calculate the minimum depth of the well using the equation provided in Figure 12.22. Minimum depth of the well (h_2) above the invert is 2D. Note: Increasing this depth will increase the efficiency of the well.

6. The total depth of the well (h_w) can be calculated using $h_w = h_1 + h_2$.

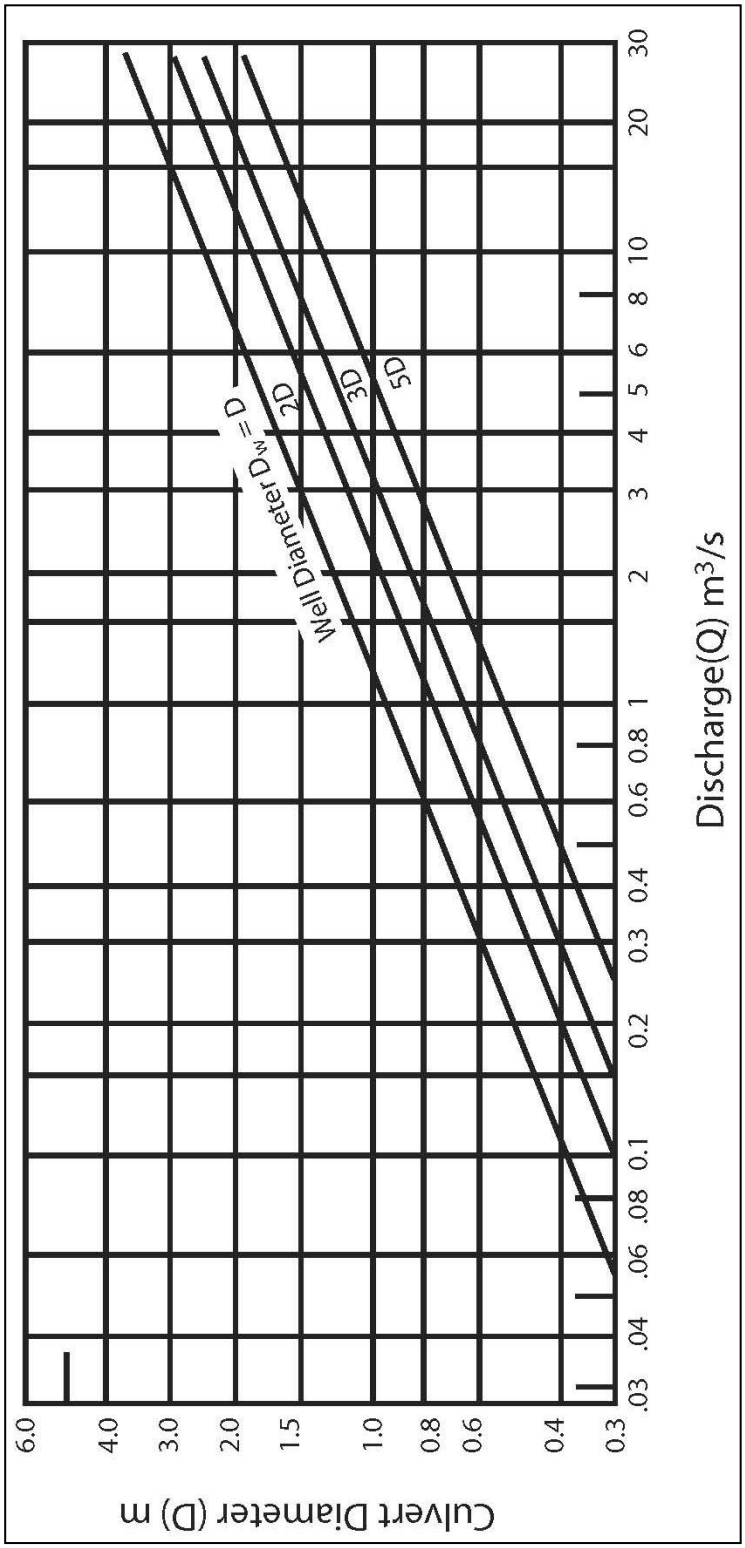
7. Riprap protection or other types of channel protection should be provided around the stilling well outlet and for a distance of $3D_W$ downstream.

Figure 12.22 Stilling Well (Typical)



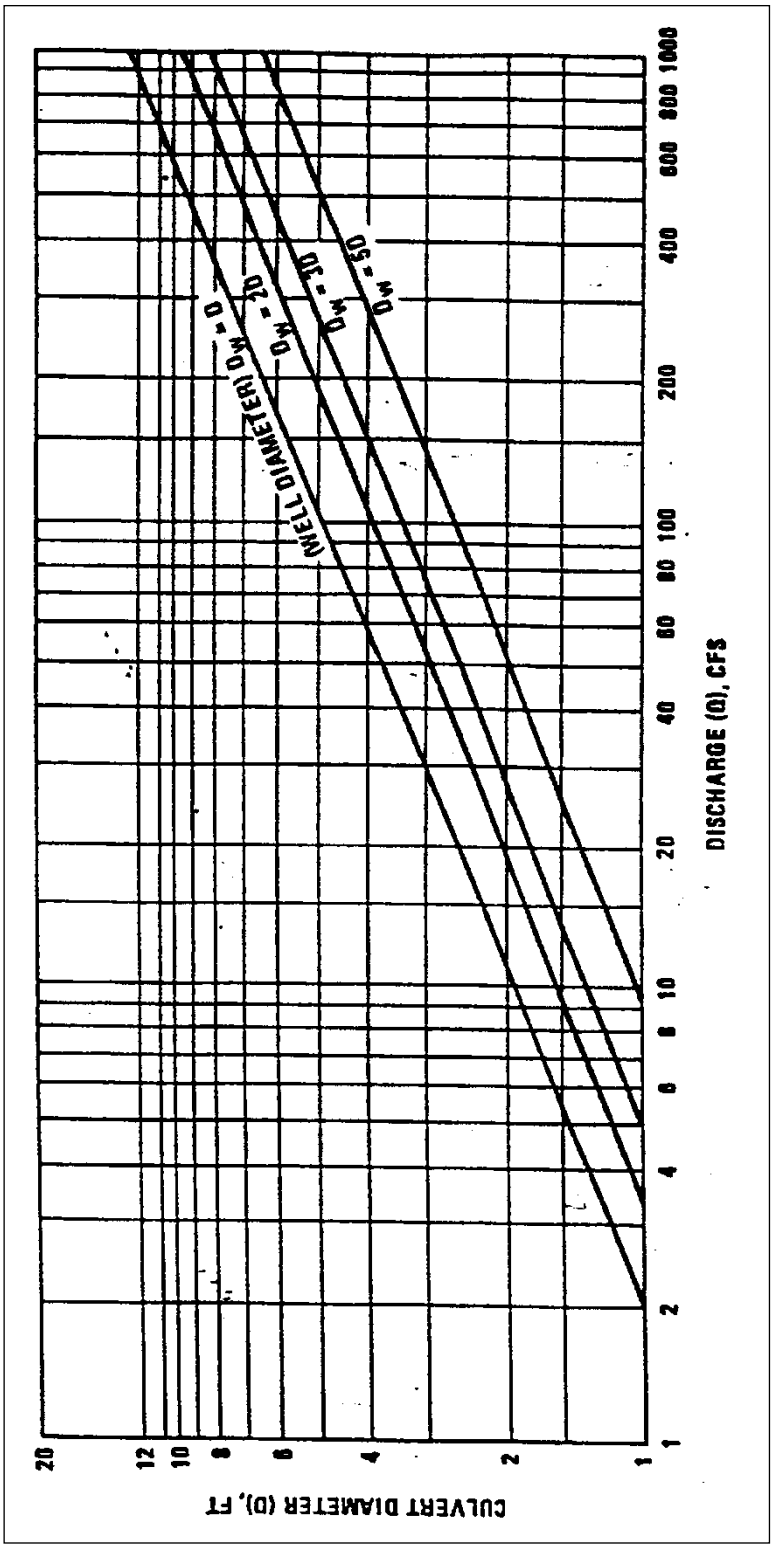
Reference: HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, 2006).

Figure 12.23 Stilling Well Diameter (Metric)



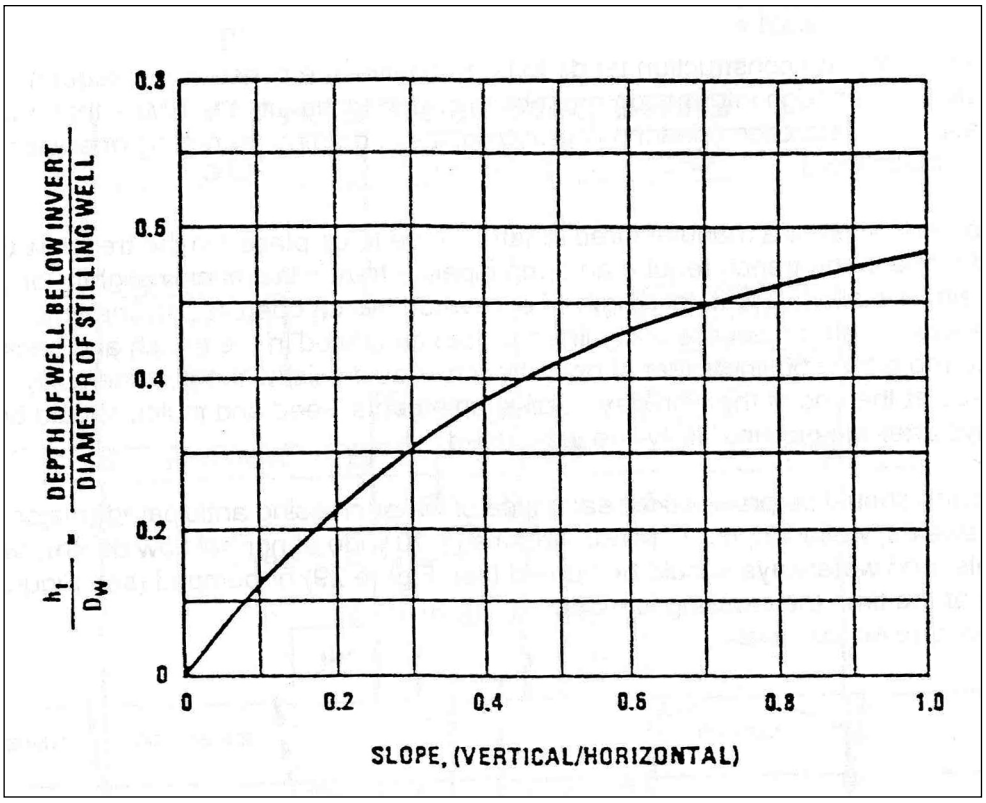
Reference: HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, 2006).

Figure 12.24 Stilling Well Diameter (U.S. Customary)



Reference: HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, 2006).

Figure 12.25 Depth of Stilling Well Above Invert



Reference: HEC-14, *Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, 2006).

M. Diversion Ditch. A diversion ditch is a channel used to convey upland water around work areas. A diversion ditch might outlet to grassed waterways, at-grade stabilization structures, natural watercourses and storm sewers.

Diversion ditches may be used where:

- The flow would damage the work or work area if not diverted.
- Excessive runoff from upstream drainage areas would interfere with the efficient operation of E&SPC or PCSM controls.
- Runoff to construction areas, cut slopes, or other temporarily disturbed sites might accelerate erosion and sedimentation problems, or interfere with the establishment of vegetation.
- Sediment-laden water needs to be directed to a specific BMP such as a Sediment Trap or Sediment Basin.

Diversion ditches are NOT appropriate in the following situations:

- Below sediment producing areas unless measures to prevent sediment accumulation are installed with or before the diversion ditches.
- In place of terraces on land where terracing is needed to control erosion.

Determine the location of the diversion ditch considering the following:

- Construction Sequencing and Phasing. Select a location that minimizes interference with construction activities.
- Right-of-Way. Select a location that will minimize right-of-way requirements and impacts.
- Earthwork. Select a location with the least overall quantity of excavation.
- Soil Type. If possible, select a location where the diversion ditch is constructed in erosion resistant soil.

Diversion ditches are to be designed to convey the peak discharge from the 2-year frequency storm event. In Special Protection watersheds, the 5-year design storm should be used for diversion ditches. To calculate the capacity of the diversion ditch, refer to Chapter 8, *Open Channels*.

Most diversion ditch cross sections are trapezoidal or triangular in shape with rounded corners. For design purposes, a trapezoidal or triangular representation is sufficient. If available channel linings are found to be inadequate for the computed velocity, try a wider channel. The diversion ditch should be designed to have stable side slopes. The channel depth to the top of the berm should be constructed a minimum of 10% greater than the final design depth to account for settling. The berm should have a minimum top width of 1.2 m (4 ft) at the design elevation. A freeboard of not less than 150 mm (6 in) should be provided for the design discharge. A larger freeboard may be required where unstable flow conditions exist. For channels greater than 600 mm (2 ft) total depth, the freeboard should be one-quarter the total depth.

Channel grades may be uniform or variable. Incorporate appropriate measures to dissipate energy and prevent scour at flow transitions (see Section 12.5.K, Outlet Protection: Rock, and Section 12.5.N., Channel Lining BMPs). Bed slopes may not be averaged when determining flow velocity, capacity, or shear stress. Diversion Ditches must be stabilized prior to receiving flow. This may require additional lining. Refer to the Channel Lining BMP for design specifics and options.

Ensure there is a stable outlet for the diversion ditch. A diversion ditch might outlet to grassed waterways, at-grade stabilization structures, natural watercourses and storm sewers.

N. Channel Lining. Channel lining is defined as a temporary or permanent erosion-resistant protection placed on channel beds and side slopes to prevent scour and erosion. Channels are lined in order to convey runoff without damage from erosion or flooding. Channel lining reduces sedimentation downstream and contributes to overall water quality. Constructed channels and ditches should be lined with an erosion resistant lining. The type of lining is contingent upon several factors including runoff velocity, soil type, and channel function. Options for channel liners include grass (vegetated channels), sod, RECPs, rock, concrete, and other methods. See Chapter 8*, Open Channels,* for comparison of channel lining methods.

A protective liner is required if the channel is located in a Special Protection watershed (HQ/EV) or if shear stress exceeds the maximum permissible for the type of soil present. Protective linings for channels and streams can be very expensive; therefore, a special effort should be made to consider the lowest-cost erosion control, including future maintenance, for the particular location. Recommended erosion control measures for ditch and channel protection are discussed in Chapter 8, *Open Channels*.

O. Rock Barrier. A rock barrier is a small, temporary, stone dam installed across a channel or swale. The primary purpose of a rock barrier is to remove sediment originating from flow in a channel before vegetation is fully established.

Rock barriers should be considered for the following situations:

- To limit sedimentation in runoff within constructed channels until a protective lining is installed and operational.
- During a temporary disturbance within the channel.
- Below construction work within an existing stream channel while flow is being diverted past the work area. In such cases, place the filter between the work area and the discharge from the bypass system.
- Where channels are 600 mm (2 ft) or greater in depth.

Rock barriers may be used in combination but are not intended to be installed in place of other E&SPC controls such as:

- Sediment basins and sediment traps.
- Appropriate channel linings. Use of rock barriers in this manner often results in overtopping of the channel during storm events, scouring of the channel bottom below the filter, and/or erosion of the channel side slopes as sediment deposits build up behind the filter.

- Adequate protective lining in sediment basin emergency spillways. If rock barriers are used in this situation, the effective discharge capacity of the spillway can be reduced and will increase the possibility of embankment failure.

Rock barriers should be designed as outlined in this manual and in accordance with Publication 72M, *Roadway Construction Standards*, and Publication 408, *Specifications*. A typical rock barrier design is shown to Standard Drawing RC-73M. The height of the filter should be equal to half the total depth of the channel. The center depression should be equal to one-half of the filter height or 150 mm (6 in). Rock barriers should not be placed within channels to reduce flow velocity or within channels less than 600 mm (2 ft) in depth. Rock barriers should be designed with a 300 mm (1 ft) thick layer (filter) of AASHTO No. 57 aggregate to be placed on the upstream side of the barrier. In Special Protection watersheds, a 150 mm (6 in) layer of compost should be placed on top of the filter stone.

Maintenance of rock barriers during construction should be considered. Remove the rock barrier and accumulated sediment upon final stabilization of the channel. The rock barrier should be inspected weekly and after each rainfall event. If the rock becomes eroded around the sides of the rock barrier, the filter should be repaired immediately. Sediment should be removed when accumulation reaches one-half the height of the filter at the center of depression. The rock barrier should also be replaced if the filter stone becomes clogged.

P. Sediment Trap. A sediment trap is a small, temporary ponding area created by construction of an earthen embankment. Two types of sediment traps are commonly used: an embankment sediment trap, and a riser sediment trap. The embankment sediment trap discharges through a stone filter that is located within the embankment of the trap. An embankment sediment trap can also discharge through a stone filter that is located over a Type M inlet. A riser sediment trap discharges through a riser and outlet pipe that is located within the embankment of the trap.

The primary purpose of sediment traps is to detain sediment-laden runoff from small, disturbed areas and provide adequate settling time to allow the majority of sediment and other particulates to settle out prior to discharge from the construction site. Sediment traps are appropriate for drainage areas 2.0 hectare (5.0 ac) or less. Embankment sediment traps are primarily designed to function as temporary facilities. If, after evaluation, a sediment trap is specified in the area where a permanent stormwater management basin is to be located, a riser sediment trap should be used. Sediment traps may also be used for the dewatering of excavation holes in lieu of a pumped water filter bag where practical. Note this is not an acceptable practice when water levels inside the sediment trap are at (or higher) than the cleanout elevation or top of the sediment storage zone.

The following should be considered when locating a sediment trap:

- Aligned the sediment trap with natural drainage patterns as much as practicable.
- Allow access for removing sediment and disposing of it properly under typical weather conditions.
- Sediment traps should avoid impacts to wetlands and streams unless impracticable.
- A sediment trap should be located such that it is not tributary to other sediment traps, collector ditches, or sediment basins.
- Allow sufficient space to construct and maintain the sediment trap. Temporary construction easements or additional right-of-way may be required. If discharge from a sediment trap cannot reach a channel within the right-of-way or cannot be contained within an implied stormwater easement, a temporary stormwater easement may be necessary.
- Collection channels carrying sediment-laden water should be directed to a sediment trap or basin.

The following is a list of design constraints for a sediment trap. Design methods and procedures are derived from the *Erosion and Sediment Pollution Control Program Manual* (PA DEP, 2000). Refer to Standard Drawing RC-71M for typical details of embankment and riser sediment traps.

- Sediment trap sizing is based on the total contributing drainage area. The maximum permissible contributing drainage area, including the trap itself, is 2.0 hectare (5.0 ac). The contributing drainage area includes all areas that will be tributary to the trap at any given time during the life of the sediment trap.
- The minimum volume of the sediment trap is 140 m^3/ha (2000 ft^3/ac) of contributing drainage area. This is divided into two parts: Dewatering Volume and Sediment Storage Volume. Both volumes are determined by the contributing drainage area.
    - Dewatering Volume: A minimum of 90 m^3/ha (1300 ft^3/ac).

  - Sediment Storage Volume: A minimum of 50 m^3/ha (700 ft^3/ac).
- Maximum exterior and interior embankment side slopes are 1V:2H (2H:1V).
- The maximum spillway side slope is 1V:2H (2H:1V).
- Sediment traps must be able to dewater the dewatering volume completely. In areas where total dewatering is required, adequate filtering of the sediment storage area must be provided.
- Minimum depth is 0.6 m (2 ft). (Minimum 0.3 m (1 ft) for sediment storage and a minimum of 0.3 m (1 ft) for dewatering zone).
- Maximum constructed embankment height is 1.5 m (5 ft) unless the embankment is constructed as a permanent stormwater management basin (refer to riser sediment trap). If the embankment height is greater than 1.5 m (5 ft), safety regulations may apply and should be investigated.
- Minimum embankment top width is 1.5 m (5 ft).
- The flow length is to be measured at the elevation which corresponds to the top of the dewatering zone. The minimum flow length (L) is 3.0 m (10 ft). The minimum flow length to width ratio is 2:1. Equation 12.4 may be used for computing minimum length in traps that are generally rectangular. For traps that have an irregular shape, the best way to determine average width is to divide the surface area at top of dewatering with the maximum length of the trap at the same elevation. Then the ratio can be verified. Where 2:1 ratio is not possible by grading only, a baffle may be used to increase flow length to the required 2:1 ratio.

(Equation 12.4)

$$L_{(min)} = 1.41(SA)^{0.5}$$

Where: $L_{(min)}$ = Minimum Flow Length – m (ft)
SA = Surface Area at the top of the dewatering volume – m^2 (ft^2)

Other general design considerations include:

- The water from the contributing drainage area may enter the sediment trap by any appropriate method. If the runoff is conveyed to the sediment trap by a collector channel or combination of slope pipe and channel, the channel and/or pipe should be sized by the methods indicated in Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10.
- Where stormwater runoff from disturbed areas enters a sediment trap from different directions, consider routing flows from the various areas to a single inlet into the sediment trap.
- The water discharged from a sediment trap must be conveyed to a downstream watercourse by means of an adequately designed channel. When the conveyance channel intersects with the downstream watercourse, appropriate methods should be used to prevent the banks of the receiving water from erosion or scour. An earthen level spreader below a sediment trap is not an acceptable practice.
- During construction, a stake should be placed in the center of the sediment trap. This stake is marked with the height corresponding to the top of the sediment storage volume (clean out elevation). Accumulated sediment should be removed from the sediment trap whenever it reaches the marked elevation on the clean-out stake. Removed sediment must be disposed of per appropriate regulatory requirements.

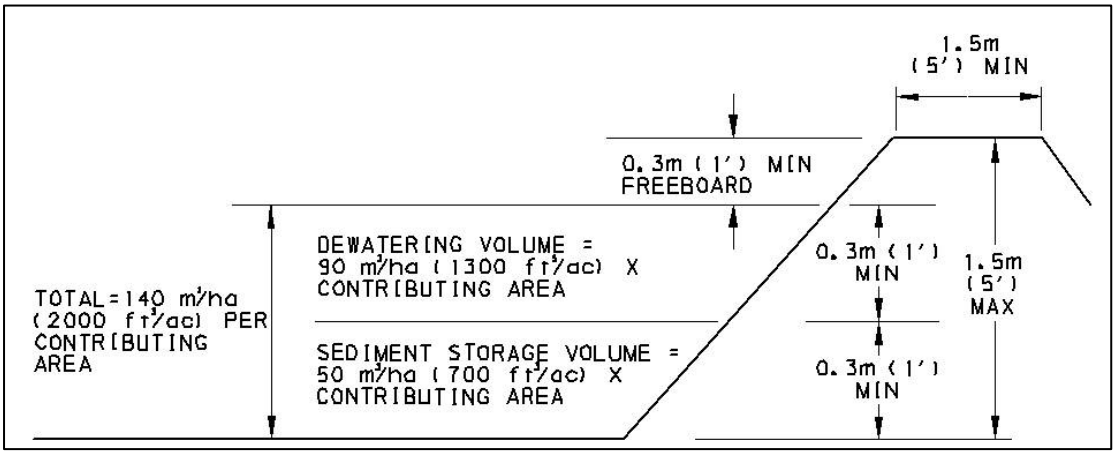
The following design considerations are specific to Embankment Sediment Traps (also refer to Figure 12.26):

- Minimum embankment spillway crest elevation is the elevation at which the required 140 m^3/ha (2000 ft^3/ac) (contributing drainage area) storage capacity is provided.
- Provide a minimum of 300 mm (1 ft) freeboard above the embankment spillway crest elevation (top of dewatering volume). This will be considered the minimum top of embankment elevation.
- Minimum embankment spillway width is 2 times the height of the spillway crest or 1.5 times the total contributing hectares (2 times the total contributing acreage), whichever is greater.
  - Example: an embankment sediment trap has a drainage area of 1.8 hectare (4.5 ac) with a bottom invert of 332.23 m (1090 ft) and spillway crest elevation of 333.45 m (1094 ft). From the spillway height portion of the equation above, the calculated spillway width is 2.44 m (8 ft). However, from drainage area portion of the equation above, the calculated spillway width is 2.7 m (9 ft). The spillway width must be greater of the two results; therefore, the spillway width should be 2.7 m (9 ft).
- The entire spillway is Rock, Class R-3. The interior of the spillway is faced with a 150 mm (6 in) layer of No. 57 coarse aggregate to help facilitate detention and filtration. In Special Protection watersheds, a 150 mm (6 in) layer of compost is placed on top of the No. 57 coarse aggregate. Geotextile, Class 3, Type B

should be securely staked on top of the No. 57 course aggregate, or the compost layer in Special Protection watersheds, up to the top of the sediment storage volume (clean-out elevation). Any excess geotextile should be staked to the bottom of the sediment trap.

- Refer to Standard Drawing RC-71M for typical details of an embankment sediment trap around a Type M inlet. A 19 mm (3/4 in) pressure treated plywood box with 50 mm by 50 mm (2 in x 2 in) pressure treated corner posts is set into the inlet grate offsets. A 600 mm (2 ft) minimum ring of No. 57 Coarse Aggregate is placed around the exterior of the plywood riser box to facilitate detention and filtration. There are ten 25 mm (1 in) perforations per vertical 300 mm (1 ft) of plywood riser. Geotextile, Class 3, Type B should be securely staked on top of the No. 57 course aggregate up to the top of the sediment storage volume (clean-out elevation). Any excess geotextile should be staked to the bottom of the sediment trap.

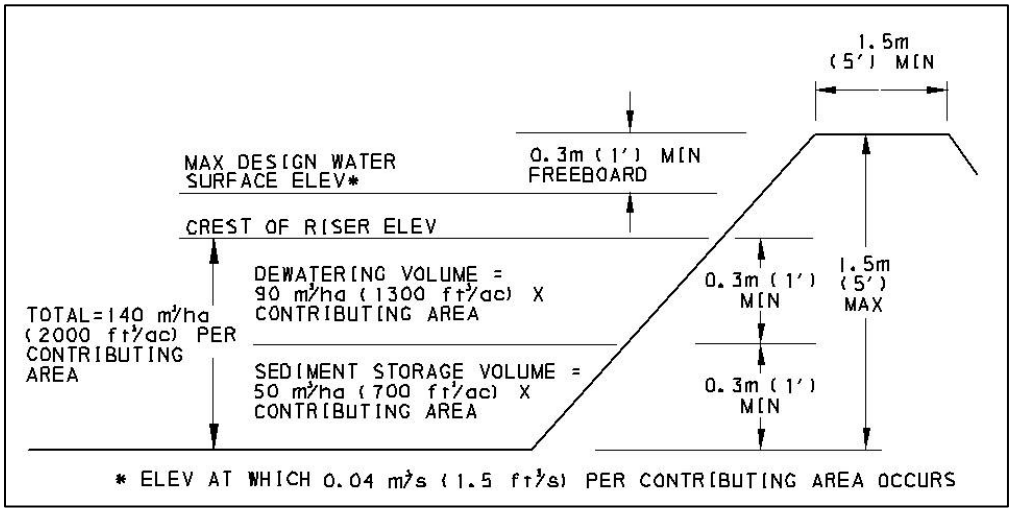
Figure 12.26 Sediment Trap (Embankment)



The following design considerations are specific to Riser Sediment Traps (also refer to Figure 12.27):

- Wherever a sediment trap discharges down a long or steep slope, consideration should be given to using a riser type spillway sediment trap (riser) in conjunction with a temporary slope pipe and adequately designed outlet protection.
- Minimum riser spillway crest elevation is the elevation at which the required 140 m^3/ha (2000 ft^3/ac) (contributing drainage area) storage capacity is provided.
- Perforations in the riser to dewater the trap should be no more than one 25 mm (1 in) perforation per vertical 300 mm (1 ft) of riser with the lowest perforation at the sediment storage volume elevation.
- The design discharge capacity of the riser and outlet pipe should be a minimum of 0.04 m^3/s per ha (1.5 cfs per acre).
- Minimum riser diameter should be 1.25 times outlet pipe diameter and should not be smaller that 300 mm (1 ft) in diameter.
- Provide a minimum of 300 mm (1.0 ft) freeboard above the maximum designed discharge capacity as defined above. This will be considered the minimum top of embankment elevation. It should be noted than an overflow spillway is not required for a riser type sediment trap. If constructed within a permanent stormwater management basin that includes an overflow spillway, the overflow spillway must be a minimum of 150 mm (6 in) above the riser crest elevation.
- Adequately sized outlet protection must be provided at end of the outlet pipe from a riser sediment trap.

Figure 12.27 Sediment Trap (Riser)



Include appropriate references details and notes for sediment traps from Publication 408, *Specifications,* and Publication 72M, *Roadway Construction Standards*. Provide dimensions and elevations to the contractor in a Summary Table (refer to Figure 12.28) on the plans with the sediment trap details.

Figure 12.28 Example Embankment Sediment Trap Summary Table

SEDIMENT TRAP (EMBANKMENT) SUMMARY TABLE

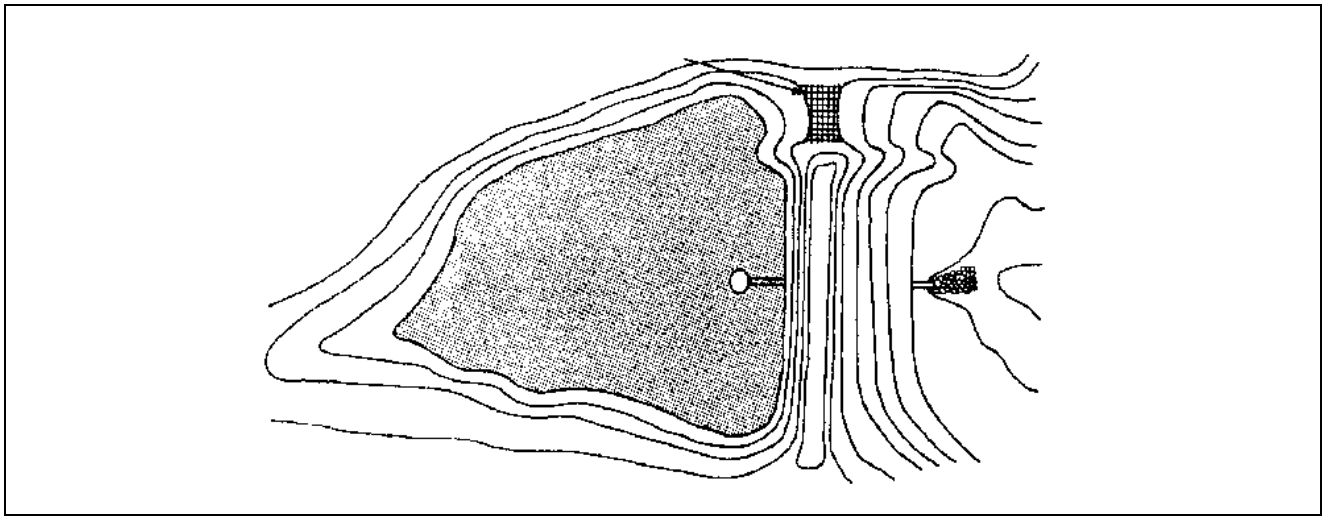
SEDIMENT TRAP NO.	LOCATION	BOTTOM ELEVATION (A)	TOP OF EMBANKMENT ELEVATION (B)	CLEANOUT ELEVATION (C)	TOP OF SPILLWAY ELEVATION (D)	SPILLWAY WIDTH (E)
ST-1	SR XXXX STA XXXX+XX, XX	XXX.X	XXX.X	XXX.X	XXX.X	XXX.X
ST-2	SR XXXX STA XXXX+XX, XX	XXX.X	XXX.X	XXX.X	XXX.X	XXX.X
ST-3	SR XXXX STA XXXX+XX, XX	XXX.X	XXX.X	XXX.X	XXX.X	XXX.X

Inspections of sediment traps should be done on a weekly basis and after each runoff event.

Q. Sediment Basin. A sediment basin is a large, temporary ponding area created by construction of an earthen embankment with an outlet structure (riser) and pipe outlet. A sediment basin discharges through an outlet structure and/or emergency spillway that is typically located within the embankment of the basin (see Figure 12.29).

In general, the primary purpose of a sediment basin is to detain sediment-laden runoff from large disturbed areas and provide adequate settling time to allow the majority of sediment and other particulates to settle out prior to discharge from the construction site. A sediment basin can be used as a perimeter control that could allow the contractor to work within the contributing drainage area without staged construction. Sediment basins are appropriate for drainage areas larger than 2.0 hectare (5.0 ac) and less than 40.0 hectare (100.0 ac). If the contributing drainage area is 2.0 hectare (5.0 ac) and less, a sediment trap or other appropriate BMP should be used. Sediment basins are designed to function as temporary facilities; however, they are often incorporated into the PCSM System upon completion of the project. In the latter case, the sediment basin is converted to a permanent stormwater management basin. Sediment basins may also be used for the dewatering of excavation holes in lieu of a pumped water filter bag where practical. It should be noted that this is not an acceptable practice when water levels inside the sediment basin are at (or higher) than the cleanout elevation or top of the sediment storage zone.

Figure 12.29 Sediment Basin



When locating a sediment basin, the following considerations should be evaluated (Refer to Figure 12.32):

- The contributing drainage area is larger than 2.0 hectare (5.0 ac) and less than 40.0 hectare (100.0 ac). If the contributing drainage area is 2.0 hectare (5.0 ac) or less a sediment trap or another acceptable BMP(s) should be used.
- The natural drainage patterns have been studied and the sediment basin is in a location appropriate to the natural drainage pattern.
- The sediment basin is located to allow access for removing sediment and disposing of it properly under typical weather conditions.
- If a permanent stormwater management basin is planned for the site, consider locating the sediment basin in the same area as this permanent structure to enable the conversion of the sediment basin to a stormwater basin at the end of construction.
- There is sufficient space to construct and maintain the sediment basin. Temporary construction easements or additional right-of-way may be required. If discharge from a sediment basin cannot reach a channel within the right-of-way or cannot be contained within an implied stormwater easement, a temporary stormwater easement may be necessary.
- Collection channels carrying sediment-laden water should be directed to a sediment trap or sediment basin.
- Sediment basins should avoid impacts to wetlands and streams unless impracticable.
- The sediment basin(s) should be located such that it is not tributary to other sediment basins, collector ditches, or sediment traps.

After the above considerations have been weighed and the decision to use a sediment basin has been made, the following is step by step list of procedures that may help the designer to complete the design of a sediment basin:

1. Determine the overall watershed classification that the drainage area to the sediment basin is within. This will help the designer to determine the correct volumes required for design. For example if the basin is in a Special Protection watershed (i.e., HQ or EV), then certain volume reductions are not applicable to the design. The designer can obtain the watershed classification information by contacting PA DEP or the CCD office in which the project site is located. Watershed classifications are also found online at www.pacode.com where the designer can browse for the watershed under PA Code, Title 25, Chapter 93.

2. Determine the type of sediment basin that is most applicable to the site needs. There are three basic types of sediment basins. Refer to Publication 408, *Specifications* and Publication 72M, *Roadway Construction Standards* for construction details and see to Standard Drawing RC-71M.

a. Temporary sediment basin. This type of sediment basin will be installed to control runoff during the construction period and then removed at the completion of earth moving activities once the site has been

stabilized. This type of sediment basin will not be converted to a permanent stormwater management basin. Typically the outlet structure configuration of this type of sediment basin consists of a Corrugated Metal Pipe (CMP) riser that is attached directly to an outlet pipe.

b. Sediment basin located within a permanent stormwater management basin. This type of sediment basin will be converted after earth moving activities cease and the site is stabilized. When locating a sediment basin in this configuration, there are typically two scenarios that fall under this category. This first is that the permanent outlet pipe is installed through the embankment and a temporary CMP riser is attached to it. Under this scenario, the permanent out structure will be installed when the basin is converted after the earth moving activities are completed and the site is stabilized. The second is where the permanent outlet structure and outlet pipe are installed and a temporary riser configuration is attached to the permanent outlet structure. It should be noted that if this scenario is used, it will be beneficial to coordinate permanent outlet structure design (Chapter 14, *Post Construction Stormwater Management*) with the design requirements for erosion control (i.e., top of structure and emergency spillway elevations).

3. After the type of sediment basin has been determined, then the designer may begin the iterative process of designing the basin. Several factors are in play when designing. It may be beneficial to use software that is designed to help the designer in basin modeling or create a spreadsheet for easy data manipulation. This iterative process can be achieved by following the suggested procedures listed below:

Step 1: Assume a "footprint" for the sediment basin and determine the total contributing area and the total disturbed area. Then determine volume requirements relative to the watershed and type of basin that is being designed. The volume requirements are listed below.

Step 2: Begin the proposed contouring process. Before starting this step, note that sediment basins should have a minimum flow length to width ratio of 2:1 at the top of the dewatering volume elevation. Refer to flow length to width ratio requirement below. Draw the proposed contours and verify that the volume requirements calculated in the step above are met by obtaining elevation/area information from each of the proposed contours that were drawn. If the volume requirements are not satisfied, then repeat this process again until they are. Note, the embankment should not exceed 4.5 m (15 ft), measured from the inside invert of the basin to the top of the embankment. Otherwise, it will be subject to additional permitting requirements (i.e., dam) by PA DEP and/or USACE. Also, note that it may be beneficial to make the assumption during this step that the total volume requirement defined below (i.e., top of dewatering) is approximately 1.0 m (3.0 ft) below the top of the embankment.

Step 3: After the proposed contours have been established, verify that the drainage area that was delineated based on the assumed "footprint" for the basin in Step 1 above is still valid. If not, adjust the drainage area and then repeat the previous steps again.

Step 4: Outlet Design Modeling. Now the designer may begin the outlet structure modeling process. Using the design criteria listed on the following pages, perform hydraulic calculations for the sediment basin. When performing this step, drawdown calculations must be performed simultaneously to verify that basin will dewater between 2 and 7 days. Note that there are additional requirements for drawdown when the basin is located within a Special Protection watershed. Also note that the emergency spillway may be included to meet the required minimum discharge rate of 0.14 m^3/s (2.0 cfs) per total contributing hectare (acre). This step may take several iterations to match the outlet structure perforation configuration to the drawdown (dewatering) requirements, which are noted on the following pages.

Step 5: Emergency Spillway Design. Every sediment basin must have an emergency spillway unless the basin is located in an infield of a circular exit in which it is surrounded by impervious pavement. If hydraulic calculations have not already been performed to size the emergency spillway, then they should be completed during this step.

Step 6: Freeboard. Verify that 0.6 m (2.0 ft) of freeboard has been provided above the maximum water surface elevation. The maximum water surface elevation corresponds to the water surface elevation achieved when the minimum discharge rate, defined in Step 4 above, is met. If this has not been achieved, go back to Step 2.

Step 7: Anti-Seep Collar. Now the designer can perform anti-seep collar calculations. Design requirements and procedures for this step are indicated on the following pages.

Step 8: Outlet Protection. Size outlet protection accordingly. Refer to Section 12.5.K.

In general, design criteria listed below is derived from the *Erosion and Sediment Pollution Control Program Manual* (PA DEP, 2000).

The following design guidelines are a simplified approach to sediment basins. Detailed hydraulic calculations are not provided and hydraulic routing is not performed in this guidance.

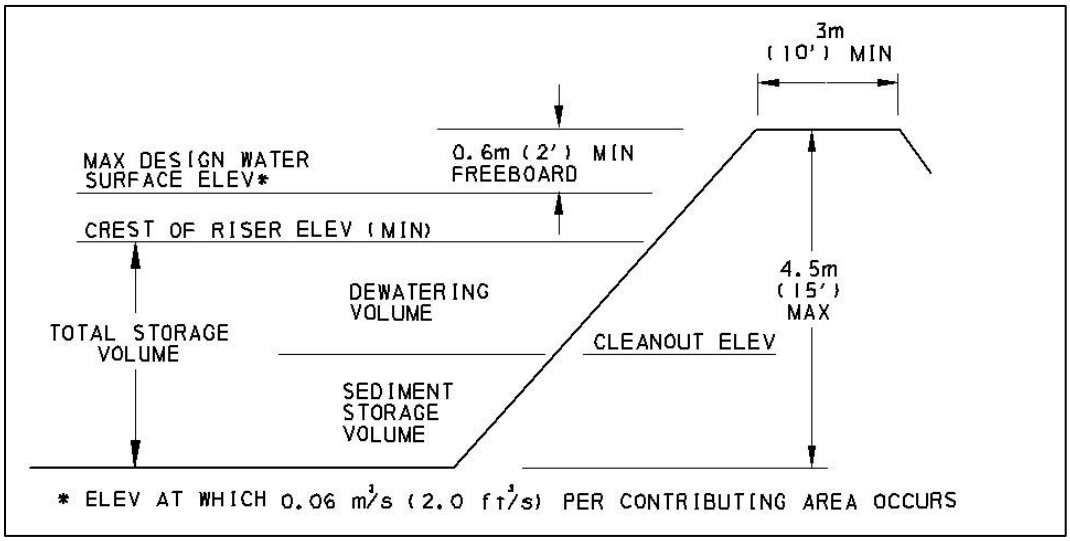
General Requirements:

- The horizontal components of the sediment basin side slope for the embankment must add to a minimum of 5.0 (i.e., interior side slope is 1V:2H (2H:1V) and exterior side slope is 1V:3H (3H:1V)). Note, if sediment basin(s) is located in an area adjacent to traffic (i.e., ramp infield, or along side roadway), side slopes may not be steeper than 1V:3H (3H:1V).
- Maximum embankment height is 4.5 m (15.0 ft), measured from the inside invert of the basin to the top of the embankment. If the embankment is greater than this height, additional permits from PA DEP and/or USACE will be required.
- The minimum top width of a sediment basin is 3.0 m (10.0 ft).

Volume Requirements:

- Sediment basin sizing is based on the total contributing drainage area and the disturbed area within total contributing area. The maximum permissible contributing drainage area, including the basin itself, is 40.0 hectare (100.0 ac). The contributing drainage area includes all areas that will be tributary to the basin at any given time during the life of the sediment basin (Refer to Figure 12.30).

Figure 12.30 Sediment Basin Volume



- The minimum sediment storage volume of the sediment basin is 70 m^3 (1000 ft^3) per disturbed hectare (acre). The corresponding elevation of this volume requirement is the cleanout elevation of the sediment basin.
- The dewatering volume of the sediment basin is 350 m^3 (5000 ft^3) per total contributing hectare (acre). The following reductions can be utilized, however the minimum required dewatering volume is 250 m^3 (3600 ft^3) per total contributing hectare (acre). If the sediment basin is located in a Special Protection watershed, no reductions are permitted. In addition, sediment basins located within Special Protection watersheds must meet the requirements listed in items 1 or 2 below (not both). The corresponding elevation of this volume requirement is the minimum crest elevation of the principal spillway and is also known as the top of dewatering volume.
    - A reduction of 50 m^3 (750 ft^3) per total contributing hectare (acre) may be utilized for basins with principal spillways that dewater from the top 150 mm (6 in) only of the water level at any given time within the dewatering zone (i.e. floating skimmer type of outlet structure).

- A reduction of 50 m^3 (750 ft^3) per total contributing hectare (acre) may be utilized for basins with permanent pools greater the 0.5 m (1.5 ft) in depth. Note that the sediment storage volume may be used to achieve this reduction allowance.
- A reduction of 25 m^3 (350 ft^3) per total contributing hectare (acre) may be utilized for basins with length to width ratios of 4:1 or greater. See length to width ratio equations listed below.
- A reduction of 25 m^3 (350 ft^3) per total contributing hectare (acre) may be utilized for basins with dewatering times between 4 and 7 days. See dewatering criteria listed below.

- The total required storage volume is defined as the required sediment storage volume plus the required dewatering volume.

DESIGN EXAMPLE 1 – Special Protection watershed (i.e., HQ or EV designation)
Total Drainage Area = 7.28 ha (18 ac)
Disturbed Area = 2.43 ha (6 ac)
Required Sediment Basin Volume: no reductions in dewatering volume are allowed.

Dewatering Volume:	350 m^3/ha x 7.28 ha = 2548 m^3
	(5,000 ft^3/ac x 18 ac) = (90,000 ft^3)
Sediment Storage Volume:	70m^3/ha x 2.43 ha = 170 m^3
	(1,000 ft^3/ac x 6 acres = 6,000 ft^3)
Total Required Storage Volume =	2,718 m^3 (96,000 ft^3)

DESIGN EXAMPLE 2 – Non-Special Protection watershed (i.e., TSF, WWF, CWF designations)
Total Drainage Area = 7.28 ha (18 ac)
Disturbed Area = 2.43 ha (6 ac)
0.5 m (1.5 ft) permanent pool provided
4:1 flow length to width ratio provided
4 to 7 day drawn down (dewatering) period provided
Required Sediment Basin Volume: reductions totaling 100 m^3 (1400 ft^3) per total contributing hectare (acre) in dewatering volume from above are allowed.

Dewatering Volume –	250 m^3/ha x 7.28 ha = 1820 m^3
	(3,500 ft^3/ac x 18 acres) = (63,000 ft^3)
Sediment Storage Volume –	70m^3/ha x 2.43 ha = 170 m^3
	(1,000 ft^3/ac x 6 ac = 6,000 ft^3)
Total Required Storage Volume =	1,990 m^3 (69,000 ft^3)

- A minimum of 0.6 m (2.0 ft) of freeboard is required for a sediment basin. The freeboard is measured above the maximum water surface elevation. This elevation will be considered the minimum top of embankment elevation. See outlet design requirements listed below for criteria for the maximum water surface elevation.

Flow Length to Ratio:

- The flow length is to be measured at the elevation of the top of the dewatering zone. The minimum flow length to width ratio is 2:1. Equation 12.5 may be used for computing minimum length in basins that are generally rectangular. For basins that have an irregular shape, the best way to determine average width is to divide the surface area at top of dewatering with the maximum length of the trap at the same elevation. Then the ratio can be verified. Where 2:1 ratio is not possible by grading only, a baffle may be used to increase flow length to the required 2:1 ratio.

(Equation 12.5)

$$L_{(min)} = 1.41(SA)^{0.5}$$

Where: $L_{(min)}$ = Minimum Flow Length – m (ft)
SA = Surface Area at the top of the dewatering volume – m^2 (ft^2)

- When a 4:1 length to width ratio is desired to obtain the reduction allowance listed above (when not located in a Special Protection watershed), Equation 12.6 may be used for computing the flow length in basins that are generally rectangular. For basins that have an irregular shape, the best way to determine average width is to divide the surface area at top of dewatering with the maximum length of the trap at the same elevation. Then the ratio can be verified. Where the 4:1 ratio is not possible by grading only, a baffle may be used to increase flow length to achieve this goal.

(Equation 12.6)

$$L_{(min)} = 2.0(SA)^{0.5}$$

Where: $L_{(min)}$ = Minimum Flow Length – m (ft)
SA = Surface Area at the top of the dewatering volume – m^2 (ft^2)

Outlet Design Requirements (riser type):

- Minimum riser diameter should be 1.25 times outlet pipe diameter. The minimum riser diameter in general is 380 mm (15 in) which corresponds to a minimum outlet pipe diameter of 300 mm (12 in).
- Perforations in the riser structure should be 25 mm (1 in) in diameter. The first row of perforations should be at the sediment storage elevation. A sufficient number of holes should be provided in the riser within the dewatering volume to provide the required 2 to 7-day drawdown time range (4 to 7 days in Special Protection watersheds or if the credit is utilized to reduce the dewatering volume as described above). The holes should be equally distributed around the riser structure in rows that are spaced 300 mm (12 in) vertically. The orifice equation, Equation 12.7 listed below, should be used to determine flows necessary to calculate the dewatering time.

Orifice Flow (HEC-22, *Urban Drainage Design Manual* (FHWA, 2001b)): *(Equation 12.7)*

$$Q_{or} = C_{or}A(2gH)^{0.5}$$

Where: Q_{or} = Flow, m^3/s (cfs)
C_{or} = 0.6 (square-edged uniform orifice entrance conditions)
0.4 (ragged-edged acetylene torched orifices in a corrugated metal pipe riser – HEC-22)
A = Cross sectional area of an orifice, m^2 (ft^2)
g = Acceleration due to gravity, 9.8 m/sec^2 (32.2 ft/sec^2)
H = Depth of water above the center of the orifice, m (ft)

- The dewatering time is defined as the time required to drawdown the basin from the crest of the principal spillway (top of dewatering volume) to cleanout elevation (top of the sediment storage volume). For Special Protection watersheds, it is calculated by multiplying the flow rate times the volume within the dewatering zone. Since the flow rate varies with head, this calculation is performed using a maximum of 300 mm (12 in) steps vertically. The time associated with each step is calculated and then totaled when the final step is at the cleanout elevation. For all other watersheds, the following rule of thumb can be used. Provide 645 mm^2 (1 in^2) of opening per acre of drainage area with all perforations 25 mm (1 in) in diameter and equally spaced vertically along the riser. The lowest row of perforations is at the sediment storage zone elevation (i.e., the cleanout elevation). The number of perforations needed may be determined by dividing the total number of acres tributary to the basin by 0.785.
- When the disturbed drainage area to the sediment basins is composed of highly erodible soils, it is suggested that the dewatering time be pushed out towards the 7-day mark.
- The elevation at which the total storage volume is provided is the minimum elevation for the crest principal spillway (riser structure).

- The riser pipe diameter (or riser structure size) is determined as a function of the head acting on the top of the riser to meet the minimum flow requirement of 0.14 m^3/s (2.0 cfs) per total contributing hectare (acre). The head is calculated as the difference between the water surface elevation and the riser crest elevation.
- Analyze the riser/outlet structure for three possible limiting flow types: weir flow, orifice flow, and pipe flow. The discharge capacity is the smallest of these three flow rates. Discharges through riser perforations may be ignored during this computation.
- The top of a riser structure functions as either a weir or an orifice depending on the stage. Therefore the riser structure must be evaluated using both the appropriate weir flow equation, Equation 12.8 or Equation 12.9, and the orifice flow equation, Equation 12.10.
- The pipe flow equation is listed below. Refer to design requirements listed below for emergency spillway design.
- Provide a minimum of 0.6 m (2.0 ft) freeboard above the maximum designed discharge capacity as defined above. This will be considered the minimum top of embankment elevation. Note, if the emergency spillway is being used to provide part of the 0.14 m^3/s (2.0 cfs) per total contributing hectare (acre) discharge, the freeboard must be provided above the design flow elevation in the emergency spillway.

Weir Flow (sharp crested with no end contractions– used for CMP or other type of pipe riser):

(Equation 12.8)

$$Q_w = C_w L H^{1.5}$$

Where:
- Q_w = Weir flow, m^3/s (cfs)
- C_w = 1.71 in Metric units (3.10 in U.S. Customary units)
- L = Circumference of riser pipe, m (ft)
- H = Depth of flow above the riser crest, m (ft)

Weir Flow (broad crested – used for concrete outlet structure (i.e. Type M inlet top)):

(Equation 12.9)

$$Q_w = C_w L H^{1.5}$$

Where:
- Q_w = Weir flow, m^3/s (cfs)
- C_w = 1.66 in Metric units (3.00 in U.S. Customary units)
- L = Perimeter of inside edges of concrete riser structure, m (ft)
- H = Depth of flow above the riser crest, m (ft)

Orifice Flow:

(Equation 12.10)

$$Q_{or} = C_{or} A (2gH)^{0.5}$$

Where:
- Q_{or} = Orifice flow, m^3/s (cfs)
- C_{or} = 0.6
- A = Cross sectional area of the riser structure, m^2 (ft^2)
- g = Acceleration due to gravity, 9.8 m/sec^2 (32.2 ft/sec^2)
- H = Depth of water above the riser crest, m (ft)

Pipe Flow: *(Equation 12.11)*

$$Q_p = A[2gh/(1 + K_m + K_p L)]^{0.5}$$

Pipe Friction: (*Equation 12.12*)

Metric:	**U.S. Customary:**
$K_p = 1245792n^2/d_i^{4/3}$	$K_p = 5087n^2/d_i^{4/3}$

Where:
- Q_p = Pipe flow, m^3/s (cfs)
- A = Cross sectional area of the outlet pipe, m^2 (ft^2)
- g = Acceleration due to gravity, 9.81 m/sec^2 (32.2 ft/s^2)
- h = Head above the centerline of the outlet end of the barrel, m (ft)
- K_m = Coefficient of minor losses, can be assumed to be 1.0 for most principal spillway systems
- K_p = Pipe friction coefficient (Equation 12.12)
- n = Manning's coefficient of roughness
- d_i = Inside diameter of the outlet pipe, mm (in)
- L = Outlet pipe length, m (ft)

- A trash rack and an anti-vortex device should be attached to the top of the riser pipe (or structure) to prevent floating debris from blocking the opening in the top of the structure. An anti-vortex device should be used to maintain the design inflow of the riser. Refer to Standard Drawing RC-72M.

Emergency spillway requirements:

- The preferred location of the emergency spillway is in undisturbed ground, if this is not possible and it must be placed in the embankment, provide rock lining or other stable permanent lining which extends to the receiving waterway, channel or other non-erosive outlet.
- The elevation of the emergency spillway crest should be a minimum of 150 mm (6 in) above the top of the dewatering zone (crest of principal spillway).
- If a emergency spillway is not provided (note, this is rare and an emergency spillway should be provided for all sediment basins unless the basin is located within the infield of an exit in which it is surrounded by pavement), the riser structure must be sized to convey the entire minimum flow requirement of 0.14 m^3/s (2.0 cfs) per total contributing hectare (acre). Note that the freeboard is still calculated above this design maximum water surface elevation.
- Provide a minimum of 0.6 m (2.0 ft) freeboard above the maximum designed discharge capacity as defined above. This will be considered the minimum top of embankment elevation. Note: if the emergency spillway is being used to provide part of the 0.14 m^3/s (2.0 cfs) per total contributing hectare (acre) discharge, the freeboard must be provided above the design flow elevation in the emergency spillway.
- Tables provided in HEC-22, *Urban Drainage Design Manual* (FHWA, 2001b), Section 8.4.4.4 or *Hydraulic Design of Spillways* (USACE, 1990) are suggested for determining emergency spillway capacity, however, the Equation 12.14 below is also an acceptable method.

Weir Flow (broad crested): *(Equation 12.13)*

$$Q_w = C_wLH^{1.5}$$

Where:
- Q_w = Weir flow, m^3/s (cfs)
- C_w = 1.55 in Metric units (2.80 in U.S. Customary units)
- L = Bottom width of the spillway at the crest, m (ft)
- H = Depth of flow above the spillway crest, m (ft)

- The maximum spillway side slope is 1V:2H (2H:1V).
- Refer to Standard Drawing RC-71M for details.

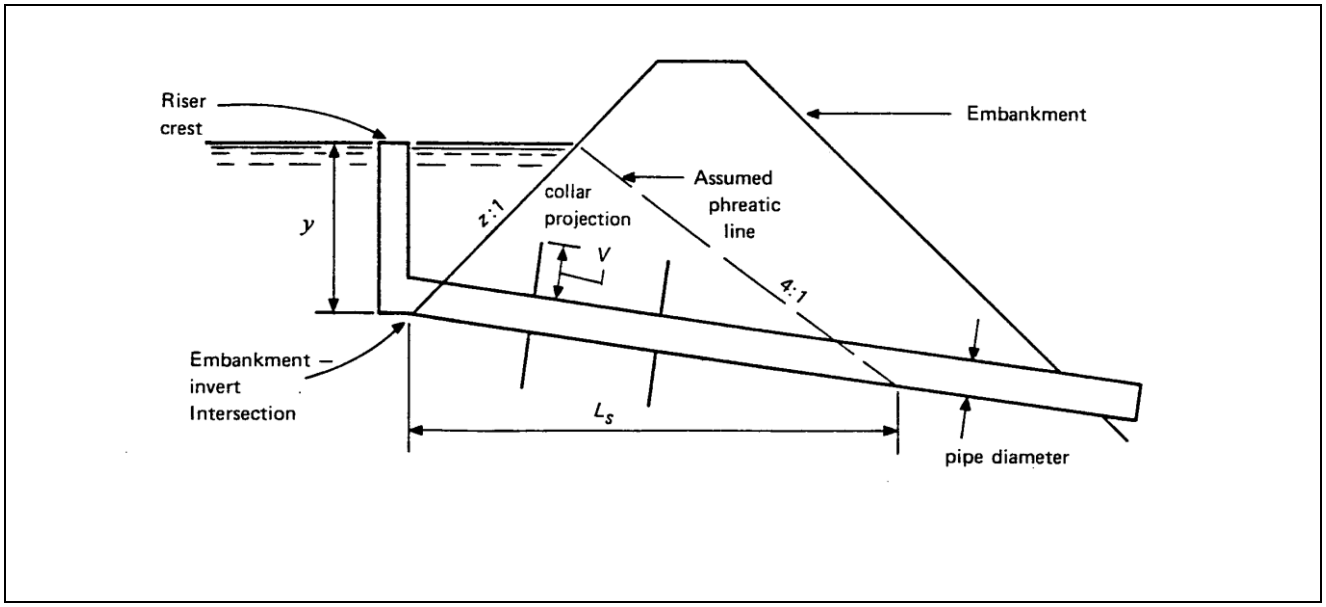
Anti-seep collar requirements:

- Anti-seep collars are used when a sediment basin has an outlet pipe associated with the principal outlet structure.
- Anti-seep collars are placed in the embankment of sediment basins to protect the integrity of the structure at the outlet point. At the outlet point of the basin, the water not only exits through the outlet pipe, but also

attempts to percolate through the embankment along the exterior surface of the pipe, thus compromising the stability of the berm. Anti-seep collars should be located below the phreatic surface in the embankment and should be spaced evenly along the pipe. Note, bedding gravel should not be placed under the outlet pipe from a sediment basin or permanent stormwater management basin.

- The phreatic surface is defined by a line drawn at a 1H:4V (4V:1H) slope from the top of the dewatering volume on the inside of the sediment basin. This line is used to estimate the embankment saturation zone. Refer to Figure 12.31.

Figure 12.31 Anti-Seep Collar Design



- Determine L_s: The intersection point is the length of the pipe in the saturation zone, L_s by the following procedure. This procedure will provide a 15% increase in the seepage length and is appropriate for the design with permanent stormwater basins.

  NOTE:
  - This procedure needs to be done in U.S. Customary Units and then converted to Metric after the collar size and spacing has been determined.
  - The following chart does provide an increase in the seepage length by 15% verses the one illustrated in the *Erosion and Sediment Pollution Control Program Manual* (PA DEP, 2000) which only increases the seepage length by 10%. A 10% increase in seepage length is only acceptable for sediment basins that are temporary (the method indicated in the *Erosion and Sediment Pollution Control Program Manual* (PA DEP, 2000) is acceptable for this).

(Equation 12.14)

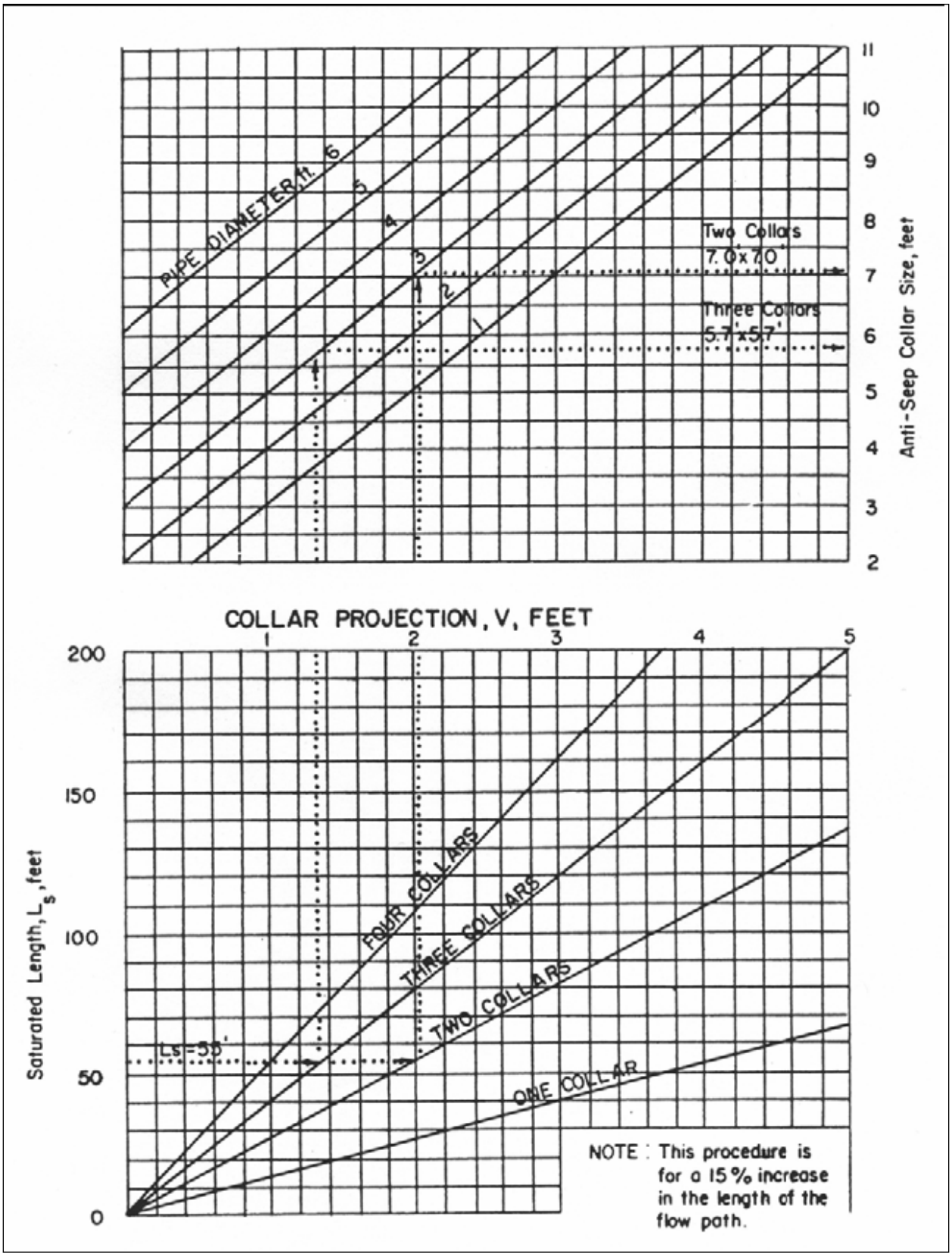
$$L_S = y(z+4)\left[1+\frac{\text{pipe slope (ft/ft)}}{0.25-\text{pipe slope}}\right]$$

Where: L_S = Length of pipe in saturated zone, (ft)
y = Vertical distance from upstream invert of principal spillway riser to top of dewatering volume, (ft)
z = Horizontal component of upstream embankment slope, (ft)

Example: y = (8 ft)
inside slope = (2.5H:1V)
embankment pipe slope = (0.1 ft/ft)
L_S = (87 ft)

1. Determine the length of outlet pipe in the saturated zone (L_s) using the Equation 12.14. Refer to Figure 12.31 for variables.
2. Using Figure 12.32, enter L_s value on the left side of the graph.
3. Draw a horizontal line to the point where it intersects with the line that matches the desired number of collars.
4. Draw a vertical line from that point to the line in the upper chart that corresponds to the diameter of the outlet pipe.
5. From that point, draw a horizontal line to the right to read V, the required size of the collar(s).
6. If this diameter does not fit in the embankment, the number of collars should be increased.
7. The maximum spacing between collars should be 14 × V.
8. They should not be located closer than 600 mm (24 in) to a pipe joint.

Figure 12.32 Graphical Determination of Anti-Seep Collar Size



Reference: USDA NRCS.

The following list of other design considerations should also be evaluated:

- The water from the contributing drainage area may enter the sediment basin by any appropriate method. If the runoff is conveyed to the sediment basin by a collector channel or combination of slope pipe and channel, the channel and/or pipe should be sized by the methods indicated in Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10.
- Where stormwater runoff from disturbed areas enters a sediment basin from different directions, consider routing flows from the various areas to a single inlet into the sediment basin.
- Adequately sized outlet protection must be provided at end of the outlet pipe from a riser sediment basin.
- The water discharged from a sediment basin must be conveyed to a downstream watercourse by means of an adequately designed channel. When the conveyance channel intersects with the downstream watercourse, appropriate methods should be used to prevent the banks of the receiving water from erosion or scour. An earthen level spreader below a sediment basin is not an acceptable practice.
- Wherever a sediment basin discharges down a long or steep slope, consideration should be given to using a temporary/permanent slope pipe and adequately designed outlet protection.
- During construction a stake should be placed in the center of the sediment basin. This stake is marked with the height corresponding to the top of the sediment storage volume (clean out elevation). Accumulated sediment should be removed from the sediment basin whenever it reaches the marked elevation on the clean-out stake. Removed sediment must be disposed of per appropriate regulatory requirements.
- Bedding gravel should not be placed under the outlet pipe from a sediment basin or permanent stormwater management basin.

Include appropriate references, details and notes from Publication 408, *Specifications* and Publication 72M, *Roadway Construction Standards*. Provide dimensions and elevations to the contractor in a summary table on the plans with the sediment basin details. Inspections of sediment basins should be done on a weekly basis and after each runoff event. If a sediment basin is to be converted to a PCSM BMP (stormwater management basin); refer to the guidance applicable in Chapter 14, *Post-Construction Stormwater Management*.

12.6 IN-CHANNEL E&SPC BMPS

BMPs identified in this section are specifically for use when the construction is to be performed in a channel with flowing water. These BMPs may isolate the work area or divert the flow from the work area.

A. Bypass Channel with Non-Erosive Lining (For Channel Work). A bypass channel can be used as a temporary stream diversion to divert flow in the natural stream channel with a base flow around the work area and back into the natural channel at a downstream location. (See Figure 12.33) If the work area is in a Special Protection or HQ/EV watershed, the type of diversion should be selected on a case by case basis by consulting with the regulatory authority.

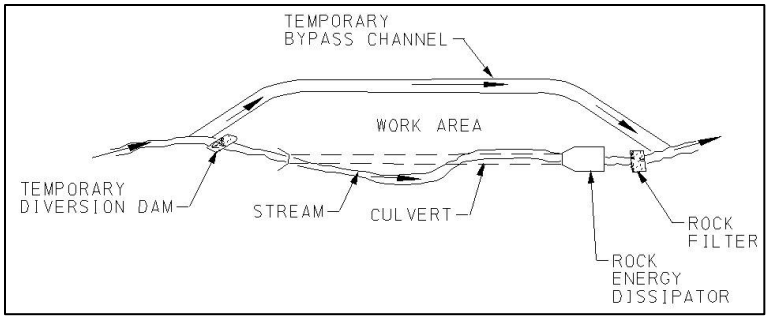
Bypass Channels may be used:

- When the contributing drainage area is any size.
- To divert small streams.
- Where there is a flat flood plain.
- Whenever it is necessary to divert flow for more than 2 days.

When restoring the channel to the natural drainage course, attention must be paid to reconstructing the walls of the channel to ensure the wall integrity. It should be noted that permit requirements should be checked prior to channel restoration.

Bypass channels are to be designed to convey the base flow around the work area for short duration projects or a 1-year storm event for longer duration projects (e.g., box culvert construction). Identify an area upstream for the bypass channel to begin and a downstream area for the flow to be reintroduced to the stream path. Ideal locations would be parallel to the existing stream path.

Figure 12.33 Example Bypass Channel Used For Culvert Installation



Design the channel using the Manning's equation as discussed in Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10. In addition, the channel should be lined as suggested in Section 12.5.N.

The location of the bypass channel should allow sufficient room to do the necessary channel work as well as install structures in the vicinities of the tie-ins (e.g., wingwalls). It is also important that the channel not be pushed against a steep slope, especially in erodible soils with low shear strength. Bypass channels should be inspected daily.

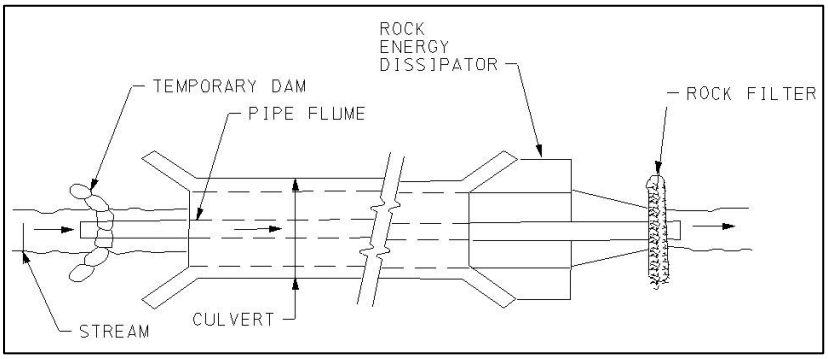
B. Temporary Stream Diversion: Flume through a Work Area. This section describes a method of temporary diversion by use of a flume to convey a stream through the work area. When in-channel work is necessary on the outside edges of a stream, a flume may be constructed to convey water through the work area. If the work area is in a Special Protection or HQ/EV watershed the type of diversion should be selected on a case-by-case basis by consulting with the regulatory authority. Figure 12.34 shows a typical flume through work area diversion.

Flume through a Work Area may be used when:

- The stream width is less than 3 m (10 ft).
- Whenever it is necessary to divert flow for more than 2 days.

Flumes are to be designed to convey the base flow around the work area for short-duration projects or a 1-year storm event for longer duration projects (e.g., box culvert construction). Using the flow calculated, size a culvert to pass the flow using the methods in Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10. Refer to *Erosion and Sediment Pollution Control Manual* (PA DEP, 2000) for rock filter design guidance.

Figure 12.34 Flume Through Work Area



Maintenance of flumes should also be considered. The flume should be inspected daily and after each rainfall event. Parts of the flume that may be damaged during a rain event should be immediately replaced.

C. Temporary Stream Diversion: Pump around In-Channel Work Area. Temporary stream diversion using a pump to divert flow from the natural stream channel and discharging it at a downstream location is a method to enable an in channel work area. When in-channel work is necessary, a pump bypass, in combination with cofferdams, can be used to move the base stream flow around the work area. If the work area is in a Special Protection or HQ/EV

Watershed, the type of diversion should be selected on a case-by-case basis by consulting with the regulatory authority. Figure 12.35 shows a typical pump around stream diversion.

Pump around diversion may be the least intrusive diversion and should be considered above other methods when the following conditions listed below apply:

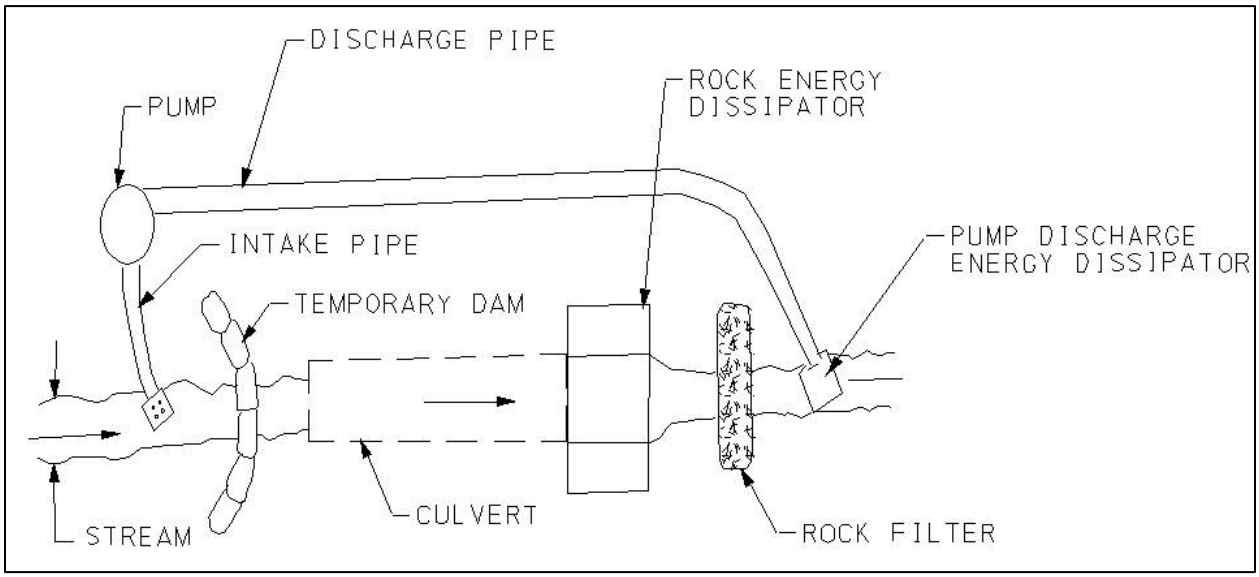
- The stream width is less than 3 m (10 ft).
- The stream will be diverted for less than 2 days.

There is no design method to be considered; however, the work should be done in dry weather. The guidelines below should be followed:

Step 1: Size the pump for base flow.
Step 2: Select a location upstream of the work area to begin the diversion.
Step 3: Select a location downstream of the work area to reintroduce the flow.

Pump-around systems should be monitored throughout the work period.

Figure 12.35 Pump Around In-Channel Work Area



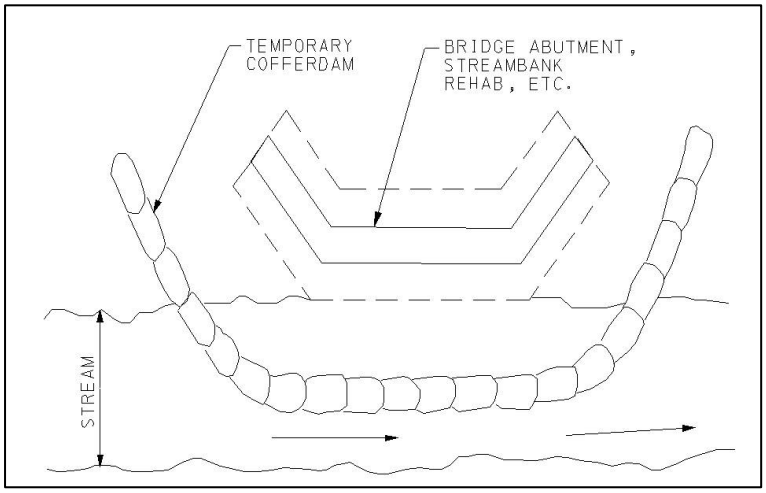
D. In-Stream Cofferdam. An in-stream cofferdam (as shown in Figure 12.36) is a temporary stream diversion device constructed in a channel to keep flow away from an in-channel work area. This will be required when in-channel work is necessary on one side of a stream at a time. If the work area is in a Special Protection or HQ/EV Watershed the type of diversion should be selected on a case-by-case basis by consulting with the regulatory authority.

In-stream temporary diversion device may be used when:

- The stream width is greater than 3.0 m (10 ft).
- The contributing drainage area is greater than 260 ha (640 ac).
- Where concrete barriers are used, normal flow depth should not exceed ½ the height of the barrier.

The work area may contain stagnant water. Evaluate the work area to see if work can be accomplished in an area of dead water or if the areas must be dewatered.

Figure 12.36 In-Stream Temporary Diversion Device



The 2-year design storm should be used to determine the depth of the flow using the methods in Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10. The height of the temporary stream diversion device should be no less than 0.2 m (0.6 ft) above the water elevation. Backwater conditions should also be evaluated when considering an in-stream cofferdam diversion. The area blocked off by the temporary stream diversion device should be minimized and it may not exceed 60% of the width of the channel at one time. The temporary stream diversion device should be constructed in low flow conditions.

12.7 GLOSSARY

Abrasion - Removal of stream bank material due to entrained sediment, ice or debris rubbing against the bank.

Act 167 Plan - Stormwater management plans partially funded by PA DEP and prepared by the county for PA DEP-designated watersheds. Act 167 is the Pennsylvania Stormwater Management Act, 32 P.S. §§ 680.1 et seq., which authorizes PA DEP to provide funding for municipal implementation of stormwater programs implementing watershed-based plans conducted by counties.

Angle of Repose - The maximum angle, as measured from the horizontal, at which granular particles can stand.

Aprons - Protective material laid on a stream bed to prevent scour commonly caused by some drainage facility. More specifically, a floor lining of such things as concrete, timber and riprap, to protect a surface from erosion, e.g., the pavement below chutes, spillways, at the toes of dams or at the outlet of culverts. Material placed on the banks is commonly termed a blanket.

Articulating Concrete Block Revetment System - A matrix of individual concrete blocks placed together to form an erosion resistant overlay that meets specific static and hydraulic performance characteristics.

Berm - A narrow shelf or ledge; also a form of dike. A more detailed description might be: 1) the space left between the upper edge of a cut and the toe of an embankment; 2) a horizontal strip or shelf built into an embankment to break the continuity of an otherwise long slope.

Best Management Practice - Activities, facilities, measures, or procedures used to minimize accelerated erosion and sediment to protect, maintain, reclaim and restore the quality of waters and the existing and designated uses of waters within this Commonwealth. (Title 25 PA Code Chapter 102).

Bypass Flow - Flow that bypasses an inlet on grade and is carried in the street or channel to the next inlet downstream.

Channel Lining - The material applied to the bottom and/or sides of a natural or constructed channel. Material may be such things as concrete, sod, grass, rock or any of several other types of commercial linings.

Channel - The term "channel" has been defined numerous ways: 1) the bed and banks that confine the flow of surface water in a natural stream or artificial channel; 2) the course where a stream of water runs or the closed course or conduit through which water runs, e.g., a pipe; 3) an open conduit either naturally or artificially created that periodically or continuously contains moving water or that forms a connecting link between two bodies of water. River, creek, run, branch, anabranch [arroyo, draw, wash] and tributary are some of the terms used to describe natural channels. Natural channels many be single or braided.

Check Dam – A relatively low dam or weir across a channel for the diversion of irrigation flows from a small channel, canal, ditch or lateral. A check dam can also be a low structure, dam or weir, across a channel for such things as the control of water stage or velocity of the control of channel bank erosion and channel bed scour from such things as headcutting.

Concentrated Flow - Flowing water that has been accumulated into a single fairly narrow stream.

Contributing Drainage Area - Total of both disturbed and undisturbed area contributing, usually upslope, runoff to a BMP. Since drainage areas may change during grading operations, roadway construction, installation of sewer lines, and construction of buildings and parking lots, the maximum contributing area is not necessarily the pre- or post-construction drainage area.

Conveyance - A measure of the ability of a stream, channel or conduit to convey water. A comparative measure of the water-carrying capacity of a channel; that portion of the Manning discharge formula that accounts for the physical elements of the channel. Conveyance is expressed as: $(1/n)AR^{2/3}$ where n is Manning's n, A is the cross section area of flow and R is the hydraulic radius. See Manning's Equation.

Countermeasure - A measure intended to prevent, delay or reduce the severity of erosion problems.

County Conservation District - A conservation district, as defined in section 3(c) of the Conservation District Law (3 P.S. § 851(c)), which has the authority under a delegation agreement executed with PA DEP to administer and enforce all or a portion of the erosion and sediment control program in this Commonwealth.

Crest - The maximum elevation of a flood at a specific location. Other definitions are 1) the top of a dam, dike, spillway or weir; 2) the overflow portion of a road or embankment; 3) the summit of a wave; and 4) the peak of a flood.

Detention - The process of temporarily collecting and holding back stormwater for later release to receiving waters.

Dewatering Zone - The zone within a sediment basin where stormwater runoff is held and released in a controlled manner.

Discharge - Volume of water passing a point during a given time. PA DEP defines it in PA Code, Title 25, Chapter 92 as "An addition of any pollutant to surface waters of this Commonwealth from a point source, including: (i) Additions of pollutants from surface runoff and stormwater which is collected or channelized; (ii) Discharges through pipes, sewers or other conveyances which do not lead to a treatment works; (iii) Discharges through pipes, sewers or other conveyances."

Diversion - A facility, including a channel, terrace or dike constructed up-slope of an earth disturbance activity for the purpose of diverting runoff away from an existing or proposed disturbed area.

Drag - The retarding force acting on a body moving through a fluid parallel and opposite to the direction of motion.

Earth Disturbance Activity - A construction or other human activity which disturbs the surface of the land, including, but not limited to, clearing and grubbing, grading, excavations, embankments, land development, agricultural plowing or tilling, timber harvesting activities, road maintenance activities, mineral extraction, and the moving, depositing, stockpiling, or storing of soil, rock or earth materials.

Edge - The revetment limits at top and bottom of slope or bank, and at leading and trailing limits or flanks of the revetment construction.

Embankment - A raised, typically earth, structure used to impound water and/or to carry a roadway.

Emergency Spillway - A rock or vegetated earth waterway around a dam or in a sediment basin (or a permanent stormwater detention facility), built with its crest above the principal spillway. Used to supplement the principal spillway in conveying extreme amounts of runoff safely past the dam to minimize damage and flood hazards.

Erosion - The natural process by which the surface of the land is worn away by water, wind or chemical action.

Erosion and Sediment Pollution Control (E&SPC) - Application of best management practices to stabilize areas disturbed by grading operations, reduce loss of soil due to water or wind action, and prevent water pollution.

Erosion and Sediment Control Plan - A site-specific plan identifying best management practices to minimize accelerated erosion and sedimentation.

Fill Slope - Side or end slope of an earth-fill embankment. Where a fill slope forms the streamward face of a spill through abutment, it is regarded as part of the abutment.

Flow - A stream of water; movement of such things as water, silt and/or sand; discharge; total quantity carried by a stream.

Freeboard - (1) The vertical distance between the level of the water surface, usually corresponding to the design flow and a point of interest such as a bridge beam, levee top or specific location on the roadway grade. (2) The distance between the normal operating level and the top of the sides of an open conduit; the crest of a dam, etc., designed to allow for wave action, floating debris, or any other condition or emergency, without overtopping the structure.

Froude Number - A dimensionless number (expressed as $F = V/(gy)^{1/2}$) that represents the ratio of inertial to gravitational forces; i.e., at a Froude number of unity the flow velocity and wave celerity are equal. High Froude numbers can be indicative of a high velocity associated with supercritical flow and thus the potential or scour and high momentum forces. Stated another way, a number that varies in magnitude inversely with the relative influence of a gravity on the flow patter; $F>1$ indicates rapid (supercritical) flow; $F<1$ indicates tranquil (subcritical) flow.

Gabion - A rectangular basket made of steel wire fabric or mesh that is filled with rock or similar material of suitable size and gradation. Used to construct such things as flow-control structures, bank protection, groins, jetties, permeable dikes and riparian spur dikes. When filled with cobbles, masonry remnants or other rock or suitable size and gradation, the gabion becomes a flexible and permeable block with which the foregoing structures and devices can be built.

Geosynthetic Cellular Confinement System - Honeycomb structure of cells filled with a variety of infill materials that protects and stabilized the underlying strata from the erosive effects of wind and water.

Geotextile - A fabric manufactured from synthetic fiber that is designed to achieve specific engineering objectives, including seepage control, media separation (e.g., between sand and soil), filtration, or the protection of other constructions elements such as geomembranes.

Grade - Three definitions are suggested: 1) the longitudinal slope of a road, channel or natural ground; 2) the finished surface of a canal bed, road bed, top of embankment or bottom of excavation; 3) any surface prepared for the support of such things as conduit paving, ties, or rails.

Hydrodynamic - Involving principles that deal with the motion of fluids and the forces acting on solid bodies immersed in fluids and in motion relative to them.

Impervious - A surface that cannot be easily penetrated; for instance, rain does not readily penetrate asphalt or concrete surfaces.

Infiltration - The passage of water through the soil surface into the ground.

Inlet - Consider four definitions:1) a surface connection to a closed drain; 2) a structure at the diversion end of a conduit; 3) the upstream end of any structure through which water may flow; 4) an inlet structure for capturing

concentrated surface flow. Inlets may be located in such places as along the roadway, a gutter, the highway median or a field.

Longitudinal Profile - The profile of a stream or channel drawn along the length of its centerline. In drawing the profile, elevations of the water surface of the thalweg are plotted against distance as measured from the mouth or from an arbitrary initial point.

Manning Equation - An empirical formula devised by Manning, based upon original work by Ganguillet and Kutter, for computing flow in open channels and pipes. In its present form it has been modified to: $v = (1/n)R^{2/3}S^{1/2}$ where v = velocity, R = hydraulic radius or A/W_p where A = cross section area and W_p = wetted perimeter and S = Hydraulic Gradient. See Manning's n.

Manning's n - A coefficient of roughness used in the Manning Equation for estimating the capacity of a channel to convey water. Generally, "n" values are determined by inspection of the channel.

Mattress - A blanket or revetment of materials interwoven or otherwise lashed together and placed to cover an area subject to erosion.

Municipal Separate Storm Sewer System (MS4) - A separate storm sewer (including roads with drainage systems, municipal streets, catch basins, curbs, gutters, ditches, manmade channels or storm drains) which is all of the following: (i) Owned or operated by a state, city, town, borough, county, district, association or other public body (created by or under State law) having jurisdiction over disposal of sewage, industrial wastes, stormwater or other wastes, including special districts under State law such as a sewer district, flood control district or drainage district, or similar entity, or a designated and approved management agency under section 208 of the Federal Act (33 U.S.C.A. § 1288) that discharges to surface waters of this Commonwealth; (ii) Designed or used for collecting or conveying stormwater; (iii) Not a combined sewer; (iv) Not part of a POTW.

National Pollutant Discharge Elimination System (NPDES) - As authorized by the Section 402 of the Federal Clean Water Act (33 U.S.C.A. 1342), the National Pollutant Discharge Elimination System (NPDES) permit program controls water pollution by regulating point sources that discharge pollutants into waters of the United States.

Notice Of Intent (NOI) - A complete form submitted for NPDES general permit coverage which contains information required by the terms of the permit and by § 92.81—92.83 (relating to general permits). An NOI is not an application.

Outlet Protection - To armor the outfall of a pipe or culvert in order to reduce stormwater velocity and dissipate the energy of the flow leaving the outlet before it empties into the receiving channel.

Perimeter Controls - BMPs placed or constructed along the perimeter of an earth disturbance area to prevent runoff from entering the disturbed area, or to capture and treat sediment runoff prior to leaving a disturbed area.

Permanent Stabilization - A minimum, uniform 70% perennial vegetative cover and density, or 100% non-vegetative cover which will resist accelerated erosion or the proper placing of other materials to avoid sliding or other movement.

Permissible Velocity - The highest velocity at which water may be carried safely in a canal or other conduit without channel bed scour or bank erosion.

Phase II - The Phase II Final Rule, published in the Federal Register on December 8, 1999, requires NPDES permit coverage for stormwater discharges from certain regulated small municipal separate storm sewer systems (MS4s); and construction activity disturbing between 1 and 5 acres of land (i.e., small construction activities).

Point Source - Any discernible, confined and discrete conveyance, including, but not limited to, any pipe, ditch, channel, tunnel, conduit, well, discrete fissure, container, rolling stock, CAFO, landfill leachate collection system, or vessel or other floating craft, from which pollutants are or may be discharged.

Principal Spillway - The structure within a sediment basin which controls the discharge of water from the facility.

Project Site - The entire area of activity, development or sale including: (i) The area of an earth disturbance activity; (ii) The area planned for an earth disturbance activity; (iii) Other areas which are not subject to an earth disturbance activity.

Revetment - Erosion resistant materials placed directly on a slope or bank to protect the slope or bank from erosion.

Rills - The formation of numerous, closely spaced streamlets due to uneven detachment of surface soils by runoff on slopes.

Riprap - A layer, facing, or protective mound of broken concrete, sacked concrete, rock, rubble, or stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also, the stone used for this purpose.

Riser - When the entrance to a culvert may become easily clogged a corrugated metal pipe or a structure made of timber or concrete with small perforations, called a riser, is installed vertically to permit entry of water and prohibit the entry of mud and debris. The riser may be increased in height as the need occurs.

Rolled Erosion Control Products (RECPs) - A temporary degradable or long-term non-degradable material manufactured or fabricated into rolls designed to reduce soil erosion and assist in the growth, establishment and protection of vegetation.

Runoff Event - A rainfall event which produces runoff.

Runoff - That part of the precipitation that runs off the surface of a drainage area after accounting for all abstractions. The portion of precipitation that appears as flow in streams; total volume of flow of a stream during a specified time.

Sag Location - A drop or depression below the surrounding area.

Scour - The displacement and removal of channel bed material due to flowing water; usually considered as being localized as opposed to general bed degradation of headcutting. The result of the erosive action of running water that excavates and carries away material from a channel bed.

Sediment - Soils or other material transported by surface water as a product of erosion.

Sedimentation - The action or process of forming or depositing sediment in waters of this Commonwealth.

Shear Stress - An internal force tangential to the section of which it acts; an action or stress resulting from applied forces that causes or tends to cause two contiguous parts of a body to slide relatively to each other in a direction parallel to their plane of contact.

Sheet Flow - The flow of rainwater or snowmelt over the land surface toward stream channels. After it enters a stream, it becomes runoff.

Side Slopes - The slope of the sides of a canal, dam, or embankment. Currently sanctioned the naming of the vertical distance first as 1 to 2 (or, frequently, 1:2) meaning a vertical distance of 1m to 2m horizontal. Another form, not as subject to misinterpretation by thoughtless transposition, is 1V on 2H.

Special Protection Watershed, High Quality Watershed or Exceptional Value Watershed - In regards to E&SPC, these are watersheds defined in Pennsylvania under 25 Pa. Code Part 93 and require the water quality of these watersheds to be maintained and protected (generally a higher standard of protection).

Spillway - A passage for spilling surplus water.

Stabilization - The proper placing, grading, constructing, reinforcing, lining, and covering of soil, rock or earth to insure their resistance to erosion, sliding or other movement.

Storm Inlet Protection - Prevents sediment-laden water from entering a storm sewer at new inlets in areas of new development or existing inlets in areas of reconstruction.

Storm Sewer - Principally a drain for conveying stormwater, but at least part of the time a drain that also conveys raw sewage is termed a storm sewer.

Stormwater - Runoff from precipitation, snow melt runoff and surface runoff and drainage.

Stormwater Discharge Associated with Construction Activity - The discharge or potential discharge of stormwater into waters of this Commonwealth from construction activities including clearing and grubbing, grading and excavation activities involving 5 acres (2 hectares) or more of earth disturbance, or an earth disturbance on any portion, part of or during any stage of a larger common plan of development or sale that involves 5 acres (2 hectares) or more of earth disturbance over the life of the project.

Tailwater - Tailwater is the depth of flow in the channel directly downstream of a drainage facility. Often calculated for the discharge flowing in the natural stream without the highway effect (but may include other local effects from development), unless there is a significant amount of temporary storage that will be (or is) caused by the highway facility; in which case, a flood routing analysis may be required. The tailwater is usually used in such things as culvert and storm drain design and is the depth measured from the downstream flow line of the culvert or storm drain to the water surface. May also be the depth of flow in a channel directly downstream of a drainage facility as influenced by the backwater curve from an existing downstream drainage facility.

Temporary Stabilization - Provides immediate control of accelerated erosion from a disturbed area pending further disturbance or stabilization between seeding and establishment of permanent vegetative cover. In this field reference, the principal temporary stabilization measures are mulch, erosion control blankets, and temporary vegetative cover.

Termination - The means of construction at revetment edges e.g. toe trench, bench, etc.

Toe - That portion of a stream cross-section where the lower bank terminates and the channel bottom or the opposite lower bank begins.

Tractive Force -The drag on a stream bank caused bypassing water that tends to pull soil particles along with the stream flow. The force or drag developed at the channel bed by flowing water. For uniform flow, this force is equal to a component of the gravity force acting in a direction parallel to the channel bed on a unit wetted area. Usually expressed in units of stress; i.e., force per unit area.

Transition - A short conduit and/or channel uniting two other conduits and/or channels having different hydraulic elements; a conversion; a variable conduit or channel section connecting one uniform conduit or channel to another of different cross section form.

Waters of the Commonwealth - Rivers, streams, creeks, rivulets, impoundments, ditches, watercourses, storm sewers, lakes, dammed water, wetlands, ponds, springs and other bodies or channels of conveyance of surface and underground water, or parts thereof, whether natural or artificial, within or on the boundaries of this Commonwealth.

12.8 CHAPTER 12 NOMENCLATURE

Symbol	Definition	Units
A	Cross sectional area or surface area	m^2 or ft^2
β	Slope angle	degrees
C_{or}	Orifice coefficient	dimensionless
C_w	Weir coefficient	dimensionless
d	Cell depth	m or ft
d	Stake diameter	m or ft
DA	Drainage area	ha or ac
d_c	Minimum acceptable depth of infill	mm or in
d_i	Inside diameter of outlet pipe	m or in
D_o	Outlet pipe diameter	m or ft (cm or in)
D_{cs}	Cross slope spacing	m or ft
D_{ds}	Downslope spacing	m or ft

FOS	Factor of Safety	dimensionless
γ	Unit weight	kn/m^2 or lb/ft^3
g	Acceleration due to gravity	m/s^2 or ft/s^2
H	Depth of flow above riser crest	m or ft
H_G	Apron depth	m or ft
K_m	Coefficient of minor losses	dimensionless
K_p	Pipe friction coefficient	dimensionless
K_p	Coefficient of passive earth	dimensionless
L	Length	m or ft
L	Stake length	m or ft
L_a	Apron length	m or ft
L_{min}	Minimum flow length	m or ft
L_s	Length of pipe in saturated zone	m or ft
MDS	Maximum Downslope Spacing	m or ft
n	Manning's coefficient of roughness	dimensionless
φ	Minimum angle of repose	degrees
P_p	Required stake resistance of individual stake	N/stake or lb/stake
Q	Discharge, flow	m^3/s or cfs
Q_{or}	Orifice flow	m^3/s or cfs
Q_p	Pipe flow	m^3/s or cfs
Q_w	Weir flow	m^3/s or cfs
R	Radius	m or ft
RSR_a	Required Stake Resistance of stake array	kg/m^2 or lb/ft^2
S	Average slope	percent
SA	Surface area	m^2 or ft^2
SS_u	Ultimate long-term seam strength	kN/m or lb/ft
v	Collar projection	m or ft
V_{cap}	Maximum permissible velocity	m/s or ft/s
W	Width	m or ft
W_a	Apron width	m or ft
W_G	Cover weight	kg/m^2 or lb/ft^2
W_{min}	Minimum filler strip width	m or ft
y	Vertical distance from upstream invert of principal spillway riser to top of dewatering zone	m or ft
z	Horizontal component of upstream embankment slope	m or ft

12.9 REFERENCES

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CHAPTER 12, APPENDIX A

E&S RELATED REGULATIONS

12A.0 INTRODUCTION

The following paragraphs summarize some of the important laws and regulations related to erosion, sediment, and stormwater control programs. Some of these laws and regulations contain specific permitting requirements for earth disturbances that have been incorporated into PA DEP's application package for the National Pollutant Discharge Elimination System (NPDES) Permit for Stormwater Discharges Associated with Construction Activities. Other laws and regulations involve permitting programs, such as 25 PA Code § 105, which are focused primarily on issues other than erosion and sediment control, but which contain requirements that permit applications include plans to minimize the potential effects of accelerated erosion and sedimentation, or that permit applications be accompanied by copies of erosion and sediment control plans with a letter from a regulatory review authority that indicates the erosion and sediment control plan is satisfactory (e.g. 25 PA Code § 105.13(f)). Additional information regarding the laws and regulations related to control of erosion and sediment can be obtained from the agencies responsible for the programs (i.e. Environmental Protection Agency (EPA), United States Army Corps of Engineers (USACE), and the Pennsylvania Department of Environmental Protection (PA DEP)).

12A.1 REGULATORY REQUIREMENTS

A. The Federal Clean Water Act, 33 U.S.C. § 1251 et seq. The Clean Water Act, which was enacted in 1972, was formerly known as the Federal Water Pollution Control Act. Its purpose is to "restore and maintain the chemical, physical, and biological integrity of the Nation's waters." The Clean Water Act sets requirements for water quality standards for the discharge of pollutants into waterways.

The three primary sections of this Act pertaining to stormwater management regulations are Sections 401, 402, and 404 (33 U.S.C. §§ 1341, 1342, and 1344 respectively).

1. Section 401. Section 401 is triggered if the construction or operation of a facility (1) requires a Federal license or approval (e.g. a Section 404 Permit from the U.S. Army Corps of Engineers) and (2) the construction or operation of the facility will cause a discharge into navigable waters. Section 401 requires that an individual applying for a Federal license or permit to discharge into navigable waters provide the permitting agency with a certification from the State (from PA DEP; in Pennsylvania) indicating that the discharge will comply with provisions in Sections 301, 302, 303, 306, and 307 of the Clean Water Act. The aforementioned sections provide the effluent limitations, water quality standards, and performance standards for certain types of activities and discharges. The PA DEP approves this certification, which is commonly referred to as a "401 Water Quality Certification" or 40 CFR Part 121.

2. Section 402. This section of the Clean Water Act requires a permit for the discharge of any pollutant, or combination of pollutants into waters of the United States. This permit is referred to as the National Pollutant Discharge and Elimination System (NPDES) Permit. The purpose of the permit is to ensure that necessary actions are taken to protect water quality and quantity. This section provides for permits for discharges associated with industrial activity and permits for discharges from municipal storm sewers (stormwater). This section defines stormwater associated with construction activity as "industrial activity." EPA has enacted regulations defining the NPDES Permit process. 40 CFR Part 122. PA DEP's NPDES program has been approved by EPA; therefore PA DEP is responsible for the issuance of NPDES Permits.

3. Section 404. This section of the Clean Water Act prohibits the discharge of dredged or fill material into waters of the United States without a permit from the USACE. This permit is referred to as a "Section 404 Permit." As part of the Section 404 Permit, a 401 Water Quality Certification is required from PA DEP.

The USACE provides for two types of Section 404 permits, general and individual, depending upon the project's complexity, environmental impacts, and location. (It should be noted that general permits include nationwide, regional, and programmatic Section 404 permits). For projects that require a Section 404 Permit

and a PA DEP Chapter 105 Permit (as discussed below), a Joint Permit Application is available through the PA DEP, and may be used and submitted to satisfy both the Section 404 and Chapter 105 Permit requirements (the use of the Joint Permit is restricted depending upon certain project conditions). The USACE regulations 33 CFR Parts 325, 320, 323, 330 and EPA regulations 40 CFR Part 122 also apply.

Details regarding Section 404 Permits can be obtained by contacting the USACE, or through their website at www.usace.army.mil. Details regarding the NPDES Permits and Section 401 Water Quality Certification can be obtained from the PA DEP through their website at www.dep.state.pa.us/efacts.

B. Pennsylvania's Clean Streams Law, Act of June 22, 1937 (P.L. 1987, No. 394) as amended 35 P.S. § 691.1 et seq. The Clean Streams Law was enacted in 1937 in order to "preserve and improve the purity of the waters of the Commonwealth for the protection of public health, animal and aquatic life, and for industrial consumption, and recreation..." This law provides for protection of water supplies and water quality, regulates discharges of sewage and industrial waste, regulates mine operations and their impact on water quality, supply and quantity, and regulates stormwater associated with construction activities. Therefore, construction activities fall under the Clean Streams Law. Through this law, the PA DEP is given the power to "establish policies for effective water quality control and water quality management in the Commonwealth of Pennsylvania and coordinate and be responsible for the development and implementation of comprehensive public water supply, waste management, and other water quality plans." This law requires permit approval for discharges from mines, discharge of sewage or industrial waste, and discharge of any other substance that would result in pollution, both directly and indirectly, into waters of the Commonwealth.

Several PA DEP permit processes have been generated through regulations promulgated in part or in whole, pursuant to the Clean Streams Law. Of these, the following list includes, but is not limited to, those permits that have a link to erosion and sediment control and stormwater management:

- NPDES Permit for Stormwater Discharges Associated With Construction Activities.
- NPDES Phase II MS4 Permit.
- E&S Control Permit.

Details regarding these permits can be obtained from the PA DEP through their website at www.dep.state.pa.us/efacts. Select the link to the "Guide to DEP Permits and Other Authorizations."

C. Pennsylvania's Stormwater Management Act, Act of October 4, 1978, P.L. 864 No. 167, 32 P.S. § 680.1 et seq. (as amended by Act 63). The purpose of Act 167 is to encourage planning and management of storm water runoff in each watershed, authorize a comprehensive program of storm water management designated to preserve and restore the flood carrying capacity of Commonwealth streams, and to encourage local administration and management of storm water consistent with the Commonwealth's duty as trustee of natural resources.

The county stormwater management plans are commonly referred to as "Act 167 Plans." The plans evaluate both the hydrologic and hydraulic characteristics of the drainage basins, and are designed to manage stormwater from a quantity and quality perspective. Act 167 Plans are adopted by counties and approved by PA DEP. After an Act 167 Plan is adopted and approved, each municipality is required to adopt and implement ordinances necessary to regulate development and activities within the municipality in a manner consistent with the Act 167 Plan. Moreover, construction using Commonwealth funds within a watershed with an approved Act 167 plan shall be completed in a manner consistent with the plan.

The Pennsylvania Stormwater Management Act (Act 167) is the legislative basis for stormwater management. Section 11 (a) of the Act states that "after adoption and approval of a watershed storm water plan in accordance with this act, the location, design and construction within the watershed of storm water management systems, obstructions, flood control projects, subdivision and major land developments, highways and transportation facilities, facilities for the provision of public utility services and facilities owned or financed in whole or part in by funds from the Commonwealth shall be conducted in a manner consistent with the watershed storm water plan." Thus, wherever an adopted and approved Act 167 plan exists, consistency with that plan is a statutory requirement for PennDOT.

Often, these Act 167 Plans overlap with requirements from the NPDES Construction Stormwater Permit. For NPDES Construction Permits, the Post Construction Stormwater Management (PCSM) Plan always needs to be

consistent with the approved Act 167 Plan. PCSM plans also need to meet the design requirements contained in the NPDES construction stormwater permit application. In the rare case that the design requirements in the NPDES construction permit application directly conflict with the requirements in an approved Act 167 plan, the requirements in the approved Act 167 plan take precedence; however, all requirements can usually be satisfied.

This Act also requires any land developer to implement measures: (1) "to assure that the maximum rate of stormwater runoff is no greater after development than prior to development activities;" or (2) "to manage the quantity, velocity and direction of the resulting stormwater in a manner which otherwise adequately protects health and property from possible injury."

D. Federal National Pollutant Discharge Elimination System Phase II. The NPDES Phase II MS4 Program is designed to ensure that government entities located within designated urbanized areas take actions to control/manage stormwater runoff and associated discharges into surface waters. An MS4 is a "municipal separate storm sewer system." Any municipality operating an MS4 within a designated urbanized area must obtain an NPDES Phase II MS4 Permit from DEP. Included within the requirements of this permit process are the development and implementation of a plan to meet six minimum control measures (MCMs); with a time schedule, series of BMPs, and measurable goals required for each MCM. The MCMs include the following:

- Public Education and Outreach.
- Public Participation and Involvement.
- Illicit Discharge Detection and Elimination.
- Construction Site Runoff Control.
- Post-Construction Stormwater Management.
- Pollution Prevention and Good Housekeeping for Municipal Operations and Maintenance.

PennDOT has an NPDES Phase II MS4 Permit which permits the discharge of stormwater from PennDOT facilities to surface waters within the designated urbanized areas. Under the MS 4 Permit, part of PennDOT's compliance with the six minimum control measures described above is to renew and update PCSM and E&S design guidance periodically. Therefore, the designer is not responsible for updating the permit, but following the design guidance provided by PennDOT.

E. Pennsylvania's Dam Safety and Encroachments Act (Act of November 26, 1978 (P.L. 1375 No. 325) as amended, 32 P.S. § 693.1 et seq.) This Act provides for the regulation and safety of dams, reservoirs, water obstructions, and encroachments in the Commonwealth of Pennsylvania. It requires that regulations be developed establishing: 1) standards and criteria for the location and design of dams, water obstructions and encroachments; 2) requirements for operation of dams; 3) requirements for monitoring, inspection, and reporting of conditions affecting the safety of dams, water obstructions, and encroachments; and 4) requirements for emergency warning and action plans, etc. It applies to dams as well as other water obstructions and encroachments located in, along, across, or projecting into any watercourse, floodway, or body of water. Types of activities under this Act's jurisdiction include, but are not limited to, placing fill in waters of the Commonwealth (e.g. wetlands and streams) or the construction of bridges, culverts, or pipes in waters of the Commonwealth. Based on this statute, the construction, operation, maintenance, modification, enlargement, or abandonment of any dam, water obstruction, or encroachment is prohibited without a permit from the PA DEP. This permit is known as either a Chapter 105 Dam Safety Permit or Water Obstruction and Encroachment Permit as described in the following heading for PA Code, Title 25, Chapter 105. The Dam Safety Permit is specific to the design, construction, maintenance, operation, modification, and/or abandonment of dams, while the Water Obstruction and Encroachment Permit is specific to the construction, maintenance, operation, modification, and/or abandonment of water obstructions and encroachments.

F. Commonwealth of Pennsylvania, PA Code, Title 25, Chapter 105: Dam Safety and Waterway Management. Chapter 105 of PA Code, Title 25 is part of the regulatory mechanism for Pennsylvania's implementation of the Dam Safety and Encroachments Act. The Chapter 105 regulations serve to "provide for the comprehensive regulation and supervision of dams, reservoirs, water obstructions and encroachments in the Commonwealth in order to protect the health, safety, welfare and property of the people" by requiring a permit for the construction, operation, maintenance, modification, enlargement, or abandonment of a dam, water obstruction, or encroachment. This permit is typically referred to as a "Chapter 105 Permit."

From a stormwater run off standpoint, a Chapter 105 Permit requires an analysis of the project's impact on Act 167 Stormwater Management Plans and a letter from the municipality commenting on the analysis. If the stormwater

analysis reveals increases in peak rates of runoff or flood elevations, the permit application must also include a description of property that may be affected and an analysis of the degree of risk to the property. Finally, except for small projects, proof of an application for an erosion and sedimentation plan must be included.

There are three types of PA DEP Chapter 105 Permits. They include general permits, small project permits, and individual permits. There are several general permits, each containing specific limits and restrictions. Copies of these permits and their conditions can be obtained from the PA DEP website at www.dep.state.pa.us. Applicants for these general permits need only register their intent to construct the project in accordance with the conditions of the permit. No additional application information is required.

A small project application is required for projects that do not qualify for a general permit, but are considered to have an "insignificant impact" on safety and protection of life, health, property and the environment as defined in Chapter 105.1 of the regulations, and that do not impact wetlands. All other projects require an Individual Chapter 105 Permit.

For projects that require both a PA DEP Chapter 105 Permit (small project or individual permit) and a USACE Section 404 Permit (as discussed above), a Joint Permit Application is available through the PA DEP, and may be used and submitted to satisfy both the Section 404 and Chapter 105 Permit requirements (the use of the Joint Permit is restricted depending upon certain project conditions).

G. Pennsylvania's Flood Plain Management Act, Act of October 4, 1978, P.L. 851, No. 166, 32 P.S. § 679.101 et seq. This Act provides for the regulation of land and water use for flood control purposes. It authorizes a comprehensive and coordinated program, based upon the National Flood Insurance Program, to preserve and restore the efficiency and carrying capacity of streams and floodplains in Pennsylvania. The adoption and administration of floodplain management regulations necessary to comply with the National Flood Insurance Program is governed by the provisions in the Pennsylvania Municipalities Planning Code.

H. Commonwealth of Pennsylvania, PA Code, Title 25, Chapter 106: Floodplain Management. Chapter 106 is part of the regulatory mechanism for implementation of the Flood Plain Management Act. It requires individuals to obtain a permit to construct, modify, remove, destroy, or abandon a highway obstruction or an obstruction in a floodplain. Its primary purpose is to prevent flooding and protect people and property from such flooding, by encouraging planning and development in floodplains that are consistent with sound land use practices. This permit is obtained (for highway projects) from the Pennsylvania PA DEP under the Chapter 105 Program.

I. Commonwealth of Pennsylvania, PA Code, Title 25, Chapter 92: National Pollutant Discharge Elimination System (NPDES) Permitting, Monitoring and Compliance. Chapter 92 was issued under Section 5 and 402 of the Clean Streams Law. Chapter 92 sets forth permitting, monitoring and compliance requirements with regard to the PA DEP implementation of the NPDES program. As part of this process all construction projects with greater than 2.0 ha (5 ac) of earth disturbance are required to obtain an individual NPDES permit to construct the project. Those projects between 0.4 - 2.0 ha (1 - 5 ac) of earth disturbance, which also have a point source of discharge are required to obtain a general NPDES permit. According the PA DEP, a point source discharge is defined as "any discernible, confined and discrete conveyance, including but not limited to, any pipe, ditch, channel, tunnel, well, discrete fissure, or container from which pollutants are or may be discharged."

If applicants meet either of the aforementioned criteria then the project is required to address post-construction stormwater management as part of the permit application package. When completing the supporting documentation for the NPDES permit and preparing a post-construction stormwater management plan applicants must consider the following items.

1. PA DEP's policy strives for a no net change in stormwater runoff in terms of volume, rate, and water quality comparing pre-construction with post-construction runoff conditions. For runoff rate, PA DEP's policy calls for the evaluation of the 1- through 100-year storm events. For runoff volume, PA DEP's policy calls for the evaluation of the 2-year 24-hour storm event or smaller.

2. Consistency with the standards of watershed-based stormwater management plans approved and implemented under the Stormwater Management Act (Act 167).

3. Application of both rate control and also volume control of stormwater runoff in High Quality or Exceptional Value watersheds. Volume increases may be permitted, if justified, in High Quality Watersheds but any increase in post-construction stormwater runoff volume must be mitigated in Exceptional Value watersheds. For a map or listing of High Quality or Exceptional Values watersheds in Pennsylvania contact the Pennsylvania Department of Environmental Protection (PA DEP). As the performance standards for post construction stormwater management controls in HQ or EV watersheds are frequently changing, designers should review current PA DEP requirements and contact the District Environmental Manager for current performance standards for a project prior to initiating the design.

For further description of the NPDES application process and how it relates to PennDOT projects refer to Publication 13M, Design Manual, Part 2, *Highway Design*, Section 13.5.B.2.

J. Commonwealth of Pennsylvania, PA Code, Title 25, Chapter 93: Water Quality Standards. The provisions of this chapter were issued under sections 5 and 402 of the Clean Streams Law. Chapter 93 sets forth water quality standards for waters of the Commonwealth, including wetlands. In addition, this Chapter provides for the implementation of antidegradation requirements to protect existing use of waters of the Commonwealth. In watersheds designated as High Quality (HQ) or Exceptional Value (EV), Chapter 93 requires that nondischarge alternatives be considered. Where no environmentally sound and cost-effective nondischarge alternatives exist, Chapter 93 requires that the applicant demonstrate that the discharge will maintain and protect the existing quality of receiving surface waters. For High Quality Waters, DEP may allow a reduction in water quality (1) if necessary to accommodate important economic or social development in the area; and (2) a demonstration is made that said reduction will support applicable existing and designated uses, e.g. WWF, TSF, CWF. For a list of special protection watersheds, contact the PA DEP.

K. Commonwealth of Pennsylvania, PA Code, Title 25, Chapter 102: Erosion and Sediment Control. Chapter 102 of PA Code, Title 25, does not deal directly with long-term stormwater management but addresses the application of the National Pollution Discharge Elimination System as it relates to construction and maintenance projects in the state of Pennsylvania. The focus of Chapter 102 is to implement and maintain BMPs to minimize potential for accelerated erosion and sedimentation during construction activities. The Erosion and Sedimentation Pollution Control (E&SPC) Plan is the product of the Chapter 102 process. The E&SPC plan is normally reviewed and approved by the County Conservation Districts.

Table 12A.1 Relationship of Activities, Regulations, and Permits for Stream Work

Activity*	Regulations	Requirements/Permits
Discharging Stormwater Into Surface Waters	PA Clean Streams Law Federal Clean Water Act, Sections 402 Federal NPDES Regulations at 40 CFR Part 122 PA Stormwater Management Act PA Code, Title 25, Chapter 92: NPDES Permitting, Monitoring, and Compliance PA Code, Title 25, Chapter 102: Erosion and Sediment Control	The NPDES regulations (40 CFR § 122), and Chapter 92 require that construction activities with disturbances of 5 or more acres, or those that disturb between 1 and 5 acres with a point source discharge be permitted, as per the NPDES Permit for Stormwater Discharges Associated With Construction Activities. Stormwater is then managed by NPDES after construction through the NPDES Phase II Permit.
Stream Encroachment or Obstruction	Federal Clean Water Act, Sections 401 and 404 PA Dam Safety and Encroachments Act PA Code, Title 25, Chapter 105: Dam Safety and Waterway Management PA Code, Title 25, Chapter 102: Erosion and Sediment Control Federal NPDES Regulations at 40 CFR Part 122 PA Code, Title 25, Chapter 92: NPDES Permitting, Monitoring, and Compliance	Work that creates or allows water runoff, effluent, or other pollutants to be discharged into navigable waters requires a Section 401 Water Quality Certification to ensure water quality standards are met. A USACE Section 404 Permit is required to allow for the discharge of dredged or fill material into navigable waterways. A Chapter 105 Permit is needed for any structure or activity that changes, expands, or diminishes the course, current, or cross section of a watercourse, floodway, or body of water. In addition, these activities will likely involve earth disturbance activities and discharges into waters, thereby requiring permits as described above in this table and an E&S Plan as per Chapter 102. The NPDES regulations (40 CFR § 122), and Chapter 92 require that construction activities with disturbances of 5 or more acres, or those that disturb between 1 and 5 acres with a point source discharge be permitted, as per the NPDES Permit for Stormwater Discharges Associated With Construction Activities. Structures that require less than 1 acre of earth disturbance will not require an NPDES permit.
Dam Construction / Removal	PA Dam Safety and Encroachments Act PA Code, Title 25, Chapter 105: Dam Safety and Waterway Management	Dam construction or removal will require a Dam Safety Permit, as well as a PA DEP Chapter 105 Permit (or qualify for a waiver). In addition, this activity may involve earth disturbance activities and discharges into waters, thereby requiring permits as described above in this table.
* Activities that occur within EV or HQ Watersheds, as per PA Code, Title 25 Chapter 93, require an Individual NPDES Permit for Stormwater Discharges Associated With Construction Activities.		

CHAPTER 12, APPENDIX B

RECOMMENDED NOTES FOR ESPC PLANS

12B.0 STANDARD NOTES

For required and optional notes refer to Publication 14M Chapter 6 CONTOUR GRADING AND DRIANAGE PLANS, EROSION AND SEDIEMTN POLLUTION CONTROL PLANS, AND POST CONSTRUCTION STORMWATER PLANS.

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CHAPTER 12, APPENDIX C

RECOMMENDED STANDARDS FOR E&SPC PLANS

12C.0 GENERAL

The Erosion and Sediment Pollution Control (E&SPC) Plans include both a set of drawings for construction and a technical narrative. The Department typically titles the drawings for construction the Erosion and Sediment Pollution Control Plans as discussed in 12C.1. The required E&SPC technical narrative is discussed further in 12C.2. The following guidance is a combination of PA DEP's recommendations for developing an E&SPC Plan, and PennDOT's standards and preferences. It is important to note, however, that preferences may vary between County Conservation Districts, and even PA DEP regional offices. Therefore, it is important to coordinate with the agency that will be reviewing the E&SPC Plan.

Standards in Publication 14M, Design Manual, Part 3, *Plans Presentation*, related to borders, title blocks, and general plan presentation must be applied. However, not all of the information pertinent to E&SPC Plans is described in Publication 14M, Design Manual, Part 3, *Plans Presentation*; complimentary information is described below.

All drawings included in the E&SPC Plan must be legible. Letters and numbers used on the plans must be readable. Symbols used should be readily distinguishable from each other, and clutter should be avoided. Drawings should be clearly labeled and dated. Revised drawings must have the date of each revision shown. Sufficient space should be provided on the top sheet for the approval stamp and signature of the reviewing agency. The location of this space should be such that the stamp and signature are visible when the drawing is folded.

At least one set of drawings submitted to any agency for review must be full size. To facilitate review, it is recommended that a composite work area plan (or roll map) be submitted in addition to the regular plans. The composite plan should show the entire project, or major portions thereof, with proposed contours, E&SPC BMPs, and staging or work area boundaries.

The name of the plan designer along with his/her contact information should be provided on the cover sheet, as well as in the narrative. Also provide the District's contact information on the cover sheet and in the narrative.

12C.1 PLAN DRAWINGS

Plan drawings contain all of the information that the contractor needs to build the project. Information not needed by a contractor, such as supporting calculations or manufacturer's test data, should be placed in the E&SPC narrative. In general, the plan drawings should contain:

- Cover sheet
- Detail sheets
- Plan sheets

The engineer responsible for the plan must place a black ink rubber stamp seal and his/her signature on the cover sheet. Either a black ink rubber stamp seal or a facsimile seal must be placed on all subsequent sheets.

Separate plans must be included for offsite borrow or disposal areas, if known, which are part of the project. If offsite borrow and disposal areas are not known, a general note should be added to the plan stating that the contractor must develop and obtain approval of a separate erosion E&SPC Plan for each work area not detailed in the plan located outside of the limits of disturbance. These drawings must include all information required on the main drawings.

A. Cover Sheet. At a minimum, the cover sheet of the drawing set should contain the following information:

- Title

- Call Before You Dig / PA One Call logo and serial number
- Location map (Type 10) with the limits of work identified, scale, and north arrow
- Sheet index
- Contact information of the County Conservation District (and PA DEP Regional Office for JPA_2 projects) and District contact
- Name and contact information of the plan preparer

The General Notes and/or other supplemental information may also be included on the cover sheet if space is available.

B. Detail Sheets. Supplemental drawings include any notes, details, etc. that are integral to the overall E&SPC Plan. Each bit of information a contractor will need in order to correctly install, operate, and maintain each of the proposed E&SPC BMPs should be placed on the drawings. This includes:

1. General Notes. Refer to Appendix 12B, *Recommended Notes for E&SPC Plans* for a list of common general notes for E&SPC Plans.

2. Sequence of BMP Installation and Removal. The order in which E&SPC BMPs are installed and removed relative to the general sequence of construction activities is critical information. Each BMP should be clearly identified in the sequence. A general statement such as "Install perimeter BMPs, as shown" is not sufficient.

3. Maintenance Plan. It is recommended that maintenance information be placed near the detail for the BMP for which it is appropriate. However, a separate Maintenance Schedule should be provided specifying responsibility for conducting inspections, frequency of those inspections, general time frames for completing repairs, and instructions for disposing of sediment cleaned from the various BMPs.

4. BMP Details. A detailed plan view (drawn to scale) should be provided for each proposed sediment basin. This plan view should include proposed contours and show all points of inflow into the basin as well as all outlet structures (principal and emergency spillways) with the proposed outlet protection for both inflows and discharges. Other features such as clean-out stakes and baffles (where needed) should also be shown. Other pertinent information should be shown on typical details. Any other E&S related structure or BMP requiring a scale drawing must also be shown on a plan drawing with all critical dimensions indicated.

Other BMPs for which scale drawings are not required (e.g., channels, stabilized construction entrances, silt fence, etc.) may be shown on typical drawings. Chapter 12, *Erosion and Sediment Pollution Control*, and Publication 72M, *Roadway Construction Standards*, contain most of the typical details that will be needed for a project. However, if there is a particular BMP that PennDOT does not have a detail for, the PA DEP E&S Manual should be consulted for the appropriate measure. Note that if a detail from the PA DEP E&S Manual is used and it is necessary to alter the detail, the term "Standard Construction Detail" cannot be used. Care should be taken to include all pertinent data with the typical details, including critical dimensions and elevations as well as accompanying notes and tables. Tables should be included where a particular BMP will be used in more than one location with differing specifications for each location.

5. Temporary and Permanent Stabilization. The means for providing temporary and permanent stabilization should be included in the supplemental drawings. Typically, this can be addressed by placing the recommended seed mixes, fertilizer, mulch application, and provisions for stabilization during non-germinating periods.

6. Recycling and Disposal of Waste Materials. Instructions for how the contractor should dispose of or recycle waste materials (e.g., concrete, metal, rock, asphalt, etc.) must be provided on the plan.

7. Means to Address Soil Limitations. The means to address the identified soils limitations must be included on the drawings. For example, a note to use only certain areas as borrow areas for fill for sediment basins or traps, or special fertilization requirements for portions of the project, etc.

8. Index Map for Work Area Drawings. When the work area extends beyond a single plan sheet, provide a sketch of the overall project area showing the location of match lines or outlines of sheets.

C. Plan Sheets. Work area drawings are those plan sheets that contain planimetric and topographic information of the work area. Each plan drawing showing the work area must have a graphic scale, north arrow, a legend identifying all symbols used, and match lines (if more than one sheet is required).

The coverage of the work area drawings(s) must include sufficient surrounding area so that the maximum-during-construction drainage areas to all disturbed areas and BMPs can be shown. Where drainage areas are too large to be shown on the plan maps, such as offsite areas draining to diversion ditches, they should be indicated on the plan and depicted on the location map(s) in the narrative.

The scale of the plan map(s) must be large enough to clearly depict the topographic features of the site. Contours must be at an interval that will adequately describe the topography of the site. A scale of 1 inch equals 50 feet or less, with 2-foot maximum contour intervals should be used unless the reviewing agency agrees that another scale and/or contour interval is appropriate (e.g., extremely steep or extremely gentle slopes). Both existing and proposed grading contours must be shown on the plan. Existing contours should be dashed and lighter in shading than proposed contours. Proposed contours should be solid and dark.

All E&S features must be indicated by symbols and a legend of symbols provided on each page (refer to Publication 14M, Design Manual, Part 3, *Plans Presentation*, Chapter 13). Symbols may be identified by use of notes and arrows as long as this does not clutter or otherwise cause confusion. Match lines must identify the adjoining work area drawing(s). An overall map should be provided for projects having many adjoining plan maps to indicate how the individual plan maps fit together.

All existing roadways, municipal boundaries, streams, watercourses, wetlands, other surface waters, FEMA floodways, structures, utilities, and identifiable landmarks within or in close proximity (that could affect or be affected by the project) should be shown on the work area drawing(s). Each roadway should be identified, along with legal right-of-way lines and any temporary or permanent easements. Stream names must be provided, and unnamed streams should be labeled "Unnamed tributary to…" Wetlands should be labeled in a manner consistent with the delineation report.

All proposed improvements, such as roads, storm sewers, utilities, etc., must be shown on the plan. Station numbers should be provided for all proposed roadways, pipelines, major utility lines, and stream channel relocations. These station numbers should be consistent with any that are specified in the Sequence of BMP Installation and Removal.

The proposed limits of disturbance must be shown on the work area drawings. All proposed earthmoving as well as all proposed E&SPC BMPs must be located within those boundaries. Any areas within those boundaries that are to remain undisturbed should be clearly delineated. Permitted sites must also clearly show the NPDES permit boundaries. It cannot be assumed that the limits of disturbance or the property line is the permit boundary.

All proposed E&SPC BMPs (including erosion control blankets), waterways, and stormwater management facilities must be shown on the plans. Where projects are to be constructed in phases, plans for each phase (showing the BMPs that will be present and functioning during that phase) should be provided. Each BMP should be clearly labeled and consistent with those used on the detail sheets and in the supporting calculations section of the narrative. Whenever channel dimensions or linings change, it must be clearly shown on the plans.

Sites discharging to Special Protection Watersheds must meet stricter standards and specifications. Where only part of a project is tributary to such a watershed, that portion of the project should be clearly delineated on the plans. Likewise, areas of special concern, such as potentially hazardous materials; areas prone to sinkhole formation, landslides, or mine subsidence; and resource areas that will be protected (e.g., natural drainageways, riparian buffers, unusual geologic features, etc.) should also be shown.

12C.2 NARRATIVE

In general, the narrative should include the items listed in the following paragraphs. Details are provided for each.

A. Project Description. An overall description of the location and proposed construction activities should be provided in the narrative. For non-permitted projects, also describe the past, present, and proposed land uses. For

projects using the general NPDES permit, refer the reader to Section B of the NPDES NOI form. An example of the NPDES NOI form Section B is provided in Appendix 4B, *Sample Notice of Intent (NOI) Application Form*.

B. Location Map. A location map that shows the relationship of the project to municipal boundaries and major highways must be provided as part of every E&SPC Plan. The location map must be reprinted or copied from the appropriate 7½ minute USGS quadrangle map(s). The name(s) of the quadrangle(s) must be included on the location map. The location map must be included in the E&SPC narrative.

C. Soils. The preferred method of showing soils boundaries within the project area is to provide a legible photocopy of the appropriate soils map(s) from the NRCS website with the project outline clearly shown and identified. The locations of all proposed sediment basins and traps should also be shown on the map(s). These soil maps should be located in the E&SPC narrative. An acceptable alternative is to plot the soil boundaries on the E&SPC Plan drawings. When this option is chosen, care should be taken to avoid error due to enlargement of the soil map from which the boundaries are taken.

The types, depth, slope, and limitations (related to the proposed application) of the soils may be included in the narrative (preferred) or included on the drawings. Potential resolutions to each of the soil limitations that are noted should also be provided. Data on the physical characteristics of the soils, such as their texture, resistance to erosion, and suitability for intended use should be included in the narrative report. Hydric soils should also be identified. This information is available in soil survey reports, published by the USDA, Natural Resources Conservation Service (formerly Soil Conservation Service), in cooperation with the Pennsylvania State University College of Agriculture and others.

D. Waters of the Commonwealth. All streams in Pennsylvania are classified based on their designated and existing water uses and water quality criteria. Designated uses for Commonwealth waters are found in PA Code, Title 25, § 93.9a through § 93.9z. Existing uses of Commonwealth waters are usually the same as the designated use, except where information has been provided to or obtained by the PA DEP which indicates that a particular water body actually attains a more stringent water use than the designated use. Existing uses are protected pursuant to PA Code, Title 25, § 93.4a through § 93.4c. Names of stream receiving runoff from the project and their designated and existing water uses must be noted in the narrative.

E. Description of E&SPC BMPs. For non-permitted projects, describe the types of BMPs being used on before, during, and after earth disturbance activities. For permitted projects, refer the reader to Section 10 of the NPDES NOI form.

F. Calculations. The area draining to each BMP must be determined. In some instances the drainage area will increase or decrease as the site grading proceeds. In such cases, the maximum drainage area to the BMP must be used to determine the design capacity. If the runoff from a project area discharges to a stream that is classified for Special Protection, more stringent criteria are used to design the BMPs for that site.

The design (for E&S purposes) peak discharge rate and outlet velocity for each point source discharge from the project should be tabulated in the narrative.

Other BMP calculations that must be provided in the narrative include those for:

- Channels
- Sediment basins
- Sediment traps
- Outlet protection

Use of the Standard Worksheets in PA DEP's E&S Manual is highly recommended. Worksheets may be printed and completed by hand, or they can be scanned and completed electronically.

CHAPTER 12, APPENDIX D

E&S INITIATIVE COORDINATION INFORMATION

12D.0 PA DEP REGIONAL OFFICES

http://www.depweb.state.pa.us/portal/server.pt/community/regional_resources/13769

Table 12D.1 Correlation between PA DEP Regions, PennDOT Engineering Districts, and Conservation Districts

PennDOT Districts	PA DEP Region					
	1 **Southeast** ***Conshohocken*** ***(610) 323-2409***	**2** **Northeast** ***Wilkes-Barre*** ***(570) 836-3952***	**3** **Southcentral** ***Harrisburg*** ***(717) 337-9820***	**4** **Northcentral** ***Williamsport*** ***(570) 966-9129***	**5** **Southwest** ***Pittsburgh*** ***(412) 938-3432***	**6** **Northwest** ***Meadville*** ***(814) 332-6942***
District 1-0						Crawford Erie Forest Mercer Venango Warren
District 2-0			Juniata Mifflin	Cameron Centre Clearfield Clinton Potter		Elk McKean
District 3-0				Bradford Columbia Lycoming Montour Northumberland Snyder Sullivan Tioga Union		
District 4-0		Lackawanna Luzerne Pike Susquehanna Wayne Wyoming				
District 5-0		Carbon Lehigh Monroe Northampton Schuylkill	Berks			
District 6-0	Bucks Chester Delaware Montgomery Philadelphia					
District 8-0			Adams Cumberland Dauphin Franklin Lancaster Lebanon Perry York			
District 9-0			Bedford Blair Fulton Hunting.		Cambria Somerset	
District 10-0					Armstrong Indiana	Clarion Butler Jefferson
District 11-0					Allegheny Beaver	Lawrence
District 12-0					Fayette Greene Washington Westmoreland	

12D.1 CCD CONTACT INFORMATION

www.pacd.org/your-district/find-your-district

Adams Conservation District	(717) 334-0636
Allegheny Conservation District	(412) 241-7645
Armstrong Conservation District	(724) 548-3425
Beaver Conservation District	(724) 378-1701
Bedford Conservation District	(814) 623-7900 ext. 4
Berks Conservation District	(610) 372-4657
Blair Conservation District	(814) 696-0877 ext. 5
Bradford Conservation District	(570) 265-5539
Bucks Conservation District	(215) 345-7577
Butler Conservation District	(724) 284-5270
Cambria Conservation District	(814) 472-2120
Cameron Conservation District	(814) 486-2244 ext. 5
Carbon Conservation District	(610) 377-4894 ext. 4
Centre Conservation District	(814) 355-6817
Chester Conservation District	(610) 925-4920
Clarion Conservation District	(814) 226-4070
Clearfield Conservation District	(814) 765-2629
Clinton Conservation District	(570) 726-3798
Columbia Conservation District	(570) 784-1310
Crawford Conservation District	(814) 763-5269
Cumberland Conservation District	(717) 240-7812
Dauphin Conservation District	(717) 921-8100
Delaware Conservation District	(610) 892-9484
Elk Conservation District	(814) 776-5373
Erie Conservation District	(814) 825-6403
Fayette Conservation District	(724) 438-4497
Forest Conservation District	(814) 755-3450
Franklin Conservation District	(717) 264-5499
Fulton Conservation District	(717) 485-3547
Greene Conservation District	(724) 852-5278
Huntingdon Conservation District	(814) 627-1627
Indiana Conservation District	(724) 471-4751
Jefferson Conservation District	(814) 849-7463
Juniata Conservation District	(717) 436-8953 ext. 5
Lackawanna Conservation District	(570) 281-9495
Lancaster Conservation District	(717) 299-5361
Lawrence Conservation District	(724) 652-4512
Lebanon Conservation District	(717) 272-3908 ext. 4
Lehigh Conservation District	(610) 391-9583
Luzerne Conservation District	(570) 674-7991
Lycoming Conservation District	(570) 433-3003
McKean Conservation District	(814) 887-4001
Mercer Conservation District	(724) 662-2242
Mifflin Conservation District	(717) 248-4695
Monroe Conservation District	(570) 629-3060
Montgomery Conservation District	(610) 489-4506
Montour Conservation District	(570) 271-1140
Northampton Conservation District	(610) 746-1971
Northumberland Conservation District	(570) 286-7114 ext. 4
Perry Conservation District	(717) 582-8988 ext. 4
Pike Conservation District	(570) 226-8220
Potter Conservation District	(814) 274-8411 ext. 4
Schuylkill Conservation District	(570) 622-3742 ext. 5
Snyder Consdervation District	(570) 837-3000
Somerset Conservation District	(814) 445-4652 ext. 5
Sullivan Conservation District	(570) 928-7057
Susquehanna Conservation District	(570) 278-4600 ext. 280
Tioga Conservation District	(570) 724-1801 ext. 3
Union Conservation District	(570) 524-3860
Venango Conservation District	(814) 676-2832
Warren Conservation District	(814) 726-1441
Washington Conservation District	(724) 228-6774
Wayne Conservation District	(570) 253-0930
Westmoreland Conservation District	(724) 837-5271
Wyoming Conservation District	(570) 836-2589
York Conservation District	(717) 840-7430

CHAPTER 12, APPENDIX E

GUIDANCE ON CHAPTER 102 REQUIREMENTS FOR ROAD MAINTENANCE ACTIVITIES

12E.0 PURPOSE

Why is this guidance necessary?

PennDOT regularly conducts maintenance on its roads. Some of the activities associated with maintenance are minor in nature, while other activities are much larger in scope. The purpose of this document is to demonstrate the types of projects that meet the criteria for roadway maintenance activities as defined in 25 PA Code Chapter 102 (Chapter 102). Earth disturbances meeting these criteria are exempted from National Pollutant Discharge Elimination System (NPDES) permits for Stormwater Discharges Associated with Construction Activities and can total up to 2.0 ha (25 acres) before requiring a Chapter 102 Erosion and Sediment Control Permit (ESCP). Guidance is also provided on calculating the area of disturbance for bridge and culvert projects.

How should this guidance be used?

The guidance provided herein is useful in the project planning, scoping, and preliminary engineering stages of project development.

In the planning stage, a number of factors are considered when estimating a programmed cost for a project. Right-of-way need is chief among these factors, as stormwater facilities may require right-of-way or easement acquisition. The stormwater permitting and design effort for a project is heavily dependent upon the characterization of the construction activities. Road maintenance activities are exempt from NPDES permits and, in general, require minimal stormwater design effort. The technical scope of work and overall project cost can be more accurately defined with this knowledge.

In preliminary engineering, the project should be evaluated as a whole in terms of the proposed project footprint and associated construction activities. The determination made during scoping of the need for an NPDES permit should be verified before Final Design begins.

12E.1 INTRODUCTION

PennDOT performs road maintenance activities (RMAs) in order to keep the state's highway system operable and safe. Structural highway components, including pavement, stormwater drainage features, and bridges, deteriorate over time due to physical wear and weathering. The purpose of maintenance is to maintain a roadway or bridge at a functional condition. As a result, the characteristics of stormwater runoff before and after maintenance are unchanged. This fact is recognized in state and federal regulations, which provide exemptions for most RMAs from NPDES permitting. The sections that follow define and provide examples of RMAs.

12E.2 DEFINITIONS

Road Maintenance Activities

The following definition is from Pa Code Title 25 Section 102.1.

> *(i) Earth disturbance activities within the existing road cross-section or railroad right-of-way including the following:*
>
> *(A) Shaping or re-stabilizing unpaved roads*
>
> *(B) Shoulder grading.*
>
> *(C) Slope stabilization.*
>
> *(D) Cutting of existing cut slopes.*
>
> *(E) Inlet and endwall cleaning.*
>
> *(F) Reshaping and cleaning drainage ditches and swales.*
>
> *(G) Pipe cleaning.*
>
> *(H) Pipe replacement.*
>
> *(I) Support activities incidental to resurfacing activities such as minor vertical adjustment to meet grade of resurfaced area.*
>
> *(J) Ballast cleaning.*
>
> *(K) Laying additional ballast.*
>
> *(L) Replacing ballast, ties and rails.*
>
> *(M) Other similar activities.*
>
> *(ii) The existing road cross-section consists of the original graded area between the existing toes of fill slopes and tops of cut slopes on either side of the road and any associated drainage features.*

The Chapter 102 definition only encompasses those activities which create an earth disturbance. Many RMAs performed by PennDOT do not disturb earth, so they are not included in disturbed area calculations. This document describes both RMAs that disturb earth and RMAs that do not. In this document, activities which meet the Chapter 102 definition of Road Maintenance Activities will be referred to as Chapter 102 RMAs.

Existing Road Cross-Section

As noted above, the term "existing road cross-section" is defined in Chapter 102 under the definition for Road Maintenance Activities. The top of cut slopes and toe of fill slopes defines the outer limit of the road cross-section, unless a drainage feature extends beyond the fill slope or runs parallel to the cut or fill slope. Maintenance from the source to the outlet of drainage features is a Chapter 102 RMA. All other earth disturbance occurring within the road cross-section is a Chapter 102 RMA.

Earth Disturbance *(from Chapter 102)*

> *A construction or other human activity which disturbs the surface of the land, including land clearing and grubbing, grading, excavations, embankments, land development, agricultural plowing or tilling, operation of animal heavy use areas, timber harvesting activities, road maintenance activities, oil and gas activities, well drilling, mineral extraction, and the moving, depositing, stockpiling, or storing of soil, rock or earth materials.*

For the purposes of calculating the amount of disturbed area on a project, milling above the subbase layer and paving on top of existing pavement are RMAs, but they are not earth disturbance activities.

Figure 12E.1 is an illustration of a typical road cross section. The existing road cross-section extends from the left of the top of cut ditch to the right of the toe of fill ditch. If neither drainage feature was present, the road cross-section would be the top of cut and toe of fill. Any earth disturbance between the road cross section and the right-of-way is not a Chapter 102 RMA; therefore, it is included in the calculation of disturbed area for the NPDES permit.

Figure 12E.1 Existing Road Cross Section - Two Way Highway

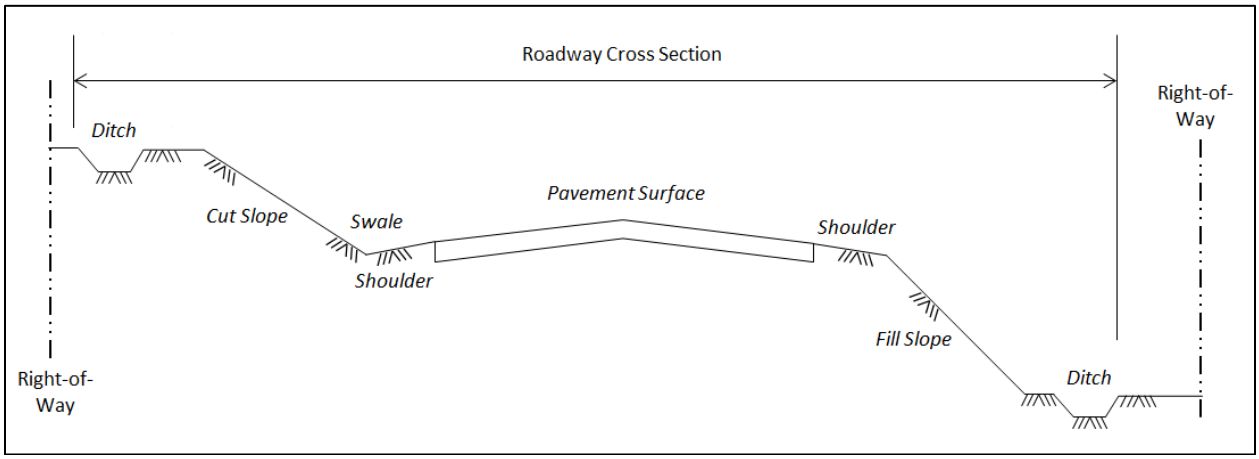
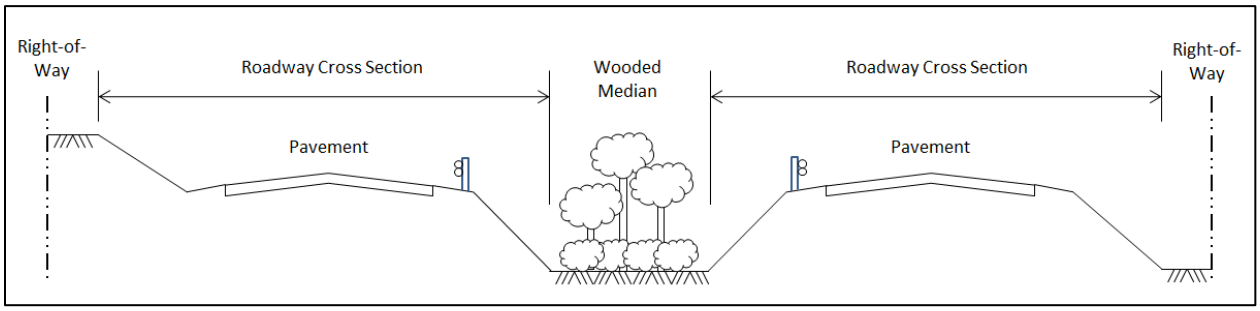


Figure 12E.2 illustrates a divided highway, where roadway has been constructed on either side of a naturally vegetated median area. These types of highways sections are common along interstate highways. Except for maintaining drainage facilities, any earth disturbance within the median area is not a Chapter 102 RMA and would be included in the disturbed area calculation for the NPDES permit.

Figure 12E.2 Existing Road Cross Section - Divided Highway



Median areas can also be highly disturbed and an integral part of the drainage system. Figure 12E.3 depicts a divided highway with a graded and vegetated median. Chapter 102 RMAs are permitted within the disturbed vegetated median (e.g., channel cleaning) because it is part of the existing road cross section.

In curbed roadway sections, the existing road cross section is defined as the area in between (and including) the curbs. Sidewalks are not part of the cross section. A typical curbed roadway section is shown in Figure 12E.4.

Figure 12E.3 Existing Road Cross Section - Divided Highway

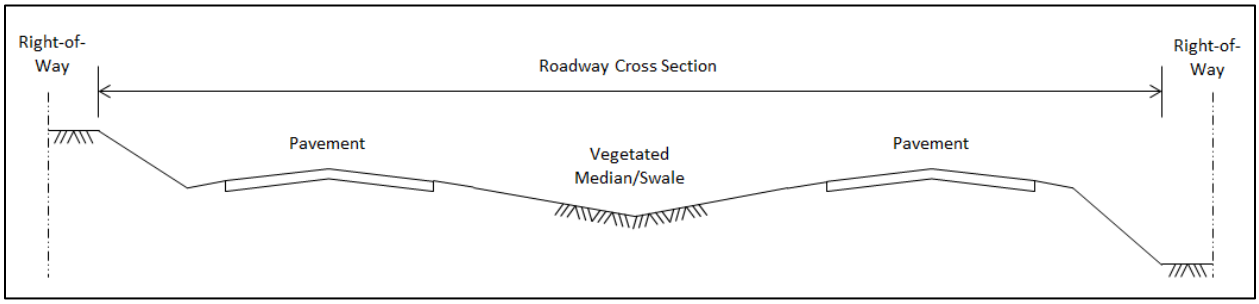
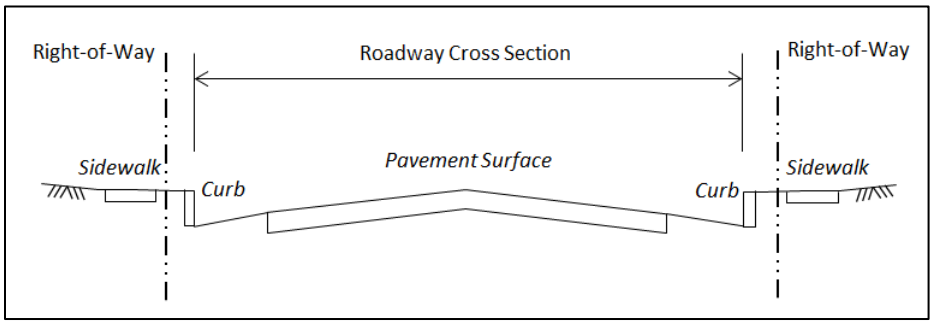


Figure 12E.4 Existing Road Cross Sections - Curbed



12E.3 EXAMPLES OF ROAD MAINTENANCE ACTIVITIES

Road maintenance activities that PennDOT regularly conducts include all of the following examples. Not all of these RMAs meet the Chapter 102 definition, which requires that the work be (1) a maintenance activity, (2) involve earth disturbance, and (3) occur within the existing road cross section (with the exception of drainage features). RMAs that do not cause earth disturbance are noted in the description of the activity.

a. Shaping or re-stabilizing unpaved roads – Maintenance of dirt and gravel roads involves periodic re-shaping of the road surface to ensure proper drainage and traffic passage by re-establishing crown and proper cross-slope; and incorporating loose stones back into the road surface (see Figure 12E.5).

Figure 12E.5 Displaced Course Stones at Edge of Road



b. Shoulder cutting and grading – Shoulder cutting (Figure 12E.6) removes excess soil and debris from unpaved shoulder areas. This improves drainage and allows water to leave the roadway. This process requires equipment such as graders, trucks, brooms, belt loaders or wheel loaders, and rollers. The grader cuts excess material from the shoulder and places it in a pile to be picked up by a loader. It is deposited into a truck to haul to a fill site. The roller follows the loading operation to compact or stabilize the exposed soil. The final piece of the operation is sweeping to clean excess material remaining on the roadway.

Figure 12E.6 Shoulder Cutting



Shoulder grading involves the shaping and stabilizing of unpaved roadway shoulder areas. A shoulder grading crew utilizes graders, dump trucks, a belt loader, a roller and usually a street sweeper. PennDOT grades shoulders to eliminate the drop-off between the roadway and the shoulder and to allow water to drain away from the road surface. If ruts are allowed to form and remain on the shoulder, water can enter and damage the edge of the pavement. The outside edge of the shoulder is shaped toward the road. The loosened soil is then spread over the shoulder to fill low areas and ruts. High areas are then cut with another grader. Any excess soil is picked up by the belt loader and loaded into trucks for use at another site. Finally, the shoulder is paved or the soil is compacted with a roller and the pavement is swept.

Converting a gravel shoulder to a paved shoulder is considered a Chapter 102 RMA, whereas converting a vegetated shoulder into a gravel or paved shoulder is not – it is new construction. Similarly, conversion of a vegetated shoulder into paved shoulder for temporary traffic control is not a Chapter 102 RMA, even if the temporary shoulder area is restored back to a vegetated shoulder. Permanent stormwater management controls are not required for areas that are paved during construction and then revegetated.

c. Slope stabilization – Rills and gullies form on roadway embankments that are not adequately protected from erosion. Unmanaged offsite runoff, inadequate drainage, poor soil conditions, and other factors can create conditions in which erosion of roadway slopes occur. Slope stabilization can be limited to repairing small problem areas, or it can involve the removal of the existing plant cover and replacing it with a more stable, long-term protective plant cover.

d. Cutting of existing cut slopes – This type of activity may be necessary when loose rocks fall into the swale or road or when large quantities of cut material slide onto the road. Steep slopes may be cut back at a more gradual grade to reduce the potential for soil or rock loss. The disturbed area outside of the original cut slopes must be separated from the calculated Chapter 102 RMA area, as it is subject to NPDES requirements if the area is greater than 0.4 ha (1 acre).

e. Inlet and endwall cleaning – The inlet and outlet ends of a culvert can accumulate debris and sediment, which reduces the hydraulic capacity of the culvert. Removal of debris and sediment is a Chapter 102 RMA.

f. Reshaping and cleaning drainage ditches and swales – Drainage channels can erode when channel lining is improperly designed or constructed. Channels can also accumulate sediment and anti-skid material if channel gradients are not sufficient to transport the particles. These situations require maintenance. Ditch cleaning is usually done using a grader or backhoe. Reshaping is not meant to increase the channel capacity beyond the original design. This work is a Chapter 102 RMA even if the drainage features are located beyond the original top of cut and toe of fill limits. The replacement of or installation of new channel outlet protection is also a Chapter 102 RMA.

g. Pipe cleaning – Stormwater pipes can collect sediment, which reduces the hydraulic capacity of the drainage system. Pipes are cleaned by flushing the system with water and collecting the sediment at the outlet. Most pipes are cleaned with a high-velocity sewer cleaner. This RMA is *not* an earth disturbance.

h. Pipe replacement – Controlling water flow is one of the most important aspects of maintaining pavements. Uncontrolled water damages both the pavement surface as well as the area under the pavement, causing deterioration. Typically, pipe is installed prior to repaving or sealing the pavement surface. Pipe and culvert replacement operations consist of cutting or sawing pavements, creating a trench in the subgrade, installing the pipe, filling the trench, compacting the fill material and replacing the pavement surface. Figure 12E.7 depicts a PennDOT pipe replacement operation. Pipe replacement can be an individual activity, such as replacing a collapsed pipe, or it can be part of a larger project, such as a drainage improvement project. This work is a Chapter 102 RMA even if the drainage features are located beyond the original top of cut and toe of fill limits. Replacing stormwater inlets and replacing existing or installing new pipe or culvert outlet protection are also Chapter 102 RMAs.

Figure 12E.7 Pipe Replacement



i. Shoulder backup (i.e., Support activities incidental to resurfacing activities such as minor vertical adjustment to meet grade or resurfaced area) – When a road is resurfaced by overlaying, the finished road surface may be higher than the original. The elevation difference results in a drop-off at the edge of the pavement section. Shoulder backup material is placed and compacted to prevent deterioration of the shoulder and accelerated erosion from concentrated flow at the base of the drop-off. Backup material may or may not be seeded. Images before and after shoulder backup material is placed are shown in Figure 12E.8. The placement of shoulder backup material is a Chapter 102 RMA.

Figure 12E.8 Overlay Before Backup (Left); Overlay After Backup (Right)



j. Mill and pave (overlay) – Mill and overlay is a common process used to extend the life of existing roadway pavements. During a mill and overlay project, the existing bituminous pavement is milled (as in Figure 12E.9) and removed from the site. Then new bituminous pavement courses are placed to the required thickness. Mill and overlay is *not* an earth disturbance activity.

Figure 12E.9 Milling Operation



k. Manual patching – Manual patching, as depicted in Figure 12E.10, is most commonly known as pothole patching. Crews patch potholes during the winter using a cold patch. Cold patch is asphalt mixed with soap, water and aggregate. The cold patch material is simply placed into the pothole with a shovel and then compacted. Although this is designed to be a temporary repair, cold patch jobs can last two or more years. Hot mix is a permanent patch for a pothole which requires a mixture of pure asphalt and aggregate heated to about 300 degrees Fahrenheit. The pothole is cut square, cleaned and then treated with a tack-coat of asphalt that acts like glue. After the tack-coat application, the hot mix is placed into the pothole and compacted using a roller or other device. Manual patching is *not* an earth disturbance activity.

Figure 12E.10 Manual Patching



l. Mechanized patching – Mechanized patching, as depicted in Figure 12E.11, is used to patch roads with extensive pothole damage or with large areas of cracked pavement and depressions. It also improves the smoothness of the road surface. Mechanized patching involves six specific steps - marking the area that needs patched, cleaning the area using a street cleaning broom, filling any large holes or low areas with patching material, compacting the material, applying a course of asphalt over the marked area, and compacting the area with a roller. Mechanized patching is *not* an earth disturbance activity.

Figure 12E.11 Mechanized Patching



m. Surface treatment – Commonly known as "oil and chip," surface treatment is used as a way to extend the life of low-traffic-volume roads for another three to five years. It seals the road surface to keep water out and restores the friction of the surface to enhance traction. First, the roadway is swept and the hot liquid asphalt is sprayed on the road. Then, fine stones are spread on top of the asphalt. Finally, the area is rolled and swept again to remove loose stones. It generally takes about two days for the stones to fully bond in the hardened asphalt. This RMA is *not* an earth disturbance. Figure 12E.12 depicts a surface treatment operation.

Figure 12E.12 Surface Treatment



n. Reconstruction – Reconstruction maintains the function of the road by replacing most or all of the pavement section. It involves one of three methods – remove and replace; cracking and seating; and rubblizing. Removal of the entire pavement section down to the subgrade, followed by construction of the new pavement section, is one method. Undercutting and backfilling the subgrade may be necessary to correct poor subgrade conditions. This method of reconstruction is a Chapter 102 RMA. The crack-and-seat method reduces the pavement to 18-inch pieces and rolls them into the base or subbase courses to seat them before applying an overlay. Rubblizing involves breaking the existing concrete pavement into 8 to 12 inch pieces, rolling them into the base or subbase courses, and placing an overlay. Cracking and seating and rubblizing are *not* earth disturbance activities.

o. Guide rail installation – Replacing damaged guide rail and resetting guide rail are Chapter 102 RMAs. Installation of new guide rail is not a Chapter 102 RMA. Guide rail installation is an earth disturbance. For purposes of calculating an area of disturbance, only the earthwork at the guide rail termini and along the shoulder (if additional material is placed) must be quantified and included in the NPDES disturbed area calculation.

p. Bridge replacement – Replacing or rehabilitating a bridge superstructure is by itself not an earth disturbance activity. Replacing bridge abutments, foundations, and approach slabs are earth disturbance activities and may be Chapter 102 RMAs, depending on the scope of the project. The portions of a typical bridge replacement project that would not be considered Chapter 102 RMAs include: widening of the approaches to accommodate a larger bridge; changing the vertical or horizontal roadway alignment; and construction of temporary stream crossings or access roads. Section 12E.4 explains how disturbed areas associated with bridge projects are calculated for NPDES permit purposes.

q. Temporary stockpiles – Stockpiles of stone, topsoil, and other materials are common on construction sites. The area occupied by the stockpile is considered earth disturbance. Stockpiling is not a Chapter 102 RMA, so this area would be included in the NPDES disturbed area calculation.

r. Stormwater management facilities – Maintenance of stormwater management facilities, or best management practices (BMPs), is a Chapter 102 RMA. Post Construction Stormwater Management Plans contain instructions for the inspection, maintenance and repair of BMPs. Maintenance items include removing debris and sediment, removing dead and invasive plants, replenishing mulch, replacing engineered soil, repairing rutted or eroded soil, and many others.

s. Wasting excess fill – Removing material from one area of the site and dumping it in another is not a Chapter 102 RMA and would be included in the NPDES disturbed area calculation. Wasting excess fill down the side of an embankment is not an acceptable practice under any circumstances, as the material is not protected from erosion. Figure 12E.13 shows an example of improper disposal of excess fill over an embankment, which is also adjacent to a stream. This particular project was supposed to involve only

RMAs; however, improper wasting of excess fill would be in violation of multiple state regulations as a result of the improper dumping.

Figure 12E.13 Excavated Material Improperly Dumped over Embankment



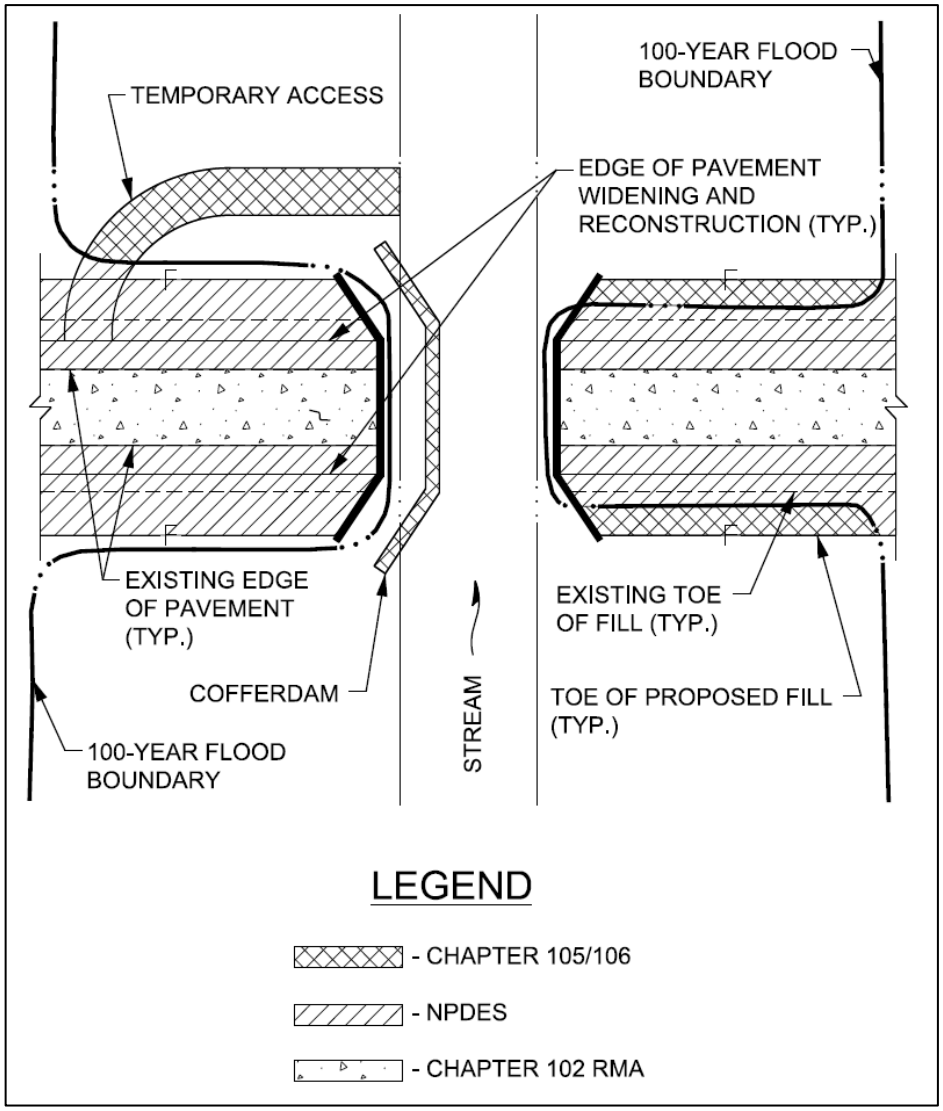
12E.4 CONSTRUCTION IN FLOODPLAINS

Maintenance activities that occur within the 100-year floodplain are given special attention in this document because of how they are counted in determining disturbed acreage for NPDES permit purposes. Earth disturbance activities that are authorized under Section 404 of the Clean Water Act (CWA) are exempt from needing to also obtain NPDES permit coverage. Section 404 authorizes the discharge of fill material into waters of the US, and it also requires a Section 401 (of the CWA) water quality certification. If a Section 401 water quality certification is required, a Pennsylvania (PA Code, Title 25) Chapter 105 permit is also required. The 100-year floodplain (because of PA Code, Title 25, Chapter 106) defines the outer limit of authorization for the Chapter 105 permit; therefore, earth disturbance activities within the 100-year floodplain are not counted towards the 1.0-acre NPDES threshold. This includes work within the stream channel, but does not include areas that naturally drain away from the stream channel and into another watershed.

The 100-year flood boundary shall be taken as the boundary delineated on Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRMs). In areas where no detailed FEMA study has been established, the 100-year flood boundary will be that which is established in the hydrologic and hydraulic study that is prepared for the project

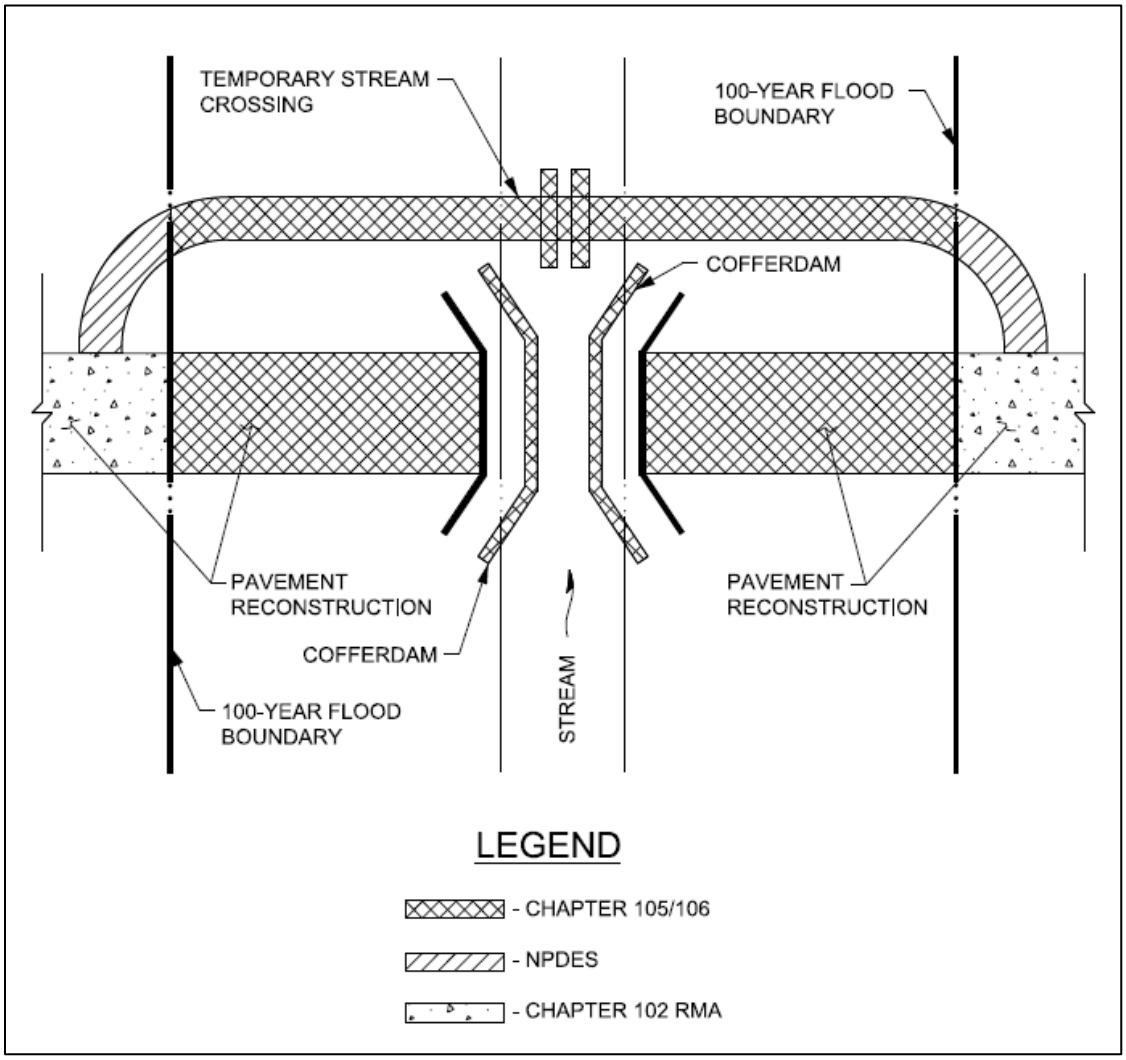
The following two bridge replacement scenarios illustrate which areas must be counted for NPDES permit purposes, and which areas are authorized under a Chapter 105 permit. Figure 12E.14 (Bridge Scenario A) depicts a bridge replacement project with reconstruction and widening of the roadway approaches. The roadway approaches are elevated above the 100-year flood, so the flood boundary is contracted through the bridge opening. A portion of the new road embankment fill on the right side is excluded from the NPDES permit calculation of disturbed acreage because the 100-year flood elevation is above the toe of the fill slopes. The cofferdam and the portion of the temporary access road within in the 100-year flood boundary are not counted towards the 1.0-acre NPDES threshold. The paved footprint of the existing approaches would also be excluded. Removal of the bridge deck itself is *not* an earth disturbance; therefore, it is not included in the disturbed acreage calculation.

Figure 12E.14 Bridge Scenario A – Roadway Approaches Above 100-Year Floodplain



Bridge Scenario B in Figure 12E.15 depicts a project where most of the project area is inundated by a 100-year flood event. All earth disturbance within the 100-year flood boundary, regardless of whether it is a Chapter 102 RMA, is excluded from the NPDES disturbed area calculation. The pavement reconstruction outside of the 100-year flood boundary is also excluded because it meets the definition of a Chapter 102 RMA. In this example, the only area of the project that would be included in the NPDES disturbed area calculation is the portion of the temporary stream crossing that is outside of the flood boundary.

Figure 12E.15 Bridge Scenario B – Roadway Approaches Below 100-Year Floodplain



12E.5 COMPOSITE PROJECTS AND CALCULATIONS

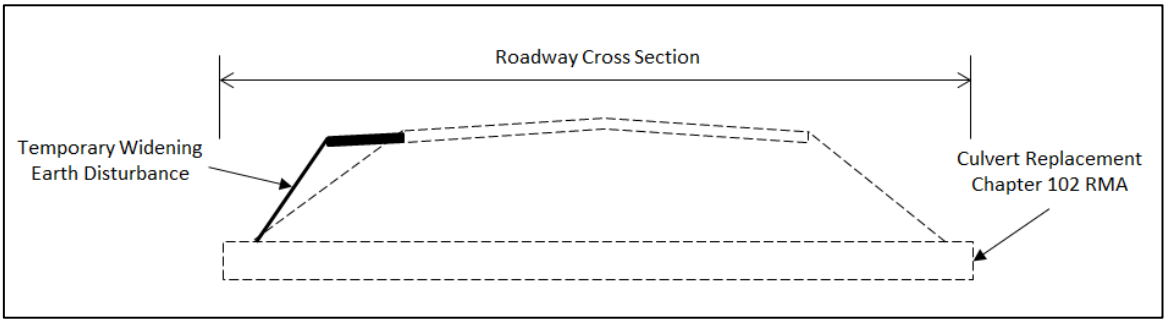
It is not uncommon for a project to include more than one of the RMAs described in Section 12E.4. As long as the total disturbed area from the RMAs does not exceed 25 acres, a Chapter 102 ESCP (permit) is not required. Projects may also include activities that are not identified in Section 12E.4; these should be separated from the Chapter 102 RMAs when looking at permit requirements. If the disturbed area on a project from all non-RMAs exceeds 0.4 ha (1 acre), an NPDES permit is required. All earth disturbances, regardless of area or classification, require implementation of proper erosion and sediment (E&S) controls per Chapter 102. For all Department force maintenance activities, follow the E&S BMPs defined in Publication 464, *Maintenance Field Reference for Erosion and Sedimentation Controls*. A written E&S plan is required for any earth disturbance in a High Quality or Exceptional Value watershed, and for earth disturbances exceeding 5,000 ft^2 in all other watersheds.

Several examples of projects are presented in this section. While these examples do not represent every possible combination of activities on a project, they serve as guidelines for evaluating and determining permit requirements for other projects.

a. Example 1 – A resurfacing project involves milling and paving 1.6 km (1 mi) of a two-lane road. Several small culverts will be replaced. Temporary widening will be constructed in the areas of the culverts to maintain two-way traffic through the work areas. A schematic of the proposed widening is shown in Figure 12E.16.

- Chapter 102 RMAs: culvert excavation and backfilling
- Non-exempt earth disturbance: temporary pavement construction (and removal, if applicable)

Figure 12E.16 Temporary Widening Schematic



The total area of earth disturbance before excluding Chapter 102 RMAs is the sum of the culvert, temporary pavement, and temporary fill areas. Since the temporary pavement construction is outside of the existing road cross-section, only the culvert replacements are considered Chapter 102 RMAs; these would obviously amount to less than 2.0 ha (25 acres) of disturbance. The temporary pavement construction must result in less than 0.4 ha (1 acre) of disturbance; otherwise, an NPDES permit would be required.

b. Example 2 – A rural roadway project includes 1.6 km (1 mi) of full-depth reconstruction of a two-lane road. The existing 0.6 m (2 ft) wide shoulders will be replaced with 1.8 m (6 ft) wide shoulders. All drainage systems, including ones that extend beyond PennDOT's legal right-of-way into municipal right-of-way, will be replaced.

- Chapter 102 RMAs: full-depth reconstruction within the existing paved footprint, replacing drainage systems
- Non-exempt earth disturbance: construction of the additional shoulder width and any associated grading

Reconstruction of existing pavement and replacing existing drainage systems are Chapter 102 RMAs; however, widening for safety, capacity, or any other reason is not. The new shoulder areas and associated grading outside of the existing pavement footprint would be disturbed area that is counted towards the 0.4 ha (1 acre) threshold for an NPDES permit. Stormwater pipes that are located outside of the roadway cross-section may be replaced as a Chapter 102 RMA as long as the primary source of runoff in the drainage system is the road and the same trench is used to remove the existing and place the new pipes.

c. Example 3 – A diamond interchange reconstruction and safety improvement project involves replacing the existing ramps (full-depth), extending acceleration and deceleration lanes, replacing existing guide rail and installing new guide rail, and replacing and reconfiguring (between ramps) the drainage facilities.

- Chapter 102 RMAs: full-depth reconstruction within the existing paved footprint, replacement of drainage systems

- Non-exempt earth disturbance: construction of the acceleration and deceleration lane extensions, grading necessary to install new guide rail, construction of any new or offline drainage

Replacing the existing guide rail is not a Chapter 102 RMA because it is not an earth disturbance. The grading associated with the installation of new guide rail is not a Chapter 102 RMA and would need to be included in the NPDES earth disturbance calculation.

d. Example 4 – A bridge replacement project involves removing and constructing new bridge abutments and a bridge deck over a stream. Approximately 91.4 m (300 ft) of roadway approaches on each side of the bridge will be reconstructed and made slightly wider due to an increase in the bridge width. Guide rail is being replaced and extended about 30.5 m (100 ft) in each direction as well. A temporary stabilized road, such as the one shown in Figure 12E.17, will be constructed from one of the approaches down to the stream for contractor access. The 100-year floodplain extends several hundred feet from the edges of the stream channel, but it does not overtop the roadway or bridge.

Figure 12E.17 Temporary Road for Stream Access



- Chapter 102 RMAs: full-depth reconstruction within the existing pavement footprint
- Non-exempt earth disturbance: roadway construction outside of the existing pavement footprint, temporary road construction

The earth disturbance within the 100-year floodplain is excluded from the NPDES disturbed area calculation because it is covered by another federal water quality permit. Reconstruction within the existing paved footprint (edge of shoulder to edge of shoulder) is a Chapter 102 RMA; all paving outside of the footprint and outside of the 100-year floodplain is counted towards the 0.4 ha (1 acre) NPDES threshold. The temporary road is most likely located within the floodplain and, if so, would not be added to the NPDES earth disturbance calculation.

e. Example 5 – A 1.6 km (1 mi) long urban 3R (resurfacing, restoration and rehabilitation) project involves milling and paving, correcting substandard cross slopes, replacing and installing new curb, replacing and installing new sidewalk, and replacing stormwater cross pipes.

- Chapter 102 RMAs: correcting substandard pavement cross slopes, curb replacement, stormwater cross pipe replacement
- Non-exempt earth disturbance: new curb installation, new and replacement sidewalk construction

The curb defines the edge of the existing road cross section; therefore, replacing the curb is a Chapter 102 RMA. Installing new curb where none exists is not a Chapter 102 RMA. Replacing and installing new sidewalk is not a Chapter 102 RMA because it is outside of the existing cross section.

f. Example 6 – A reconstruction project involves replacing the existing pavement section and shoulder widening in both directions on a divided highway. The roadway is divided by a 12.2 m (40 ft) wide grass median, which collects runoff in swale in the center of the median. The inside shoulders will increase in width from 1.8 m (6 ft) to 3.0 m (10 ft), or 1.2 m (4 ft) of widening into the grass median. The outside shoulders will be paved to a total width of 2.4 m (8 ft); however, the outside shoulder widening will occur over an existing compacted gravel area, which essentially functions as an extension of the existing shoulder.

- Chapter 102 RMAs: Full-depth reconstruction within the existing paved footprint, pavement widening into existing gravel shoulder area
- Non-exempt earth disturbance: Shoulder widening into grass median

The entire median area is considered part of the existing road cross section because it was constructed when the road was built and it serves an essential drainage function. However, the pavement footprint ends at the existing inside edge of shoulder, and any widening into the grass median is not a Chapter 102 RMA. This area would be counted towards the 1.6 km (1 acre) NPDES threshold. However, the pavement footprint on the outside edges of the highway extend to the outside of the compacted gravel area. Constructing bituminous or concrete shoulders in this area is a Chapter 102 RMA because the existing area is effectively impervious and provides the same function as a shoulder.

12E.6 SUMMARY

Roadway maintenance is essential to the preservation of state highways and bridges. The construction activities that create earth disturbance and that are necessary to maintain the functionality of the road are defined by 25 PA Code Chapter 102 as Road Maintenance Activities (RMAs). The preceding information on PennDOT RMAs should serve as a guide for determining a project's permit requirements related to earth disturbance. Permit requirements should be determined as early as possible in the project planning and development process. Projects often include a combination of Chapter 102 RMAs and non-Chapter 102 RMAs; each activity should be delineated and its area should be calculated. In order to avoid any type of earth disturbance permitting for a project, the sum of all RMA areas must be less than 2.0 ha (25 acres), and the sum of all non-Chapter 102 RMA areas must be less than 0.4 ha (1 acre). Any earth disturbance located within the 100-year floodplain and draining to that body of water is authorized by the Section 401 Water Quality Certification issued with the Water Obstruction and Encroachment Permit for that project and, therefore, does not require authorization under an NPDES permit.

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CHAPTER 13

STORM DRAINAGE SYSTEMS

13.0 OVERVIEW

A. Introduction. This chapter provides guidance on storm drain design and analysis. The quality of a final in-place system depends upon careful attention given to every aspect of the design. The quality of construction and level of maintenance also greatly affects the function of the system. Most aspects of storm drain design (e.g., system planning, pavement drainage, gutter flow calculations, inlet spacing, pipe sizing, hydraulic grade line calculations) are included.

The design of a drainage system must address the needs of the traveling public and 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 water courses and absence of side channels.
- The more costly property impacts damages that 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 may interfere with or possibly halt the passage of highway traffic.

B. Inadequate Drainage. The most serious effects of an inadequate roadway drainage system are:

- Impact to surrounding or adjacent property, resulting from water overflowing the roadway curbs 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.

13.1 POLICY AND GUIDELINES

A. Introduction. Highway storm drainage facilities collect stormwater runoff and convey it through the roadway right-of-way in a manner that 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, inlets, pipes, ditches, channels and culverts. The placement and hydraulic capacities of storm drainage facilities should be designed to secure as low a degree of risk of traffic interruption by flooding as is consistent with the importance of the road and the design traffic service requirements, with minimization of the potential for damage to adjacent property.

Following is a summary of policies that should be followed for storm drain design and analysis. For additional technical information, refer to *Highway Drainage Guidelines*, Chapter 9 (AASHTO, 2003), HEC-21 (FHWA, 1993) and HEC-22 (FHWA, 2001).

B. Bridge Decks. Zero gradients, sag vertical curves and superelevation transitions with flat pavement sections should be avoided where possible on bridges. Publication 15M, Design Manual, Part 4, *Structures*, PP3.2.3 discusses the policy for bridge deck drainage. Many short, single-span bridges do not require drainage facilities. Where drainage facilities are needed, the preferred design is to capture the runoff with inlets at the end(s) of the bridge. Quantity and quality of runoff should be maintained as required by applicable stormwater regulations. Publication 15M, Design Manual, Part 4, *Structures*, and Section 13.4.G. discusses the maximum length of deck permitted without drainage facilities.

C. Curbs and Inlets. Curbs and inlets are used where runoff from the pavement would erode fill slopes and/or to reduce right-of-way needed for shoulders or channels. Where the combination of inlets and storm pipes are necessary, pavement sections are often curbed.

D. Design Frequency. The flow for storm drainage systems is based upon design frequency, as defined by the return period (in years), and is influenced by the allowable water spread on the pavement and the design speed of the roadway. This design criteria is included in Section 13.6.

E. Detention Storage. Reduction of peak flows can be achieved by the storage of runoff in detention basins, storm drainage pipes, swales and channels and other detention storage facilities. Reduction of peak flows should be considered at locations where existing downstream conveyance facilities are inadequate to handle peak-flow rates from highway storm drainage facilities. In many locations, highway agencies or developers are not permitted to increase runoff over existing conditions. Additional benefits of detention storage include the reduction of downstream pipe sizes and the improvement of water quality by removing sediment and/or pollutants. See Chapter 14, *Post-Construction Stormwater Management*.

F. Gutter Flow Calculations. Gutter flow calculations are necessary to relate the quantity of flow to the spread of water on the shoulder, parking lane or pavement section. Composite gutter sections have a greater hydraulic capacity for normal cross slopes than uniform gutter sections. Refer to Section 13.7 for additional information and procedures.

G. Hydrology. The Rational method is the most common method in use for the design of storm drainage systems when the momentary peak-flow rate is desired. Its use should be limited to systems with drainage areas of 80 ha (200 acres) or less. The minimum acceptable time of concentration is five minutes. Drainage systems involving detention storage, pumping stations and large or complex storm systems require the development of a runoff hydrograph. The Rational method is briefly described in Section 13.3, and both the Rational method and Hydrograph methods are described in Chapter 7, *Hydrology*.

H. Hydroplaning. See Section 13.5 for design criteria related to hydroplaning.

I. Inlets. The term "inlets" refers to all types of inlets (e.g., grate inlets, curb inlets, slotted inlets). Drainage inlets are sized and located to limit the spread of water on traffic lanes to tolerable widths for the design storm in accordance with the design criteria specified in Section 13.8. The width of water spread on the pavement at sags should not be substantially greater than the width of spread encountered on continuous grades.

Grate inlets and depression of curb opening inlets should be located outside the through traffic lanes to minimize the shifting of vehicles attempting to avoid them. All grate inlets shall be bicycle safe in those areas where bicycle traffic (or wheelchairs) are anticipated. Type C curb inlets are preferred to Type M or Type S inlets at major sag locations because of the debris handling capabilities, provided by the 75 mm (3 in) throat in the curb section.

In locations where significant ponding may occur (e.g., at underpasses or sag vertical curves in depressed sections), recommended practice is to place flanking inlets on each side of the inlet at the low point in the sag (see Section 13.9.A). See Section 13.9.A for information pertaining to the location of inlets. Publication 72M, *Roadway Construction Standards*, RC-45M and RC-46M should be referenced for Inlet design standards.

J. Manholes. The maximum spacing of access structures, whether manholes, junction boxes or inlets, should be as specified in Section 13.10.B. Refer to Section 13.10 for additional manhole criteria.

K. Pavement Drainage. To facilitate flow along the gutter and minimize ponding, desirable longitudinal gutter grades should not be less than 0.5%. Cross slope considerations are presented in Section 13.4.C. In many locations optimal longitudinal grades are not available for gutters. Ultimately, gutter capacity is a function of allowable spread and the design flow. If gutter capacity is inadequate for a given design flow, then the drainage system design must be modified to meet the design criteria. One option in such a location is to reduce the inlet spacing or reduce the amount of flow bypassing an upstream inlet.

L. Roadside and Median Ditches. Large amounts of runoff shall be intercepted before reaching the highway to minimize the deposition of sediment and other debris on the roadway and to reduce the amount of water that must be carried in the gutter section. Median areas and inside shoulders shall be sloped to a center depression to prevent runoff from the median area from running across the pavement. Surface channels should have adequate capacity for the design runoff and should be located and shaped in a manner that does not present a traffic hazard. Vegetative lining should be used where velocity and shear stress are not prohibitive. Appropriate linings may be necessary

where vegetation will not control erosion. Right-of-way restrictions/costs in urban areas often render roadside ditches impracticable.

M. Storm Pipes. A storm pipe is defined as that portion of the storm drainage system that receives runoff from inlets and conveys the runoff to some point where it is then discharged overland, into a channel, water body or piped system. It consists of one or more pipes connecting two or more inlets. A storm drain may be a closed-conduit, open-conduit or some combination of the two. They may be designed with future development in mind, if appropriate. A higher design frequency (or return interval) may be used if desired be used for storm drain systems located in a sag vertical curve to decrease the depth of ponding on the roadway and bridges and potential inundation of adjacent property. Where feasible, the storm pipes shall be designed to avoid existing utilities. Attention shall be given to the storm drain outfalls to ensure that the potential for erosion is minimized. Drainage system design should be coordinated with the proposed staging of large construction projects to maintain an outlet throughout the construction project.

N. System Planning. System planning prior to commencing the design of a storm drain system is essential. The basics required are discussed in Section 13.2 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.

13.2 SYSTEM PLANNING

A. 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 policies associated with that design.

B. General Design Approach. The design of a storm drain system is generally a process that evolves as a project develops. The primary ingredients to this process are listed below in a general sequence by which they may be implemented. This manual does not attempt to name all the participants of this process because it will vary with each agency; however, the Designer will play a major role.

1. Collect data (see Section 13.2.C.).
2. Coordinate with other agencies.
3. Prepare preliminary sketch (Section 13.2.D.).
4. Determine inlet location and spacing (Sections 13.8 and 13.9).
5. Plan layout of storm drain system:
   - Locate main outfall.
   - Determine direction of flow.
   - Locate existing utilities.
   - Locate connecting mains.
   - Locate manholes.
6. Size the pipes (Section 13.11.E).
7. Review hydraulic grade line (Section 13.12).
8. Prepare the plan.
9. Provide documentation (*Model Drainage Manual,* Chapter 4 (AASHTO, 2005)).

C. 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 through storm drainage systems) to some existing outfall location. Furthermore, there should be an understanding of the nature of the outfall because it may have a significant influence on the performance of 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 USGS, NRCS, many municipalities, some county governments and even private developers, are also valuable sources of the data for a proper storm drainage design. MPO/RPOs 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. Act 167 Stormwater Management plans and local flood plain ordinances should be reviewed when available.

D. 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 that 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.

E. Location and Size of Storm Drain. In general, storm drain pipes should not decrease in size in a downstream direction. Section 13.11.E should be further consulted for pipe sizes guidance.

Locate the storm drain to avoid conflicts with utilities, foundations or other obstacles. Minimizing the depth of the storm drain may produce a significant cost savings. Coordination with utility owners during the design phase is necessary to determine if an adjustment to the utilities or the storm drain system is required. The location of the storm drain may affect construction activities and phasing. The storm drain should be located to minimize traffic disruption during construction. Dual trunklines along each side of the roadway may be used in some cases where it is difficult or more costly to install laterals. Temporary drainage may be needed to avoid increases in flood hazards during construction.

F. Outfall Policy. Highway systems may increase peak discharge and volume due to increases in the impervious area and decreases in the time of concentration or lag time. Accumulation or diversion of flow may also result in an increase in stormwater runoff created by the placement of storm drainage system outfalls in locations where concentrated flow did not previously exist. Some local governments may have regulations regarding the management of post-construction stormwater runoff which may relate to the drainage design of highway projects. However, PennDOT is not obligated to adhere to these local regulations concerning stormwater runoff and is only required to comply with those stormwater management provisions enacted by Act 167 stormwater management plans. The following items are common issues related to stormwater management which design professionals should be aware of prior to commencing with the design of drainage systems for highway projects.

- Hydrologic computations should be developed to quantify any increase stormwater runoff caused by highway construction projects and must be prepared to support the design of all new drainage systems. The hydraulic computations shall evaluate those flood frequencies or design events which are prescribed in Chapter 7, *Hydrology* and Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, based upon roadway classification.
- Whenever possible, highway projects should make provisions to limit increases in the peak discharge from storm outfalls to avoid increases in downstream flood hazards and the creation of environmental impacts in downstream receiving waterways.
- Storage facilities shall be designed according to the principles and procedures set forth in Chapter 14, *Post-Construction Stormwater Management*. If detention or other post-construction stormwater management control is not provided to limit the increase in peak discharge, the outfall will be increased in size to accommodate the increase in discharge.
- For streams with detailed Flood Insurance Studies (FIS)/AE-zones, the 100-year floodplain is divided into two zones, the floodway and the floodway fringe. The floodway is that area that must be kept open to convey flood waters downstream. Conversely, the floodway fringe is that area that can be developed in accordance with FEMA standards as adopted in the local law. There is a two-tiered system of technical evaluation for proposed projects in the floodplain. All proposed floodplain projects must meet the "no adverse affect" criteria, while proposed floodway projects must also meet the "no-rise" criteria.

Detailed floodplain and floodway mapping of outfall channels are required if the drainage system increases the water surface elevation as specified above. A detailed floodplain map of the outfall channel and hydraulic computations shall be submitted to the governing planning agency and FEMA, to support any changes within a 100-year floodplain in zones AE.

- During routine maintenance operations highway maintenance personnel should observe and make note of changes in development that may increase stormwater runoff to a highway's drainage system. If a problem is known or anticipated to occur, the District maintenance staff will notify the municipality or property owners that are responsible for the problem so that those owners may correct the problem and restore or maintain the capacity of PennDOT's drainage system. If the cause of the problem is not obvious to District maintenance personnel, the District should initiate a hydraulic study to assess the problem and develop mitigation measures, which are able to resolve problems occurring with the drainage system.
- Publications 170 and 282, and the PA Code Chapters 441 and 459 should be referenced related to Highway Occupancy Permit (HOP) procedures.

A change in the timing of peak discharges from highway projects can result in an increase in the peak discharge to receiving waters. Storage facilities can limit runoff peaks from a highway project to existing conditions at the outfall, but changes in the peak timing can create increases in peak discharge and stage elevation in the receiving waters. Highway construction projects which increase post-construction stormwater runoff can have an impact on downstream structures. Any project that increases post-construction stormwater runoff should complete a hydrologic analysis to insure adequate capacity for combined flows at those structures. Land developments should not increase flows onto PennDOT right-of-ways, and a hydrologic analysis may be required to insure combined flows as discussed above are not increased. Developers and land owners who submit an application for a Highway Occupancy Permit must demonstrate that peak discharge rates at PennDOT's drainage facilities are not increased as a result of the proposed project.

A hydrologic analysis of receiving waters provides useful information in assessing flood hazard impacts due to changes in peak timing. Hydrograph methods are the most reliable when estimating the effects of changes in peak runoff rates to receiving waters with storage facilities located on upstream tributaries. For major storm drain systems, the hydrologic analysis must include the effects of any uncontrolled increases in runoff to the downstream drainage systems.

Stability of outfall channels must be assessed when there are significant changes in discharges from highway projects. Storage facilities may limit the peak discharge to predevelopment conditions for the larger flood frequencies; however, a significant increase in the rate of volume of runoff at lower frequencies (e.g., the 2-year discharge) can significantly impact channel stability. An increase in the 1.1- to 2.5-year frequency discharges can increase the sediment transport rate such that changes in channel stability may occur. Chapter 8, *Open Channels*, provides guidance on channel stability. Publication 13M, Design Manual, Part 2, *Highway Design*, provides guidelines for channel protection measures. Outfall channel stability assessment must be made for significant increases in peak discharge. Channel stability measures must be employed if found to be necessary by the outfall channel stability assessment.

13.3 HYDROLOGY

A. Introduction. The Rational method is the most common method in use for the design of storm drainage systems as it is easy to calculate and provides a momentary peak-flow rate necessary to size pipes and inlets.. Its use should be limited to systems with drainage areas of 80 ha (200 ac) or less. The rational method differs from other hydrologic methods in that it cannot create a hydrograph or determine the volume of stormwater runoff necessary to design storage facilities, pumping stations, and other features which are affected by runoff volume. Both the rational method of calculating an instantaneous peak flow and other hydrologic methods of calculating stormwater runoff hydrographs are described in Chapter 7, *Hydrology*.

B. Rational Method. The Rational Equation is written as follows:

(Equation 13.1)

Metric	**U.S. Customary**
$Q = 0.2755CIA = (\sum CA)\ 0.2755I$	$Q = CIA = (\sum CA)\ I$

where: Q = discharge, m^3/s (cfs)
C = runoff coefficient (see Table 7.6)
I = rainfall intensity, mm/h (in/h) (see PDT-IDF curves in Chapter 7, *Hydrology*)
A = drainage area, km^2 (ac)

1. Runoff Coefficient (C). The runoff coefficients for various types of surfaces are discussed in Chapter 7, *Hydrology*, Section 7.5.E., and Table 7.6 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:

$$\text{Weighted } C = [A_1C_1 + A_2C_2 + A_3C_3] / [A_1 + A_2 + A_3]$$

where: A_1, A_2, C_1, C_2, etc. are the drainage areas and C values of the subareas making up the contributing watershed.

2. Rainfall Intensity (I). Rainfall intensity is the intensity of rainfall in millimeters per hour (in/h) 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 volume of rain; i.e., an intensity of 130 mm/h (5 in/h) for a duration of five minutes indicates a total rainfall volume of (130)(5/60) = 10.8 mm ((5)(5/60) = 0.4 in). See Chapter 7, *Hydrology* for a more complete discussion and data to be used for determining the intensity of rainfall. Rainfall intensities shall be obtained from rainfall intensity duration frequency curves from approved references as listed in Chapter 7, *Hydrology*.

3. Time of Concentration (t_c). 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 should be concerned with two different times of concentration; one for inlet spacing and the other for pipe sizing. There is a major difference between the two times of concentration.

a. 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 to the back of the curb, 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 five minutes, a minimum t_c of five minutes 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. For a constant roadway grade and relatively uniform contributing drainage area, the time of concentration for each succeeding inlet could also be constant.

b. 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. In storm drain applications, time of concentration generally consists of two components: (1) the time to flow to the inlet, which can consist of sheet flow, shallow concentrated flow and channel or gutter flow segments, and (2) the time to flow through the storm drain to the point under consideration.

The sheet flow time of concentration (t_c) segment is typically developed using the kinematic wave approach. Channel and storm drain times of concentration can be developed using Manning's equation or the HEC-22 (FHWA, 2001) triangular gutter approach. Travel time within each component and storm drain pipes can be estimated by the relation:

(Equation 13.2)

$$t_t = L / 60V$$

where: t_t = travel time, minutes
L = distance or length of pipe in which runoff must travel, m (ft)
V = estimated or calculated normal velocity, m/s (ft/s)

All of these methods are further described in Chapter 7, *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 estimate the contributing area, A_c, is as follows:

(Equation 13.3)

$$A_c = A\,[t_{c1} / t_{c2}]$$

Where: $t_{c1} < t_{c2}$, and A is the area of the basin with the longest t_c.

A minimum time of concentration of five minutes is recommended for calculation of runoff.

C. Other Hydrologic Methods. The use of other hydrologic methods may be desirable for the following reasons:

- Provide compatibility with studies of systems adjacent to or connected with the storm drain system.
- As required by government agencies.
- Produce a more detailed analysis.
- Provide a more realistic simulation of detention ponding, channel routing or pipe routing.
- Provide an analysis of peak timing conditions of receiving waters.

For a list of accepted hydrologic methods, the reader should refer to Chapter 7, *Hydrology*.

D. Detention. Estimation of the effects of detention requires a reservoir routing procedure such as that presented in Chapter 14, *Post-Construction Stormwater Management*. By introducing detention ponds, the designer is able to attenuate the peak of the runoff hydrograph, thus reducing the outflow design discharge rate. The same approach for peak-flow attenuation is valid and particularly useful in a storm drainage system in which there are substantial lengths of large diameter pipes. In such systems, the storage capacity of the pipes can have a substantial effect on the final shape of the runoff hydrograph.

13.4 PAVEMENT DRAINAGE

A. Introduction. Roadway features considered during gutter, inlet and pavement drainage calculations include:

- Longitudinal and cross slope.
- Curb and gutter sections.
- Pavement texture/surface roughness.
- Roadside and median ditches.
- Bridge decks.

The pavement width, cross slope, profile and pavement texture control the time it takes for stormwater to drain to the gutter section. The gutter cross section and longitudinal slope control the quantity of flow that can be carried in the gutter section.

B. Longitudinal Slope. A minimum longitudinal gradient is more important for curbed pavement than for uncurbed pavement because it is susceptible to the spread of stormwater against the curb. Flat gradients on uncurbed pavements can also lead to a spread problem if vegetation is allowed to build up along the pavement edge.

C. Cross Slope. Publication 13M, Design Manual, Part 2, *Highway Design* and Publication 14M, Design Manual, Part 4, *Structures,* provide the geometric standards for cross slope for roadway sections and bridges respectively.

The design of pavement cross slope is often a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort. *Pavement and Geometric Design Criteria for Minimizing Hydroplaning* (FHWA, 1984) reports that cross slopes of 2% have little effect on driver effort in steering, especially with power steering or on friction demand for vehicular stability. Use of a cross slope steeper than 2% on pavements with a central crown line is not desirable. Reference should be made to Publication 13M, Design Manual, Part 2, *Highway Design*, Section 1.5.

Median areas should not be drained across traveled lanes. A careful check should be made of designs to minimize the number and length of flat pavement sections in cross slope transition areas, and consideration should be given to increasing cross slopes in sag vertical curves and crest vertical curves and in sections of flat longitudinal grades. Where curbs are used, depressed gutter sections can be effective at increasing gutter capacity and reducing spread on the pavement.

D. Pavement Texture. The pavement texture is an important consideration for roadway surface drainage. Although the designer will have little control over the selection of the pavement type or its texture, it is important to know that pavement texture does have an impact on the buildup of water depth on the pavement during rain storms. A good macrotexture provides a channel for water to escape from the tire/pavement interface and, thus, reduces the potential for hydroplaning.

A high level of macrotexture may be achieved by tining new portland cement concrete pavements while it is still in the plastic state. Re-texturing of an existing portland cement concrete surface can be accomplished through pavement grooving and cold milling. Both longitudinal and transverse grooving are very effective in achieving macrotexture in concrete pavement. Transverse grooving aids in surface runoff resulting in less wet pavement time. Publication 242, *Pavement Policy Manual*, defines PennDOT's pavement policy and is the reference for standards on this subject.

E. Curb and Gutter. Publication 72M, *Roadway Construction Standards,* RC-64M, RC-65M, and RC-67M should be referenced for curb and gutter standards. Curbing at the outside edge of pavements is normal practice for low-speed, urban highway facilities. They serve several purposes that include containing the surface runoff within the roadway and away from adjacent properties, preventing erosion, providing pavement delineation and enabling the orderly development of property adjacent to the roadway. Curbs may be either vertical barrier or mountable type, and they are typically portland cement concrete, although bituminous curb is used occasionally. A curb and gutter forms a triangular channel that can be an efficient hydraulic conveyance facility that can convey runoff of a lesser magnitude than the design flow without interruption of the traffic. When a design storm flow occurs, there is a spread or widening of the conveyed water surface and the water spreads which may include not only the gutter width, but also parking lanes or shoulders and portions of the traveled surface. This is the width the designer is most concerned with in curb and gutter flow, and limiting this width becomes a very important design criterion. This will be discussed in further detail in Section 13.7. Publication 72M, *Roadway Construction Standards,* RC-64M and RC-65M should be referenced for design standards.

Where practicable, it is desirable to intercept runoff from cut slopes and other areas draining toward the roadway before it reaches the highway, to minimize the deposition of sediment and other debris on the roadway and to reduce the amount of water that must be carried in the gutter section. Shallow swale sections at the edge of the roadway pavement or shoulder offer advantages over curbed sections where curbs are not needed for traffic control. These advantages include a lesser hazard to traffic than a vertical curb, and hydraulic capacity that is not dependent on spread on the pavement. These swale sections without curbs are particularly appropriate where curbs have historically been used to prevent water from eroding fill slopes.

F. Roadside and Median Channels. Roadside channels are commonly used with uncurbed roadway sections to convey runoff from the highway pavement and from areas that drain toward the highway. 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. Where practicable, the flow from major areas draining toward curbed highway pavements shall be intercepted by a roadside channel prior to intersecting the roadway.

It is preferable to slope median areas and inside shoulders to a center swale to prevent drainage from the median area from running across the pavement. Flow in the median can be collected by median inlets and conveyed through storm sewer laterals to the outside of the roadway This is particularly important for high-speed facilities and for facilities with more than two lanes of traffic in each direction.

G. Bridge Decks. Publication 15M, Design Manual, Part 4, *Structures*, is PennDOT's standard for bridge decking and should be referenced. Drainage of bridge decks is similar to other curbed roadway sections. It is often less efficient, because cross slopes are flatter, parapets collect large amounts of debris, and small drainage inlets or scuppers have a higher potential for clogging by debris. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried over several spans to the bridge end for disposal or can be carried by scuppers.

The gutter spread should be checked to ensure compliance with the design criteria in Publication 13M, Design Manual, Part 2, *Highway Design*, Publication 15M, Design Manual, Part 4, *Structures*, and Section 13.7. Zero gradients and sag vertical curves should be avoided on bridges. Scuppers are the recommended method of deck drainage because they can reduce the problems of transporting a relatively large concentration of runoff in an area of generally limited right-of-way. They also have a low initial cost and are relatively easy to maintain. However, the use of scuppers should be evaluated for site-specific concerns. Free-fall scuppers should not be located over embankments, slope pavement, slope protection, driving lanes or railroad tracks. Runoff transported to the end of the bridge should generally be collected by inlets and conveyed by storm sewer to a stabilized outlet. Runoff should also be handled in compliance with applicable stormwater quality regulations.

Many bridges will not require any drainage structures at all. To determine the length of deck permitted without drainage structures consult Publication 15M, Design Manual, Part 4, *Structures*, PP3.5.2.7.5

H. Shoulder Gutter and/or Curbs. Although sheet flow is typically the preferred method of conveyance down fill slopes, shoulder gutter and/or curbs may be appropriate to protect fill slopes from erosion in areas where stormwater runoff from the roadway is concentrated. In such cases, gutter and/or curbs can be used to protect fill slopes higher than 6 m (20 ft), with standard slopes of 1V:2H (2H:1V). They may also be used on fill slopes higher than 3 m (10 ft) with standard slopes between 1V:6H (6H:1V) and 1V:3H (3H:1V) if the roadway grade is greater than 2%. In areas where permanent vegetation cannot be established, shoulder gutter and/or curbs are recommended on fill slopes higher than 3 m (10 ft) regardless of the grade. Inspection of the existing/proposed site conditions and contact with maintenance and construction personnel shall be made by the designer to determine if vegetation will survive.

Shoulder gutter and/or curbs may be appropriate at bridge ends where concentrated flow from the bridge deck would otherwise run down the fill slope. This section of gutter should be long enough to include the guiderail and roadway transitions.

I. Median/Median Barriers. Medians are commonly used to separate opposing lanes of traffic on divided highways. It is preferable to slope median areas and inside shoulders to a center depression to prevent drainage from the median area from running across the traveled pavement. Where median barriers are used and, particularly on horizontal curves with associated superelevations, it is necessary to provide inlets and storm pipes to collect the water that accumulates against the barrier. Slotted inlets adjacent to the median barrier can also be used for this purpose.

J. Impact Attenuators. The location of impact attenuator systems should be reviewed to determine the need for drainage structures in these areas. With impact attenuator systems, it is necessary to have a clear or unobstructed opening as traffic approaches the point of impact. Therefore, it is essential that the ponding of stormwater or the presence of drainage devices such as inlets, swales or dikes do not impede the proper function of these systems. If the impact attenuator is placed in an area where superelevation or other grade separation occurs, inlets and/or storm sewer may need to be placed to prevent water from running through the clear opening and crossing the highway lanes or ramp lanes. Curb, curb-type structures or swales cannot be used to direct water across this clear opening because vehicular vaulting could occur when the impact attenuator system is utilized.

13.5 HYDROPLANING

Hydroplaning conditions can develop for relatively low vehicular 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. Wide pavements during high intensity storms produce depths of flow

of water. 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. Controlling factors for hydroplaning include:

- Vehicular speed.
- Tire conditions (pressure and tire tread).
- Pavement micro and macrotexture.
- Roadway geometrics (pavement width, cross slope, grade).
- Pavement conditions (rutting, depressions, roughness).

Speed is 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. In many respects, hydroplaning conditions are analogous to ice or snow on the roadway.

Designers do not have control over all factors involved in hydroplaning. However, many remedial measures can be included in development of a project to reduce hydroplaning potential. The following are practical measures for limiting conditions that are conductive to hydroplaning:

- Pavement Sheet Flow:
  - Maximize transverse slope (2%).
  - Maximize pavement roughness (opposite of International Roughness Index (IRI)).
- Gutter Flow:
  - Limit street spread (by decreasing inlet spacing).
  - Maximize interception of gutter flow above superelevation transitions.
- Sag Areas:
  - Limit roadway ponding duration and depth.
- Overtopping:
  - Limit depth and duration of overtopping flow.

If suitable measures cannot be implemented to address an area of high potential for hydroplaning, or an identified existing problem area, consideration should be given to installing advance warning signs.

13.6 DESIGN FREQUENCY AND SPREAD

A. Design Frequency. Chapter 7, *Hydrology* and Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10 should be referenced for design storm frequency and duration. The design storm frequency for pavement drainage may not be consistent with the frequency selected for other components of the storm drain system. For example, a 10-year frequency may be selected to limit spread on grade and a 50-year frequency may be used at a sag location. The trunkline and laterals on grade may be sized for the 10-year frequency where the trunkline or outfall from a sag area may be sized to accommodate a 50-year frequency.

B. Spread. In general, the spread should be held to the specified width for design frequencies. For storms of greater magnitude, the spread can be allowed to utilize "most" of the pavement as an open channel. For multi-laned curb and gutter, or guttered roadways with no parking, it is not practical to avoid travel lane flooding when longitudinal grades are flat (0.2% to 1%). However, flooding should not exceed one-half the travel lane width.

C. Selection. 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 minimized 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 mph), even a shallow depth of water on the pavement can cause hydroplaning. Design speed is recommended for use in evaluating hydroplaning potential.

Other considerations include inconvenience, hazards and nuisances to pedestrian traffic and buildings adjacent to roadways that are located within the splash zone. These considerations should not be minimized and, in some locations (e.g., commercial areas), may assume major importance.

D. Design Criteria. Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10 provides the design criteria for various roadway classifications, specifically Section 10.2. For the storm frequency, which is defined as return period (in years), culvert cross pipes shall be designed according to the events listed in Section 10.2.C.2. The second paragraph in this section, refers designers to different sections of the chapter based upon a project's objective:

- Publication 13M, Design Manual, Part 2, *Highway Design*, Section 10.6.E.
- Publication 13M, Design Manual, Part 2, *Highway Design*, Table 10.6.1.
- Publication 13M, Design Manual, Part 2, *Highway Design*, Section 10.3.C.

Table 10.6.1 describes the minimum return period to be used to develop a design flow for different roadway classifications.

13.7 GUTTER FLOW CALCULATIONS

A. Introduction. Gutter flow calculations are necessary 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 Figures 13.1(a) and 13.1(b) can be utilized to solve uniform cross slope channels, composite gutter sections and V-shape gutter sections. Figure 13.3 is also useful in solving composite gutter section problems. Computer programs are also useful for this computation and inlet capacity. Composite gutter sections have a greater hydraulic capacity for normal cross slopes than uniform gutter sections. Example problems for each gutter section are shown in the following sections.

B. Manning's *n* for Pavements. The roughness of the pavement surface affects water spread. The methods for determining spread provided in this chapter use Manning's roughness coefficient (*n*). Refer to Table 13.1 for recommended values.

Table 13.1 Manning's *n* for Street 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

Source: *Design Charts for Open-Channel Flow* (FHWA, 1961).

C. Uniform Cross Slope Procedure. The nomograph in Figures 13.1(a) and 13.1(b) is used with the following procedures to find gutter capacity for uniform cross slopes:

CONDITION 1: Find spread, given gutter flow:

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, given spread:

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 Q_n 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, the gutter capacity times n (Q_n) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

D. Composite Gutter Sections Procedure. Figure 13.2 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, given flow:

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 (2 ft); $n = 0.016$; $Q = 0.057$ m^3/s (2.0 cfs); try $Q_s = 0.020$ m^3/s (0.7 cfs).

Step 2 Calculate the gutter flow in W (Q_w), using the equation:

(*Equation 13.4*)

Metric	**U.S. Customary**
$Q_w = Q - Q_s$	$Q_w = Q - Q_s$
$Q_w = 0.057 - 0.020 = 0.037$ m^3/s	$Q_w = 2.0 - 0.7 = 1.3$ cfs

Step 3 Calculate the ratios Q_w/Q and S_w/S_x, and use Figure 13.2 to find an appropriate value of W/T from Figure 13.2*:*

Metric	**U.S. Customary**
$Q_w/Q = (0.037$ m^3/s) / (0.057 m^3/s) = 0.65	$Q_w/Q = (1.3$ cfs) / (2.0 cfs) = 0.65

$S_w/S_x = 0.06/0.02 = 3$

Therefore, $W/T = 0.27$

Step 4 Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3:

Metric	**U.S. Customary**
$T = 0.6$ m / 0.27 = 2.22 m	$T = 2$ ft / 0.27 = 7.41 ft

Step 5 Find the spread above the depressed section (T_s) by subtracting W from the value of T from Step 4:

Metric	**U.S. Customary**
$T_s = 2.2$ m – 0.6 m = 1.62 m	$T_s = 7.41$ ft – 2.0 ft = 5.41 ft

Step 6 Use the value of T_s from Step 5 and Manning's n, S and S_x to find the actual value of Q_s from Figure 13.1:

$Q_s = 0.014$ m^3/s (0.5 cfs)

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:

Figure 13.1(a) Flow in Triangular Gutter Sections (Metric)

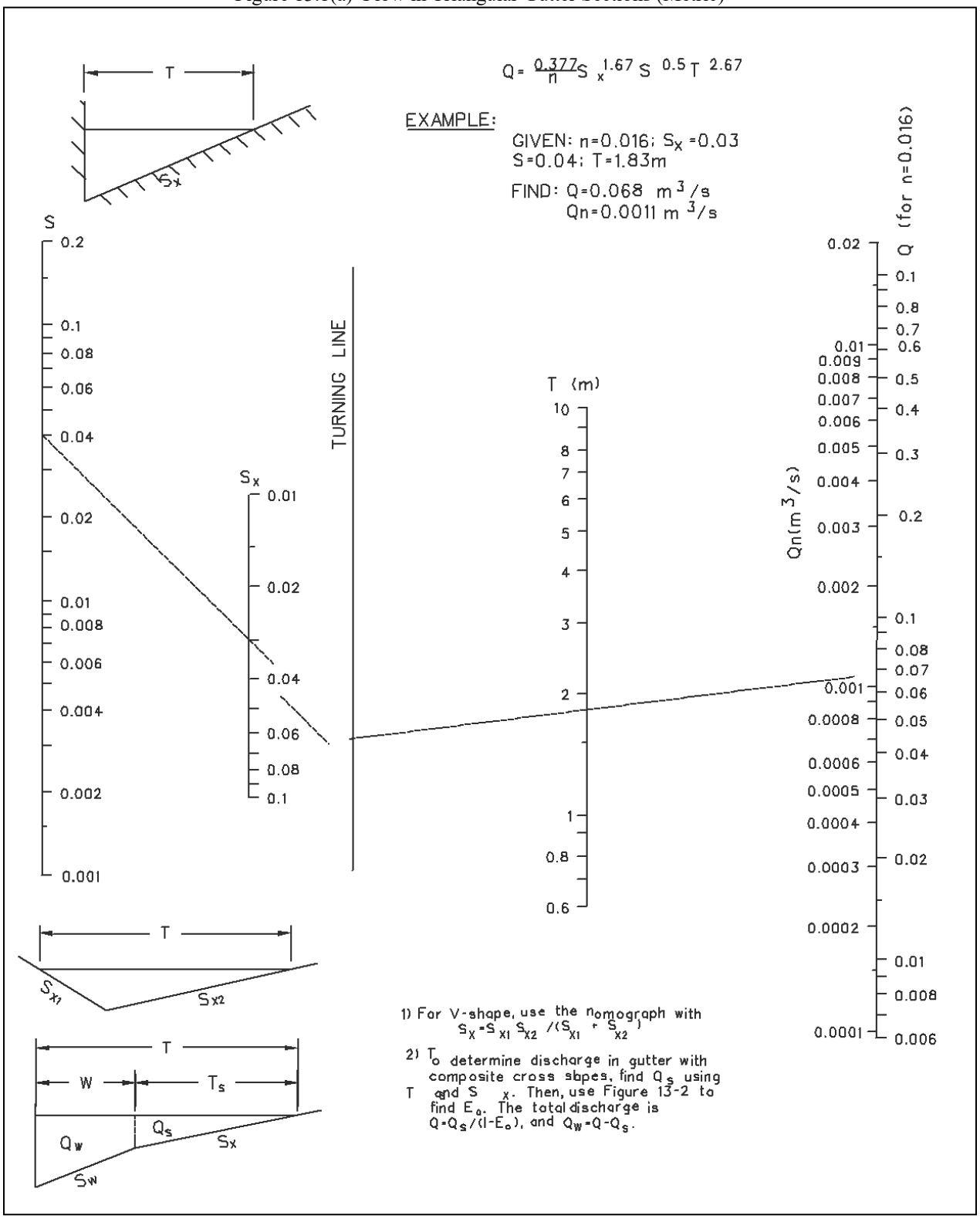
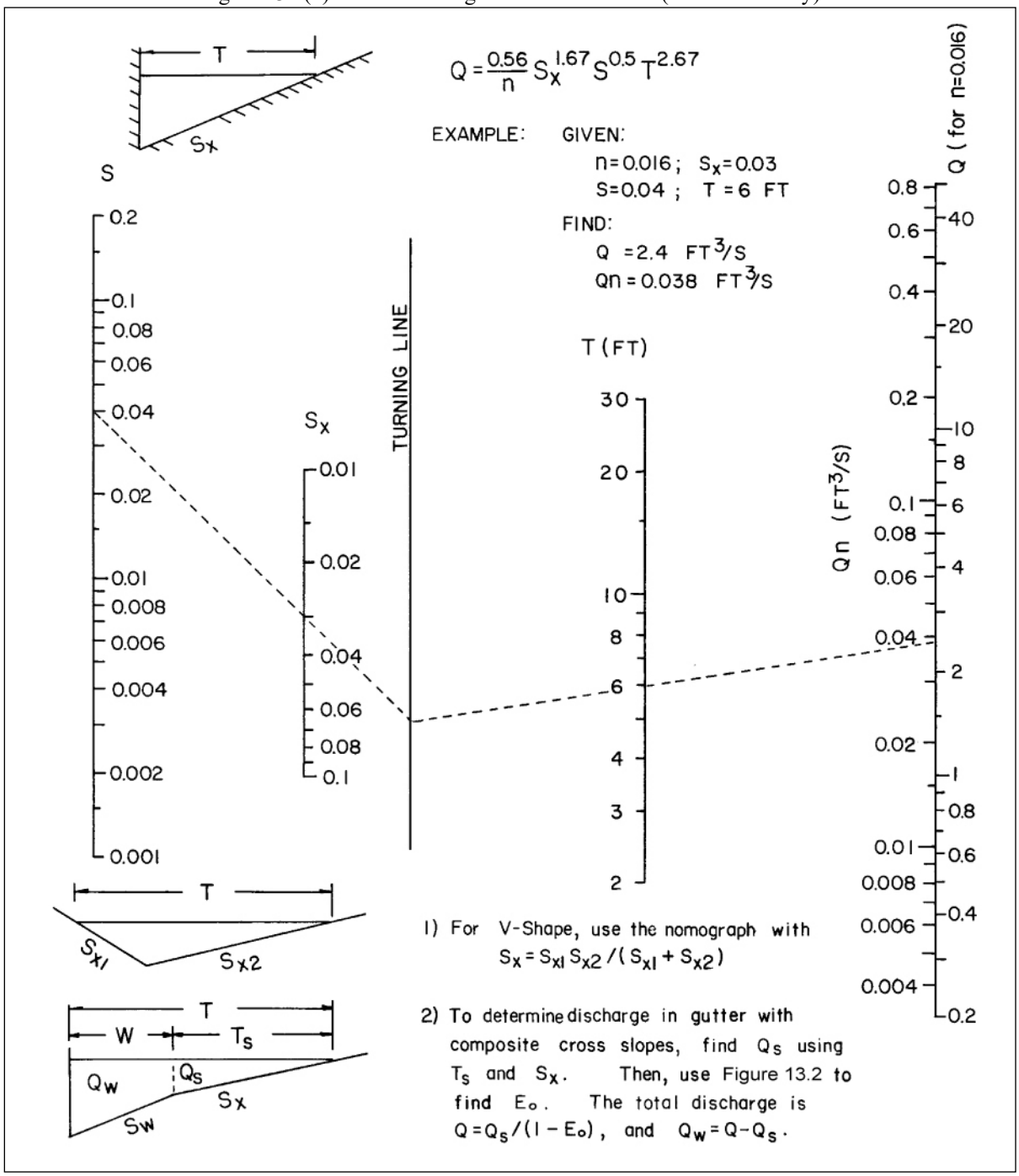
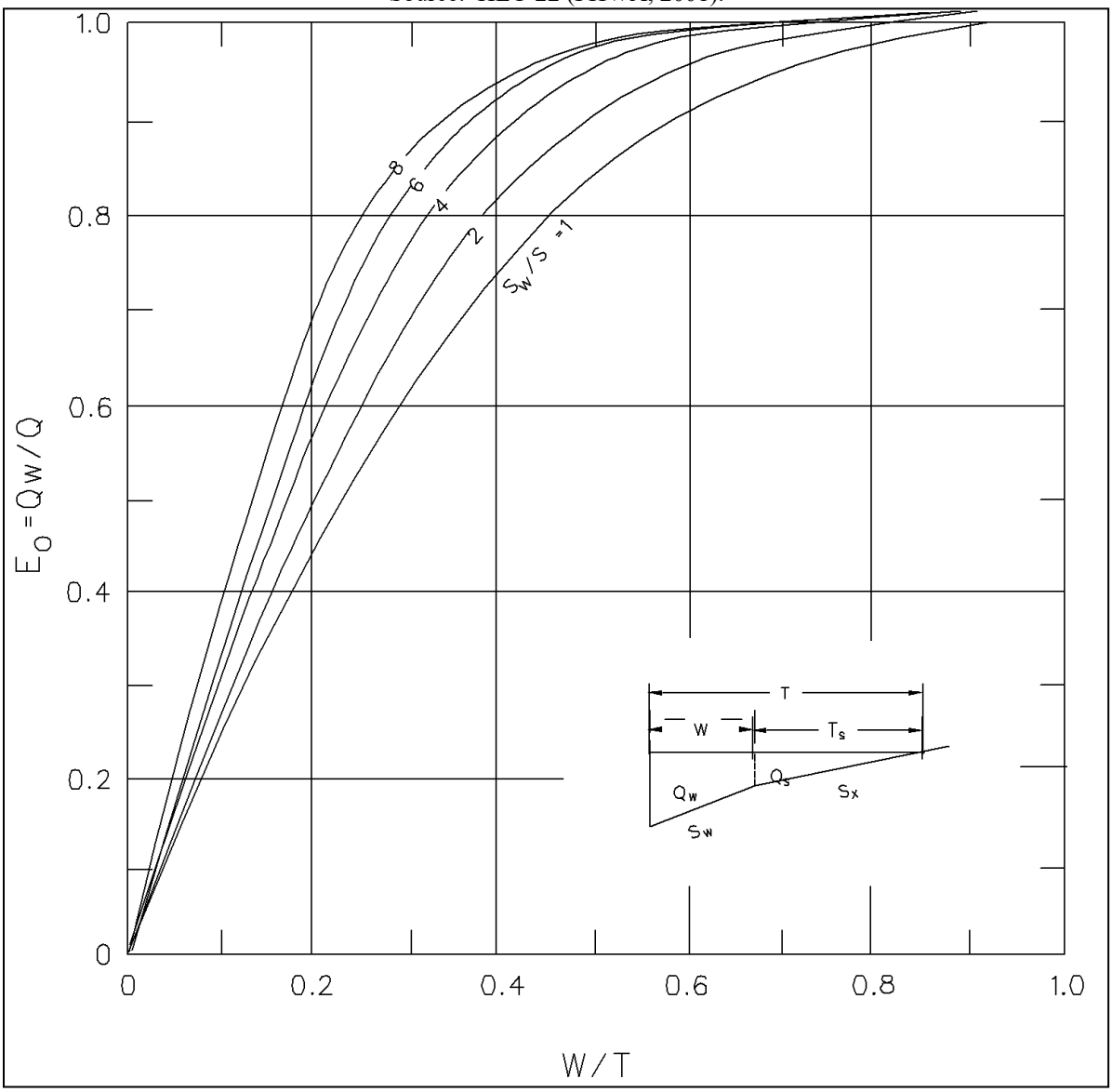


Figure 13.1(b) Flow in Triangular Gutter Sections (U.S. Customary)



Source: HEC-22 (FHWA, 2001).

Figure 13.2 Ratio of Frontal Flow to Total Gutter Flow
Source: HEC-22 (FHWA, 2001).



Metric	**U.S. Customary**
Compare 0.014 m^3/s to 0.020 m^3/s "no good"	Compare 0.5 cfs to 0.7 cfs "no good"
Try Q_s = 0.023 m^3/s	Try Q_s = 0.8 cfs
$Q_w = Q–Q_s$ = 0.057m^3/s - 0.023m^3/s = 0.034m^3/s	$Q_w = Q–Q_s$ = 2.0 cfs - 0.8 cfs = 1.2 cfs
Q_w/Q = 0.034 / 0.057 = 0.6	Q_w/Q = 1.2 cfs / 2.0 cfs = 0.6
From Figure 13.2, W/T = 0.23	From Figure 13.2, W/T = 0.23
T = 0.6m/0.23 = 2.61m, T_s = 2.61m - 0.6m = 2.01m	T = 2.0ft / 0.23 = 8.7ft, T_s = 8.7ft - 2.0ft = 6.7ft
From Figure 13.1, Q_s = 0.023 m^3/s - OK	From Figure 13.1, Q_s = 0.8 cfs - OK
ANSWER: Spread T = 2.61 m	ANSWER: Spread T = 8.7 ft

CONDITION 2: Find gutter flow, given spread:

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 m (10 ft); W = 0.6 m (2 ft); T_s = 3.05 m – 0.6 m = 2.44 m (T_s = 10 ft – 2 ft = 8 ft); S_x = 0.04; S = 0.005 m/m; S_w = 0.06; n = 0.016; d = 0.13 m (0.43 ft).

Step 2 Use Figure 13.1 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 13.1, Q_s = 0.085 m^3/s (3.0 cfs)).

Step 3 Calculate the ratios W/T and S_w / S_x and, from Figure 13.2, find the appropriate value of E_o (the ratio of Q_w /Q). (W/T = 0.6 m / 3.05 m = 0.2 (W/T = 2 ft / 10 ft = 0.2); S_w / S_x = 0.06/0.04 = 1.5; from Figure 13.1, E_o = 0.46).

Step 4 Calculate the total gutter flow using the equation:

(*Equation 13.5*)

$$Q = Q_s/(1 - E_o)$$

where: Q = gutter flow rate, m^3/s (cfs)
Q_s = flow capacity of the gutter section above the depressed section, m^3/s (cfs)
E_o = ratio of frontal flow to total gutter flow (Q_w/Q)

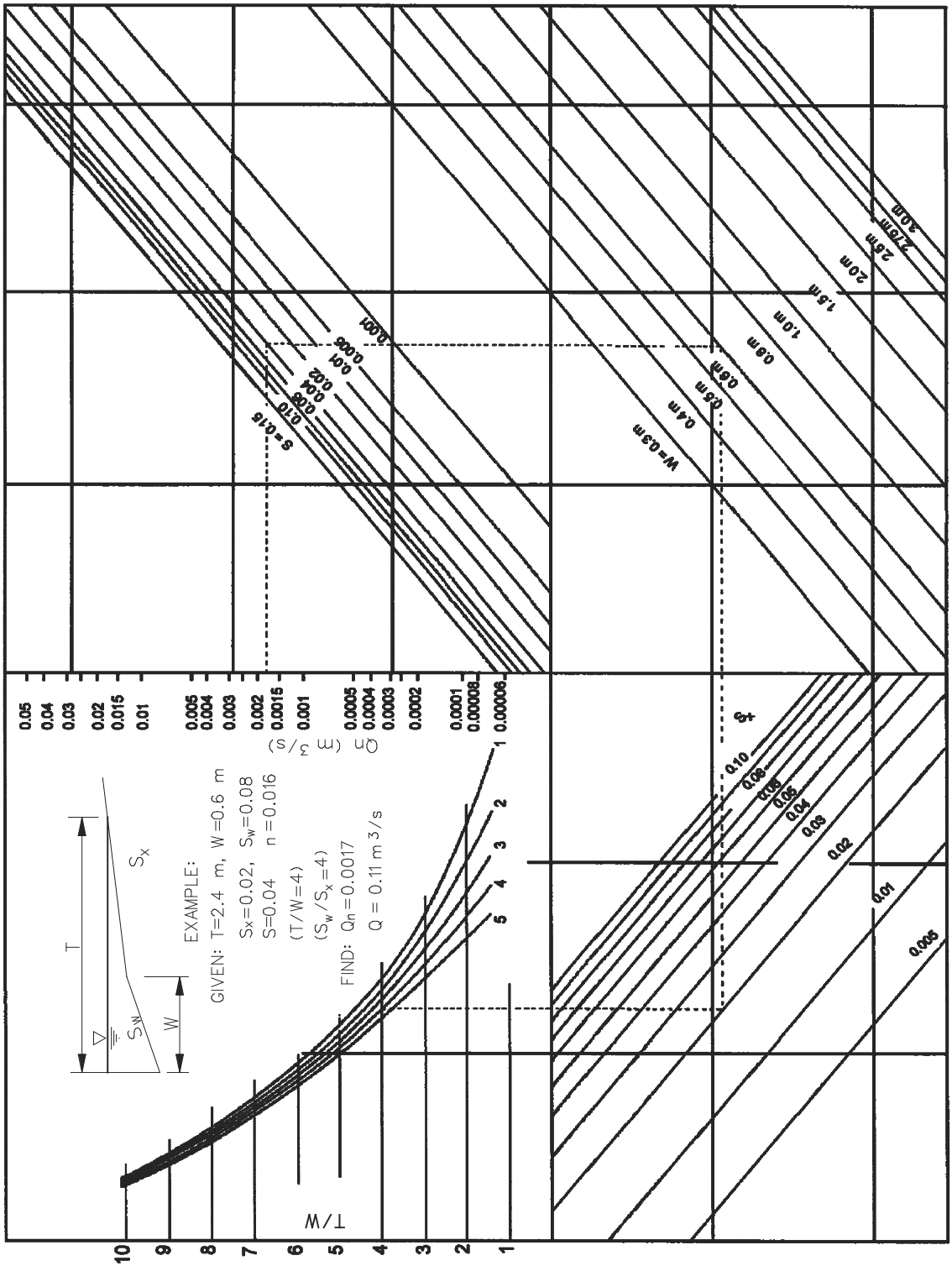
Metric	**U.S. Customary**
Q = 0.085 / (1 - 0.96) = 0.157 m^3/s	Q = 3.0 / (1 - 0.46) = 5.6 cfs

Step 5 Calculate the gutter flow in the depressed section using Equation 13.4:

Metric	**U.S. Customary**
Q_w = Q - Q_s = 0.157 - 0.085 = 0.072 m^3/s	$Q_w = Q - Q_s$ = 5.6 - 3.0 = 2.6 cfs

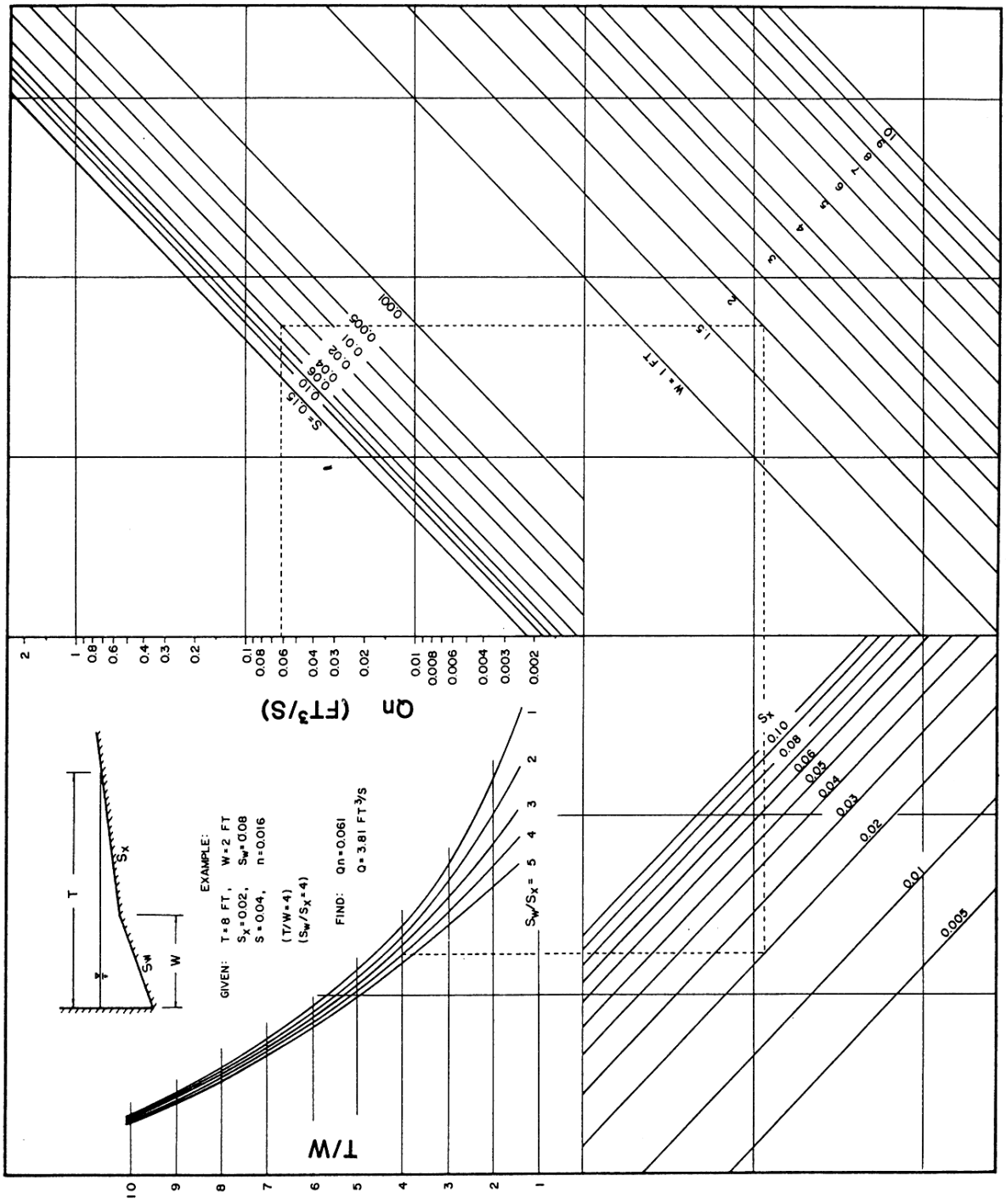
Note: Figure 13.3 can also be used to calculate the flow in a composite gutter section.

Figure 13.3(a) Flow in Composite Gutter Sections (Metric)



Source: HEC-12

Figure 13.3(b) Flow in Composite Gutter Sections (U.S. Customary)



Source: HEC-12

E. V-Type Gutter Sections (Procedures). Figure 13.1 can also be used to solve V-type channel problems. The spread (T) can be calculated for a given flow (Q), or the flow (Q) can be calculated for a given spread (T). This method can be used to calculate approximate flow conditions in the triangular channel adjacent to concrete median barriers. It assumes that the effective flow is confined to the V-type channel with spread T_1. Figure 13.4 illustrates the following procedure for a 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: $S = 0.01$, $S_{x1} = 0.06$, $S_{x2} = 0.04$, $S_{x3} = 0.015$, $n = 0.016$, $Q = 0.057$ m^3/s (2.0 cfs), shoulder width = 1.8 m (6 ft)).

Figure 13.4(a) V-Type Gutter (Metric)

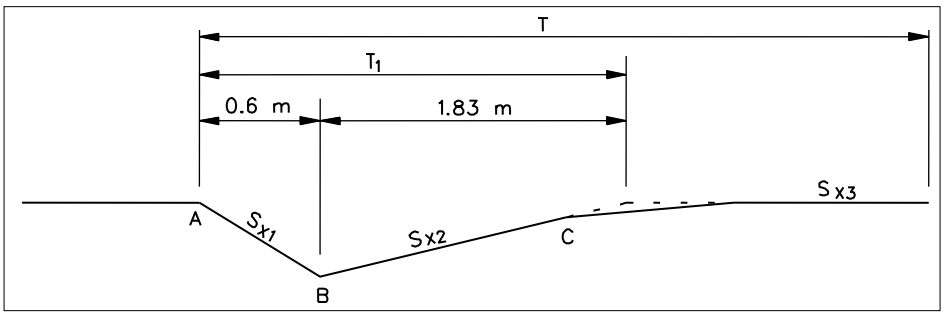
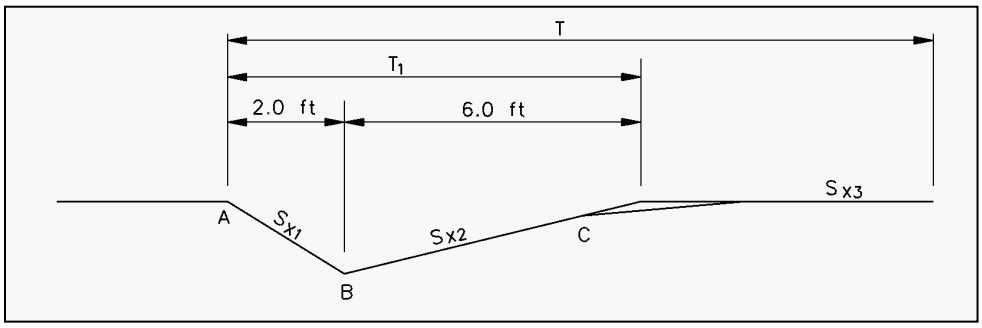


Figure 13.4(b) V-Type Gutter (U.S. Customary)



Step 2 Calculate S_x:

$$S_x = S_{x1}\ S_{x2}/(S_{x1} + S_{x2})$$
$$S_x = (0.06)(0.04)/(0.06 + 0.04) = 0.024$$

Step 3 Solve for T_1 using the nomograph on Figure 13.1:

T_1 is a hypothetical width that is correct if it is contained within S_{x1} and S_{x2}. From the nomograph, T_1 = 2.56 m (8.4 ft); however, because the shoulder width of 1.8 m (6 ft) is less than 2.56 m (8.4 ft), S_{x2} is 0.04, and the pavement cross slope S_{x3} is 0.015. T will actually be greater than 2.56 m (8.4 ft); 2.56 - 0.6 = 1.96 m (8.4 ft - 2.0 = 6.4 ft), which is > 1.83 m (> 4.0 ft); therefore, the spread is greater than 2.56 m (8.4 ft).

Step 4 To find the actual spread, solve for depth at Points B and C:

	Metric	**U.S. Customary**
Point B:	1.96 m at 0.04 = 0.078 m	6.4 ft at 0.004 = 0.26 ft
Point C:	0.078 m - (1.83 m at 0.04) = 0.0048 m	0.26 ft - (6.0 ft at 0.04) = 0.016 ft

Step 5 Solve for the spread on the pavement (Pavement cross slope = 0.015):

Metric	**U.S. Customary**
$T_{0.015}$ = 0.0048 m / 0.015 = 0.32 m	$T_{0.015}$ = 0.016 ft / 0.015 = 1.05 ft

Step 6 Find the actual total spread (T):

Metric	U.S. Customary
T = 0.6 m + 1.83 m + 0.32 m = 2.75 m	T = 2 ft + 6 ft + 1.06 ft = 9.06 ft

CONDITION 2: Given spread (T), find flow (Q):

Step 1 Determine input parameters such as longitudinal slope (S), cross slope (S_x), Manning's n and allowable spread. (Example: n = 0.016, S = 0.015, S_{x1} = 0.06, S_{x2} = 0.04, T = 1.8 m (6 ft)).

Step 2 Calculate S_x:

$$S_x = S_{x1}S_{x2}/(S_{x1} + S_{x2})$$
$$S_x = (0.06)(0.04)/(0.06 + 0.04) = 0.024$$

Step 3 Using Figure 13.1, solve for Q: For T = 1.8 m (6.0 ft), Q = 0.031 m^3/s (1.1 cfs)

The equation shown on Figure 13.1 can also be used.

13.8 INLETS

A. General. Inlets are drainage structures utilized to collect surface water through grate or curb openings and convey it to storm pipes or to culverts. Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10 and Publication 72M, *Roadway Construction Standards*, RC-45M and RC-46M specify the standards for inlet types and should be referenced during the design process. All inlets, regardless of capacity, which exceed 2.75m (9 ft) in depth require special details and designs. Bicycle safe grates, should be used in those locations where bicycles (or wheelchair traffic) are anticipated to be present, all other locations should use standard inlet grates as indicated in Publication 72M, *Roadway Construction Standards*, RC-45M and RC-46M so as to not decrease the capacity of the inlet.

This section discusses the various types of inlets in use and recommends guidelines on the use of each type.

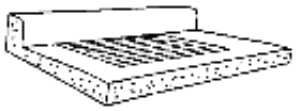
B. Types. The following four classes of storm drain inlets may be used in PennDOT applications:

1. Grate Inlets. Grate inlets (Types C, D-H, M, and S) perform satisfactorily over a wide range of gutter grades. Grate inlets generally lose capacity with an increase in grade, but to a lesser degree than curb opening inlets. The principal advantage of grate inlets is that they are installed along the roadway where the water is flowing. Their principal disadvantage is that floating trash or debris may clog them. For safety reasons, preference should be given to grate inlets. Additionally, where bicycle traffic occurs or is expected to occur, bicycle safe grates shall be provided. Curved vane grate inlets provide the best capture efficiency.

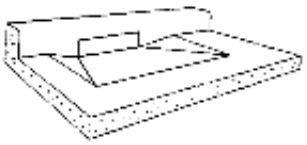
2. Curb-Opening Inlets. Curb opening inlets are not preferred on the state highway system and should be considered in special cases only. Curb opening inlets are most effective on flatter slopes, in sags, and with flows which typically carry significant amounts of floating debris. The interception capacity of curb-opening inlets decreases as the gutter grade increases. Consequently, the use of curb-opening inlets may be considered in sags and on grades less than 3%.

3. Combination Inlets. Combination inlets result in a high capacity inlet which offers the advantages of both grate and curb opening inlets. When the curb opening precedes the grate in a "sweeper" configuration, the curb-opening inlet acts as a trash interceptor during the initial phases of a storm. Used in a sag configuration, the sweeper inlet can have a curb opening on both sides of the grate.

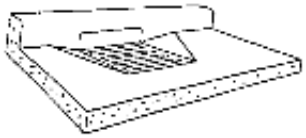
Inlet Types



Grate Inlet



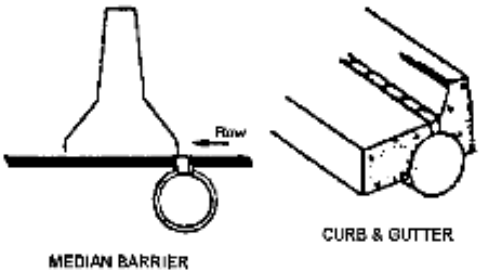
Curb-Opening Inlet



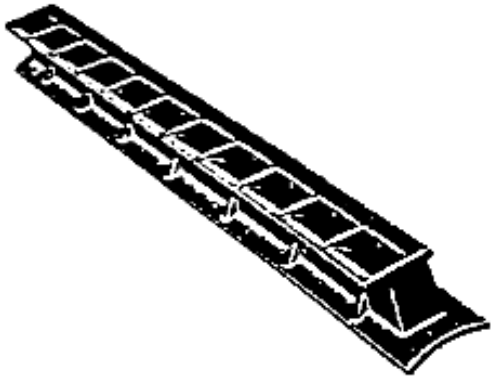
Combination Inlet

4. Slotted Drains. Slotted drains may be considered in areas where it is desirable to intercept sheet flow before it crosses onto a section of roadway. Their principal advantage is their ability to intercept flow over a wide section. However, slotted inlets are very susceptible to clogging from sediments and debris, and are not recommended for use in environments where significant sediment or debris loads may be present. Slotted drains on a longitudinal grade do have the same hydraulic capacity as curb openings when debris is not a factor. Publication 408, *Specifications*, Section 617 describes the construction of slotted drains.

Slotted Drains



Vertical Riser Type



Vane Type

C. Inlet Locations. Inlets are required at locations needed to collect runoff within the design controls specified in the design criteria (see Section 13.6). In addition, there are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations should be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity or runby. Examples of such locations are as follows:

- Sag points in the gutter grade.
- Upstream of median breaks, entrance/exit ramp gores, cross walks and street intersections.
- Immediately upstream and downstream of bridges.
- Immediately upstream of cross slope reversals.
- On side streets at intersections.
- At the end of channels in cut sections.
- Behind curbs, shoulders or sidewalks to drain low areas.
- Where necessary to collect snow melt.

Inlets should not be located in the path where pedestrians are likely to walk.

13.9 INLET SPACING

A. General. Inlet spacing is based upon inlet capacity and allowable spread. Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10 should be referenced for the standards in regards to inlet spacing, with specific

spacing limits set forth based upon location, condition and inlet type. On shoulder (without swale) or curb sections, the maximum spacing of inlets shall not exceed 140 m (450 ft), except where a Type D-H Inlet or the alternate single unit inlet is used. Inlet spacing in depressed median sections and shoulder swale areas shall not exceed 280 m (900 ft).

13.10 STORM DRAIN MANHOLES

A. Location. Manholes are used to provide entry to continuous underground storm drainage pipes for inspection and cleanout. Grate inlets may be used in lieu of manholes when entry to the system can be provided at the grate inlet, 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 pipes 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, where it is impossible to avoid locating a manhole in a traffic lane, care should be taken to ensure that it is not in the normal vehicular wheel path.

B. Spacing. The spacing of manholes shall be in accordance with the criteria in Table 13.2.

Table 13.2 Manhole Spacing Criteria

Size of Pipe - mm (in)	Maximum Distance - m (ft)
450–600 (18–24)	100 (300)
675–900 (27–36)	125 (400)
1050–1350 (42–54)	150 (500)
≥ 1500 (≥ 60)	300 (1000)

C. Types and Sizing. A manhole, with appropriate frame and lid, should be provided based upon Publication 408, *Specifications*, Section 605.2 and is shown on the Publication 72M, *Roadway Construction Standards*. The diameter of the manhole is based on the following storm drain sizes:

- Where the storm drain pipe diameter is 750 mm (30 in) or less, a 1200 mm (48 in) diameter circular manhole shall be provided.
- Where the storm drain pipe diameter is 750 mm - 1050 mm (30 in - 42 in) inclusive, a modified box manhole should be provided with a minimum width of 1372 mm (54 in).
- Where the storm drain pipe diameter is 1050 mm (42 in) or larger, a modified box manhole should be provided with a minimum internal width that is flush with the outside diameter of the pipe.

For drain pipes less than 750 mm (30 in) in diameter placed in a circular manhole, 300 mm of horizontal clearance is must be provided between the outside surfaces of the pipes. Pipes not at the same depth vertically shall be located at least one-half the maximum outside diameter of the drain pipe apart.

13.11 STORM PIPES

A. 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 pipe connects with other pipes or the outfall.

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 is reduced. 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 Chapter 7, *Hydrology* for a discussion on time of concentration.

For ordinary conditions, storm pipes 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.

B. Design Procedures. The Storm Sewer Design computation tables from Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10 may be utilized for design. The design of storm drainage systems is generally divided into the following operations:

Step 1 Preliminary Layout. The first step in storm drain design is to develop a conceptual storm drain layout, including inlet, manhole, and pipe locations. This is usually completed on a plan that shows the roadway, adjacent land use conditions, intersections, and under/overpasses. Surface utilities, underground utilities, and any other storm drain systems shall also be shown. Tentative inlets, junctions, and access locations shall be identified based primarily on obvious project requirements and limitations, such as low points and intersections.

Step 2 Determine inlet location and spacing as outlined in Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10 and earlier in this chapter.

Step 3 Determine drainage areas, runoff coefficients and a time of concentration to the first upstream inlet. Using an Intensity-Duration-Frequency (IDF) curve, determine the rainfall intensity (I). Calculate the discharge (Q) using Equation 13.1.

Step 4 Pipe Sizing. Given the preliminary layout, it is possible to begin the hydraulic analysis necessary to size the storm drain system. The method used to size storm pipes is based on a gravity (non-pressure) flow concept, which shall generally be adopted for design on the Department's projects. In areas of an extreme flat grade, where a realistic size cannot be attained by means of the usual gravity flow design, a special design based on a pressure flow concept may be considered.

Step 5 Calculate travel time in the pipe to the next inlet or manhole by dividing pipe length by the velocity. This travel time is added to the time of concentration for a new time of concentration and a new rainfall intensity at the next entry point.

Step 6 Calculate the new area (A) and multiply by the runoff coefficient (C), add to the previous (CA), divide by 3.63 and multiply by the new rainfall intensity (I) to determine the new discharge (Q, Metric). Calculate the new area (A) and multiply by the runoff coefficient (C), add to the previous (CA) and multiply by the new rainfall intensity to determine the new discharge (Q, U.S. Customary). Determine the size of pipe and slope necessary to convey the discharge.

Step 7 Continue this process to the storm drain outlet. Utilize the equations and/or nomographs to accomplish the design effort.

Step 8 Complete the design by calculating the hydraulic grade line as described in Section 13.12. The design procedure should include the following:

- Storm drain design computations can be made on forms provided in Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10.
- All computations and design sheets should be clearly identified. The designer'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.

C. Sag Point. As indicated above, the storm drainage system that drains a major sag point shall have an inlet placed at the low point on sag vertical curves with flanking inlets on each side of the low point at a distance not to exceed 30 m (100 ft) or at a grade not greater than 60 mm (0.20 ft) above the sag inlet. If warranted, the inlet may also be sized to accommodate the runoff from a larger rainfall event. This can be done by actually computing the bypass flow occurring at each inlet during larger event and accumulating it at the sag point. The inlet at the sag point and the storm drain pipe leading from the sag point must be sized to accommodate this additional bypass flow within the criteria established. See Section 13.6. To design the pipe leading from the sag point, it may be helpful to convert the additional bypass flow created by the larger event into an equivalent CA that can be added to the design CA.

In some cases designers may want to design separate systems to prevent the above-ground system from draining into the depressed area. This concept may be more costly but, in some cases, may be justified. Another method would be to design the upstream system for a larger design flow to minimize the bypass flow to the sag point. Each case must be evaluated on its own merits, and the impacts and risk of flooding a sag point assessed.

D. Hydraulic Capacity. The most widely used formula for determining the hydraulic capacity of storm pipes for gravity and pressure flows is the Manning's formula, expressed by the following equation:

(Equation 13.6)

Metric

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

U.S. Customary

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

where: V = mean velocity of flow, m/s (ft/s)
n = Manning's roughness coefficient
R = hydraulic radius, area of flow divided by the wetted perimeter (A/WP) (m, ft)
S = the slope of the energy grade line, m/m (ft/ft)

In terms of discharge, the above formula becomes:

(Equation 13.7)

Metric

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

U.S. Customary

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

where: Q = rate of flow, m^3/s (cfs)
A = cross sectional area of flow, m^2 (ft^2)

For circular storm pipes flowing full, $R = D/4$ and Equations 13.6 and 13.7 become:

(Equation 13.8)

Metric

$$V = \frac{0.397}{n} D^{2/3} S^{1/2}$$

$$Q = \frac{0.312}{n} D^{8/3} S^{1/2}$$

U.S. Customary

$$V = \frac{0.590}{n} D^{2/3} S^{1/2}$$

$$Q = \frac{0.463}{n} D^{8/3} S^{1/2}$$

where: D = diameter of pipe, m (ft)

The Manning's roughness coefficients for corrugated metal pipes are included in Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Appendix F.

The nomograph solution of Manning's formula for full flow in circular storm pipes is shown on Figures 13.5(a); and 13.5(b); Figures 13.6(a) and 13.6(b); and Figures 13.7(a) and 13.7(b). Figure 13.8 has been provided to assist in the solution of Manning's Equation for partial full flow in storm pipes.

For additional information regarding pipe sizing of storm sewer systems see Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Section 10.3.B.

E. Minimum Pipe Sizes. The minimum diameter of storm pipe shall be 450 mm (18 in) for circular pipe (or equivalent size pipe arch), except pipes under a 7.6 m (25 ft) or greater fill height shall not be less than 600 mm (24 in). Storm pipes are provided in 75 mm (3 in) increments up to the 900 mm (36 in) diameter size and 150 mm (6 in) increments for those exceeding 900 mm (36 in) diameter size.

In general, the size of a downstream storm pipe should not be smaller than that of the upstream storm pipe(s). This requirement is not an absolute criterion and sound engineering judgment should be exercised in making the determination. As an example, it would not be economical to purposely adopt a long stretch of downstream storm pipes larger than an upstream slope pipe where the size of the slope pipe is hydraulically determined by entrance conditions.

F. Minimum Grades. The minimum slope in a pipe shall not be less than 0.35%. All storm pipes shall be placed on the most economical slope and at the most economical depth.

To minimize sediment deposition, storm pipes 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 ensure that there is sufficient velocity in all storm pipes to deter settling of particles. Pipes with smooth interiors, such as concrete and smooth plastic, exceed the minimum velocity at slopes equal to or greater than the minimum allowable pipe slope (0.35%). Pipes with greater interior roughness, such as corrugated metal pipes, should be checked if the pipe diameter is 900 mm (36 in) or less. Minimum slopes required for a velocity of 0.9 m/s (3 ft/s) can be calculated by rearranging the Manning formula:

(Equation 13.9)

Metric

$$S = \frac{(nV)^2}{R^{4/3}}$$

U.S. Customary

$$S = \frac{(nV)^2}{1.486R^{4/3}}$$

Avoid abrupt changes in direction or slope of pipe. Where such abrupt changes are required, place an inlet or manhole at the point of change.

G. Curved Alignment. Curved alignment of storm pipes 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 used if there is not room for the long-radius bends. Deflecting the joints to obtain the necessary curvature is not desirable except in very minor curvatures. Using large manholes solely for changing direction may not be cost effective on large-size storm pipes.

H. Placement. Provide 300 mm (12 in) minimum cover from the top of pipe barrel to bottom of base course. Refer to Publication 72M, *Roadway Construction Standards*, RC-30M for details concerning minimum cover over pipe under pavements. In addition, refer to Publication 219M, *Standard Drawings for Bridge Construction* and Publication 13M, Design Manual, Part 2, *Highway Design.* For special design and modeling of pipes, refer to Publication 15M, Design Manual, Part 4, *Structures,* Appendix H.

Ductile iron pipe may be used when: (1) a minimum cover is at least 75 mm (3 in) but less than 150 mm (6 in) from the top of pipe barrel to the subgrade elevation; and (2) high impact and concentrated heavy loadings are involved. Under these conditions, provide a ductile iron pipe with a minimum 3-edge bearing strength of 17.8 kN/m (4000 lb/ft) times the diameter in meters (feet).

A minimum drop of 50 mm (2 in) shall be provided in the inlet or other junction structure between the lowest inlet pipe invert elevation and the outlet pipe invert elevation. When there is a change in pipe size in an inlet or other junction structure, the elevation of the lowest invert of the incoming pipes shall not be less than that of the outlet pipe.

Figure 13.5(a) Manning's Formula for Full Flow in Storm Pipes (Metric)
Source: HEC-22 (FHWA, 2001).

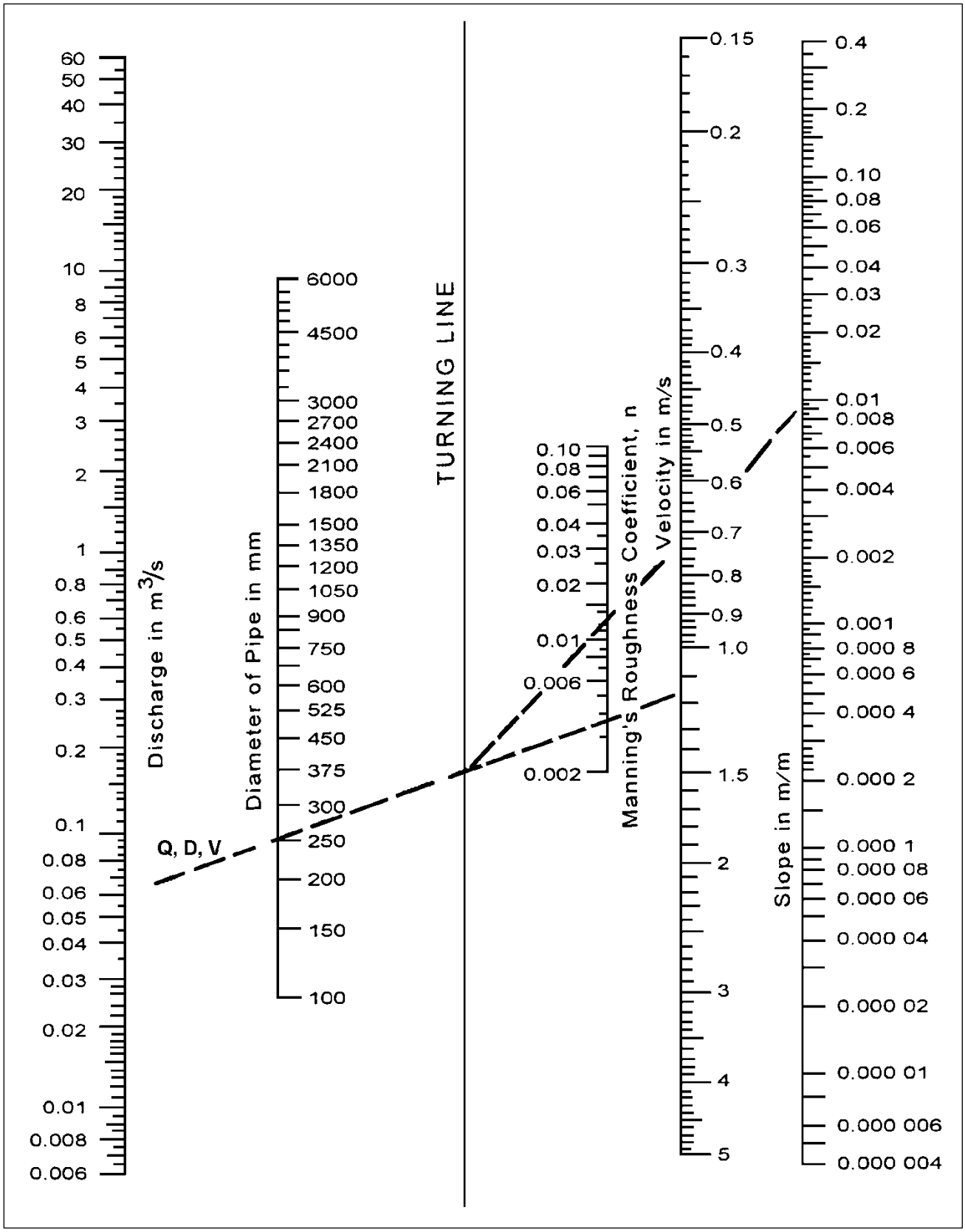


Figure 13.5(b) Manning's Formula for Full Flow in Storm Pipes (U.S. Customary)
Source: HEC-22 (FHWA, 2001).

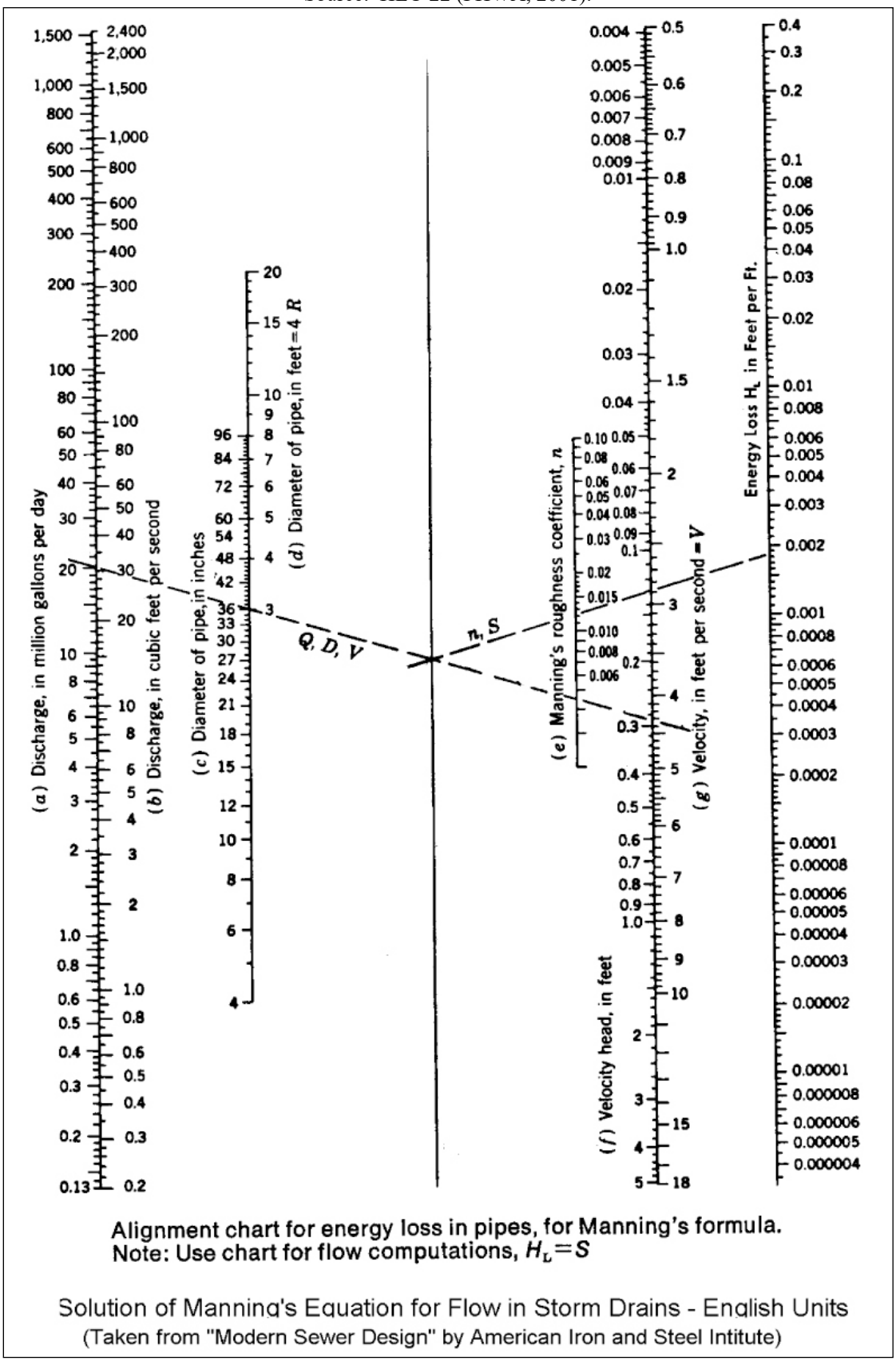


Figure 13.6(a) Nomograph for Computing Required Size of Circular Pipes for Full Flow (Metric)
(n = 0.013 or 0.015)

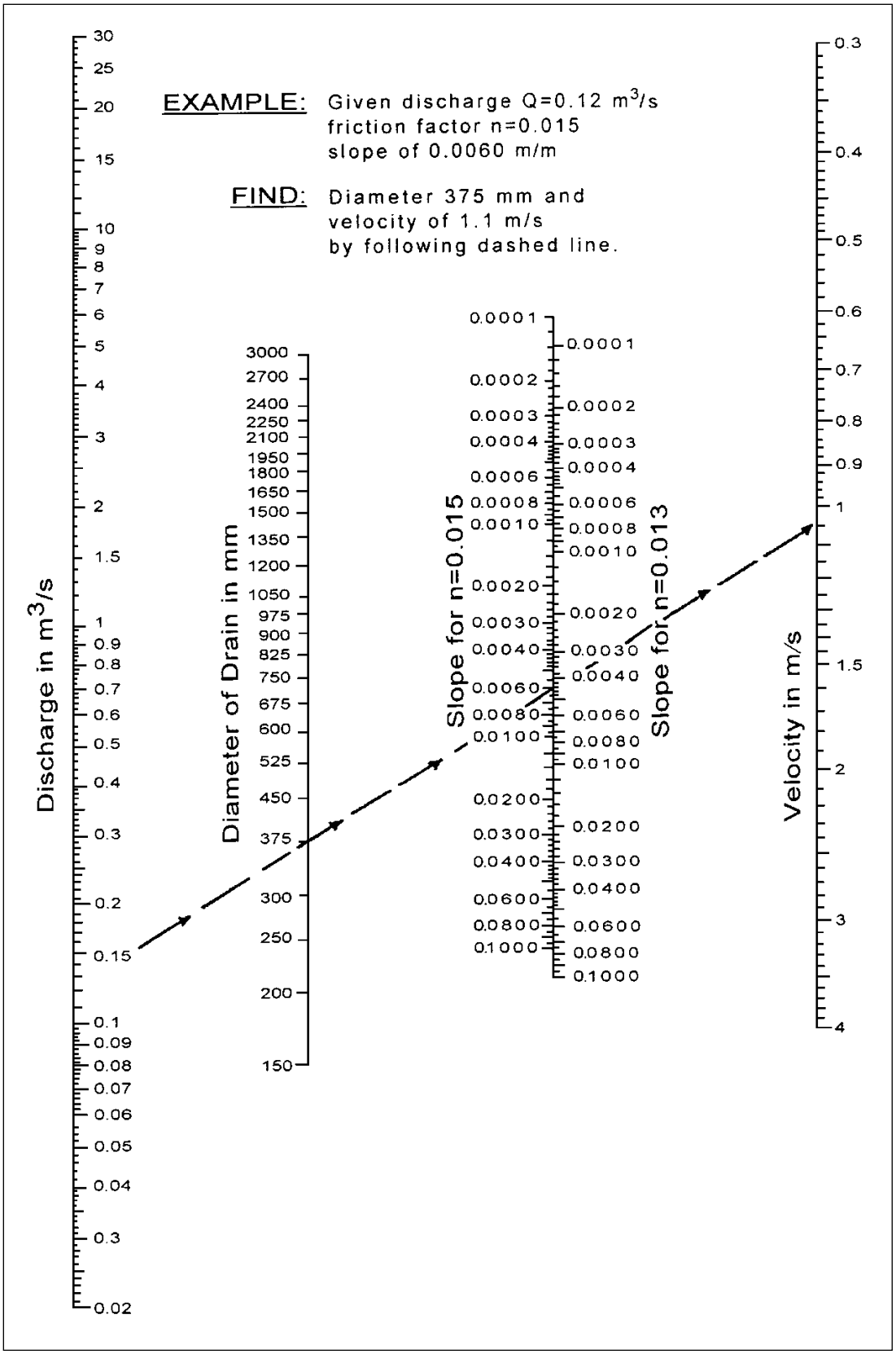


Figure 13.6(b) Nomograph for Computing Required Size of Circular Pipes for Full Flow (U.S. Customary) (n = 0.013 or 0.015)

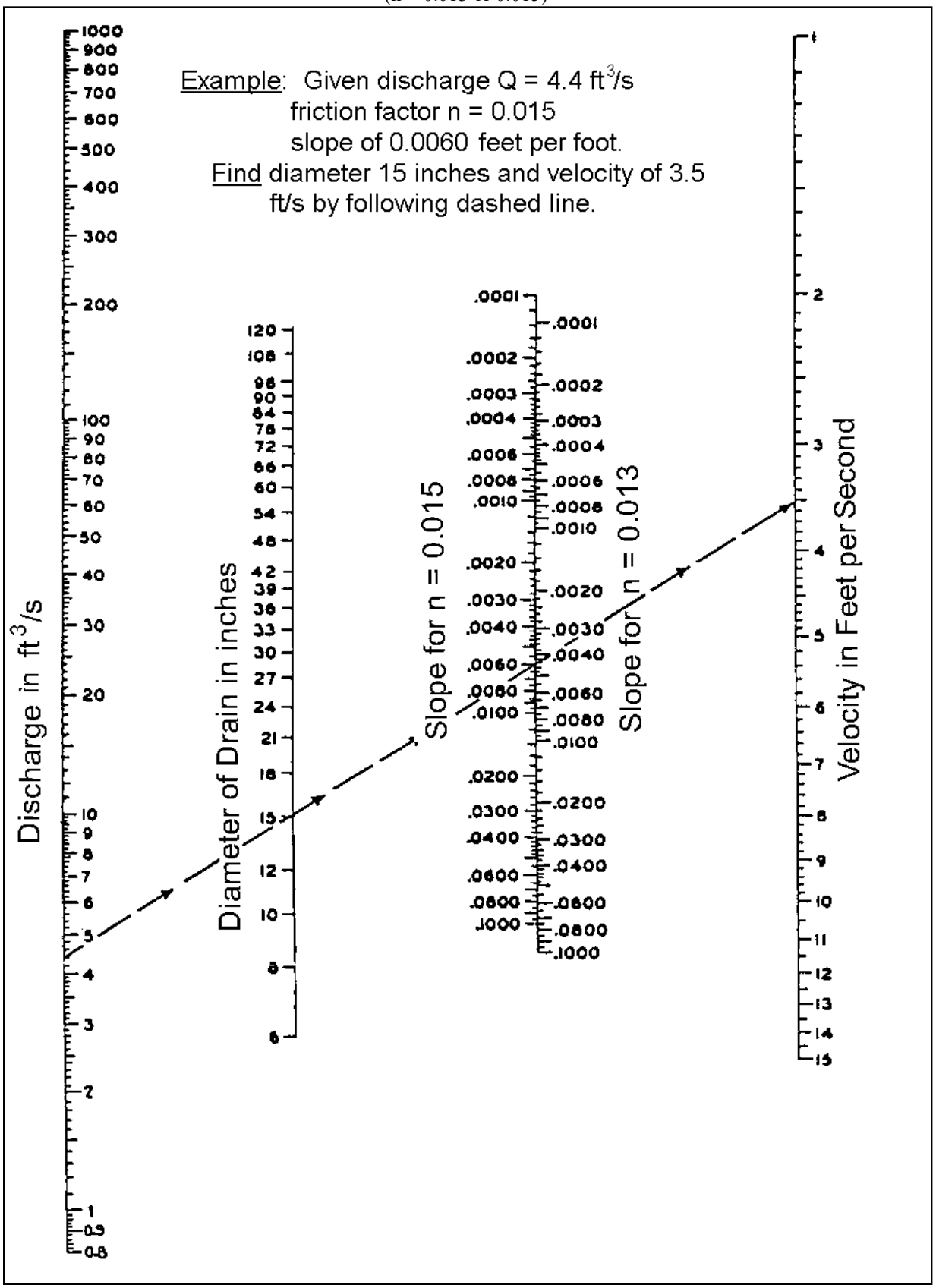


Figure 13.7(a) Concrete Pipe Flow Nomograph (Metric)
Source: *Open Channel Hydraulics* (Chow, V. T., 1959).

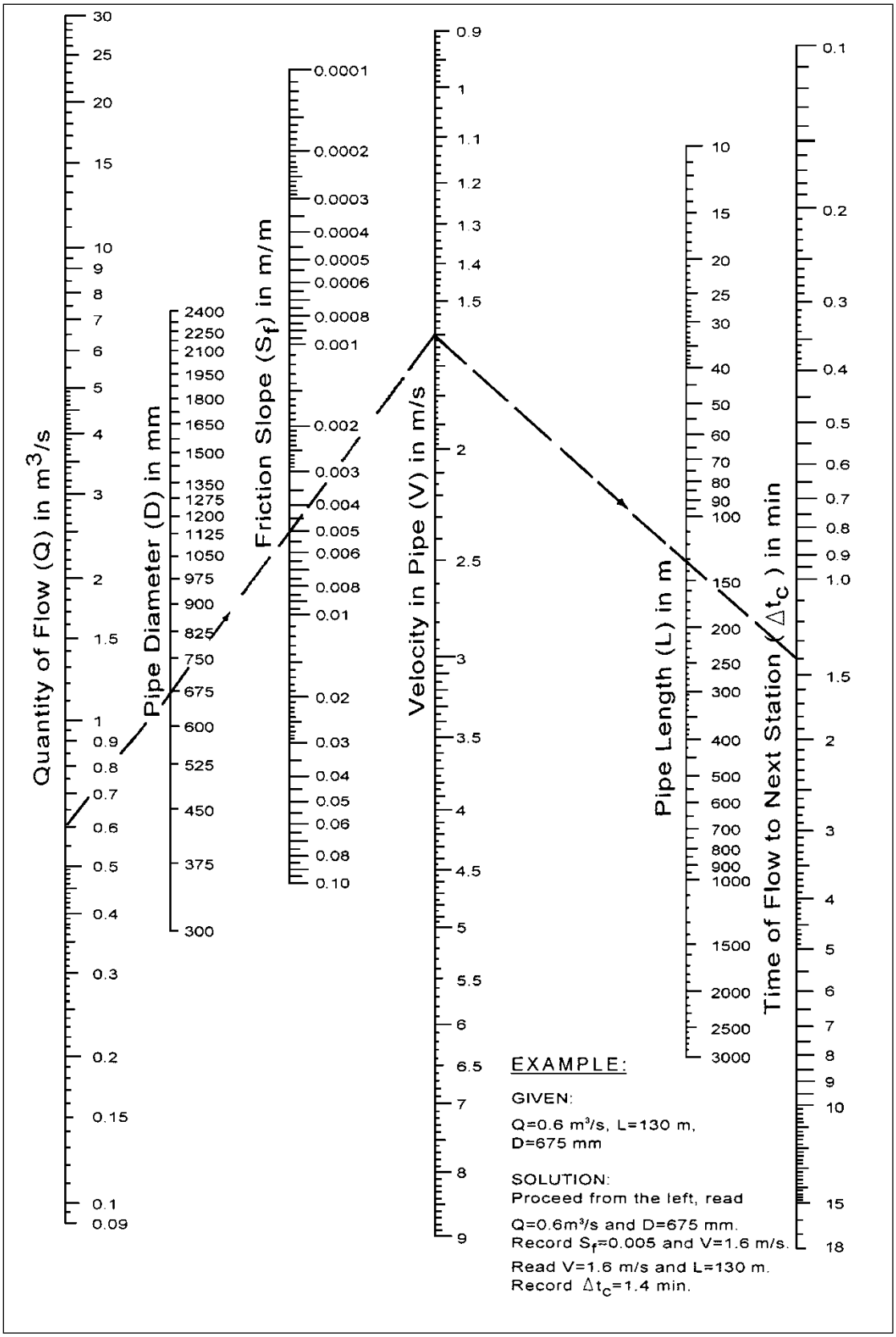


Figure 13.7(b) Concrete Pipe Flow Nomograph (U.S. Customary)
Source: *Open Channel Hydraulics* (Chow, V. T., 1959).

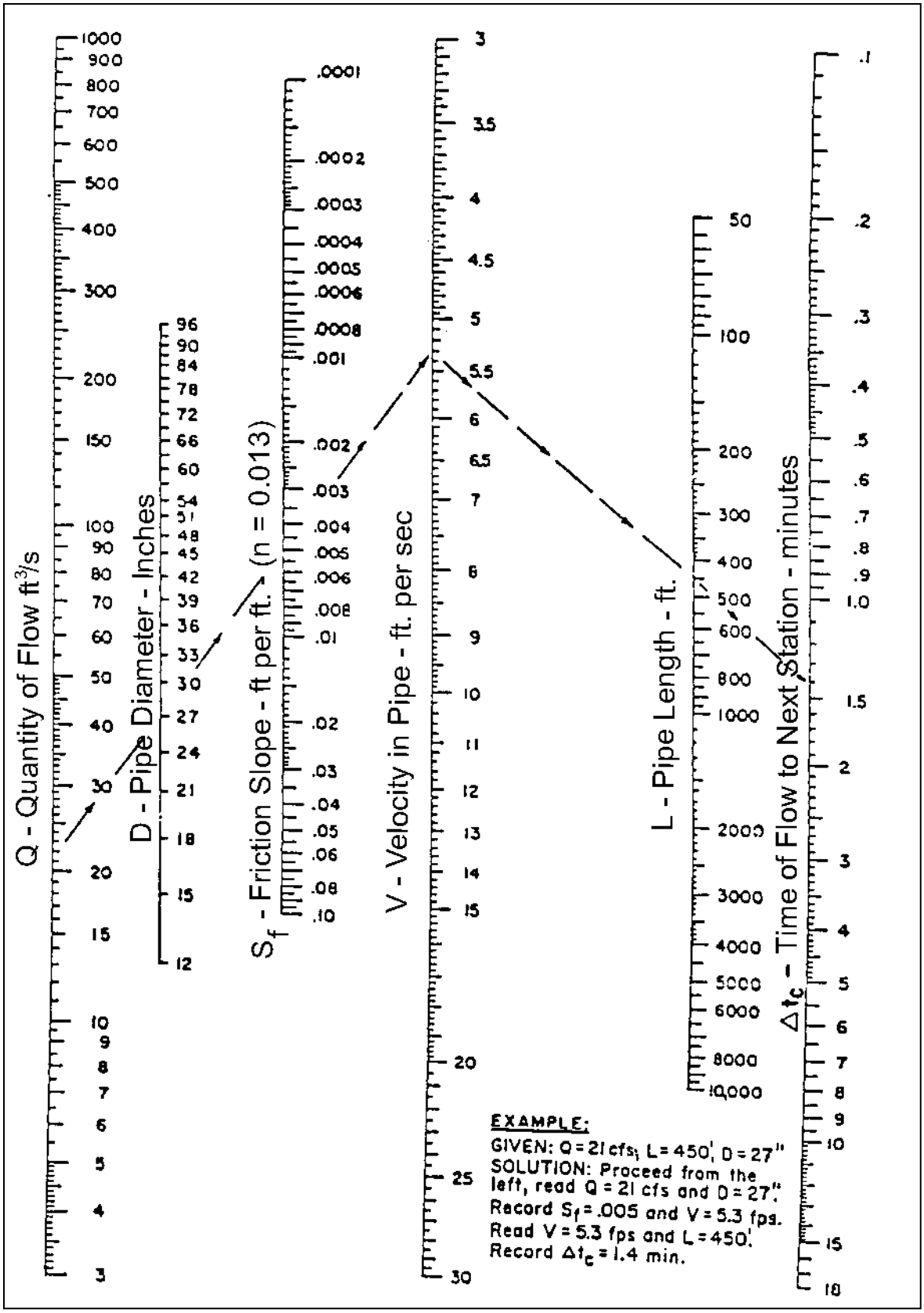
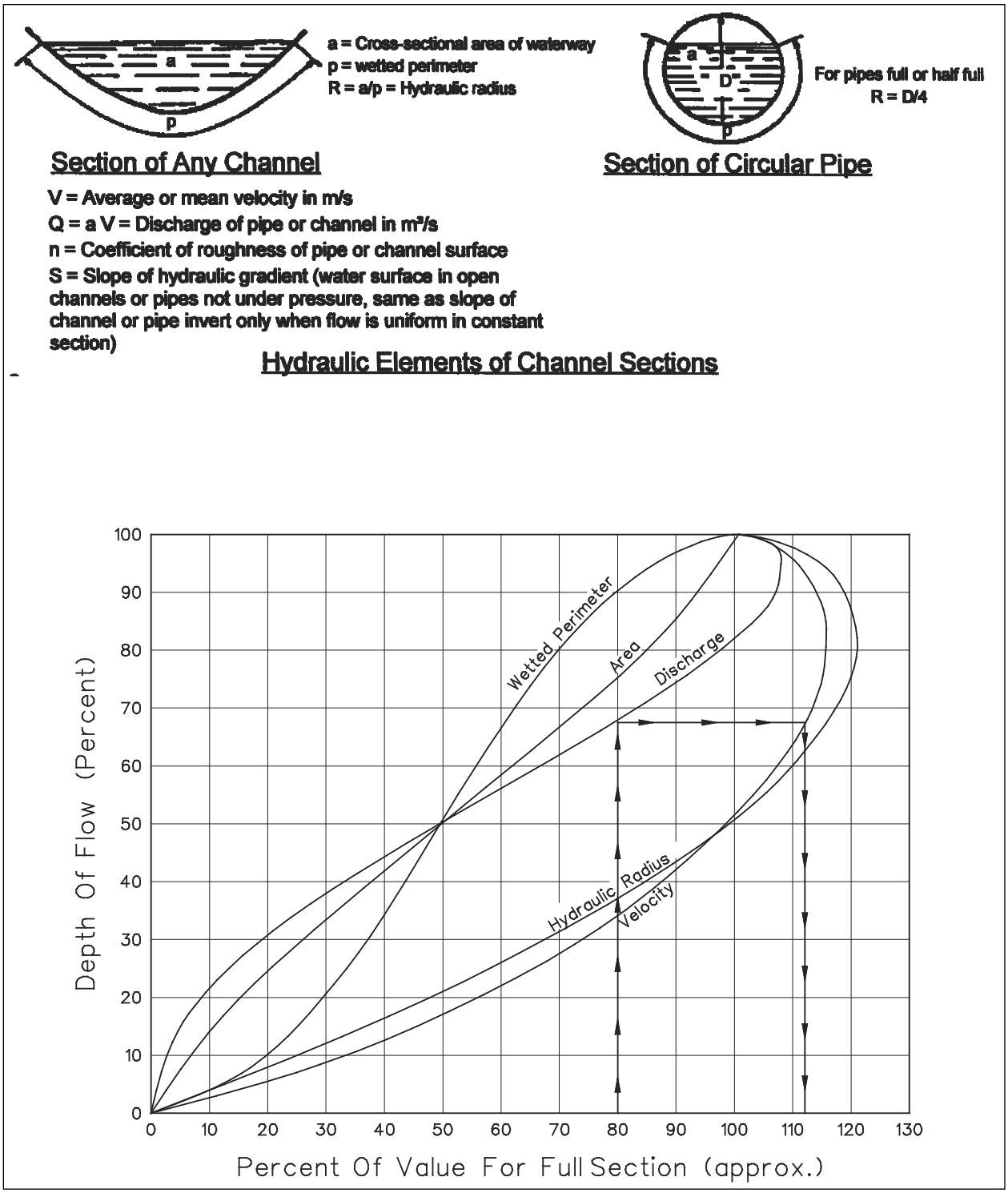


Figure 13.8 Values of Hydraulic Elements of Circular Section for Various Depths of Flow



Storm drain computation sheets, in both Metric and U.S. Customary Units, can be found in Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10, Section 10.3.B.

13.12 HYDRAULIC GRADE LINE

A. Introduction. The hydraulic grade line (HGL) is the last important feature to be established for the hydraulic design of storm pipes. This grade line 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 pipes 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 in-place 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. Usually, it is helpful to compute the energy grade line (EGL) first, then subtract the velocity head ($V^2/2g$) to obtain the HGL. Many methods are based on energy loss such that the EGL is computed and the velocity head ($V^2/2g$) is then subtracted from the energy grade to obtain the HGL.

Water quality treatment facilities placed in the storm drain system may significantly reduce the system capacity. The computation of the hydraulic grade line shall include the head loss caused by the treatment facility to determine if the facility causes an intolerable reduction in system capacity. If the hydraulic grade line upstream of the treatment facility is excessive, the treatment facility may be lowered in some cases to reduce the hydraulic grade line upstream of the device. If the device causes excessive head loss and cannot be lowered to reduce the upstream hydraulic grade line, then an alternative device having a lower head loss shall be considered.

B. Tailwater. For most design applications, the tailwater either will be above the crown of the outlet or can be considered to be between the crown and critical depth. To determine the EGL, begin with the tailwater elevation or $(d_c + D)/2$ (use only as an approximation for hand calculations; it is not valid if the designer is determining friction losses by computing backwater through the pipe), whichever is higher, add the velocity head for full flow and proceed upstream to compute all losses (e.g., exit losses, friction losses, junction losses, bend losses, entrance losses) as appropriate.

An exception to the above might be an 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 designer may consider the joint or coincidental probability of two events occurring at the same time. For 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. When applying this approach, it is necessary to perform two independent sets of hydraulic grade line computations for the combinations of frequencies. This is discussed in Section 13.2.G and Chapter 7, *Hydrology*, Section 7.2.E.

The hydraulic grade line using a 100-year flood stage of the receiving waters should be checked to determine if reverse flow conditions may occur. If reverse flow conditions are possible, then flap gates and/or a shut-off gate may be considered.

C. 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:

(Equation 13.10)

Metric

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

U.S. Customary

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

where: V = average outlet velocity, m/s (ft/s)
V_d = channel velocity downstream of outlet, m/s (ft/s)
C_o = exit loss coefficient (1.0 Metric, 0.3 U.S. Customary)

Note that, when $V_d = 0$ as in a reservoir, the exit loss is one velocity head. For partial full flow where the pipe outlets into a channel with moving water, the exit loss may be reduced to virtually zero.

D. Bend Loss. The bend loss coefficient for storm drain design is minor but can be evaluated using the formula:

(Equation 13.11)

$$h_b = 0.0033(\Delta)\left(\frac{V_o^2}{2g}\right)$$

where: Δ = angle of curvature, degrees

E. 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 in meters (feet). Energy losses from pipe friction may be determined by rewriting Manning's Equation with terms as previously defined:

(Equation 13.12)

Metric

$$S_f = \left(\frac{Q_n}{AR^{2/3}}\right)^2$$

U.S. Customary

$$S_f = \left(\frac{Q_n}{1.49AR^{2/3}}\right)^2$$

The head losses due to friction may be determined by the formula:

(Equation 13.13)

$$H_f = S_f L$$

Manning's Equation can also be written to determine friction losses for storm pipes as follows:

(Equation 13.14)

Metric

$$H_f = 6.35\frac{n^2V^2L}{D^{4/3}}$$

U.S. Customary

$$H_f = 2.87\frac{n^2V^2L}{D^{4/3}}$$

(Equation 13.15)

Metric

$$H_f = \frac{19.62 n^2 L}{R^{4/3}} \left(\frac{V^2}{2g} \right)$$

U.S. Customary

$$H_f = \frac{64.4 n^2 L}{R^{4/3}} \left(\frac{V^2}{2g} \right)$$

where:
H_f = total head loss due to friction, m (ft)
n = Manning's roughness coefficient
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/s^2 (32.2 ft/s^2)
S_f = slope of hydraulic grade line, m/m (ft/ft)

F. Manhole Losses. The head loss encountered from one pipe to another through a manhole 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^2 / 2g). Experimental studies have determined that the K value can be approximated as follows:

(Equation 13.16)

$$K = K_o C_D C_d C_Q C_p C_B$$

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_p = correction factor for plunging flow
C_B = correction factor for benching

1. Relative Manhole Size. K_o is estimated as a function of the relative manhole size and the angle of deflection between the inflow and outflow pipes (see Figure 13.10):

(Equation 13.17)

$$K_o = 0.1 \left(\frac{b}{D_o} \right) (1 - \sin\theta) + 1.4 \left(\frac{b}{D_o} \right)^{0.15} \sin\theta$$

where:
θ = the angle between the inflow and outflow pipes, degrees
b = manhole diameter, mm (in)
D_o = outlet pipe diameter, mm (in)

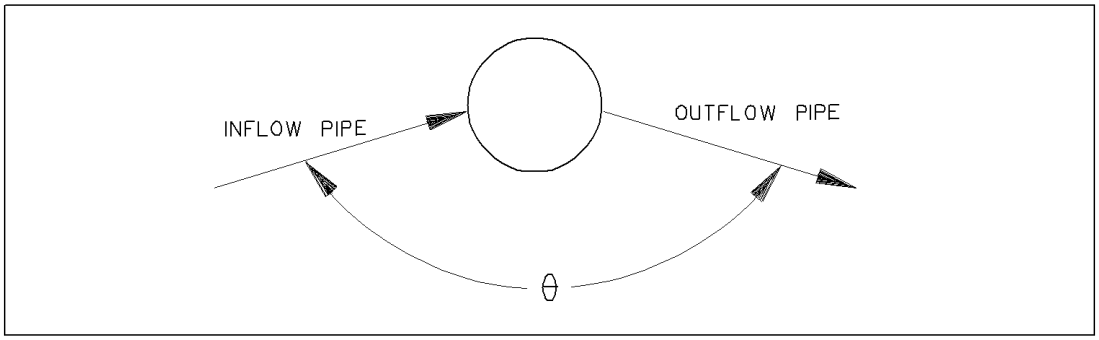
2. Pipe Diameter. A change in head loss due to differences in pipe diameter is only significant in pressure-flow situations where the depth in the manhole to outlet pipe diameter ratio, d/D_o, is greater than 3.2. Therefore, it is only applied in such cases:

(Equation 13.18)

$$C_D = \left(\frac{D_o}{D_i} \right)^3$$

where: D_i = incoming pipe diameter, mm (in)
D_o = outgoing pipe diameter, mm (in)

Figure 13.9 Deflection Angle



3. Flow Depth. The correction factor for flow depth is significant only in free surface flow or low pressures, where the d/D_o ratio is less than 3.2, and is only applied in such cases. Water depth in the manhole is approximated as the level of the hydraulic grade line at the upstream end of the outlet pipe. The correction factor for flow depth, C_d, is calculated by the following:

(Equation 13.19)

$$C_d = 0.5 \left[\frac{d}{D_o} \right]^{0.6}$$

where: d = water depth in manhole above outlet pipe invert, m (ft)
D_o = outlet pipe diameter, m (ft)

4. Relative Flow. The correction factor for relative flow, C_Q, is a function of the angle of the incoming flow and the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed as follows:

(Equation 13.20)

$$C_Q = (1 - 2\sin\theta) \left(1 - \frac{Q_i}{Q_o} \right)^{0.75} + 1$$

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 (cfs)
Q_o = flow in the outlet pipe, m^3/s (cfs)

To illustrate this effect, consider the manhole shown in Figure 13.10 and assume the following two cases to determine the impact of Pipe 2 entering the manhole:

Metric

$$C_{Q3-1} = (1 - 2\sin 180°) \left(1 - \frac{0.09}{0.12} \right)^{0.75} + 1 = 1.35$$

U.S. Customary

$$C_{Q3-1} = (1 - 2\sin 180°) \left(1 - \frac{3.2}{4.2} \right)^{0.75} + 1 = 1.34$$

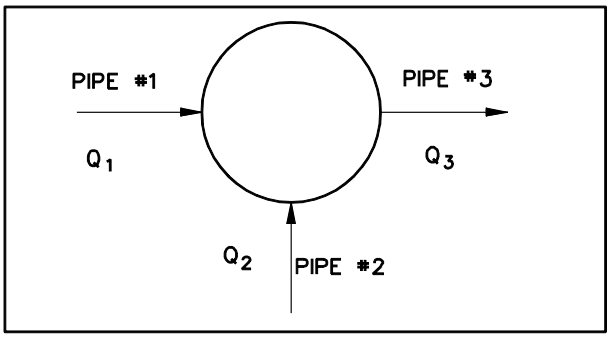
Case 1

Metric	U.S. Customary
Q_1 = 0.09 m^3/s, Q_2 = 0.03 m^3/s	Q_1 = 3.2 cfs, Q_2 = 1.0 cfs
Q_3 = 0.12 m^3/s, then C_Q = 1.35	Q_3 = 4.2 cfs, then C_Q = 1.34

Case 2

Metric	U.S. Customary
Q_1 = 0.03 m^3/s, Q_2 = 0.09 m^3/s	Q_1 = 1.0 cfs, Q_2 = 3.2 cfs
Q_3 = 0.12 m^3/s, then C_Q = 1.81	Q_3 = 4.2 cfs, then C_Q = 1.81

Figure 13.10 Relative Flow Effect



5. Plunging Flow. The correction factor for plunging flow, C_p, is calculated by the following:

(Equation 13.21)

$$C_p = 1 + 0.2 \left[\frac{h}{D_o}\right] \left[\frac{(h-d)}{D_o}\right]$$

where: C_p = correction for plunging flow
h = vertical distance of plunging flow from flow line of incoming pipe to the center of outlet pipe, m (ft)
D_o = outlet pipe diameter, m (ft)
d = water depth in manhole, m (ft)

This correction factor corresponds to the effect of another inflow pipe or surface flow from an inlet plunging into the manhole on the inflow pipe, for which the head loss is being calculated. Using the notations in Figure 13.10 for the example, C_p is calculated for Pipe 1 when Pipe 2 discharges plunging flow. The correction factor is only applied when $h > d$.

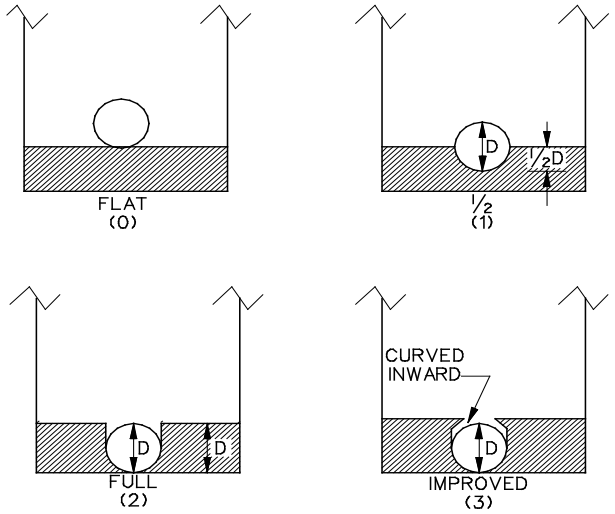
6. Benching. The correction for benching in the manhole, C_B, is obtained from Table 13.4. Benching tends to direct flows through the manhole, resulting in reductions in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed.

Table 13.3 Correction for Benching

Bench Type	Correction Factors, C_B	
	Submerged[1]	Unsubmerged[2]
Flat floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07
Improved	0.40	0.02

[1]Pressure flow, $d/D_o > 3.2$
[2]Free surface flow, $d/D_o < 1.0$

Figure 13.11 Schematic Representation of Benching Types



7. Summary. In summary, to estimate the head loss through a manhole 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.

G. Hydraulic Grade Line Design Procedure. The equations and charts necessary to manually calculate the location of the hydraulic grade line are included in this chapter. A step-by-step procedure is given to manually compute the HGL. Table 13.5 can be used to document the procedure.

If the HGL is above the pipe crown at the next upstream manhole, pressure-flow calculations are indicated; if it is below the pipe crown, then open channel flow calculations should be used at the upstream manhole. The procedures outlined here assume full flow. The process is repeated throughout the storm drain system. If all HGL elevations are acceptable, then the hydraulic design is adequate. If the HGL exceeds an inlet elevation, then adjustments to the trial design must be made to lower the water surface elevation.

See Figure 13.12 for a sketch depicting the use of energy losses in developing a storm drain system:

Step 1 Enter in Column 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 Column 2 the tailwater elevation if the outlet will be submerged during the design storm; otherwise, refer to the tailwater discussion in Section 13.12.B. for procedure.

Step 3 Enter in Column 3 the diameter, D_o, of the outflow pipe.

Step 4 Enter in Column 4 the design discharge, Q_o, for the outflow pipe.

Step 5 Enter in Column 5 the length, L_o, of the outflow pipe.

Step 6 Enter in Column 6 the outlet velocity of flow, V_o.

Step 7 Enter in Column 7 the velocity head, $V_o^2/2g$.

Step 8 Enter in Column 8 the exit loss, H_o.

Step 9 Enter in Column 9 the friction slope (SF_o) in m/m (ft/ft) of the outflow pipe. This can be determined by using the equation given in Publication 15M, Design Manual, Part 4, *Structures*, PP3.5.2.7.5. *Note: Assumes full-flow conditions*.

Step 10 Enter in Column 10 the friction loss (H_f) that is computed by multiplying the length (L_o) in Column 5 by the friction slope (SF_o) in Column 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 Column 11 the initial head loss coefficient, K_o, based on relative manhole size as computed by Equation 13.7.

Step 12 Enter in Column 12 the correction factor for pipe diameter, C_D, as computed by Equation 13.18.

Step 13 Enter in Column 13 the correction factor for flow depth, C_d, as computed by Equation 13.19. *Note: This factor is only significant where the d/D_o ratio is less than 3.2*.

Step 14 Enter in Column 14 the correction factor for relative flow, C_Q, as computed by Equation 13.20.

Step 15 Enter in Column 15 the correction factor for plunging flow, C_p, as computed by Equation 13.21. *Note: This correction factor is only applied when $h > d$.*

Step 16 Enter in Column 16 the correction factor for benching, C_B, as determined in Table 13.5.

Step 17 Enter in Column 17 the value of K as computed by Equation 13.16.

Step 18 Enter in Column 18 the value of the total manhole loss, $KV_o^2/2g$.

Step 19 If the tailwater submerges the outlet end of the pipe, enter in Column 19 the sum of Column 2 (TW elevation) and Column 7 (exit loss) 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 (dc + D)/2.

Step 20 Enter in Column 20 the sum of the friction head (Column 10), the manhole losses (Column 18), and the energy grade line (Column 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.

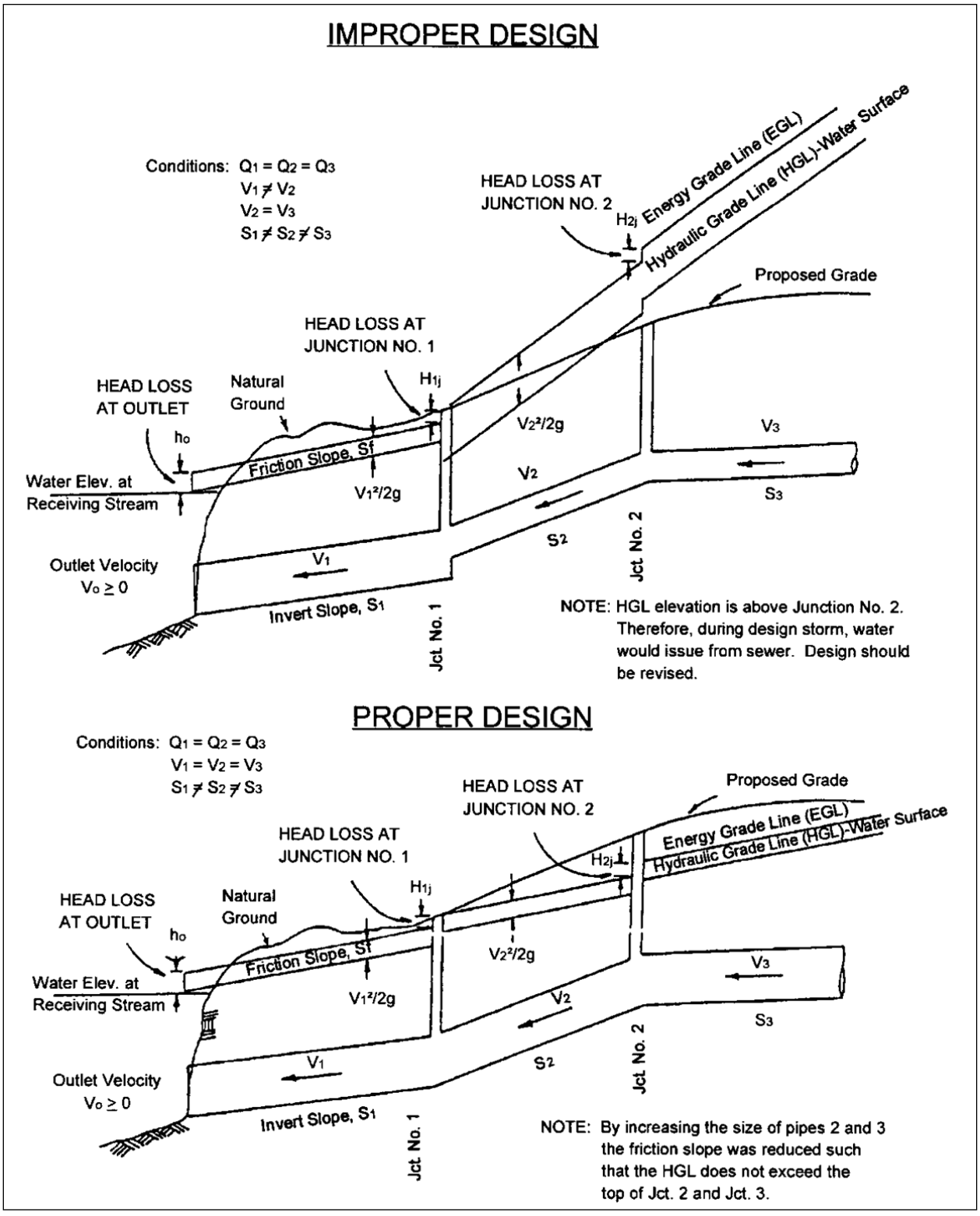
Step 21 Determine the HGL (Column 21) throughout the system by subtracting the velocity head (Column 7) from the EGL (Column 20).

Step 22 Check to make certain that the HGL is below the level of allowable highwater at that point. If the HGL is above the finished grade elevation, water will exit the system at this point for the design flow. *Note: TOC is top of curb.*

Table 13.4 Hydraulic Grade Line Computation Form

Station	TW	D_o	Q_o	L_o	V_o	$V_o^2/2g$	H_o	SF_o	H_f	K_o	C_D	C_d	C_Q	C_p	C_B	K	$K(V_o^2/2g)$	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)

Figure 13.12 Use of Energy Losses in Developing a Storm Drain System



Calculating backwater for partial full-flow conditions through the pipe is tedious and may require many iterations to determine if gravity-flow or pressure-flow conditions exist. The junction losses may be quite different depending on flow types. Momentum methods may be required to estimate hydraulic jump conditions. The above procedure is simplistic and may not apply well to gravity-flow conditions, especially if supercritical flow is encountered.

The above procedure applies to pipes that are flowing full, as should be the condition for the design of new systems. If a partial full-flow condition exists, the EGL is located one velocity head above the water surface.

13.13 WATER QUALITY TREATMENT

Storm drain systems in high quality or exceptional value watersheds may be required to be fitted or retrofitted with stormwater treatment facilities. The treatment facilities may remove oils, grease, floatables and sediment. The amount of pollutants that treatment facilities can remove varies depending on the type of facility installed. Commercially available facilities typically provide design data for use in selecting the size and type needed. Chapter 11, *Surface Water Environment*, also provides some additional aspects of stormwater quality relating to storm drainage systems. Various treatment facility types may not provide an "equivalent" amount of pollutant removal or level of service. The criteria for pollutant removal must be established before the alternative facilities are designed to provide for alternative bidding. Substitutions of other treatment systems must be designed to meet the design criteria used for those alternatives shown in the contract plans.

13.14 INVERTED SIPHONS

An inverted siphon carries the flow under an obstruction such as sanitary sewers, water mains or any other structure or utility lane that is in the path of the storm drain line. The storm drain invert is lowered at the obstacle and is raised again after the crossing. In certain instances it may be useful to provide two barrels through the inverted section in order to maintain the desired conveyance under the obstacle. Regardless of the number of pipes used for the inverted siphon, to minimize sediment deposition and promote self cleansing, pipes should be designed to maintain a minimum velocity of greater than 0.9 m/s (3 ft/s). Specific criteria for designing inverted siphons can be found in most hydraulics textbooks.

13.15 UNDERDRAINS

In certain areas, groundwater can be a significant problem because it attacks foundations, substructures, subgrades and other aspects of highway components. In most soils where groundwater is a problem a system of underdrains is installed for the removal of excess moisture. Underdrains may consist of networks of perforated (or otherwise permeable) pipe or collector fields. Where such appurtenances are needed, the additional expense in their installation may be fully justified in terms of future savings in roadway and structure maintenance costs. Publication 13M, Design Manual, Part 2, *Highway Design*, and Publication 72M, *Roadway Construction Standards*, should be referenced for specific design parameters related to underdrains and there configuration with respect to the highway environment.

Percolation rates for groundwater may be obtained from NRCS offices or measured at the site using field tests. Collector pipe sizes and networks may then be established for the removal of that water. French drains can be very useful where the unwanted groundwater percolation rates are relatively high. Collector fields may be useful where reasonable outfalls for groundwater are not available. All of the above appurtenances may be enhanced by the use of some type of geotextile filter material.

13.16 COMPUTER PROGRAMS

To assist with storm drain system design, it is recommended that HEC-22 (FHWA, 2001) be consulted prior to completing the design of systems situated in difficult or problematic locations or conditions.

Storm drainage software has a wide range of capabilities and the selection of software may be based on the various features that the software provides. Hydrologic methods are of primary importance. In addition to the rational

method, some software provides various hydrographic techniques such as the NRCS curve number method, or Kinematic Wave, which may be better suited to a specific condition or location. Hydraulic capabilities of storm drain software also vary and may include detailed inlet capture modeling, junction loss and hydraulic grade line analysis. Some programs have design capabilities that can optimize inlet spacing or pipe sizes. The quality of the analysis provided by the different software programs may vary significantly; therefore, the user must be aware of the limitations of the software and must carefully review the results to determine if the methods are properly applied and if the results are reasonable. Many software packages are interfaced with Geographic Information Systems to automate and assist in the importing and exporting of data into and out of the modeling programs. Some software packages provide details of the storm system suitable for construction plans, and some software packages are integrated with the roadway design software packages. For guidance on acceptable software programs and methods, one should reference Chapter 7, *Hydrology* and Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10.

13.17 DEFINITIONS

Following are discussions of concepts that will be important in a storm drainage analysis and design. These concepts will be used throughout the remainder of this chapter in addressing different aspects of storm drainage analysis:

Check Storm - The use of a less frequent event to assess hazards at critical locations where water can pond to appreciable depths is commonly referred to as a check storm or check event.

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

Crown - The crown, sometimes known as the soffit, is the top inside of a pipe.

Culvert - A culvert is a closed conduit whose purpose is to convey surface water under a roadway, railroad or other impediment. It may have one or two inlets connected to it to convey drainage from the median area.

Curb-Opening - A drainage inlet consisting of an opening in the roadway curb.

Drop Inlet - A drainage inlet with a horizontal or nearly horizontal opening.

Equivalent Cross Slope - An imaginary straight cross slope having conveyance capacity equal to that of the given compound cross slope.

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

Flow - Flow refers to runoff that is moving during a storm.

Frontal Flow - The portion of the flow that passes over the upstream side of a grate.

Grate Inlet - A drainage inlet composed of a grate in the roadway section or 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 that is utilized to convey stormwater runoff. A composite gutter section consists of the section immediately adjacent to the curb, parking lane, shoulder or pavement that is at a constant cross slope and is used for conveyance. See Section 13.7 for additional information.

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). In an open channel, the HGL is equal to the elevation of the channel bottom plus the flow depth.

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

Invert - The invert is the inside bottom of the pipe.

Lateral Line - A lateral line, sometimes referred to as a lead, has inlets connected to it but has no other inlets or storm pipes connected into it. It may be smaller in diameter than its trunkline and may convey a lesser flow.

MPOs - Metropolitan Planning Organization

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. Pressure occurs when pipes and inlets are flowing full.

RPO - Regional Planning Organization

Runby/Bypass - Carryover flow that 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 runby for one design storm and larger or smaller amounts for other storms.

Sag Point - A low point in a vertical curve.

Scupper - A vertical hole through a bridge deck for deck drainage. Sometimes, a horizontal opening in the curb or barrier is called a scupper.

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

Slotted Drain Inlet - A drainage inlet composed of a continuous slot built into the top of a pipe that serves to intercept, collect and transport the flow. Two types in general use are the vertical riser and the vane type.

Storm Drain - A storm drain is a closed conduit that conveys stormwater that has been collected by inlets to an adequate outfall. It generally consists of laterals or leads and trunk lines or mains.

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

Spread - The width of stormwater flow in the gutter measured laterally from the roadway curb. See Section 13.7 for methodology to compute.

Trunk Line - A trunk line is the main storm drain line. Lateral lines may be connected at inlet structures or manholes. A trunk line is sometimes referred to as a "main."

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

13.18 CHAPTER 13 NOMENCLATURE

Symbol	Definition	Units
A	Area of cross section of flow	m^2 (ft^2)
A	Watershed drainage area	ha (ac)
a	Depth of depression	mm (in)
B	Barrel width	m (ft)
C	Runoff coefficient	dimensionless
C_o	Exit loss coefficient (1.0 Metric 0.3 U.S. Customary)	dimensionless
C_B	Correction factor for benching	dimensionless
C_d	Correction factor for flow depth	dimensionless
C_D	Correction factor for pipe diameter	dimensionless
C_P	Correction factor for plunging flow	dimensionless
C_Q	Correction factor for relative flow	dimensionless

D	Culvert diameter or barrel height	m (ft)
d	Depth of flow	m (ft)
E_o	Ratio of frontal flow to total gutter flow	dimensionless
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 (subscript indicates section)	m (ft)
I	Rainfall intensity	mm/h (in/h)
K_E	Entrance loss coefficient	dimensionless
K_o	Initial head loss coefficient based on relative manhole size	dimensionless
L	Length of culvert, length of deck	m (ft)
n	Manning's roughness coefficient	dimensionless
P	Wetted perimeter	m (ft)
Q	Discharge, flow	m^3/s (cfs)
Q_i	Flow in inflow pipe	m^3/s (cfs)
Q_o	Flow in outflow pipe	m^3/s (cfs)
Q_s	Gutter capacity above the depressed section	m^3/s (cfs)
Q_w	Gutter capacity in the depressed section	m^3/s (cfs)
R	Hydraulic radius (A/P)	m (ft)
S	Slope of culvert	m/m (ft/ft)
S_f	Slope of hydraulic grade line	m/m (ft/ft)
S_L	Longitudinal slope of pavement	m/m (ft/ft)
S_w	Depressed section slope	m/m (ft/ft)
S_x	Pavement cross slope	m/m (ft/ft)
T	Total spread on pavement	m (ft)
T_S	Spread above depressed section	m (ft)
t_c	Time of concentration	minutes
t_t	Travel time	minutes
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)
W	Width of depression	m (ft)
W_{dk}	Width of drained deck	m (ft)
θ	Angle between piers	degrees

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CHAPTER 14

POST-CONSTRUCTION STORMWATER MANAGEMENT

14.0 INTRODUCTION

A. Overview. The purpose of this chapter is to provide design guidelines for post-construction stormwater management (PCSM) on Department projects. The goals of PCSM, risk considerations, and descriptions of Department-approved stormwater control measure (SCM) types are described in this chapter. This chapter also provides general design criteria for detention and retention storage facilities (basins), procedures for sizing basins and performing routing calculations, and procedures for observing soil profiles and infiltration testing.

B. Background. The Department performs a broad spectrum of activities while maintaining and improving the Commonwealth's transportation system. Highway improvement projects involve, to varying degrees, altering the existing landscape through a combination of clearing, compaction, and impervious cover. These activities can disrupt the natural hydrologic processes that reduce surface runoff, such as interception and infiltration. The development of land into less pervious areas generally leads to an increase in stormwater runoff volume, higher peak flows, higher average temperature of runoff, collection of a larger mass of pollutants (due to lack of infiltration capacity), and an increased flooding hazard for downstream waterways. These factors can contribute to degradation – changes in the physical, chemical, and biological properties – of the receiving waters.

Not all roadway improvement projects are equal in terms of their effect on stormwater runoff quantity, quality, and patterns. The Department's projects tend to have less of an effect on runoff characteristics than other types of development, as most projects involve improvements within an existing legal right-of-way, which has already been largely disturbed in order to construct the highway facility. The effects are generally proportional to the amount of additional impervious area being proposed.

Among Pennsylvania's water quality standards are antidegradation requirements, which are described in Section 93.4a of Title 25 of the PA Code. The antidegradation requirements are aimed at protecting the existing instream uses of surface waters, in addition to maintaining and protecting the water quality of High Quality (HQ) and Exceptional Value (EV) waters. Stormwater runoff from roadways is considered a point source discharge that has the potential to impact existing uses and water quality, so it is regulated by PA DEP.

The most direct way to achieve antidegradation is to mimic the natural, pre-development hydrologic conditions, which are usually dominated by infiltration and evapotranspiration (ET - see definitions in Section 14.21). However, the characteristics of highways and highway projects often limit opportunities to address runoff quantities via infiltration and ET. The Department’s policies and design guidelines on PCSM and antidegradation are appropriate for the highway environment.

14.1 TARGETED PARAMETERS FOR PCSM

Three key measures are used to assess the potential for impacts from stormwater runoff – rate, volume, and quality. The goal of PCSM is to prevent or minimize any increase in the quantity (rate and volume) of runoff while also minimizing the factors affecting the quality. The most commonly cited concerns related to potential effects of roadway runoff on water resources are:

- Stream channel erosion and flooding resulting from increases in runoff rate and volume.
- Water quality impacts to streams from particulates, floatables, hydrocarbons, and deicing materials.
- Thermal impact on streams from heat transfer, base flow reduction, and loss of riparian buffer vegetation.

The items listed above are discussed below in more detail.

A. Runoff Rate and Volume. It is well documented that a direct relationship exists between the imperviousness of

a watershed and the impairment of its surface waters. Unmitigated increases in the rate and volume of runoff discharging from developing areas have a cumulative effect, which has been shown to cause flooding and erosion of streams. Increases in the rate and volume of runoff are mostly dependent on the amount of impervious area replacing pervious area, the amount of disturbance, and the time it takes for the runoff to concentrate and leave the site. Some types of projects add relatively little (or no) impervious area and require minimal disturbance, while other types of projects create large areas of impervious cover and disturbance. Increased discharges can often be prevented in the former case by implementing qualitative and non-structural measures; whereas the latter case usually requires structural measures for peak flow and volume mitigation.

B. Constituents in Runoff. Chapter 7 of the PA Stormwater Best Management Practices Manual (PA DEP, 2006), herein referred to as the "BMP Manual," lists common pollutant constituents in highway runoff. Table 2.1 of the NCHRP Report 728 lists a number of additional common pollutant sources and constituents in highway runoff and potential receiving water impacts. Section 2.2 of the same report provides detail of each constituent, particularly sediments, metals, organic compounds, litter, debris, nutrients, chlorides, and indicator bacteria. Many, if not most of these constituents occur in relatively small concentrations and are usually addressed when the increases in the volume of runoff are mitigated.

In urban areas, part of the pollution load generated by traffic and winter road maintenance enters and is stored in roadside snow and is eventually released during snowmelt. Pollutant accumulation in roadside snow increases with traffic volume and snow residence time. The fine sediments that make up anti-skid materials can be carried into an adjacent stream or accumulate in and clog an infiltration facility. In addition, chlorides and other soluble chemicals in deicing materials and salts can spike concentrations in groundwater. The Department's MS4 permit lists several good housekeeping approaches that PennDOT utilizes to minimize pollutant loadings from winter maintenance materials:

- Monitoring and minimizing the volume of winter maintenance materials used.
- Protecting salt storage and loading areas from weather influences.
- Cleaning around the area where materials are dispensed immediately after deicing operations have ceased.

The highest pollutant concentration and loads occur in the first portion of the storm event, a phenomenon termed the "first flush." Knowing how to address the first flush of pollutants goes a long way in improving water quality. The peak rate and volume control achieved through application of the design standards described in Section 13.7.C of Pub. 13M results in the treatment of a major portion of pollutants associated with particulates from impervious surfaces, in addition to flood and stream channel protection during most storms. It should be noted, however, that solutes will continue to be transported in runoff throughout the storm, regardless of its magnitude.

C. Thermal Impact. In warm months, heat transferred from stormwater runoff to cold-water streams can be a potential source of thermal impacts. This type of effect is pronounced in urban areas. Runoff is heated as it passes over impervious surfaces with large heat storage due to solar radiation. As the magnitudes of impervious surfaces and open water ponds or basins increase, or when riparian areas (i.e., vegetated buffer zones) decrease, there is a potential for increasing summer stream temperatures. Studies have shown similar effects in the winter, except that impervious areas cool the runoff below the stream's ambient temperature. Thermal impacts are also particularly important for surface waters that have a fishery classification of Cold Water Fishes or Trout Stocking; this includes waters that are High Quality waters due to an existing or designated use as a Class A wild trout stream by the PA Fish and Boat Commission. Strategies to reduce the potential for thermal impacts include:

- Limit the use of curb and gutter sections to the extent practicable.
- Utilize vegetated channels in lieu of storm sewers to the extent practicable.
- Consider vegetated islands in-lieu of concrete islands.
- Maintain naturally occurring vegetation (i.e., buffer zones, including wetland and riparian) along streams, rivers and other surface waters for shading and thermal protection.

Two additional factors that should be considered when evaluating a project's potential for thermal impacts are the distance from the impervious areas to the surface water and the size of the surface water relative to the amount of runoff generated by the impervious areas. Generally speaking, the longer the travel time through vegetated or shaded areas, the cooler the runoff will be when it eventually reaches the surface water. Although the use of vegetated swales for stormwater conveyance is preferred, storm sewers are buried and generally stay cool; thus, a significant amount

of heat loss can take place in a long sewer run before the runoff reaches the surface water. The size of the receiving surface water is also an important factor due to mixing phenomena. Large highway projects that are adjacent to relatively small streams or bodies of water, such as headwaters, have the potential to adversely affect the temperature regime because runoff from the highway may produce a significant percentage of the total surface flow into the nearby stream. In this type of situation, it is particularly important to address potential thermal impacts using the strategies outlined above. However, it is more likely that the runoff produced by the road during a storm is insignificant compared to the flow in the receiving surface water. Additionally, the water quality criteria do not preclude the allowance of a reasonable mixing zone if there is no significant effect on the ambient temperature of the stream outside the mixing zone.

14.2 RISK CONSIDERATIONS

There are many factors that may preclude the use of an SCM, even if it otherwise appears to be applicable. The most common factors limiting their use include karst topography, high groundwater table, limiting soil zones, shallow depth to bedrock, and compacted soils. A few of these factors are described below. It should be noted that the presence of limiting factors does not exempt a project from analyzing the post-construction stormwater conditions and potential impacts to receiving waters.

A. Infiltration SCMs. The use of infiltration SCMs is challenging in cold-climate states such as Pennsylvania. Frozen soils can dramatically reduce, or stop, the rate of infiltration, chlorides may pose a risk to groundwater, and sand used as abrasives on roads may clog infiltration facilities. Consequently, designers need to make modifications to these SCMs to make them effective in cold climates. Consider areas with higher soil infiltration rates (from base design criteria) to account for the clogging potential from road abrasives and to a certain degree, account for the reduced infiltration rates during the winter season. When infiltration practices are used next to a road or pavement, they should be set back in order to avoid potential frost heave conditions. Infiltrated water can contribute to ice lenses that form beneath the road surface, aggravating frost heave and potentially causing damage. The maximum ponding elevation in a facility should be no higher than the minimum subgrade elevation of the road. Setback restrictions can be avoided by using other measures to protect pavement. For example, pavement can be insulated or underlain with a very thick gravel to protect against frost damage. Refer to Publication 242, *Pavement Policy Manual*, for additional guidance.

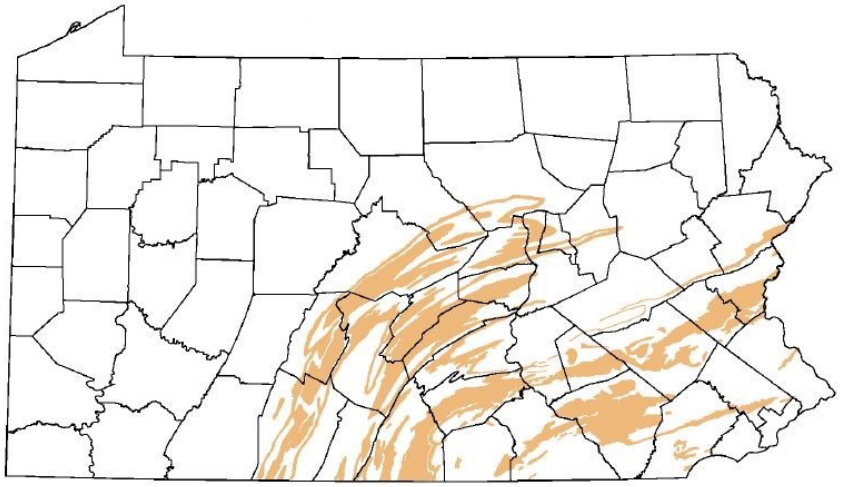
Roadway runoff can generate high levels of suspended solids. Discharge directly to an infiltration system should be avoided. Infiltration SCMs are appropriate for roadway systems but must be designed in conjunction with a pre-treatment measure (structural or non-structural) that reduces the amount of sediment and other particulate matter in roadway runoff prior to infiltration. There are a variety of options that will reduce sediment loads, including:

- Vegetated systems such as grassed swales, filter strips, and bioretention.
- During construction, sediment filter bags on inlets and other erosion and sediment pollution control best management practices (or ESPC BMPs).
- Maintenance measures such as street sweeping and vacuuming.

Using one or more of these measures before discharging to an infiltration SCM will minimize the accumulation of sediment that could lead to failure of the SCM. All measures for sediment reduction require regular maintenance.

B. Karst Topography. Karst terrain is characterized by well drained soils, closed depressions, sinkholes, springs, and disappearing and reappearing streams. Karst landscapes are underlain by bedrock strata with frequent occurrences of crevices, cavities, and caves. These landscapes exhibit high surface runoff losses through soils and sinkholes which alter "typical" surface runoff rates common in non-karst areas. In Pennsylvania, most karst lands are underlain by soluble limestone and dolomite (soluble in geologic time), collectively referred to as "carbonate rock." Thick sequences of carbonate bedrock underlay a large area in central and southeastern Pennsylvania (see Figure 14.1 below). In karst regions, consideration must be given to appropriate evaluation of surface runoff, characterization of site specific karst risks, and care in the design of infiltration SCMs.

Figure 14.1 Carbonate Bedrock in Pennsylvania



It is important to consider the appropriateness of infiltration SCMs in karst areas on a project by project basis. In general, areas with less than 4 feet of soil cover over carbonate bedrock should be avoided, and ponding depths in infiltration system should be shallow. Infiltration trenches are not recommended in areas with pronounced karst topography due to the potential for sinkhole formation and groundwater contamination. However, these limitations should not preclude infiltration altogether. Infiltration SCMs may be provided where runoff can be spread over a large area with a shallow maximum ponding depth. This should be done at existing, undisturbed grades, if possible, to avoid excavation and maintain sufficient soil depth above the bedrock. Where possible, consideration should be given to use of numerous smaller upland SCMs instead of one larger SCM at the downstream point of interest to spread out the infiltration function. Where infiltration is not feasible, maximize the use of non-structural practices and consider SCMs with high evapotranspiration characteristics, such as extended release bioretention facilities.

The impervious area loading ratio (SCM tributary impervious area to basin bottom area) is an important benchmark. The BMP Manual provides guidance on impervious area loading ratios recommended in karst areas. These values are often difficult to achieve as part of roadway projects due to inherent space limitations. Designers should be aware that exceeding the loading ratios will usually result in a shorter SCM design life because of the increased risk of sinkhole formation.

In karst areas, greater vigilance needs to be applied to inspection and maintenance of stormwater SCMs. SCMs collect runoff from large drainage areas and can increase the volume and rate of water entering karst systems. Prior to and shortly after project construction water may move downward through soil relatively slowly; subsurface erosion may be minimal, and no surface evidence of subsurface erosion may be visible for several years. Over time, as these small amounts of subsurface erosion continue the rate of vertical water movement and erosion may gradually increase resulting in shallow surface depressions. If these surface depressions go unnoticed and are allowed to progress, they can ultimately result in creation of cover collapse sinkholes. Therefore, it is very important to implement regular and frequent inspection of stormwater facilities in karst regions. In addition, emergency action plans should be put in place to define actions and responsibilities if evidence of depressions or sinkholes are observed in any conveyance or SCMs. The emergency action plan should identify appropriate contacts and the actions to be taken if depressions, sinkholes, or other failure modes are observed.

14.3 LEGAL

A. Federal Clean Water Act (CWA), Section 303d. States are required to assess and classify water bodies and streams based upon their water quality. If a stream is found to be "impaired," a Total Maximum Daily Load (TMDL) must be established for the impairing pollutant(s). The TMDL is an estimate of the total loading of the particular pollutant from both point and nonpoint sources that the water body can receive without exceeding applicable water quality standards. This standard includes a "margin of safety." The TMDL must then be met through implementation of best management practices and retrofits.

B. Federal CWA, Section 401. Requires DOT projects that may result in a discharge of pollutants to obtain a state water quality certification if the project requires a CWA Section 404 permit. The water quality certification may place conditions on the construction of the project.

C. Federal CWA, Section 402. Referred to as the NPDES permit, this section requires authorization for the discharge of any pollutant into navigable waters.

D. Federal NPDES Regulations at 40 CFR Part 122. Contains provisions for implementing the NPDES program.

E. Endangered Species Act (ESA) of 1973. The US Fish and Wildlife Service and the National Marine Fisheries Service are responsible for determining critical habitat for endangered species for terrestrial and native freshwater species and for marine and anadromous species respectively. If a project will affect this habitat, the Department must develop mitigation measures, which may potentially include retrofitting.

F. PA Clean Streams Law. Establishes the general authority for the PA DEP to establish standards for the protection, maintenance and restoration of the Commonwealth's water resources.

G. 25 PA Code Chapter 92a. This chapter sets forth permitting, monitoring, and compliance requirements with regard to the PA DEP implementation of the NPDES program. The PA DEP permit process for the NPDES Permit for Stormwater Discharges Associated with Construction Activities is promulgated, in part, through this regulation. The NPDES Permit regulates discharge or potential discharge of stormwater into waters of this Commonwealth from construction activities.

H. 25 PA Code Chapter 93. Chapter 93 establishes requirements to protect designated and existing water quality uses of surface waters. It also provides for the implementation of antidegradation requirements for activities that have the potential to discharge to Special Protection watersheds (HQ and EV). For special protection waters, non-discharge alternatives must be evaluated that are environmentally sound and will (1) minimize accelerated erosion and sedimentation during the earth disturbance activity, and (2) achieve no net change from pre-development to post-development volume, rate and concentration of pollutants in water quality. Where no environmentally sound and cost-effective non-discharge alternatives exist, Chapter 93 requires that the applicant demonstrate that the discharge will maintain and protect the existing quality of receiving surface waters. This is accomplished when a new, additional or increased discharge uses ABACT. The Chapter 102 permit application form includes an Antidegradation Analysis Module for evaluating impacts to designated and existing uses in HQ and EV watersheds.

I. 25 PA Code Chapter 96. The purpose of this chapter is to establish the process for achieving and maintaining water quality standards.

J. 25 PA Code Chapter 102. This chapter of the PA Code addresses requirements for earth disturbances, including erosion and sediment control plans, post-construction stormwater management plans, and riparian buffers.

K. 25 PA Code Chapter 105. Section 105.17 defines EV wetlands. Section 105.18a outlines the requirements that must be met in order for PA DEP to grant a Chapter 105 (water obstructions and encroachments) permit. The requirement that is most directly related to stormwater discharges applies to wetlands and provides, "The project will not cause or contribute to pollution of groundwater or surface water resources or diminution of resources sufficient to interfere with their uses."

14.4 SCM TOOLBOX – NON-STRUCTURAL

This section includes brief descriptions of each of the non-structural SCMs available for use. These descriptions should supplement the technical guidance that can be found in the BMP Manual and other similar publications listed under References in Section 14.24.

A. Minimize Compaction. The post-construction runoff from a project can be reduced by minimizing the amount of area that is compacted. Compaction of a previously undisturbed area can significantly reduce the infiltration capacity of that soil. Compaction is normally a planned construction activity, but it can also occur unintentionally, such as by the weight of construction vehicles. Well-planned staging of construction activities can reduce the need to disturb uncompacted areas outside of the construction footprint. Areas specifically designated for staging and temporary construction measures should be described in the ESPC plans and clearly marked in the field by the contractor.

B. Preserve Trees and Re-vegetate Using Native Species. Clearing of forested areas, including riparian buffers, should be limited to only those areas that are essential for construction operations. Well-planned staging of construction activities can reduce the need to disturb wooded areas outside of the construction footprint. Similar to minimizing compaction, this non-structural SCM can be applied to almost every project. Highway projects involving new alignments sometimes involve abandoning existing sections of highway. These areas may provide opportunities to offset the stormwater-related impacts of the new alignment. By removing pavement and planting the abandoned areas with native vegetation, the abandoned areas can be made to resemble pre-existing conditions.

C. Impervious Disconnection. A number of potential stormwater impacts can be reduced or eliminated by installing SCMs in between impervious areas and storm sewers. Direct connection often occurs with curbed roadways, where runoff collects along the curb and gutter, is directed into a catch basin, and then discharged to a nearby channel. This common type of design quickly and efficiently removes runoff from the roadway to prevent ponding hazards. However, peak rates of runoff generally increase due to a decreased time of concentration, and there is no mechanism to remove pollutants in runoff. Two ways to achieve disconnection are to eliminate curbs and gutters and to redirect road and driveway runoff into grassed swales or other vegetated systems designed to receive stormwater. Where curb and gutter cannot be eliminated for safety, right-of-way, or other practical reasons, carefully designed curb cuts may be used to allow runoff to spill into an adjacent vegetated SCM.

D. Slope Roughening. Known as a time of concentration (T_c) practice, slope roughening increases the time it takes for runoff to flow across a site to the drainage point or an SCM. Slowing runoff velocity potentially reduces erosion and increases the potential for infiltration. This SCM can include stair step grading, grooving, tracking, and other similar methods of creating stabilized irregularities in graded slopes. Instead of allowing runoff to sheet flow down an embankment, these surface features reduce erosion potential by slowing down the flow; create pockets of small depressions that capture and reduce the total volume of runoff; and encourage infiltration on the slope. When applied to slopes at bridge sites, the turbulence that this SCM creates aids in oxygenating the runoff before it discharges into the stream. Surface roughening has traditionally been used as an ESPC measure to reduce erosion potential and prepare a slope to receive vegetation. Slopes steeper than 2H:1V should be benched or stepped. Surface roughening should be applied to slopes 3H:1V or steeper and on all slopes within 50 feet of a surface water if rolled erosion control products are not used.

14.5 SCM TOOLBOX – STRUCTURAL

This section includes brief descriptions and technical information summaries of each of the available restoration and structural SCM types. Many of these SCMs can be used in series, commonly referred to as a "treatment train", or combined with other SCMs to improve the system's efficiency. The first four SCMs listed in this section are restoration SCMs, which can be used to return disturbed areas to more natural, vegetated areas with high infiltration and/or evapotranspiration potential. The remaining SCMs in this section are engineered measures for controlling runoff; their descriptions in this section include a technical summary that can be used as a quick reference. Typical siting and maintenance constraints for the SCMs can also be referenced in Table 14.1 at the end of Section 14.7. This information may be useful in the planning stages of design, but the technical guidance that can be found in the BMP Manual and other similar publications listed under References in Section 14.24 should be used to design the SCMs.

A. Riparian Buffer Restoration. Riparian buffers are areas adjacent to streams, ponds, etc., that protect those water resources from pollution, prevent bank erosion, provide wildlife food and cover, and shade the adjacent water, moderating temperatures for aquatic species. Buffers are transition areas between aquatic and upland environments. Department projects located in special protection watersheds may have mandatory riparian buffer requirements per Act 162 and 25 PA Code Section 102.14. Other projects that are adjacent to bodies of water with depleted riparian buffers may also consider enhancement. In both instances, the Department may take credit for the buffer as an SCM. Riparian buffers are complicated natural features that require a diverse group of expertise to effectively design enhancement strategies. Enhancement design should be coordinated with PA DEP early in project development.

B. Landscaping and Planting. Landscape restoration is the general term used for actively sustainable landscaping practices that are implemented outside riparian (or other specially protected) buffer areas. Landscape restoration includes the restoration of forest (i.e., reforestation/tree plantings, RTP) and/or meadow and the conversion of turf to meadow (LRM). In a truly sustainable site design process, this SCM should be considered only after the areas of development that require landscaping and/or revegetation are minimized. The remaining areas that do require landscaping and/or revegetation should be driven by the selection and use of vegetation (i.e., native species) that does not require significant chemical maintenance by fertilizers, herbicides, and pesticides.

C. Soil Amendment Restoration. Soil amendments, which include both soil conditioners and fertilizers, make the soil more suitable for the growth of plants and increase water retention capabilities. Compost amendments and soils for water quality enhancement are also used to enhance native or disturbed and compacted soils. These measures change the physical, chemical, and biological characteristics of the soil allowing it to more effectively reduce runoff volume and filter pollutants. Vegetated swales and grass filter strips can be treated with soil amendments to improve performance and increase their permeability. A variety of techniques are included as potential soil amendments including aerating; fertilizing; and adding compost, other organic matter, or lime to the soil.

D. Stream Restoration and Stabilization. Stream restoration and stabilization are terms for correcting unstable, altered or degraded stream habitat. Existing problems may include stream channel or bank erosion, erosive damage to public utilities or facilities (e.g., parks), or sediment transport adversely affecting downstream resources (e.g., Chesapeake Bay). Stream stabilization typically involves corrections to the streambank either at a specific site (e.g., protection of public infrastructure with riprap), or on a watershed level using a combination of grading and rock, log, or natural structures (e.g., vanes or deflectors) to reduce sediment transport, improve stream habitat, or maintain channel capacity before a complete restoration is necessary. Stream restoration is best described as a reconnection of the channel to its floodplain for events both large and small. Most construction occurs within the riparian corridor utilizing unique materials to improve ecological habitat while restoring pre-development hydraulic conditions. These SCMs are generally reserved for special projects outside the context of Chapter 102 permitted highway projects. Due to the complicated design process requiring state and federal permits, District and Central Office consultation should be sought early in the project if stream restoration or stabilization SCMs are being considered.

E. Vegetated Swale. Vegetated swales are one of the most commonly used SCMs along roads because of their ability to fit within limited right-of-way space while providing both drainage and PCSM functions. Vegetated swales are broad, shallow, typically trapezoidal channels that receive runoff from adjacent impervious surfaces and are designed to slow it down, promote infiltration, and filter pollutants and sediments in the process of conveying the runoff. Vegetated swales can receive runoff from concentrated sources (e.g., pipe outfalls), as well as from lateral sheet flow along the length of the channel. They are well suited for use along roads, either as swales in a cut section of a shoulder or in the median receiving runoff from both sides of a divided highway. They can also be used for storm sewer outlet channels and top of cut ditches.

Figure 14.2 Vegetated Swale Summary

SCM Type Code	VSW – Vegetated Swale VSC – Vegetated Swale w/ Check Dams	
Location	• Median • Swale in cut section • Top of slope ditch	
Key Design Elements	• Drainage area ≤ 10 acres • Min. 24 in. between bottom and bedrock/seasonal high groundwater table if designed as infiltration system • Longitudinal slopes from 1-6% • Use 6-12 in. high check dams to increase retention • Side slopes from 3H:1V to 5H:1V • Bottom width 2-8 ft	Vegetated swale in the median of I-180 in Lycoming County

The simplest form of a vegetated swale consists of a band of dense vegetation that can include a variety of trees, shrubs, and/or grasses. Under the vegetated surface layer is approximately 24 in. of permeable soil (minimum 0.5 in/hr infiltration rate) containing a high level of organic matter. An acceptable variation is known as a dry swale, which is essentially a vegetated swale with an infiltration trench. Check dams can also be used to reduce velocities in channels that have a longitudinal slope greater than 3 percent. Check dams used in swales parallel to the roadway must be designed such that the maximum ponding elevation in the swale does not exceed the adjacent subgrade elevation. Turf reinforcement mats can be used to provide enhanced stabilization within the channel to prevent erosion. Salt-tolerant vegetation, such as creeping bentgrass and switchgrass, should be considered in areas with regular deicing of roads in the winter.

Examples of retrofitting existing swales include: replacing or modifying poorly draining soils in the swale; adding an organic layer (i.e., compost) to encourage bioretention; replanting with species that offer greater evapotranspiration opportunities; and retrofitting ditches with check dams to provide storage in the channel. A rock-lined ditch that discharges directly into a surface water may be a good candidate for retrofitting with a turf reinforcement mat and vegetation.

F. Bioretention. Bioretention is a method of treating stormwater by pooling water on the surface and allowing filtering and settling of suspended solids and sediment at the mulch or stone layer, prior to entering the plant/soil/microbe complex media for infiltration and pollutant removal. Bioretention cells, also called raingardens, cause retention of runoff through exfiltration into the subsoil (if subsoil has adequate permeability), subsurface storage below the underdrain (if present), and evapotranspiration by vegetation. Detention storage is provided through a combination of surface ponding with control structures and subsurface storage in soil and gravel layers above the underdrain. Common areas of application for highway projects include medians, areas adjacent to local/urban roads and intersections, and parking or median islands. Several examples of road and parking application are shown in the *BMP Manual*.

Figure 14.3 Bioretention Summary

SCM Type Code	BRE – Bioretention BRU – Bioretention w/ Underdrain
Location	• Median • Swale in cut section • Top of slope ditch
Key Design Elements	• Drainage area ≤ 0.5 acre • Min. 24 in. between bottom and bedrock/seasonal high GWT when designed as infiltration system • Native, perennial vegetation • Provide overflow system



Bioretention along SR 3009 in Philadelphia County

G. Vegetated Filter Strip. Vegetated filter strips are a common and often overlooked SCM. They are gently sloping, densely vegetated areas that filter, slow, and infiltrate sheet flowing stormwater. They are essentially buffers between runoff from impervious areas and a receiving body of water. Filter strips can be best utilized along roads and next to parking areas where runoff flows off the pavement via sheet flow and into a filter strip. This scenario is possible for roads at grade or in a fill condition; it does not work for a section in cut. The width of a filter strip is measured perpendicular to the direction of flow, while its length is measured parallel to flow.

The effectiveness of filter strips can be improved by adding a pervious berm at the toe of the slope. Level spreaders can also be used to spread flow over a larger area to avoid creating a point-source discharge. Finally, under special project constraints where minimal room for traditional SCMs exists (e.g., roadway abuts sensitive area), filter strips can be further modified for placement on slopes as steep as 2H:1V to minimize disturbance of sensitive resources below fill slopes. While still qualifying as a stand-alone water quality SCM, steep slope modifications include a rock armor level spreader between the roadway and filter strip slope, and a VFS surface consisting of turf reinforcement mat, topsoil, and a "no-mow" dense low maintenance grass mix.

Figure 14.4 Vegetated Filter Strip Summary

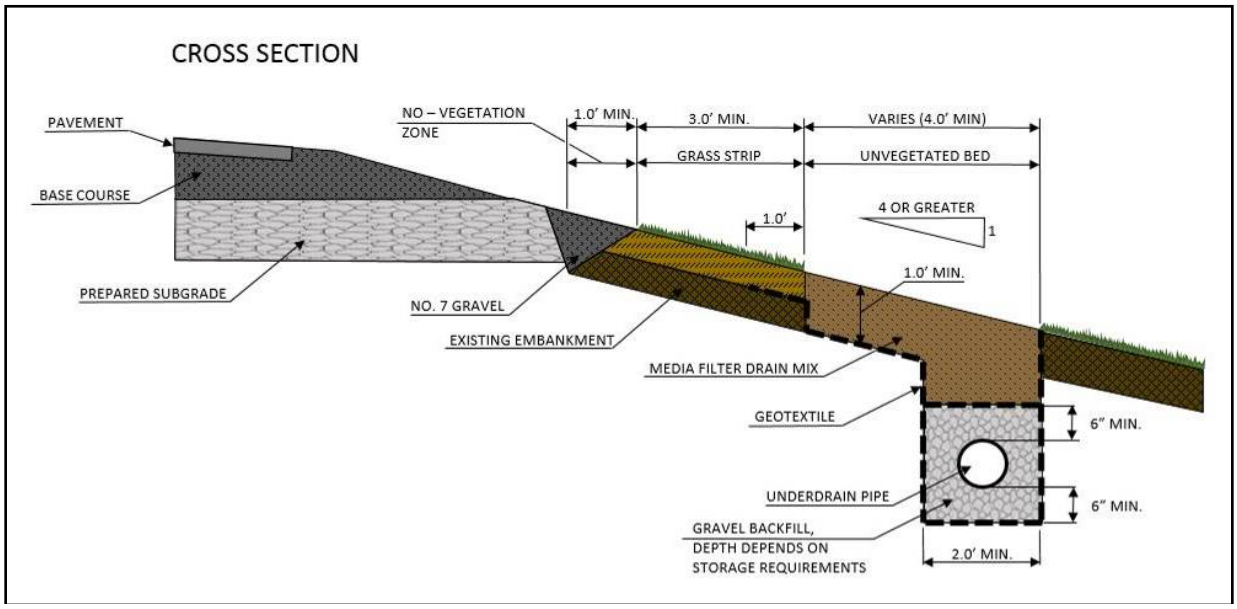
SCM Type Code	VFS – Vegetated Filter Strip VSS – Vegetated Filter Strip Steep Slope
Location	• Adjacent to inside/outside roadway shoulder • Adjacent to parking lot
Key Design Elements	• Drainage area slope ≤ 5% • Slope < 5% preferred; max. 8% • Min. width = width of drainage area • Min. length = width of travelway and shoulder in contributing drainage area • Cover may include dense turf, meadow, or indigenous woods • Soil type and level of disturbance impacts filter strip function • Steep slope (< 2H:1V) modifications available to protect sensitive resources



Vegetated filter strip adjacent to shoulder along SR 3024 in Centre County

H. Media Filter Drain. Media filter drains (also called "ecology embankments" or “bioslopes”) are embankments that treat unconcentrated runoff by rapid filtering through a porous rock-based media commonly known as a media filter drain mix. Media filter drains use a variety of physical, chemical, and biological processes to improve water quality. They are similar to vegetated filter strips, but instead of filtering runoff via sheet flow through thatch and surface soils, runoff is rapidly infiltrated into an unvegetated bed of media filter drain mix consisting of a mixture of crushed rock, dolomite, gypsum, and perlite. A media filter drain’s footprint is usually contained within the roadway embankment; however, it can also be sited in medians or as an end of pipe treatment when combined with a level spreader. Media filter drains should be designed as offline systems anytime direct sheet flow is not the source of runoff. The SCM should be placed on slopes of 4H:1V or less unless modifications are made to ensure stability (3H:1V max).

Figure 14.5 Media Filter Drain Schematic



Media filter drains cause retention of runoff through exfiltration into the subsoil (if subsoil has adequate permeability) and storage in the gravel trench below the underdrain (if present). Credit for volume reduction should not be taken for any portions of the footprint that are above compacted soils. Media filter drains have minimal detention storage because they do not allow ponding and because the media filter drain mix drains rapidly. However, peak discharges are still reduced because of movement across the grass strip pretreatment area, percolation through the media filter drain mix, and infiltration into the subsoil (if subsoil has adequate permeability). While vegetating the surface of the media filter drain mix is an option, it is preferred the mix be left unvegetated. This will increase the visibility of the SCM and maintain the permeability of the surface, while avoiding the potential brownout of a vegetative cover during dry periods. Erosion concerns involving the unvegetated bed are mitigated in properly maintained facilities through a combination of the high permeability of the bed material and delivery of sheet flow to the bed via the gravel no vegetation zone and grass strip.

Figure 14.6 Media Filter Drain Summary

SCM Type Code	MFD – Media Filter Drain	
Location	Embankment slopes	
Key Design Elements	• Side slope: max. 3H:1V if terraced; 4-7H:1V preferred • Long. slope: max. 4% • Slope length: max. 30 ft • Unvegetated bed to increase visibility and maintain permeability	Media filter drain along SR 167 in King County, WA (WSDOT)

I. Subsurface Infiltration Trench. An infiltration trench is an excavated trench lined with filter fabric and backfilled with stone. These systems encourage stormwater infiltration into subsurface soils and work well in space-limited applications. Stormwater can enter a trench via sheet flow from open-section roadways or by channelized flow from swales or storm drain outlets. When located adjacent to roadways, the subsurface drainage direction should be to the downhill side (away from pavement subbase), or located lower than the impervious subbase layer. Proper measures should be taken to prevent water infiltrating into the pavement subbase. Infiltration trenches may be used in conjunction with vegetated swales, roadway drainage systems, or both (i.e., a swale over a pipe running between inlets). A common application is to place a flat run of continuously perforated storm sewer in an infiltration trench. The design storm is conveyed through the system the same way it would through a normal storm sewer; however, smaller rain events have time to drain through the perforations and into the gravel bed. Pretreatment of runoff prior to discharging into the infiltration trench is required in order to increase the life and effectiveness of the facility. Sediment traps in the storm sewer inlets (inlet invert is 6-12 in. below pipe invert) and vegetated filters are examples of pretreatment.

Figure 14.7 Subsurface Infiltration Trench Summary

SCM Type Code	SIT – Subsurface Infiltration Trench	
Location	• Median • Shoulder swales • Between curb and sidewalk	
Key Design Elements	• Min. 24 in. between bottom and bedrock/seasonal high GWT • Min. 5 ft from edge of road • Include underdrain if infiltration rate is < 1 in/h • Trench invert elev. < than road subgrade elev. • 5:1 max. impervious-to-infiltration area	Curb opening (impervious disconnection) into an infiltration trench

J. Infiltration Detention Basin. Infiltration basins are shallow, impounded areas designed to temporarily store and infiltrate stormwater runoff. Sizes and shapes can vary from a single large basin to multiple, smaller basins

throughout a project site. Infiltration basins reduce the volume of stormwater runoff by infiltration and evapotranspiration. Poor soils may be amended by adding sand or gravel to the surface layer to increase the permeability.

Traditional stormwater management basins can be combined with infiltration basin concepts to provide peak flow detention for larger storms and the required volume control; however, care should be exercised so infiltration area loading ratios remain within recommended limits. The combined detention/infiltration basin can be utilized at the discharge points of drainage systems or placed in large median areas where positive basin outflow is provided and subbase drainage is not impeded. Alternatively, infiltration basins may be designed as "offline" facilities where a flow splitter is used to allow the design storm to enter the basin while bypassing larger events.

Figure 14.8 Infiltration Detention Basin Summary

SCM Type Code	BID – Basin, Infiltration Detention	
Location	• Median • Interchange areas • Rest stop/park-and-ride	
Key Design Elements	• Uncompacted subgrade • Min. 24 in. between bottom and bedrock/seasonal high GWT • 5:1 max. impervious-to-infiltration area • Do not install on fill placed within the last 5 years	Infiltration basin along SR 0309 in Montgomery County

During the winter months, there will be many occasions when the soil beneath an infiltration basin is frozen and long-duration ponding will occur; therefore, infiltration basins must be placed in areas where this does not create a safety hazard. When runoff containing salt-based deicers is directed to an infiltration basin, soil may become less fertile and less capable of supporting vegetation. Using salt-tolerant plants and incorporating mulch into the soil can help to mitigate this problem. Infiltration basins should not be used to store snow from highways or parking lots because the sand in the snow can clog the basin, and the chlorides and other pollutants can contaminate the groundwater.

K. Infiltration Berm. An infiltration berm is a mound of compacted earth with sloping sides that is usually located along (i.e., parallel to) a contour in a moderately sloping area. Berms create shallow depressions that collect and temporarily store stormwater runoff, allowing it to infiltrate into the ground and recharge groundwater. Berms are ideal in areas where runoff is free to discharge over slopes. The berm can be installed parallel to the road and intercept runoff prior to being discharged into adjacent areas or bodies of water. Berms can be constructed on disturbed slopes and revegetated as part of the construction process. Infiltration berms may also be constructed in combination with a subsurface infiltration trench at the base of the berm to increase the retention capacity.

Figure 14.9 Infiltration Berm Summary

SCM Type Code	IBE – Infiltration Berm	
Location	Side slope	
Key Design Elements	• Constructed parallel to contours • Min. 24 in. between bottom and bedrock/seasonal high GWT • 24 in. max. height • Side slope 4H:1V max. • Can be retrofit on slopes w/o causing significant disturbance • 5:1 max. impervious-to-infiltration area	Infiltration berm along SR 0074 in York County

L. Wet Detention Basin. A wet basin, also known as a wet pond or retention basin, includes a permanent pool for water quality treatment and additional capacity above the permanent pool for temporary runoff storage (extended detention). Wet basins are effective for pollutant removal and peak rate mitigation, but do not achieve significant groundwater recharge and volume reduction. Unlike infiltration basins, the permanent pool is a key feature and infiltration is discouraged. Wet basins should have low permeability soils at the bottom and, where possible, be excavated close to or below the groundwater table. When necessary, facilities may include an impermeable liner. Interchanges are usually ideal for wet basins because the basin is surrounded by pavement and receives runoff from all directions. Trees and other types of vegetation should be planted around the perimeter to keep the water in the basin cool and reduce potential thermal impacts. A properly designed and maintained wet basin should be an unsuitable habitat for mosquitoes. Most mosquitoes breed in shallow standing water (i.e., less than 3 feet) or deeply vegetated waters.

Wet basins are generally not the preferred SCM when the facility discharges directly into temperature-sensitive water (such as those with temperature TMDLs). To maximize peak rate reduction and minimize thermal impacts, facilities should be designed with the following considerations:

- Maintain a permanent pool.
- Preserve existing shade trees and plant fast growing trees along the perimeter, but not on the constructed embankment.
- Align basin in a north-south direction.
- Avoid excessive riprap and concrete channels that can transfer heat to runoff.

Figure 14.10 Wet Detention Basin Summary

SCM Type Code	BWD – Basin, Wet Detention
Location	• Interchange areas • Between outside embankment and R/W • Rest stop/park-and-ride
Key Design Elements	• 5-10 acre min. drainage area • Needs a high GWT, poorly drained soils, or engineered liner • Avg. permanent pool depth 3-6 ft • 2:1 min. length to width ratio • Sediment forebay pretreatment • Vegetation type and location is key for habitat and thermal impacts



Wet detention basin under construction along SR 0228 in Butler County

M. Stormwater Wetland. Stormwater wetlands are shallow marsh systems planted with emergent vegetation that are designed to treat stormwater runoff. They can provide considerable aesthetic and wildlife benefits, but require a relatively large amount of space and an adequate source of inflow to maintain the permanent water surface. Stormwater wetlands improve runoff quality through settling, filtration, uptake, chemical and biological decomposition, volatilization, and adsorption. They are effective at removing many common stormwater pollutants including suspended solids, heavy metals, total phosphorus, total nitrogen, toxic organics, and petroleum products. Peak rate is primarily controlled through the transient storage above the normal water surface. Although not typically considered a volume-reducing SCM, stormwater wetlands can achieve some volume reduction through evapotranspiration, especially during small storms. Stormwater wetlands are a good option for retrofitting existing detention basins.

Figure 14.11 Stormwater Wetland Summary

SCM Type Code	SWE – Stormwater Wetland
Location	• Median • Interchange areas • Rest stop/park-and-ride lot
Key Design Elements	• 5-10 acres min. drainage area needed or sustained base flow (e.g., springs) • 2:1 length to width ratio • Poorly drained soils or engineered liner • Sediment forebay at inlet • Can be combined with wet basin design



Stormwater wetland next to highway

N. Dry Detention Basins. Dry detention basins are large, above-ground earthen impoundments that control stormwater runoff from large drainage areas. They are designed to manage peak flow rates for extreme events

(normally up to a 100-year return period) and dry cycle between storms. The runoff volume that is temporarily stored is released through an engineered outlet structure. Dry detention basins were the primary stormwater management measure employed prior to around 2006. Subsequently, water quality features were implemented in dry detention basin designs, such as pretreatment (e.g., forebays and micropools), elongated flow paths, irregular basin shapes, and smaller low flow orifices to extend the release of smaller storm volumes. Water quality SCMs are usually needed to compliment a dry detention basin because of their low pollutant removal capabilities. Naturalized detention basins utilize native grasses, shrubs, and trees to provide greater water quality and habitat benefits relative to standard turfgrass dry detention basins.

Figure 14.12 Dry Detention Basin Summary

SCM Type Code	BDD – Basin, Dry Detention BED – Basin, Dry Extended Detention BND – Basin, Naturalized Detention
Location	• Medians on divided highways • Between embankment and R/W • Interchange areas • Rest stop/Park-and-rides
Key Design Elements	• Extreme event peak rate control • Smaller low flow orifice used to extend release of smaller storms • Forebays and micropools can enhance water quality treatment • Native plantings can enhance water quality and habitat



Dry detention basin along SR 0011 in Cumberland County

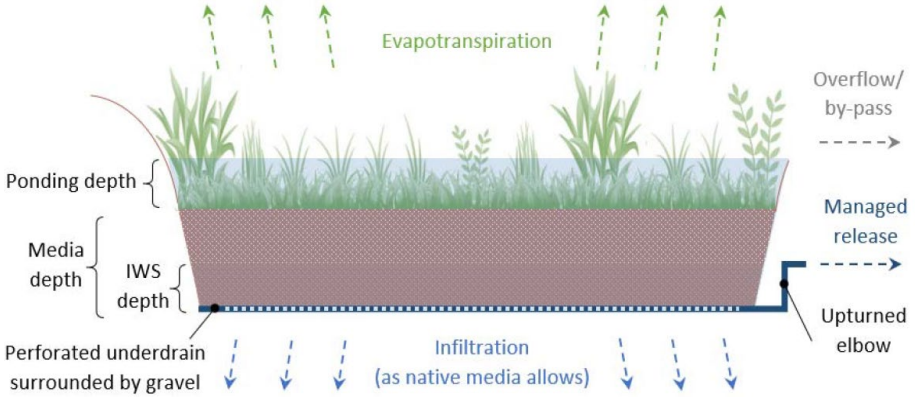
O. Dry Ultra-Extended Detention Basins. Ultra-extended detention basins provide peak rate control for extreme storm events and employ a managed release concept (MRC) to manage volume and water quality for smaller events. Ultra-extended detention basins (with MRC) should be considered only after it has been determined infeasible for infiltration SCMs to manage the net change in the pre- vs. post-development runoff volume from the 2-year/24-hour storm.

MRC comprises the collection, management, and filtration of captured runoff through a vegetated SCM and includes release of a portion of the captured runoff through an underdrain. It is intended to be used where infiltration alone is not feasible to manage volume. Basins should only be lined in sensitive areas (e.g., environmental or geologic conditions, downstream structures) where infiltration is a concern to take advantage of any existing infiltration capabilities, even if minimal.

MRC design requires temporary impoundment of runoff from a 1.2-inch/2-hour storm which is used by vegetation, filtered through a soil media, infiltrated through undisturbed soils (to the highest degree feasible), and released through an underdrain at a rate similar to the lateral unsaturated flow movement to the receiving waters from undeveloped areas. The peak rate for the 2-year/24-hour storm is managed back to the 1-year/24-hour storm to protect and improve downstream geomorphologic processes. A portion of the soil media is designed as internal water storage to further water quality and evapotranspiration benefits.

The runoff volume permanently removed is a combination of the infiltration, evapotranspiration, and volume released through the underdrain for the 1.2-inch/2-hour storm. The volume managed through MRC is credited as the net change in the pre- vs. post-development runoff volume from the 2-year/24-hour storm. Always refer to PA DEP's current design standards for MRC.

Figure 14.13 Dry Ultra-Extended Detention Basin Summary

SCM Type Code	BUD – Basin, Dry Ultra-extended Detention	Key Design Elements	• Dry detention basin with managed release concept (MRC) • Must demonstrate infiltration alone not feasible for volume control • Min. 24 in soil media depth • Internal water storage (IWS) • Max. 2 ft ponding depth for 2-yr/24-hr storm; max. 4 ft for larger storms • Max. 72 hr drawdown for all storms
Location	• Medians on divided highways • Between embankment and R/W • Interchange areas • Rest stop/park-and-rides		
			
Dry Ultra-extended Detention Basin with Managed Release Concept			

P. Level Spreader and Flow Dispersion. Level spreader and flow dispersion SCMs provide a means to discharge concentrated flows in a non-erosive manner by redistributing them as sheet flow and discharging to stabilized areas (dispersion area). Most often implemented at the outlet of a drainage system, they allow one to obtain a water quality benefit where alternative SCMs cannot be sited. They are particularly useful on project sites where a high seasonal groundwater table limits the implementation of infiltration SCMs. The spreader or dispersion device may consist of a level concrete sill, curb, earthen berm, trench, or perforated pipe. If installed level along a contour, the device provides a shallow depth of ponding behind it to ensure even redistribution of the concentrated runoff. At a minimum the device must be sized to discharge flows from the 10-year storm event while bypassing larger events (e.g., offline SCM with a flow splitter) without causing erosion downstream.

Figure 14.14 Level Spreader / Flow Dispersion Summary

SCM Type Code	LSO – Level Spreader Outfall FDF – Flow Dispersion, Forest FDV – Flow Dispersion, Vegetated Filter Strip	
Location	• Outlets of drainage systems • Entrance to basin SCMs • Areas where concentrated discharge is not desired	
Key Design Elements	• Must be level • Discharge location must be protected, stable, and discourage channelization • Size for min. 10-year event • Downstream vegetation type can vary	Level spreader/flow dispersion application

The discharge location (or dispersion area) downstream of the level spreader must consist of a stabilized area that discourages channelization. The slope, vegetation type, and coverage may vary depending on whether the area was disturbed during construction. Downstream vegetation may consist of forest, meadow, or a restored area similar to the vegetated filter strip SCM described earlier in this section. Ideally, the dispersion area should be sized or designed so no runoff leaves the area. The entire dispersion area must be protected from future development (either by right-of-way acquisition, easement or other agreement) in order to receive credit. Special attention must be taken when directing additional runoff to existing dispersion areas to avoid adverse impacts to downstream properties.

Q. Regenerative Step Pool. Regenerative step pools consist of a series of alternating shallow constructed riffles and pools similar to a stream environment. When used in combination with native vegetation and/or filter media, the SCM can provide conveyance and water quality, infiltration, and ecological benefits. Typically sized to treat one inch of runoff from the contributing area, regenerative step pools lend themselves to construction at the outfalls of drainage systems on steeper slopes, or as retrofits in eroded channels where more traditional SCMs (e.g., vegetated swales with check dams) are not feasible. This essentially allows stormwater management to occur within the conveyance system. Riffle sections consist of a reinforced (e.g., rock lined) sloped (5% max.) section that provides non-erosive conveyance of both small and larger outfall discharges to the pool sections and downstream. The riffle section cascades up to five vertical feet into a pool consisting of flat shallow ponded areas typically designed to cumulatively store and treat one inch of runoff from the contributing drainage area. They may incorporate vegetation, filter media and outlet control (e.g., weir) to treat smaller storm events while safely discharging larger storm events without causing erosion. Systems can be blended into the existing landscape providing better aesthetics than a typical rock lined outfall channel.

Figure 14.15 Regenerative Step Pool Summary

SCM Type Code	RSP – Regenerative Step Pool	
Location	• Drainage system outfalls • Existing steep eroded channels • Ephemeral or intermittent culvert outfall	
Key Design Elements	• Max. 5% slope and 8 ft length in riffle sections • Max. 5 ft drop at 50% slope between riffle and pool sections • Min. 18 in. pool depth at 0% slope • Min. pool length is twice riffle length	Center for Watershed Protection pilot regenerative step pool project in Lancaster County

14.6 SCMS REQUIRING SPECIAL APPROVAL

This section includes brief descriptions and technical information summaries for select engineered SCMs requiring special approval. Unlike the rest of the PCSM measures in PennDOT's SCM toolboxes, these SCMs cannot be proposed for implementation within Department right-of-way without prior approval from the applicable Engineering District. Typical siting and maintenance constraints for the SCMs in this section can be referenced in Table 14.1 at the end of Section 14.7.

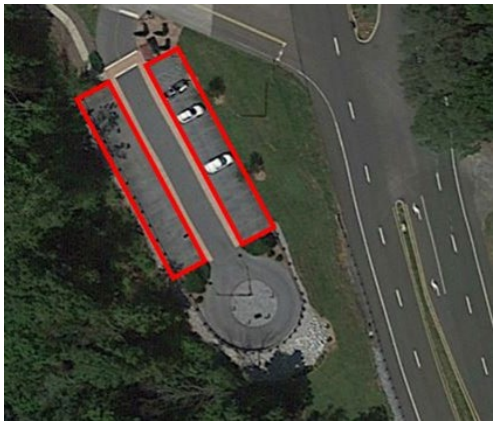
A. Street Sweeping. Street sweeping is a source control SCM for removing pollutants before they are carried away by runoff into storm sewers or adjacent surface waters. Applications are limited to projects with highly impervious surroundings and no opportunities for vegetative or structural BMPs. The maintenance manager in the District where the project is located should be consulted prior to submitting a permit where this SCM is proposed.

B. Pervious Pavement. Pervious pavement consists of a pervious surface course underlain by a uniformly graded stone bed that provides temporary storage for peak rate control. Where the native underlying soils yield favorable infiltration rates, some of the volume of stored water can be retained until it infiltrates. Pipes, vaults, and other type of structures may be installed within the stone bed to increase storage capacity.

In northern climates, pervious pavements have less of a tendency to form black ice and often require less plowing. The surface course may consist of porous asphalt, porous concrete, or various porous structural pavers. Pervious pavements are best suited for areas that will not be subject to high traffic volumes or high rates of travel speed. Paver blocks are not suitable for high rate travel speeds because of the block design, and the open graded asphalt and concrete in pervious pavements does not wear well in travel lanes. Publication 242, *Pavement Policy Manual*, Appendix K – Pervious Surface Course Design, provides recommended surface course depths for concrete and asphalt sidewalk and parking applications.

Proper construction is critical for pervious pavement to function properly and, therefore, must be undertaken in such a way as to prevent (1) compaction of underlying soil, (2) contamination of stone subbase with sediment and fines, (3) tracking of sediment onto pavement, and (4) drainage of sediment-laden waters onto pervious surface or into constructed bed. Pervious pavement can provide significant stormwater benefits as long as the pavement surface is kept free of debris and sediment. Maintenance typically involves removing sediments from the pavement by means of a vacuum truck. Sweeping may actually exacerbate the problem by working sediment down into the porous surface.

Figure 14.16 Pervious Pavement Summary

SCM Type Code	PPA – Pervious Pavement, Asphalt PPC – Pervious Pavement, Concrete PPP – Pervious Pavement, Pavers	
Location	• Park-and-rides • Walking paths • Parking lots • Sidewalks • Pull offs	
Key Design Elements	• 1-3 ft typical infiltration bed depth • Uncompacted subgrade • Frequent maintenance required to prevent pavement surface clogging • Backup drainage system needed	Pervious pavement (PPA) location (outline) at park-and-ride in Lebanon County

C. Constructed Stormwater Filter. Constructed stormwater filters utilize filter media placed in an excavation or structure to improve water quality by removing sediment and other pollutants from runoff. Infiltration may be incorporated if site conditions allow; otherwise, an underdrain must be incorporated into the design. Typical filter media may consist of sand, compost, granular activated carbon, or other media that is specifically designed for the pollutant concerns of a site. The SCM should be designed to safely convey larger storm events by either bypassing the filter or creating an offline facility. Filters can be designed to have either a vegetated or non-vegetated (exposed bed) surface depending on the pollutants of concern and expected maintenance concerns of the SCM site. This SCM type is more common in land development applications; it is generally not recommended for Department highway projects due to maintenance requirements. Pretreatment of stormwater inflow is required to be included for any constructed stormwater filter built in the right-of-way.

Figure 14.17 Constructed Stormwater Filter Summary

SCM Type Code	CSF – Constructed Stormwater Filter	
Location	Requires approval for use in special circumstances	
Key Design Elements	• Min. 12 in. depth of filter media • Min. 24 in. between bottom and bedrock/seasonal high GWT when designed as infiltration system • Pretreatment required • Safely convey larger storm events • Frequent maintenance required to prevent filter clogging	Constructed stormwater filter placed in a structure adjacent to a parking lot

D. Manufactured Treatment Devices. Manufactured treatment devices consist of proprietary products used in-line with a storm sewer system to provide water quality treatment. Their use on Department projects is limited due to safety, inspection and maintenance needs, and proprietary product approval. The volume of traffic on most state roads makes using items that are not standard industry products very risky, particularly ones that have not been in place long enough to verify their long-term properties. If a widely-used and proven alternative to a manufactured product exists, it should be given first consideration.

The manufacturer's recommended inspection and maintenance procedures associated with proprietary products are usually more intensive than the Department's standard program for other SCM types.

Projects with federal-aid funds have strict requirements related to proprietary products. The process of getting a proprietary item approved for use is often long and requires significant justification. Section 635.411 of 23 CFR (Code of Federal Regulations) states that federal funds cannot be used to purchase proprietary items unless the proprietary item is either obtained through competitive bidding with equally suitable unpatented items; is certified by the Department to be essential for synchronization with the existing highway facility, or that no equally suitable alternate exists; or is used for research or for a distinctive type of construction on relatively short sections of road for experimental purposes.

Figure 14.18 Manufactured Treatment Devices Summary

SCM Type Code	MTD – Manufactured Treatment Device	
Location	Requires approval for use in special circumstances	
Key Design Elements	• Follow manufacturer's installation instructions • Frequent inspection and maintenance required per manufacturer's recommendations • Approval for use on federal-aid projects arduous	Maintenance crew replacing an inlet filter

E. Subsurface Detention Storage. Subsurface detention storage is extended detention where peak runoff storage occurs below ground in an excavated system. The system typically utilizes a vault, tank, pipes, or proprietary system surrounded by stone to obtain the necessary storage. Note that non-proprietary storage alternatives should be given first consideration. Due to expense and constructability, subsurface detention storage is usually reserved for projects where there is insufficient area for a properly sized surface detention basin. Pretreatment of stormwater inflow is required both to reduce maintenance needs in the subsurface storage itself and to provide some level of water quality treatment.

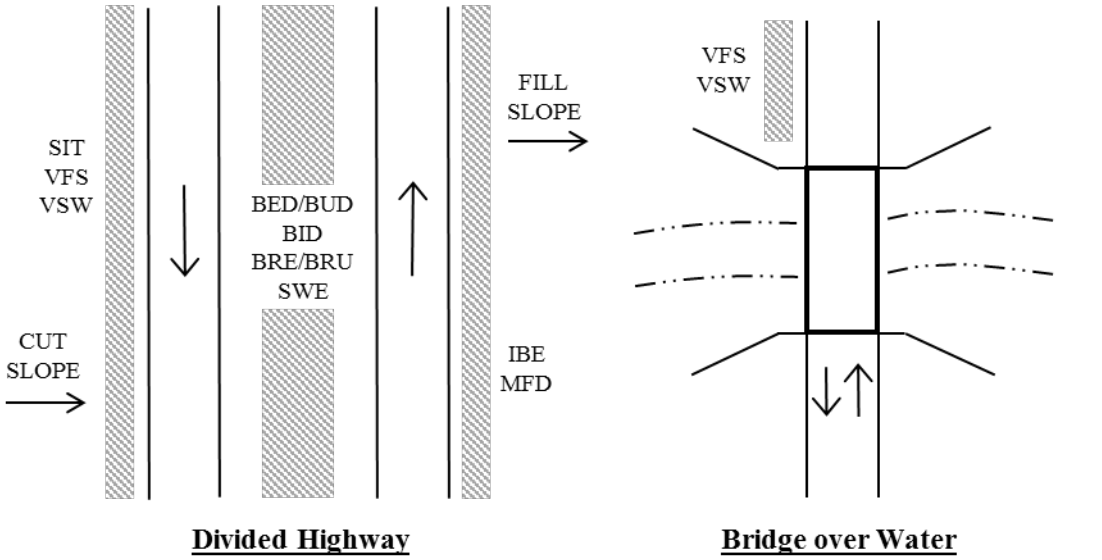
Figure 14.19 Subsurface Detention Storage Summary

SCM Type Code	SDS – Subsurface Detention Storage	
Location	• Rest stop/park-and-rides • Pull-offs • Beneath shoulders and sidewalks	
Key Design Elements	• Follows BED design guidance • Include positive overflow • Avoid siting under travelway • Pretreatment of inflow required to reduce maintenance needs	Subsurface detention storage location (outline) at park-and-ride in Lebanon County

14.7 AREAS OF APPLICATION

Figure 14.20 depicts four typical types of Department projects and the structural SCMs that would most often be applicable for each type of project. This figure does not preclude the use of other SCMs in any of these areas if adequate design documentation is provided.

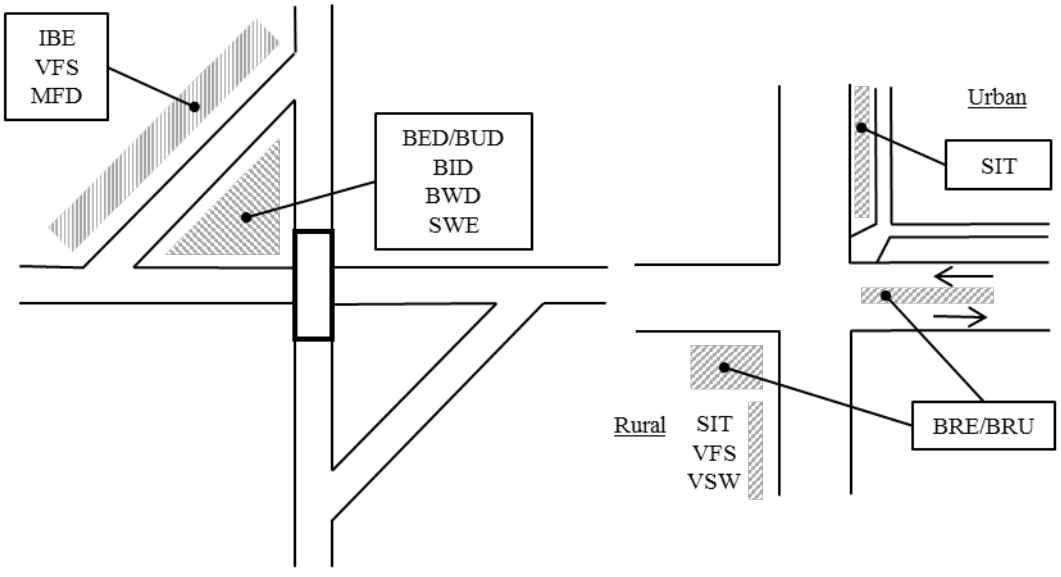
Figure 14.20 Common Structural SCM Applications



Divided Highway **Bridge over Water**

SCM Type Code Key

BED	Basin, Dry Extended Detention	MFD	Media Filter Drain
BID	Basin, Infiltration Detention	SIT	Subsurface Infiltration Trench
BRE/BRU	Bioretention	SWE	Stormwater Wetland
BUD	Basin, Dry Ultra-Ext. Detention	VFS	Vegetated Filter Strip
BWD	Basin, Wet Detention	VSW	Vegetated Swale
IBE	Infiltration Berm		



Highway Interchange **Intersections**

Table 14.1 Siting and Maintenance Constraints of SCMs

SCM	SCM Type Code	Size of Drainage Area	Space Required	Suitable with Shallow Water Table	Suitable with Shallow Bedrock	Suitable in Poorly Drained Soils	Pretreatment Required	Routine Maintenance Effort
Vegetated Swale	VSW	Small-Large	Low	Yes	Yes	Yes	No	Low
	VSC	Small	Low	No	No	No	No	Low
Bioretention	BRE	Small	Low	No	No	No	Yes	Medium
	BRU	Small	Low	No	Yes	Yes	Yes	Medium-High
Vegetated Filter Strip	VFS / VSS	Small	Low	Yes	Yes	Yes	No	Low
Media Filter Drain	MFD	Small	Low	Yes	Yes	Yes	No	Low-Medium
Subsurface Infiltration Trench	SIT	Small	Low	No	No	No	Yes	Medium-High
Infiltration Detention Basin	BID	Small-Large	Medium-High	No	No	No	Yes	Medium
Infiltration Berm	IBE	Small	Low	No	No	No	No	Medium
Wet Detention Basin	BWD	Medium-Large	Medium-High	Yes	Yes	Yes	Yes	Medium
Stormwater Wetland	SWE	Medium-Large	Medium-High	Yes	Yes	Yes	Yes	Medium
Dry Detention Basin	BED / BND	Medium-Large	Medium-High	No	Yes	Yes	No	Low-Medium
Dry Ultra-Ext. Detention Basin (MRC)	BUD	Small-Large	Low-High	Yes[1]	Yes[1]	Yes[1]	Yes	Medium-High
Level Spreader	LSO / FDF / FDV	Small	Low	Yes	Yes	Yes	No	Low
Regenerative Step Pool	RSP	Small-Medium	Low	No	Yes	Yes	No	Low
Pervious Pavement[2]	PPA / PPC / PPP	Small	Low	No	No	No	No	High
Constructed Stormwater Filter[2]	CSF	Small	Low-Medium	No	No	No	Yes	High
Manufactured Treatment Device[2]	MTD	Small	Low	Yes	Yes	Yes	No	High
Subsurface Detention Storage[2]	SDS	Small-Medium	Low-Medium	Yes	Yes	Yes	Yes	High

[1] Suitability of BUD at sites with shallow water table, depth to bedrock, and/or poorly drained soils is highly dependent on design.
[2] SCM requires special approval before they can be implemented on PennDOT projects. See Section 14.6.

14.8 STORAGE FACILITIES

A. Introduction. The use of storage facilities to provide improved stormwater management has increased dramatically in recent years. Initially these controls were primarily focused on controlling the rate of runoff but in recent years the focus and use of these features has changed from providing solely rate control to addressing both the quality and quantity (rate and volume control) of the stormwater runoff.

B. Quality. Control of stormwater quality using storage facilities offers the following potential benefits:

- Decreased downstream channel erosion and degradation.
- Maintained biologic diversity in receiving watercourses.
- Improved water quality through:
  - Stormwater filtration.
  - Capture, reduction and removal of non-point source pollutants.

C. Quantity. Controlling the quantity of stormwater using storage facilities can provide the following potential benefits:

- Reduction of peak runoff rate caused by development.
- Reduction in the volume of runoff caused by development.
- Mitigation of downstream drainage capacity problems.
- Recharge of groundwater resources which can help maintain baseflow in streams.
- Reduction or elimination of the need for downstream outfall improvements.
- Maintenance of historic low-flow rates by controlled discharge.

D. Objectives. The objectives for managing stormwater quantity using 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 through a watershed wide stormwater management program or Act 167 Stormwater Management Plan (i.e., release rate or management district target resulting in a reduction of the post-development peak flow based on a percentage of the existing or pre-development flow).

The consequence of using only detention storage to mimic pre-development flow rates is especially critical when considering the cumulative impacts of these facilities upon the entire watershed. Under normal conditions a dry detention facility only serves to truncate the peak of the post-development hydrograph, transferring the volume of runoff discharged during the peak of the event to a point that occurs after the end of the normal pre-development flow. This practice essentially extends the peak pre-development flow in a post-development condition for a longer period. The cumulative effect of this practice on a watershed is that the peak runoff from sites throughout the watershed are added to one another making the resultant post-development flow for the entire watershed higher than the pre-development flow.

In certain instances it may not be advantageous to have detention. This can be especially applicable for those areas of the watershed that are close to main tributaries in the lower portions of the watershed which are able to peak and pass their flows prior to the watershed peak arriving in the lower portions of the waterway. Because of the interaction of hydrograph timing between sites in a watershed it is important that the Department's design professionals design facilities to be consistent with applicable watershed management plans that are enacted for a particular location.

For watersheds without an adequate outfall (e.g., unstabilized channel, flood-prone area, etc.), the total volume of runoff is critical and retention storage facilities may be used to store the increase in volume and to control discharge rates.

E. Detention and Retention. Urban stormwater storage facilities are often referred to as either detention or retention facilities. Detention facilities are those facilities that are designed to reduce the peak discharge rate from a particular drainage area by detaining runoff and then releasing it at a controlled rate. The primary function of these facilities is to detain runoff for a short period and then completely drain shortly after a storm has passed. Retention facilities are different from detention facilities in that they are designed to store the runoff for very long period of time or indefinitely until it can be used or returned to the environment through a means other than release downstream. Retention facilities may or may not be designed to contain a permanent pool of water. Because most of the design procedures are the same for detention and retention facilities, the term storage facilities, as used in this chapter, is meant to include both detention and retention facilities. If special procedures or considerations are needed for detention or retention facilities related to the special aspects of their respective designs, these will be specifically cited in this chapter.

Storage facilities may be small in terms of storage capacity and embankment height where serving a single outfall from a watershed of a few acres, or they may be larger providing stormwater management control for larger portions of a watershed. Although the same principles apply to all storage facilities, certain devices are more suited to smaller drainage areas and others function better with larger drainage areas.

F. Underground Storage. Storage of stormwater flows beneath the surface has become an increasingly viable solution for developing sites with large impervious areas. The advantage of underground storage is that the area directly above the storage facility can be used for parking, recreation areas, etc. The storage volume is typically provided by burying prefabricated storage units or ordinary storm sewer pipes. Use of underground storage on PennDOT highway projects is not encouraged, as these facilities typically require special maintenance equipment and training. Underground storage may be considered for projects that discharge to sensitive areas and all other practicable control measures have been exhausted. Sensitive areas include HQ or EV surface waters, EV wetlands, stormwater-impaired surface waters, combined sewer systems, and surface waters containing threatened and endangered species and critical habitat for threatened and endangered species. Use of underground storage facilities beneath travel lanes and shoulders is not recommended.

G. Computer Programs. Routing calculations needed to design storage facilities, although not extremely complex, may be time consuming and repetitive. To assist with these calculations, many computer software programs, and spreadsheets have been created to automate the reservoir routing and support the design of storage facilities. Although some models perform one type of computation better than other available models, it is necessary for the Department to concentrate its efforts on the programs in PennDOT's H&H toolbox in order to develop and maintain agency-wide expertise. If a model not included in the forgoing list is used for a specific problem, then the project engineer should ensure that model is appropriate and that approvals are obtained from the Department. Several routing methodologies have been developed to evaluate the capacity and performance of storage facilities, with the storage indication method one of the most common and simplest to use. Regardless of the size, location or function of the facility, all storage facilities should be designed and analyzed using reservoir routing calculations to demonstrate that the storage facilities function as intended. For additional information regarding the Storage-Indication Routing Method, see Chapter 7, Hydrology.

14.9 DESIGN CRITERIA

A. General Criteria. Storage may be concentrated in large regional facilities or distributed throughout a watershed in smaller more localized facilities. The primary benefits of a large basin are reduced maintenance costs and consistent application of stormwater management controls for a large portion of the watershed. The negative aspects of the facility are the large amount of land area required and capital necessary to construct such a facility. Conversely, the advantage of smaller facilities is that they require less area to construct and can be placed in locations where large regional basins will not fit. These smaller facilities may be constructed in depressed areas or parking lots, adjacent to road embankments and freeway interchanges, incorporated into parks and recreational areas and contiguous to small lakes, ponds and depressions within urban developments. The benefit of this approach is that small pieces of land area that are not particularly useful for other purposes, because of their size or other limitations, may be incorporated into a project's stormwater management controls.

The utility of any storage facility depends on the amount of storage, its location within the watershed and its operational characteristics. An analysis of such storage facilities should consist of comparing the design flow at a point or several points of interest downstream of the proposed storage site with and without storage. Potential points of interest may include an existing downstream bridge, culvert or storm sewer; a confluence with a downstream

tributary; a key drainage feature such as a dam, channel obstruction or stream crossing; or simply a point of concentrated flow at or near the right-of-way.

Complete analysis of storage facilities should include the evaluation of several design storms to properly evaluate the performance of a proposed storage facility. Analysis for a storage facility may include design events ranging between the 2-year and the 100-year design storms (0.50 and 0.01 exceedance probability, respectively). 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.
- Freeboard above the maximum water surface elevation.
- Time to dewater.
- Provisions for maintenance (such as slope limitations and access ramps).

B. Release Rate. If an approved Act 167 Stormwater Management Plan is available for the watershed in which a project is situated, then all storage facilities and outlet works should be designed to be consistent with the appropriate release rates or management directives identified by the Plan. If an approved Act 167 Stormwater Management Plan is not available for the respective watershed, then the release rate targets outlined in Section 14.2 apply.

C. Storage. In those portions of the Commonwealth that do not have an approved Act 167 Stormwater Management Plan, the minimum performance standard for storage facilities should be based upon the needs of the design. Typically the design event, based upon the roadway classification, is an appropriate condition to start the analysis. However, the design event should then be adjusted based upon the needs of the project. In other words, the design storm cannot be arbitrarily set but must be considered on a case-by-case basis, based upon the projects needs. In certain instances it may be necessary to provide additional storage capacity for other design storms depending on the downstream system's conveyance capacity. If a stormwater storage facility is to be used as sediment basin, or as part of a project's erosion and sediment pollution controls, or if sediment is deposited in the basin during construction causing loss of storage volume, the original design dimensions of the basin should be restored once the drainage area is permanently stabilized and before completion of the project.

D. Grading and Depth. Following is a discussion of the general grading and depth criteria for storage facilities followed by broad criteria related to detention and retention facilities.

1. General. The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Vegetated embankments should be less than 4.5 m (15 ft) in height, as measured from the downstream toe of the embankment to the top of the berm and should have side slopes no steeper than 1V:3H (3H:1V). Riprap-protected embankments should be no steeper than 1V:2H (2H:1V). Geotechnical slope stability analysis is recommended for all embankments greater than 3 m (10 ft) in height, and is mandatory for embankment slopes steeper than those given above. Procedures for performing slope stability evaluations can be found in most soil engineering textbooks.

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 freeboard. Aesthetically pleasing features are also important in urbanized areas which can be augmented with landscaping, walking paths, natural areas and other appurtenances. Fencing of basins is addressed in subsequent sections of this chapter.

2. Detention. Areas above the normal high-water elevations of storage facilities should be sloped toward the facilities to allow drainage and prevent standing water from ponding in surface depressions. Typically, careful finish grading above the storage facility is required to avoid creation of upland surface depressions that may retain runoff. This is especially critical in areas which are graded at a slope of less than five percent (5%).

If not specifically designed to retain water in the bottom of the storage facility, the bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. Although low-flow or pilot channels are discouraged for most applications they may be constructed in special cases where standing water

problems are known or anticipated to occur. Typically these channels are aligned across the facility from the inlet to the outlet and function to convey low flows through the facility and prevent standing water conditions. Low flow channels may assist in the complete draining of a facility; however, constructing these channels out of impervious materials may be deleterious to the environment. Therefore, if a low flow channel is deemed necessary to the design it is recommended that such a feature be constructed of pervious materials in a meandering alignment to maximize the flow length within the basin. Check dams may be provided across the channel to slow the water through the facility, and provide limited removal of non-point source pollutants conveyed in the stormwater runoff. This approach allows for the complete emptying of the basin while at the same time providing a measure of water quality treatment of the stormwater runoff conveyed through the basin.

3. Retention. Similar to detention, areas above the normal high-water elevations of retention facilities should be sloped toward the facilities to allow drainage and prevent standing water from ponding in surface depressions. 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 the growth of weeds, without creating undue potential for anaerobic bottom conditions, should be considered. A depth of 0.9 m to 1.8 m (3 to 6 ft) is generally reasonable.

Retention facilities can be designed with forebays at the inlet into the storage facilities to prevent the deposition of sediment within the storage facility and preserve the storage capacity of the basin. The typical volume of storage within a forebay is between 10 and 15% of the total permanent pool volume and should be between 1.2 and 1.8 m (4 and 6 ft) deep.

Normally, the volume of water contained within a ponds permanent pool is not available for use as part of the facility's stormwater management controls and must be removed from the available stormwater storage capacity of the basin.

4. Emergency Spillway. Each facility should be designed with an emergency spillway with the outlet works designed to prevent the regular use of the emergency spillway. For large storage facilities, selecting a flood magnitude for sizing the emergency outlet should be consistent with a risk analysis evaluating the potential threat to downstream life and property if the basin embankment were to fail. Whenever possible the emergency spillway should be constructed on undisturbed earth and not placed in fill.

Impoundment depths greater than 4.5 m (15 ft) or volumes greater than 61,675 m^3 (50 ac-ft) are subject to the requirements of PA Code Title 25, Chapter 105, Subchapter B as administered by the Pennsylvania Department of Environmental Protection, Bureau of Waterways Engineering, Division of Dam Safety. Storage facilities of this magnitude should be avoided on Department projects.

E. Outlet Works. Outlet works for storage facilities typically include a principal spillway and an emergency overflow, which must be configured to accomplish the performance objectives of the facility. Outlet works can consist of single stage comprised of a drop inlet, pipe, weir or orifice or may consist of several stages comprised of any combination of the aforementioned control devices set at different elevations. Typically, small orifices less than 75 mm (3 in) in diameter, small pipes less than 450 mm (18 in) in diameter and slotted riser pipes are discouraged from use in permanent stormwater facilities because of the potential for clogging. Perforated metal pipes are acceptable if they are of sufficient gage and are galvanized, coated, or aluminized. To facilitate the construction of these features dimensions should be set at values which are easily defined and measured, such as whole numbers set in increments of 25 mm (1 in) for an orifice or weir and increments of 75 mm (3 in) for pipes less than 900 mm (36 in) in diameter. Pipes larger than 900 mm (36 in) in diameter which are used in the design of storage facilities should be sized in increments of 150 mm (6 in). The Publication 72M, *Standards for Roadway Construction*, RC-70M, may be referenced for standard drawings related to storage facilities and outlet works.

The principal spillway is intended to convey the design storm without allowing flow to enter the emergency outlet. All design storms should be routed through the principal spillway reserving the emergency spillway for extreme events and in case of clogging/malfunction of the principal spillway. Sizing of the outlet works for a storage facility should be based on the results of hydrologic routing calculations.

F. Location. In addition to controlling the peak discharge through the outlet works, storage facilities will change the timing of the hydrograph for the area draining from a storage facility. The construction of storage facilities not only effects the location of the peak with respect to the hydrograph timing but also the length of time in which pre-

development flow rates are exceeded in a post-development condition. If not properly planned, the aggregate effects of several storage facilities within the same drainage basin may create more flooding problems than if no stormwater controls were provided. Thus it is very important to determine what effects a particular facility may have on combined hydrographs downstream of the facility. The effects of hydrograph timing upon the watershed are assessed as part of the Act 167 planning process. Therefore, it is vital that if an Act 167 Stormwater Management Plan is approved that all proposed facilities be consistent with the enacted plan. When consistent with an approved *basin wide* stormwater management plan no further downstream analysis is necessary to demonstrate the acceptable performance of a proposed storage facility.

14.10 GENERAL PROCEDURE

A. Data Needs. Three essential types of data are necessary to evaluate the routing of stormwater through a storage facility and assess the effectiveness of a proposed basin. This data includes:

- Inflow hydrographs for all selected design storms.
- Stage-storage curve for proposed storage facility.
- Stage-discharge curve for all outlet control structures based upon the configuration of the outlet control works.

Using this data, a systematic design procedure can be followed to route the inflow hydrograph through the storage facility to establish an outflow hydrograph below the storage area. If the desired outflow results are not achieved, either the basin or the outlet geometry must be altered to yield a new stage-storage and/or a stage-discharge curve to obtain an acceptable outflow hydrograph. As it is usually necessary to control the flow from events of different return intervals (i.e., 2-, 10-, 25-, 50-, 100-year) typically, an iterative approach is needed to achieve the prescribed performance targets, requiring successive routings until the desired outflow hydrographs are achieved (see Example 14.9).

B. Stage-Storage Curve. A stage-storage curve defines the relationship between the depth of water and the available storage volume in a storage facility. Ordinarily these curves plot the storage along the x-axis and the stage against the y-axis. The data for the stage-storage curve is usually developed using a topographic map and one of the following formulas: the average-end area, frustum of a pyramid, or prismoidal formula.

The average-end area formula is most often used on non-geometric areas. The average-end area formula is expressed as:

(Equation 14.1)

$$V_{1,2} = [(A_1 + A_2)/2]d$$

where: $V_{1,2}$ = storage volume between Elevations 1 and 2, m^3 (ft^3)
$A_{1,2}$ = surface area at Elevations 1 and 2 respectively, m^2 (ft^2)
d = change in elevation between Points 1 and 2, m (ft)

The incremental change in elevation d is typically set at 0.25 m (1 ft) for use with the average-end area formula when designing stormwater storage facilities. Smaller values of d result in more accurate estimation of the available storage.

The frustum of a pyramid is expressed as:

(Equation 14.2)

$$V = d/3\ [A_1 + (A_1A_2)^{0.5} + A_2]$$

where: V = volume of frustum of a pyramid, m^3 (ft^3)
d = change in elevation between Points 1 and 2, m (ft)
$A_{1,2}$ = surface area at Elevations 1 and 2 respectively, m^2 (ft^2)

The prismoidal formula for trapezoidal basins is expressed as:

(Equation 14.3)

$$V = LWD + (L + W)\ ZD^2 + 4/3\ Z^2\ D^3$$

where: V = volume of trapezoidal basin, m^3 (ft^3)
L = length of basin bottom, m (ft)
W = width of basin bottom, m (ft)
D = depth of basin, m (ft)
Z = side slope factor, ratio of horizontal to vertical

C. Stage-Discharge Curve. A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility based upon the configuration of the basin's outlet control features. Normally the discharge is plotted on the x-axis with the stage plotted on the y-axis. A typical storage facility has two spillways, a principal spillway and an emergency spillway, and the stage-discharge curve should be developed using the cumulative discharge from both spillways. The stage-discharge curve is affected by the size and configuration of the outlet pipe culvert, weir, orifice or other outlet control device which is typically located in the principal spillway or outlet. To accurately model the outflow hydrograph from a storage facility, tailwater influences and structure losses must also be considered when developing discharge curves. As with the stage-storage curve, computation of smaller incremental changes in head will result in a more accurate stage-discharge relationship.

D. Procedure. A general procedure for using the above data in the design of storage facilities is presented below.

Step 1 Compute inflow hydrograph for runoff from various return periods being assessed design storms using the procedures outlined in Chapter 7, *Hydrology*. Both pre- and post-development hydrographs are required for all of the design storms of interest.

Step 2 Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1.

Step 3 Determine the physical dimensions of the storage facility necessary to hold the estimated volume from Step 2. Extra storage volume will be needed to account for freeboard. The maximum storage requirement calculated from Step 2 should be used to set the preliminary size of the storage facilities.

Step 4 Size and configure the outlet structure to control the rate and flow from the storage facility in order to achieve the facility's stormwater management objectives. For initial computations assume the peak stage will occur at the estimated volume from Step 2 for the event analyzed with the largest return period (least probability of occurrence).

Step 5 Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using the storage routing equations. If any of the routed post-development peak rates of discharge exceed the pre-development peak discharges, or exceed those release rates prescribed by the watershed's stormwater management plan, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to Step 3.

Step 6 Design an emergency overflow for the storage facility with the capacity to safely convey the largest inflow that is capable of entering the basin, safely through the basin, in the event the principal spillway malfunctions. Often times large events are not capable of entering a storage facility because of conveyance limitations associated with other aspects of the drainage system, for example when storm drainage systems are designed for a return interval less than the 100-year event.

Step 7 Evaluate the downstream effects of detention outflow in a manner consistent with the methodology discussed in the Design Criteria section of this chapter to ensure that the routed hydrograph does not cause downstream degradation problems.

Step 8 Evaluate the outlet velocity of stormwater runoff conveyed through the outlet control structure and discharged to the environment to ascertain if channel/bank stabilization immediately downstream of the storage facility is necessary to control excessive velocities which can cause erosion problems.

The design of storage facilities is an iterative process which can involve a significant number of reservoir routing calculations to obtain the desired results.

14.11 OUTLET HYDRAULICS

A. Outlets. Sharp-crested weir flow equations for no end contractions, two end contractions and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, V-notch weirs, weirs and orifices. If culverts are used as outlets works, procedures presented in Chapter 9, *Culverts* should be used to develop stage-discharge data. When analyzing release rates, the tailwater influence of the principal spillway culvert on the outlet control structure (orifice and/or weirs) must be considered to determine the effective head on each opening. Although the configuration of outlets may take on many different geometric shapes and forms, slotted riser pipes should not be used for the outlet of permanent stormwater facilities because of their propensity to clog. Perforated metal risers may be used provided the metal is of sufficient gage and is galvanized, coated, or aluminized.

B. Sharp-Crested Weirs. A sharp crested weir is a vertical device placed across a flow area, which controls the rate of flow, through an opening or over the top of a device, based upon the amount of head above the invert of the flow area through the weir. Sharp crested weirs differ from broad crested weirs in that they the cause flow lines exiting the weir to bend or curve downward. A sharp-crested weir with no end contractions is illustrated in Figure 14.3. The discharge equation for this configuration is :

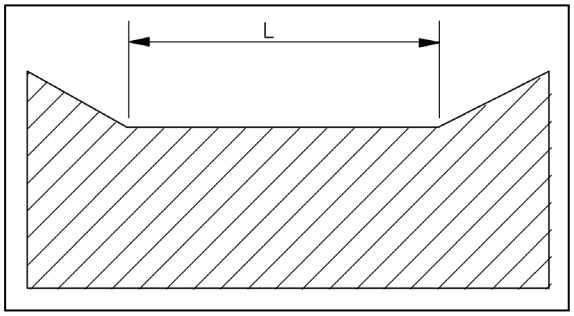
(Equation 14.4)

Metric:
$$Q = [1.805 + 0.221(H/H_c)]\ LH^{1.5}$$

U.S. Customary:
$$Q = [3.22 + 0.44(H/H_c)]\ LH^{1.5}$$

where: Q = discharge, m^3/s (cfs)
H = head above weir crest excluding velocity head, m (ft)
H_c = height of weir crest above channel bottom, m (ft)
L = horizontal weir length, m (ft)

Figure 14.21 Sharp-Crested Weir (No End Contractions)



A sharp-crested weir with two end contractions is illustrated in Figures 14.22 and 14.23. The discharge equation for this configuration is:

(Equation 14.5)

Metric:
$$Q = [1.805 + 0.221(H/H_c)]\ (L - 0.2H)\ H^{1.5}$$

U.S. Customary:
$$Q = [3.22 + 0.44(H/H_c)]\ (L - 0.2H)\ H^{1.5}$$

where: variables are the same as Equation 14.4.

Figure 14.22 Sharp-Crested Weir (Two End Contractions)

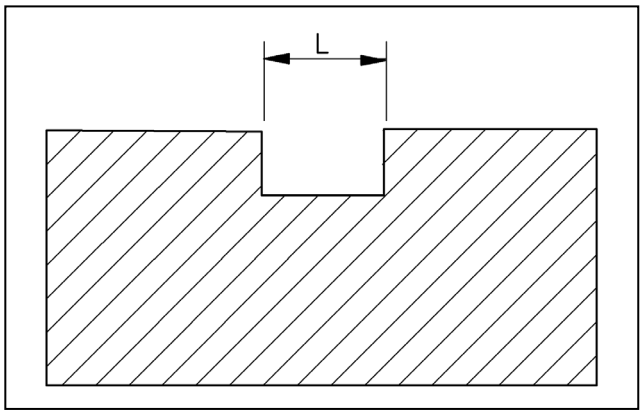
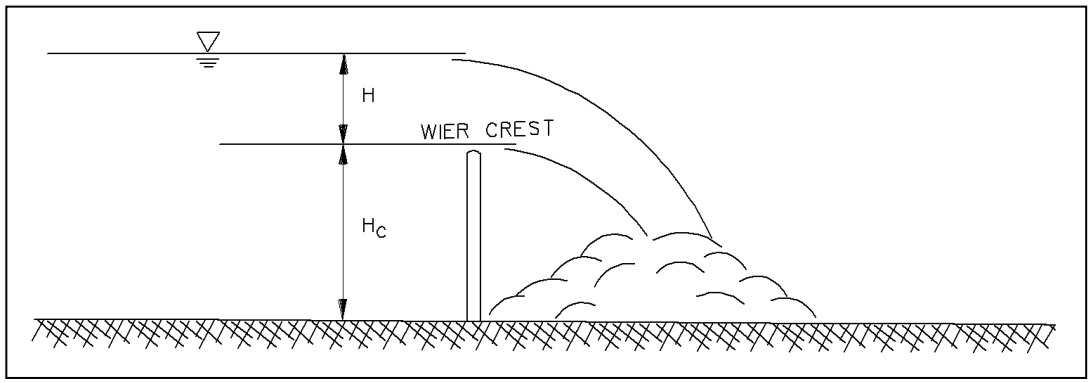


Figure 14.23 Sharp-Crested Weir and Head



A sharp-crested weir will be affected by submergence when the tailwater rises above the weir-crest elevation. The result will be that the discharge over the weir will be reduced by the rising tailwater. The discharge equation for a sharp-crested submerged weir is:

(Equation 14.6)

$$Q_S = Q_f(1 - (H_2/H_1)^{1.5})^{0.385}$$

where: Q_s = submergence flow, m^3/s (cfs)
Q_f = free flow, m^3/s (cfs)
H_1 = upstream head above crest, m (ft)
H_2 = downstream head above crest, m (ft)

C. Broad-Crested Weirs. Broad crested weirs function similar to sharp crested weirs in that the flow over the weir is a function of the amount of head above the invert of the weir opening. Broad crested weirs differ from sharp crested weirs in that the flow exiting the weir leaves the weir in a horizontal direction and does not bend downward like with a sharp crested weir. The equation generally used for a broad-crested weir is:

(Equation 14.7)

$$Q = CLH^{1.5}$$

where: Q = discharge, m^3/s (cfs)
C = broad-crested weir coefficient
L = broad-crested weir length, m (ft)
H = head above weir crest, m (ft)

If the upstream edge of a broad-crested weir is rounded 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 1.704 ($C_{U.S.\ Customary} = 3.087$). For sharp corners on the broad-crested weir, a minimum C value of 1.435 ($C_{U.S.}$

$_{Customary}$ = 2.6) should be used. Additional information on C values as a function of weir crest breadth and head is given in Table 14.6.

Table 14.2(a) Broad-Crested Weir Coefficient C-Values as a Function of Weir Crest Breadth and Head (m)

Measured Head, *H*[1] (m)	**Breadth of the Crest of Weir (m)**										
	0.15	0.23	0.30	0.46	0.61	0.76	0.91	1.22	1.52	3.05	4.57
0.06	1.55	1.52	1.49	1.45	1.40	1.37	1.35	1.31	1.29	1.37	1.48
0.12	1.61	1.55	1.50	1.46	1.44	1.44	1.42	1.40	1.38	1.41	1.49
0.18	1.70	1.60	1.52	1.46	1.44	1.44	1.48	1.49	1.49	1.49	1.49
0.24	1.82	1.68	1.57	1.48	1.44	1.44	1.47	1.48	1.48	1.49	1.46
0.30	1.83	1.73	1.65	1.52	1.47	1.46	1.46	1.47	1.48	1.48	1.45
0.37	1.83	1.77	1.70	1.58	1.49	1.46	1.46	1.47	1.47	1.49	1.46
0.43	1.83	1.80	1.77	1.61	1.53	1.48	1.46	1.46	1.46	1.47	1.46
0.49	1.83	1.82	1.81	1.69	1.60	1.52	1.48	1.47	1.46	1.46	1.45
0.55	1.83	1.83	1.83	1.69	1.59	1.51	1.48	1.47	1.46	1.46	1.45
0.61	1.83	1.83	1.82	1.67	1.57	1.52	1.52	1.48	1.46	1.46	1.45
0.76	1.83	1.83	1.83	1.81	1.69	1.60	1.55	1.50	1.47	1.46	1.45
0.91	1.83	1.83	1.83	1.83	1.77	1.68	1.61	1.51	1.47	1.46	1.45
1.07	1.83	1.83	1.83	1.83	1.83	1.76	1.64	1.52	1.48	1.46	1.45
1.22	1.83	1.83	1.83	1.83	1.83	1.83	1.69	1.54	1.49	1.46	1.45
1.37	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.59	1.51	1.46	1.45
1.52	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.69	1.54	1.46	1.45
1.68	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.59	1.46	1.45

Table 14.2(b) Broad-Crested Weir Coefficient C-Values as a Function of Weir Crest Breadth and Head (ft)

Measured Head, *H*[1] (ft)	**Breadth of the Crest of Weir (ft)**										
	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[1]Measured at least 2.5*H* upstream of the weir.
Source: Reference (Brater and King, 1976)

D. V-Notch Weirs. The discharge through a V-notch or triangular weir can be calculated from the following equation:

(Equation 14.8)

Metric:	**U.S. Customary:**
$Q = 1.38 \tan(\theta/2) H^{2.5}$	$Q = 2.5 \tan(\theta/2) H^{2.5}$

where: Q = discharge, m^3/s (cfs)
θ = angle of V-notch, degrees
H = head on apex of notch, m (ft)

E. Orifices. An orifice is a vertical or horizontal opening through which water may pass. Orifices may be configured in any variety of shapes and sizes, and are not limited to circular shapes only. In certain instances flow through pipes may be calculated using the orifice equation. Pipes smaller than 300 mm (12 in) may be analyzed as a submerged orifice if *H/D* is greater than 1.5. Pipes with an H/D ratio of less than 1.5 may be flowing only partially full and may be acting under outlet control. Thus, the discharge through the pipe may be less than that determined using the orifice equation. For sharp-edged entrance conditions the discharge through the orifice can be calculated from:

(Equation 14.9)

$$Q = CA(2gH)^{0.5}$$

where: Q = discharge, m^3/s (cfs)
A = cross-section area of orifice/pipe, m^2 (ft^2)
g = acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2)
H = head on pipe, from the center of pipe to the water surface, m (ft) *
C = discharge coefficient

* *Where the tailwater is higher than the center of the opening, the head is calculated as the difference in water surface elevations*

The discharge coefficient used in the orifice discharge equation will vary based upon the shape and configuration of the outlet according to the values provided in Table 14.3.

Table 14.3 Orifice Discharge Coefficients

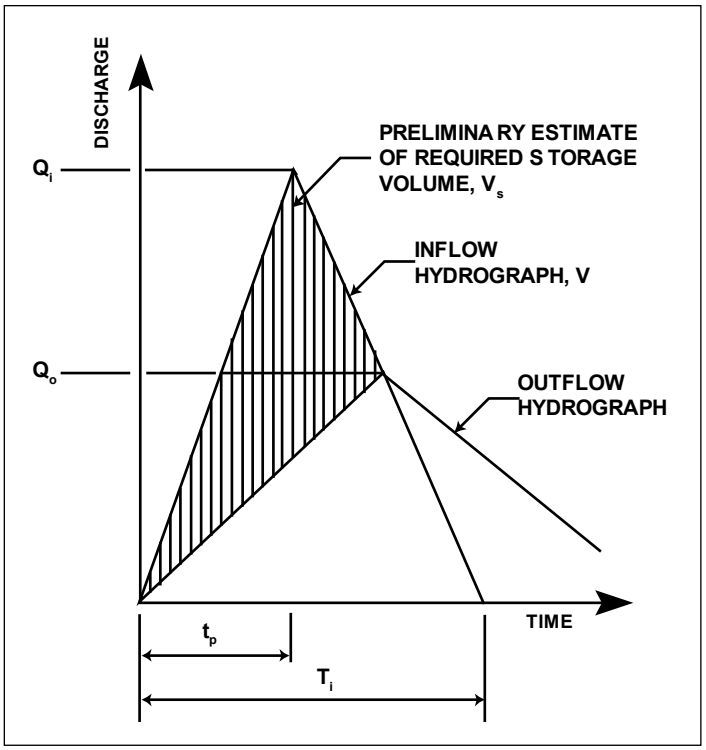
Description	**Discharge Coefficient**
Sharp-edged	0.62
Round edged	0.98
Short tube (fluid separates from walls)	0.61
Sharp tube (no separation)	0.82
Sharp tube with rounded entrance	0.97
Projecting entrance tube, (projection<½D)	0.54
Projecting entrance tube, (projection>½D)	0.72
Nozzle, Smooth and well tapered	0.98

Source: P.E. Reference Manual (Lindeburg, 2003)

14.12 PRELIMINARY DETENTION CALCULATIONS

A. Storage Volume. 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 the standard triangular shapes shown in Figure 14.24. Irrespective of the preliminary storage volume calculations the Department's design professionals are obligated to prove the size of storage facilities are adequate, by routing post-development flows through the basin, and demonstrating the applicable performance standards are achieved.

Figure 14.24 Triangular Shaped Hydrographs (For Preliminary Estimate of Required Storage Volume)



The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

(Equation 14.10)

$$V_S = 0.5T_i(Q_i - Q_o)$$

where: V_S = storage volume estimate, m^3 (ft^3)
Q_i = peak inflow rate, m^3/s (cfs)
Q_o = peak outflow rate, m^3/s (cfs)
T_i = duration of basin inflow, s

B. Alternative Method. An alternative preliminary estimate of the storage volume required for a specified peak-flow reduction can be obtained by the following regression equation procedure:

1. Determine input data, including the allowable peak outflow rate, Q_o, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, 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:

(Equation 14.11)

$$V_S/V_r = [1.291(1 - Q_o/Q_i)^{0.753}]/[(t_b/t_p)^{0.411}]$$

where: V_S = volume of storage, m^3 (ft^3)
V_r = volume of runoff, m^3 (ft^3)
Q_o = outflow peak flow, m^3/s (cfs)
Q_i = inflow peak flow, m^3/s (cfs)
t_b = time base of the inflow hydrograph, hour (Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak)
t_p = time to peak of the inflow hydrograph, hour

3. Multiply the volume of runoff by the ratio of the volume of storage to volume of runoff calculated in Step 2 to determine the estimated basin storage volume.

C. Peak-Flow Reduction. A preliminary estimate of the potential peak-flow reduction for a selected storage volume can be obtained by the following procedure:

1. Determine the following:

- Volume of runoff, V_r
- Peak-flow rate of the inflow hydrograph, Q_i
- Time base of the inflow hydrograph, t_b
- Time to peak of the inflow hydrograph, t_p
- Storage volume, V_S

2. Calculate a preliminary estimate of the potential peak-flow reduction for the selected storage volume using the following equation:

(Equation 14.12)

$$Q_o/Q_i = 1 - 0.712(V_S/V_r)^{1.328}(t_b/t_p)^{0.546}$$

3. Multiply the peak-flow rate of the inflow hydrograph, Q_i, times the potential peak-flow reduction calculated from Step 2 to obtain the estimated peak outflow rate, Q_o, for the selected storage volume (see Section 14.10.C of the example).

D. Preliminary Storage Dimensions.

- Plot the control structure location on a contour map.
- Select a desired depth of ponding for the design storm based upon site constraints, risk analysis, and jurisdictional regulations.
- Divide the estimated storage volume needed by the desired depth to obtain the surface area required of the reservoir.
- Based on site conditions and contours, estimate the geometric shape(s) required to provide the estimated basin surface area.

14.13 ROUTING PROCEDURE

The following procedure is used to perform routing through a storage facility (Puls Method of storage routing):

Step 1 Develop an inflow hydrograph, stage-storage curve and stage-discharge curve for the proposed storage facility. An example stage-storage curve is shown in Figure 14.8 and a stage-discharge curve is shown in Figure 14.9.

Step 2 Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph ($t < T_c/5$).

Step 3 Use the storage and discharge data from Step 1 to develop storage characteristics curves that correlate values of $S \pm (O/2)\Delta t$ to stage for a given basin configuration and outlet control structure design. An example tabulation of storage characteristics curve data is shown in Figure 14.7.

Step 4 Determine the value of $S_{i+1} + (O_{i+1}/2)\Delta t$ from the following equation:

(Equation 14.13)

$$S_{i+1} + (O_{i+1}/2)\Delta t = [S_i - (O_i/2)\Delta t] + [(I_i + I_{i+1})/2]\Delta t$$

where:
S_{i+1} = storage volume at end of time interval, m^3 (ft^3)
O_{i+1} = outflow rate at end of time interval, m^3/s (cfs)
Δt = routing time period, s
S_i = storage volume at beginning of time interval, m^3 (ft^3)
O_i = outflow rate at beginning of time interval, m^3/s (cfs)
I_i = inflow rate at beginning of time interval, m^3/s (cfs)
I_{i+1} = inflow rate at beginning of next time interval, m^3/s (cfs)

Other consistent units are equally appropriate.

Step 5 Between interval 1 and 2, both I_i and I_{i+1} are known. At the start of the storm the inflow into the basin is just beginning to occur and the stage is the bottom of the basin. Therefore, at the start of the initial time interval the outflow from the basin and storage within the basin are both 0 and the resulting value of $S_i - (O_i/2)\Delta t$ is also 0. Combining the appropriate values, the right side of Equation 14.13 reduces to $(I_i+I_{i+1})\Delta t/2$, at the beginning of the event. Since, $S_i - (O_i/2)\Delta t = 0$, the left side of the equation equals $(I_i+I_{i+1})\Delta t/2$, the stage may be determined from Figure 14.7 using the $S+(O/2)\Delta t$ curve. This represents the stage at the end of the time interval. The storage and discharge at the end of the interval may then be read off of Figure 14.7, respectively. The data used to create the respective curves is provided in Table 14.21.

Step 6 The stage, discharge and storage values calculated in the previous step are then used to calculate the unknown values for the next time interval. This process is repeated using the new value for I_i, I_{i+1}, O_i, and S_i to calculate a value for the left hand side of Equation 14.13. Figure 14.9 and the $S+(O/2)\Delta t$ curve is used to determine the stage at the end of the interval between periods 2 and 3. Figure 14.7 can be used to determine the discharge and storage at the end of the respective time interval.

Step 7 Step 5 and 6 may be repeated using the known values from the previous steps to determine the stage storage and discharge of the current step until the entire hydrograph is routed through the storage facility.

14.14 ROUTING EXAMPLE PROBLEM

A. Example. This example demonstrates the application of the routing methodology presented in this chapter for the design of a typical detention storage facility. Example inflow hydrographs and associated peak discharges for both pre- and post-development conditions can be developed using hydrologic methods from Chapter 7, *Hydrology*.

Table 14.4(a) Stage-Discharge-Storage Data (Metric)

Stage (m)	**Discharge, O (m^3/s)**	**Storage, S (m^3)**	**S+OΔt/2 (m^3)**
0.00	0	0	0
0.27	0.3	320	372
0.46	0.6	520	618
0.57	0.9	690	837
0.65	1.1	850	1049
0.77	1.4	1000	1239
0.87	1.7	1150	1447
0.97	2.0	1295	1644
1.07	2.3	1445	1842
1.17	2.6	1580	2012
1.25	2.9	1725	2214
1.38	3.4	2010	2593
1.47	3.7	2160	2800
1.54	4.0	2310	2989
1.61	4.2	2441	3187
1.69	4.6	2590	3378
1.77	4.9	2740	3569
1.81	5.1	2885	3784
1.95	5.7	3195	4172
2.07	6.2	3490	4574
2.13	6.5	3640	4770
2.25	7.0	3960	5189

Note: Δt = 6 min (360 sec)

Table 14.4(b) Stage-Discharge-Storage Data (U.S. Customary)

Stage (ft)	**Discharge, O (cfs)**	**Storage, S (ac-ft)**	**S+OΔt/2 (ac-ft)**
0.0	0	0.00	0.00
0.9	11	0.26	0.30
1.4	21	0.42	0.51
1.8	30	0.56	0.68
2.2	40	0.69	0.86
2.5	49	0.81	1.01
2.9	61	0.93	1.18
3.2	71	1.05	1.34
3.5	81	1.17	1.51
3.7	88	1.28	1.65
4.0	99	1.40	1.81
4.5	118	1.63	2.12
4.8	130	1.75	2.29
5.0	139	1.87	2.44
5.3	151	1.98	2.61
5.5	160	2.10	2.76
5.7	169	2.22	2.92
6.0	182	2.34	3.09
6.4	201	2.58	3.41
6.8	220	2.83	3.74
7.0	230	2.95	3.90
7.4	250	3.21	4.24

Note: Δt *= 6 min (360 sec)*

Figure 14.25(a) Example Storage Characteristics Curve (Metric)

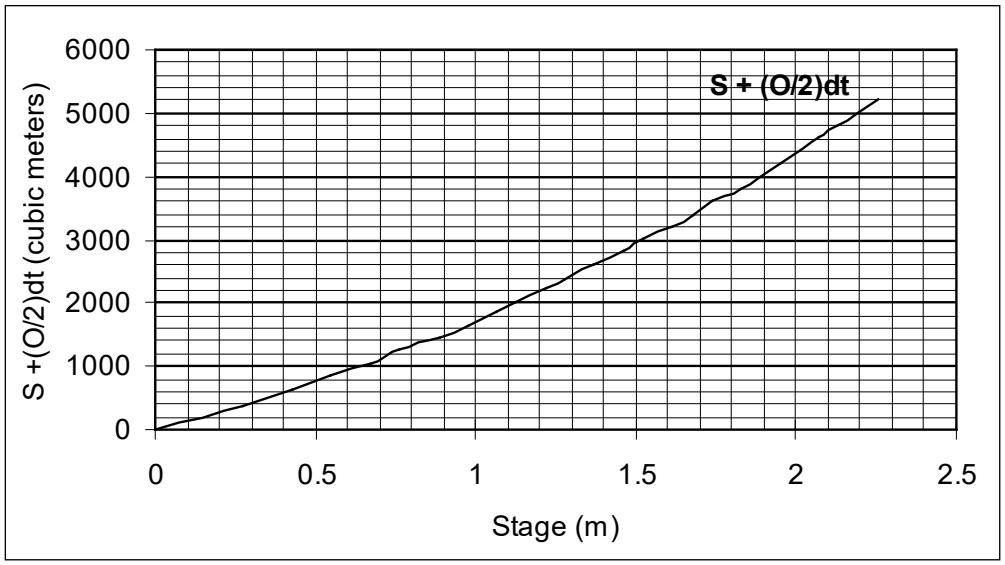


Figure 14.25(b) Example Storage Characteristics Curve (U.S. Customary)

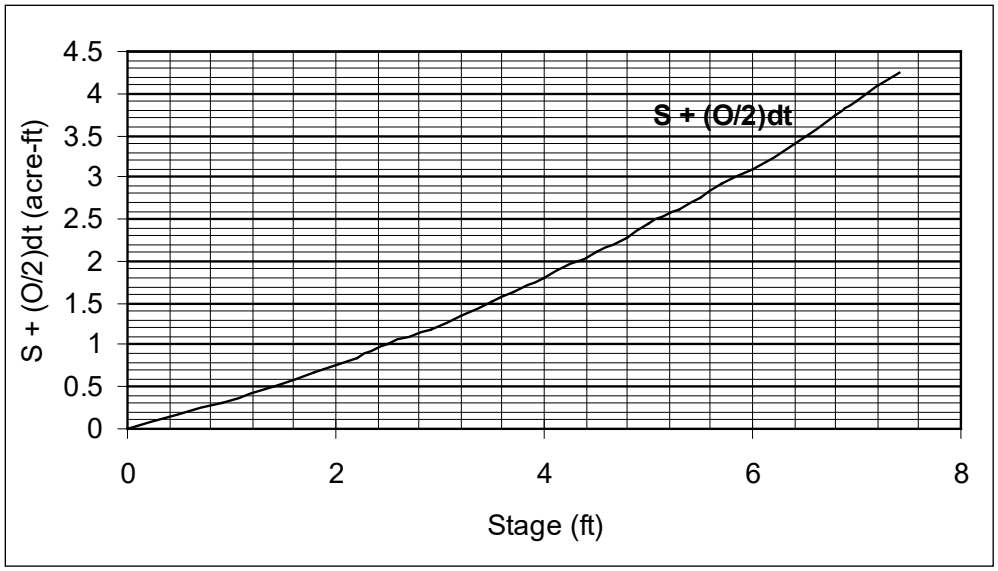


Figure 14.26(a) Example Stage-Storage Curve (Metric)

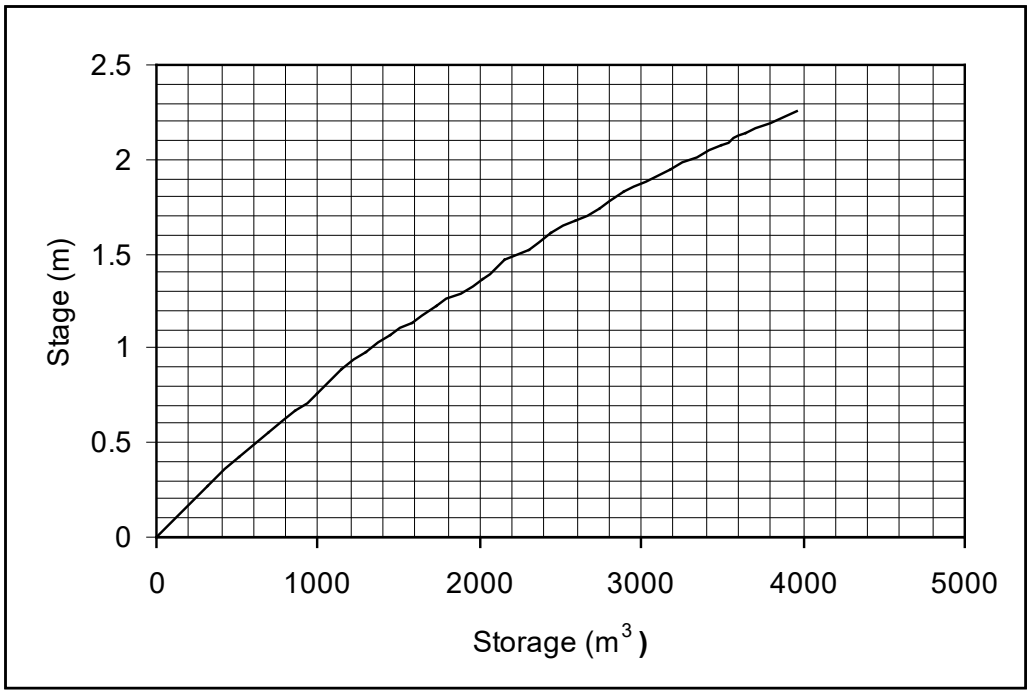


Figure 14.26(b) Example Stage-Storage Curve (U.S. Customary)

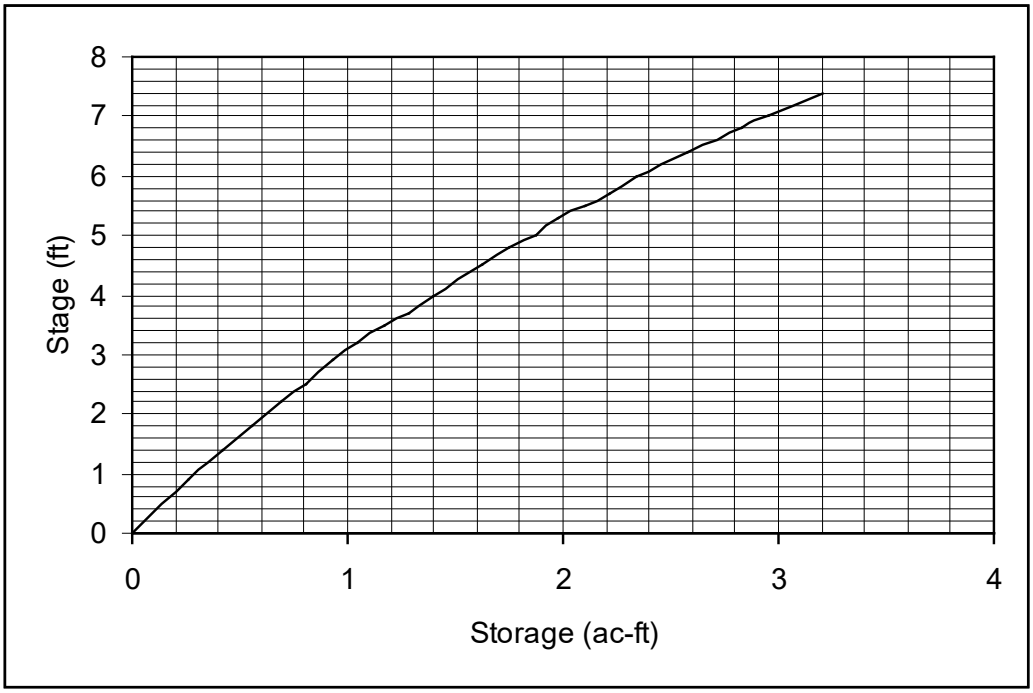


Figure 14.27(a) Stage-Discharge Curve (Metric)

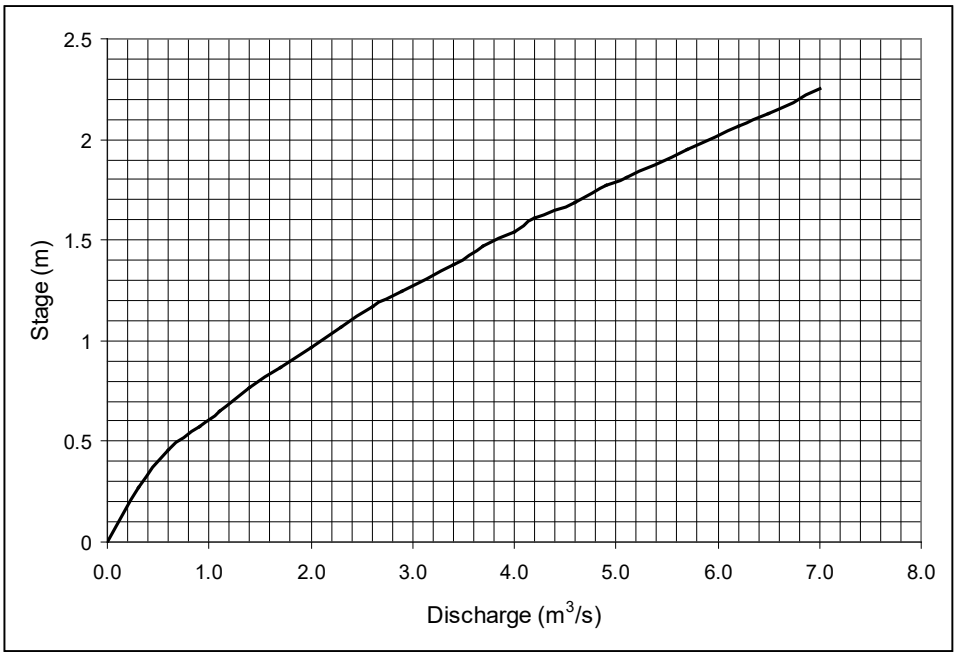
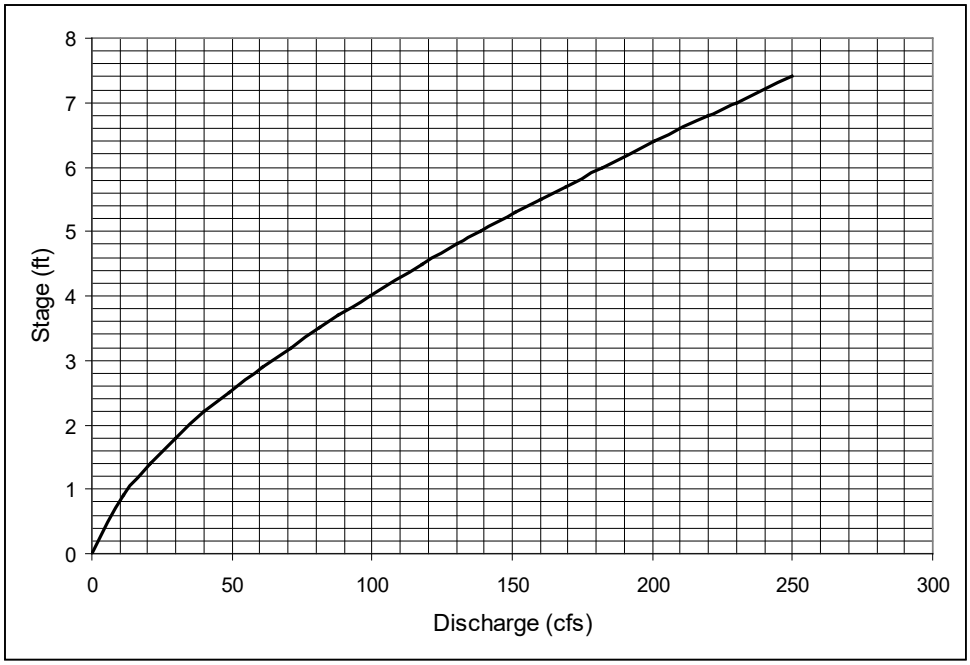


Figure 14.27(b) Stage-Discharge Curve (U.S. Customary)



B. Design Discharge and Hydrographs. Storage facilities are ordinarily designed to control runoff from a range of design storms to ensure that the facility can accommodate runoff from several events of varying sizes without creating problems associated with uncontrolled stormwater runoff. However, for this example, only peak discharges for the 10-year design storm event are presented for demonstration purposes. The respective hydrographs for the pre- and post-development conditions are as follows:

- Pre-development 10-year peak Q = 5.66 m^3/s (200 cfs)

- Post-development 10-year peak Q = 7.08 m^3/s (250 cfs).

Therefore, the allowable design discharge is 5.66 m^3/s (200 cfs) for the 10-year storm. This maximum allowable discharge limit is established so that the post-development peak discharge does not exceed the pre-development peak discharge.

Example runoff hydrographs are shown in Table 14.5. The inflow duration for the post-development hydrograph is approximately 1.25 hours.

Table 14.5(a) Example Runoff Hydrographs (Metric)

Time (h)	Pre-Development Runoff (m^3/s)	Post-Development Runoff Without Storage (m^3/s)
0.0	0	0
0.1	0.68	1.42
0.2	2.29	5.04
0.3	4.81	7.08*
0.4	5.66	4.67
0.5	4.25	2.55
0.6	2.69	1.42
0.7	1.73	0.82
0.8	1.13	0.45
0.9	0.79	0.25
1.0	0.51	0.14
1.1	0.42	0.08
1.2	0.37	0.03

*Note: The peak discharge without stormwater management controls is higher than the allowable discharge of 5.66 m^3/s for the 10-year design storm.

Table 14.5(b) Example Runoff Hydrographs (U.S. Customary)

Time (h)	Pre-Development Runoff (cfs)	Post-Development Runoff Without Storage (cfs)
0.0	0	0
0.1	24	50
0.2	81	178
0.3	170	250*
0.4	200	165
0.5	150	90
0.6	95	50
0.7	61	29
0.8	40	16
0.9	28	9
1.0	18	5
1.1	15	3
1.2	13	1

*Note: The peak discharge without stormwater management controls is higher than the allowable discharge of 200 cfs for the 10-year design storms.

C. Preliminary Volume Calculations. A preliminary estimate of the required storage volume is obtained using the simplified method outlined in Figure 14.26. For runoff from the 10-year storm, the required storage volume, V_S, is computed using Equation 14.10:

$$V_S = 0.5T_i(I - O)$$

$$V_S = [0.5(1.25)(3600)(7.08 - 5.60)] = 3195 \text{ m}^3 \text{ (2.58 ac-ft)}$$

The stage-storage data for the proposed storage facility is provided in Table 14.21 and a graphical presentation of the storage curve is provided in Figure 14.26.

D. Design and Routing Calculations. A stage-storage and stage-discharge relationship for a proposed storage facility is shown in Figures 14.26 and 14.27, respectively. The proposed basin is capable of providing adequate peak-flow attenuation a 10-year design storm. The storage-discharge relationship was developed for the proposed facility by providing, at a minimum, the estimated storage volume for 10-year design storm at a stage where the peak discharge occurs. Discharge values were computed by solving the broad-crested weir equation using the head, H, acting upon the weir and a discharge coefficient of 1.7 ($C_{\text{U.S. Customary}} = 3.1$), assuming a constant weir length of 1.2 m (4 ft) and no tailwater submergence.

Storage routing was conducted for runoff from the 10-year design storm to confirm the preliminary storage volume estimate and to establish a design water surface elevation. Routing results using the Storage Characteristics Curve given previously in Figure 14.25 and the Stage-Discharge-Storage data given in Table 14.4 using 0.1-hour incremental time steps are shown in Table 14.5 for runoff from the 10-year design storm. The preliminary design provides adequate peak discharge attenuation for the 10-year design storm.

For the routing calculations, Equation 14.13 was used:

$$S_{i+1} + (O_{i+1}/2)\Delta t = [S_i - (O_i/2)\Delta t] + [(I_i + I_{i+1})/2]\Delta t$$

Because the routed peak discharge is lower than the maximum allowable peak discharge, the storage and outlet design are acceptable. If the discharge was significantly lower than permitted, the weir length could be increased or the storage decreased to avoid the design of a facility that is larger than needed. However, if either the stage-discharge or

stage-storage relationship is altered, routing calculations must be repeated to verify that the design meets the prescribed stormwater control guidelines.

Although not shown for this example, runoff from the project specific design storm should be routed through the storage facility to establish freeboard and to evaluate emergency overflow capacity and stability requirements. In addition, the example problem presents hydraulic details only. Final design should consider site constraints (e.g., depth of water, side slope stability, maintenance, grading) to prevent standing water and provide for public safety.

Table 14.6(a) Storage Routing for the 10-Year Storm (Metric)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (h)	I Inflow (m^3/s)	$[(I_1 + I_2)]\Delta t/2$ (m^3)	Stage (m)	S Storage (m^3)	$S_i - (O_i/2)\Delta t$ (m^3)	$S_{i+1} + (O_{i+1}/2)\Delta t$ (m^3)	O Outflow (m^3/s)
0.0	0.00	254.86	0.00	0.00	0.00	254.86	0.00
0.1	1.42	1162.17	0.19	218.13	181.43	1343.61	0.20
0.2	5.04	2181.61	0.82	1065.71	787.87	2969.51	1.54
0.3	7.08	2115.34	1.51	2272.48	1575.56	3690.92	3.87
0.4	4.67	1299.79	1.78	2801.43	1912.04	3211.84	4.94
0.5	2.55	713.61	1.61	2441.14	1668.68	2382.30	4.29
0.6	1.42	402.68	1.28	1838.25	1294.29	1696.98	3.02
0.7	0.82	229.37	0.99	1324.74	952.55	1181.93	2.07
0.8	0.45	127.43	0.73	947.32	712.75	840.18	1.30
0.9	0.25	71.36	0.55	688.89	536.79	608.16	0.85
1.0	0.14	40.78	0.42	506.41	404.72	445.50	0.56
1.1	0.08	20.39	0.32	377.02	308.58	328.97	0.38
1.2	0.03	5.10	0.24	281.54	234.16	239.26	0.26
1.3	0.00	0.00	0.17	204.79	170.34	170.34	0.19

Table 14.6(b) Storage Routing for the 10-Year Storm (U.S. Customary)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (hour)	I Inflow (cfs)	$[(I_i + I_{i+1})]\Delta t/2$ (ft^3)	Stage (ft)	S Storage (ft^3)	$S_i - (O_i/2)\Delta t$ (ft^3)	$S_{i+1} + (O_{i+1}/2)\Delta t$ (ft^3)	O Outflow (cfs)
0.0	0	9,000	0	0	0	9,000	0
0.1	50	41,040	0.61	7703	6,407	47,447	7
0.2	178	77,040	2.68	37,634	27,822	104,862	55
0.3	250	74,700	4.95	80,249	55,638	130,338	137
0.4	165	45,900	5.83	98,928	67,520	113,420	174
0.5	90	25,200	5.30	86,205	58,926	84,126	152
0.6	50	14,220	4.20	64,915	45,705	59,925	107
0.7	29	8,100	3.26	46,781	33,637	41,737	73
0.8	16	4,500	2.40	33,453	25,169	29,669	46
0.9	9	2,520	1.80	24,327	18,956	21,476	30
1.0	5	1,440	1.37	17,883	14,292	15,732	20
1.1	3	720	1.04	13,314	10,897	11,617	13
1.2	1	180	0.79	9,942	8,269	8,449	9
1.3	0	0	0.55	7,232	6,015	6,015	7

Calculation Procedure:

1. At the start of the storm $I_i = 0$, $H_i = 0$, $S_i = 0$ and $O_i = 0$. Thus, 0 is entered into column 4, column 5 and column 8.
2. Calculate $[(I_i + I_{i+1})]\Delta t/2$, and record in column 3.
3. Calculate $S_i - (O_i/2)\Delta t$ and record in column 6.
4. Sum column 3 and column 6 and record in column 7.

5. The outflow and storage at time increment i+1 is determined by finding the stage using the $S + (O_i/2)\Delta t$ and stage relationship. Record the stage in column 4 for time increment i+1.
6. Use the stage at time increment i+1 to determine the storage and discharge at time increment i+1 from the stage-storage and stage-discharge relationship. Record these values in column 5 and column 8 for time increment i+1.
7. Advance to next time increment and repeat steps 2 through 7.

Figure 14.28(a) Runoff Hydrographs (Metric)

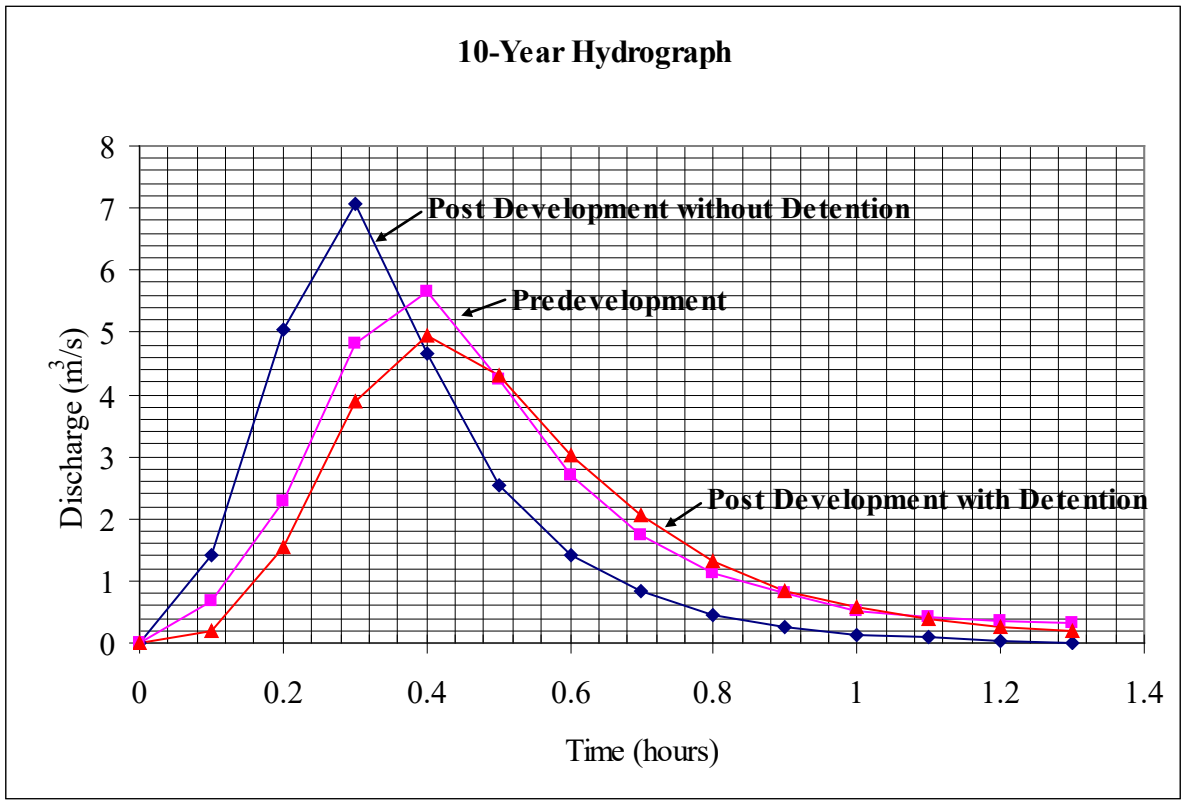
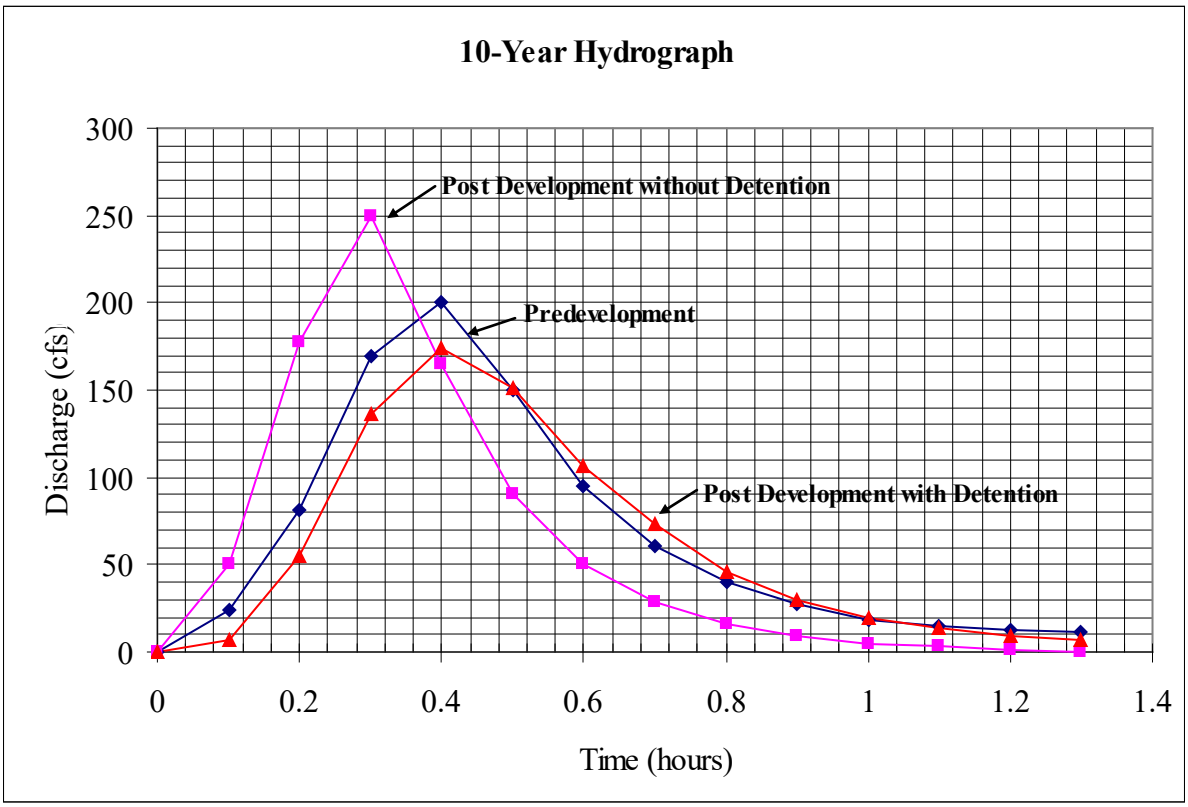


Figure 14.28(b) Runoff Hydrographs (U.S. Customary)



E. Downstream Effects. An estimation of the potential downstream effects (i.e., increased peak-flow rate and recession time) of storage facilities may be obtained by comparing the recession limbs of the pre-development and routed post-development hydrographs. A comparison of the hydrographs used in this example is provided in Figure 14.28 for the 10-year design storm.

It is important to be aware that as a watershed is developed it typically becomes more impervious with less water being infiltrated into the ground and greater volumes of water released into waterways and conveyed downstream. Detention basins simply hold the water and release it over a longer period of time, thus reducing the peak rate of runoff. Although the post-development peak rate of runoff is kept from exceeding the pre-development peak rate of runoff with a detention basin, the volume of runoff is usually increased and the length of time the peak is held is typically extended. This increase in volume of water being released may contribute to bed and bank degradation in the receiving channel.

14.15 DRY POND (DETENTION BASIN)

A. Introduction. Detention basins are depressed areas that store runoff during wet weather and release it at a controlled rate to reduce the impact of changes in land cover and land use on stormwater runoff. As implied by their name, under normal weather conditions these ponds are typically designed to be dry when it is not raining, or shortly thereafter. They are very popular because of their comparatively low cost, few design limitations, ability to serve large and small watersheds, and depending on their size, their ability to be incorporated into other uses (e.g., recreational areas).

With few design constraints these facilities are not restricted to a maximum drainage area limit. To ensure the greatest probability of success and construction of a functional dry pond, the following should be considered when locating and designing a dry detention basin.

1. Site Selection.

a. Locate the pond to maximize flow to the facility (i.e., at the low point in the topography in close proximity to an existing waterway or drainage course).

b. Fit the pond into the existing terrain. Design the pond as if it were a natural part of the landscape while being cognizant that intricate grading plans may be difficult and costly to construct. Generally, simpler geometric configurations are preferred over more complex configurations and should be selected as long as the project's water quality objectives can be achieved.

c. Pond awareness. Situate the pond in a location where it does not become a hazard or in areas without known problems. Without special considerations or features incorporated into the design, dry detention basins should not be located near or within recreation facilities. Dry detention basins can be potentially dangerous during storm events with water depths rapidly increasing in short spans of time.

2. Pond Grading. Grade dry ponds into the proposed site according to the following design criteria:

- 1% (.01ft/ft) Minimum longitudinal slope of basin bottom (flow from inlet to outlet).
- 3 m (10 ft) Minimum pond embankment top width.
- 4.5 m (15 ft) Maximum pond embankment height without potentially requiring a dam permit.
- 1V:3H (3H:1V) Maximum outer side slopes.
- 1V:2H (2H:1V) Maximum inside side slopes.
- 1V:4H (4H:1V) Maximum steepness for mowable side slopes.

3. Pond Shape. Dry ponds are most effective when they contain an irregular curvilinear pattern. Shapes such as squares, rectangles, and pointed triangles are easy to construct, but often do not appear natural and are less aesthetically pleasing than the aforementioned geometric shapes. As best as possible, the pond shape should blend into the existing environment. The shape of the pond can affect the pollutant-removal efficiency. Typically, the length-to-width ratio should be at least 2L:1W.

4. Maintenance Access. An access road with a minimum width of 3 m (10 ft) should be provided so that maintenance personnel can easily access the pond to facilitate the removal of debris and sediment and rehabilitate the BMP as needed.

5. Fencing/Safety. Safety considerations to reduce the chance of drowning include fencing the basin, reducing the maximum ponding depth, and including ledges or mild slopes. Fencing around dry ponds should only be considered when inside side slopes do not meet the criteria mentioned above and the pond is considered a hazard. Clear zone requirements and/or guide rail are necessary regardless if fences are present or not. When fences are proposed, they should be located outside of the clear zone for traffic safety reasons.

6. Emergency Spillway. All basins should be equipped with an emergency spillway. The emergency spillway provides relief in the event the principal spillway becomes blocked or fails. Flows larger than the design event are usually conveyed by the emergency spillway. Spillway side slopes should be no steeper than 1V:3H (3H:1V) with a minimum bottom width of 3 m (10 ft). All spillways should be designed to prevent erosive exit velocities by stabilizing the spillway with suitable vegetated or rock lining. In certain instances the top of the outlet control structure may be used as an emergency spillway. For a case where the top of the principal spillway is used as an emergency spillway, a secondary emergency spillway should be considered in the event the principal spillway outlet fails. Freeboard above the design depth of the spillway, to the top of the basin embankment, should be no less than 150 mm (0.5 ft).

7. Principal Spillway. As the primary outlet of the pond, the principal spillway system includes the riser assembly (outlet structure), the outfall pipe, outfall scour protection, weirs, orifices, maintenance drains, trash racks or any other components necessary for proper stormwater release. Outlets for dry ponds can be designed in a wide variety of configurations, but most outlets use riser structures constructed from concrete. Corrugated metal risers are acceptable for long-term operations if they are of sufficient gage and are galvanized, coated, or aluminized. These risers can be designed to control different storms using a single or combination of openings set at different elevations on the riser.

a. Riser Assembly (outlet structure). Risers are designed to satisfy the post-construction stormwater management objectives for a project using orifices, weirs, or a combination of these elements set at different elevations on the riser. For example, a small orifice is used to control the water quality volume or provide extended detention and a series of larger orifices or weirs set at different elevations may be used to control the larger events where rate controls are required. Larger flows may be controlled by allowing stormwater to flow through the top of the riser, using the entire riser diameter to convey the stormwater out of the facility. In some cases, an antivortex design may be necessary. The outlet structure should be designed to convey the appropriate design flow and dewater the basin completely within 48 hours after the end of each event. The design flow is not set arbitrarily for all a facilities, but is developed based on the project needs. Typically, the design process is initiated using a design storm based upon the roadway classification and then adjustment are made to select the appropriate event based upon the project's needs. As part of the design process for the riser assembly, a check should be made to assess the impact of the extended dewatering upon two successive 24-hour events and appropriate provisions be made to prevent overtopping of the facility if necessary.

b. Outfall Pipe. An outfall pipe functions to convey flow from the riser assembly to the outfall located downstream of the basin. Outfall pipes should convey the maximum outflow from the basin for the design event. The pipe should be reinforced concrete pipe or corrugated plastic pipe at the size and length dictated by design and necessary to provide the needed conveyance capacity. All pipes should be watertight and properly installed in accordance with Publication 408, *Specifications*. The minimum pipe diameter is 450 mm (18 in). If the principal spillway is to be used as an emergency spillway, the outfall pipe should be sized to convey the peak inflow that is capable of entering the basin.

c. Outfall Scour Protection. To prevent scour at the pond outfall, outlet protection should be provided and sized using the maximum discharge from the basin.

d. Weirs and Orifices. Any combination of weirs and orifices may be used to control the discharge of stormwater to meet the applicable performance standards. Orifice diameters should be no smaller than 75 mm (3 in) for clean out and maintenance purposes. Weirs and orifices should be designed such that the size and shape of the outlet are not compromised during the life of the outlet structure. To facilitate the construction of these features, the dimensions of weirs and orifices should be set at values which are easily defined and measured, such as whole numbers set in increments of 25 mm (1 in).

e. Trash Racks. Trash racks should be provided as necessary to protect orifices from clogging. Although trash racks are required for the lowest orifice, they may also be provided for other orifices if debris is a potential problem. Trash racks should be provided for the large opening in the top of the riser structure when a grate is not attached to provide for safety and prevent the deposition of debris in the outlet control structure, especially if the top of the structure is used to convey the larger events through the basin. Publication 72M, *Roadway Construction Standards*, RC-70M, may be referenced for standard drawings concerning trash racks, storage facilities and outlet works.

B. Design Objective.

1. Quality. A detention basin may be designed to improve its water quality features. This may be accomplished by constructing a series of BMPs within the basin and by designing the basin's outlet controls to manage the extended detention or retention of runoff. Incorporating pocket wetlands, shallow marshes or wet bottoms into the bottom of dry detention basins can provide an opportunity to meet the water quality objectives.

2. Quantity. The minimum performance standard with respect to quantity is that the pond should be designed to reduce the post-construction peak flow for the design storm event to levels that satisfy the PCSM targets, or if applicable, the release rate or district management objectives set forth in an approved watershed-wide Act 167 Stormwater Management Plan. To control these storms, the basin's storage should be at approximately equal to the area between the pre- and post-construction hydrographs. After a storage volume has been determined for each event, the design storm should be routed through the facility to ensure that the peak flows from the post-construction watershed do not exceed the permissible rates.

3. Quantity and Quality Combined. Combining the two design objectives (quality and quantity) will yield a dual-purpose detention basin. Several design variations may be considered to enhance the capabilities of the

facility. One consideration is shaping the basin to improve its pollutant-removal capabilities by extending the flow length within the basin. Inlet, outlet and side slopes should be stabilized with vegetation to prevent erosion, or in those areas subject to high velocities and shear stress, riprap or other stabilization measures. The basin floor should be stabilized with dense vegetation to prevent erosion and promote biological uptake of stormwater and certain non-point source pollutant. Impervious low flow channels or pilot channels through the basin are strongly discouraged except in the cases where severe ponding is an issue. 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 with a very flat bottom where sediment is easily deposited) can be used to trap the sediment before it enters the basin (see Figure 14.30). A check dam is typically placed downstream of the forebay area to separate the forebay from the remainder of the basin. Check dam sizing is based upon forebay size and depth with a spillway typically provided to facilitate the conveyance of water from the forebay into the main chamber of the basin.

Figure 14.29 Concrete Riser Example

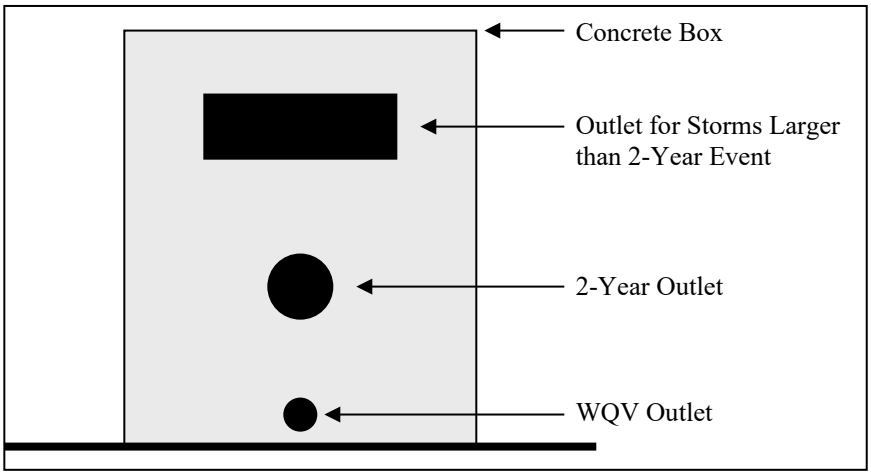
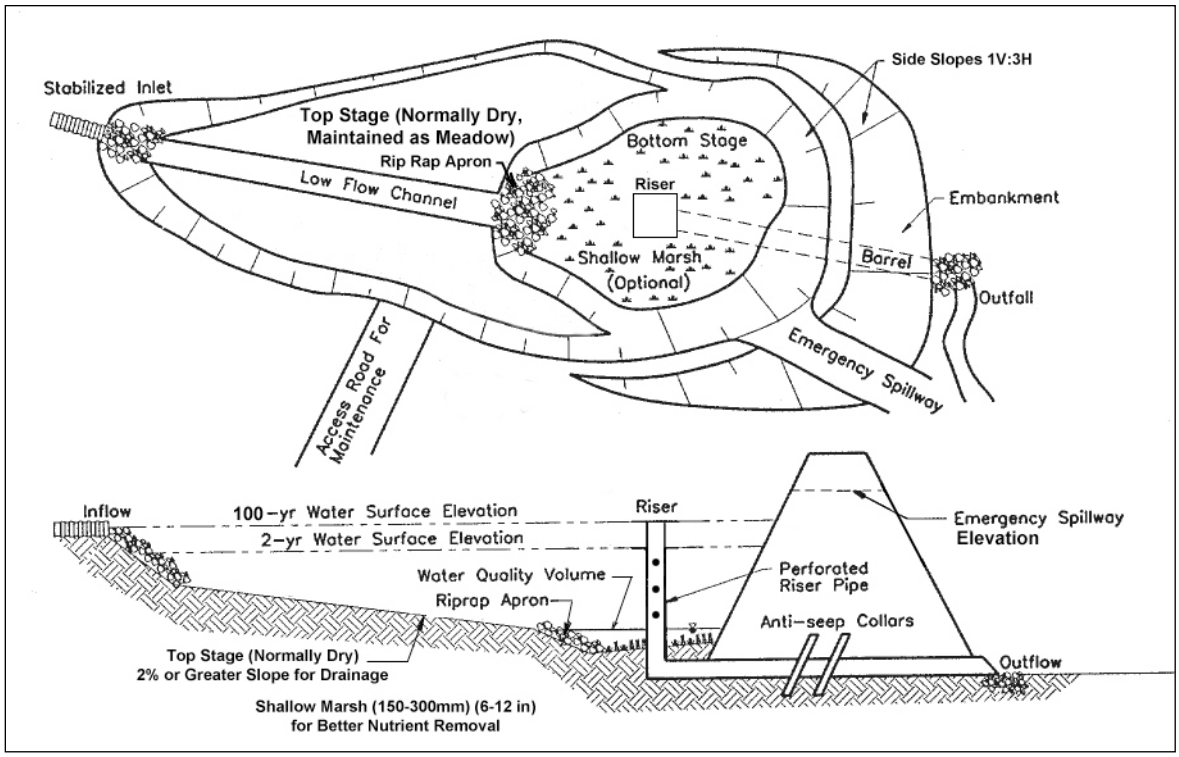


Figure 14.30 Example of Dry Pond Configuration



Reference: Schueler, 1987

14.16 WET POND

A. Introduction. A wet pond is very similar to a dry detention basin in that it detains stormwater, but it is different in that it maintains a permanent pool during dry weather. The benefit of a wet pond is that it may be used for both water quantity and water quality control, and because of the permanent pool, they may also be more aesthetically pleasing. Wet ponds are usually more expensive than dry detention basins to construct and maintain and are typically designed to serve large watersheds, in excess of 4-6 ha (10-15 ac). To ensure the greatest probability of success and a design that is fully functional, which is accepted by the public, the following standards should be adhered to in the wet pond design.

1. Site Selection

a. Locate the pond to maximize flow to the facility (i.e., the low point in the topography that is in close proximity to an existing waterway or drainage course). Ordinarily wet ponds are located below a spring or adjacent to a waterway so that a regular source of water is available to flow through the pond and prevent the water from stagnating.

b. Existing soils should be either of Hydrologic Soil Group C or D or an impervious liner is needed to create and retain the permanent pool.

c. Fit the pond into the original terrain. Design the pond as if it were a natural part of the landscape while being cognizant that intricate grading plans may be difficult and costly to construct. Generally simpler geometric configurations are preferred over more complex configurations and should be selected as long as the projects water quality objectives can be achieved.

d. Pond awareness. Locate the pond in an area such that it does not become a hazard.

2. Pond Grading. Grade wet ponds into the proposed site according to the following design criteria:

- 3.0 m (10 ft) Minimum pond embankment top width.
- 4.5 mm (15 ft) Maximum pond embankment height without potentially requiring a dam permit.
- 1.5 - 3.0 m (5 - 10 ft) Normal pond depth should be between.
- 1V:3H (3H:1V) Maximum outer side slopes.
- 1V:2H (2H:1V) Maximum inside side slopes.
- 1V:4H (4H:1V) Maximum steepness for mowable side slopes.

3. Pond Shape. The shape of the pond can significantly affect the pollutant-removal efficiency of a wet pond and a minimum length-to-width ratio of 2L:1W should be provided. Figure 14.31 shows several pond configurations that may be used to increase the length-to-width ratio of the pond. Although many shapes of ponds are possible care should be taken to avoid designing a pond which cannot be physically constructed. Normally, the further a pond diverges from regular geometric shapes the more difficult and costly the feature becomes to construct.

4. Safety/Vegetation Ledge. All ponds that maintain a permanent wet pool require a 3 m (10 ft) minimum safety bench (ledge). The safety bench (ledge) should be 300 mm (1 ft) below the normal pool elevation of the pond for establishment of vegetation. An additional safety bench/ledge should be provided 300 mm (1 ft) above the normal pool elevation. This bench should be a minimum of 3 m (10 ft) wide and should have a reverse cross slope.

5. Maintenance Access. Provide an access road with a minimum width of 3 m (10 ft). Provide access such that maintenance personal can access various sections of the wet pond (i.e., spillway, outlet structure, and safety/vegetation ledge). All wet ponds should be designed with a drain pipe which can be opened to drain the pond to facilitate maintenance of the pond and cleaning of the permanent pool. The drain should be capable of draining the entire pond within 24 hours.

6. Fencing. Fencing around wet ponds should be considered when inside side slopes do not meet the criteria mentioned above, safety benches are not constructed within the pond, and the facility is considered a potential hazard. Clear zone requirements and/or guide rail will be necessary regardless if fences are present or not to prevent access to the basin by pedestrians and others. When fences are proposed, they should be located outside of the clear zone for traffic safety considerations.

7. Emergency Spillway. All ponds should be equipped with an emergency spillway. The emergency spillway provides relief in the event the principal spillway becomes blocked or fails. Flows larger than the design event are usually conveyed by the emergency spillway. Spillway side slopes should be no steeper than 1V:3H (3H:1V) with a minimum bottom width of 3 m (10 ft). All spillways should be designed to prevent erosive exit velocities by stabilizing the spillway with suitable vegetated or rock linings. In certain instances the top of the outlet control structure may be used as an emergency spillway. For a case where the top of the principal spillway is used as an emergency spillway, a secondary emergency spillway should be considered for installation in the event the principal spillway outlet fails. Freeboard above the design depth of the spillway, to the top of the basin embankment, should be no less than 150 mm (0.5 ft).

8. Outlet. A principal spillway is the primary outlet of the pond. The principal spillway system includes the riser assembly (outlet structure), the outfall pipe, outfall scour protection, weirs, orifices, maintenance drains, trash racks or any other component necessary for proper stormwater release. Similar to dry detention basins, outlets for wet ponds can be designed in a wide variety of configurations, but most outlets use riser structures constructed from concrete. Although corrugated metal risers are acceptable for short-term operations, because of there susceptibility to damage and corrosion concrete structures are preferred. The primary components of a principal spillway system include:

a. Riser Assembly. Risers should be designed to satisfy the post-construction stormwater management objectives for a project using orifices, weirs, or a combination of these elements set at different elevations on the riser. For example, a small orifice is used to control the water quality volume or provide extended detention and a series of larger orifices or weirs set at different elevations may be used to control the larger events, where rate controls are required. Larger flows may be controlled by allowing stormwater to flow through the top of the riser, using the entire riser diameter to convey the stormwater out of the facility. In some cases, an antivortex design may be necessary. If the smallest orifice is easily clogged from floating

debris, or if heated pond water causes problems downstream during the summer, the outlet can be modified so that it will release water from below the surface of the pond. The outlet structure should be designed to dewater the volume of water above the permanent pool completely within 48 hours after the end of each stormwater event. As part of the design process for the riser assembly a check should be made to assess the impact of the extended dewatering upon two successive 24-hour events and appropriate provisions be made to prevent overtopping of the facility if necessary.

b. Outfall Pipe. An outfall pipe functions to convey flow from the riser assembly to the outfall located downstream of the pond. Outfall pipes should be designed to convey the maximum outflow from the pond. The pipe should be reinforced concrete pipe or corrugated plastic pipe at the size and length dictated by design and necessary to provide the needed conveyance capacity. All pipes should be watertight and properly installed in accordance with Publication 408, *Specifications*. The minimum pipe diameter should be no less than 450 mm (18 in). If the principal spillway is to be used as an emergency spillway the outfall pipe should be sized to convey the peak inflow to the pond.

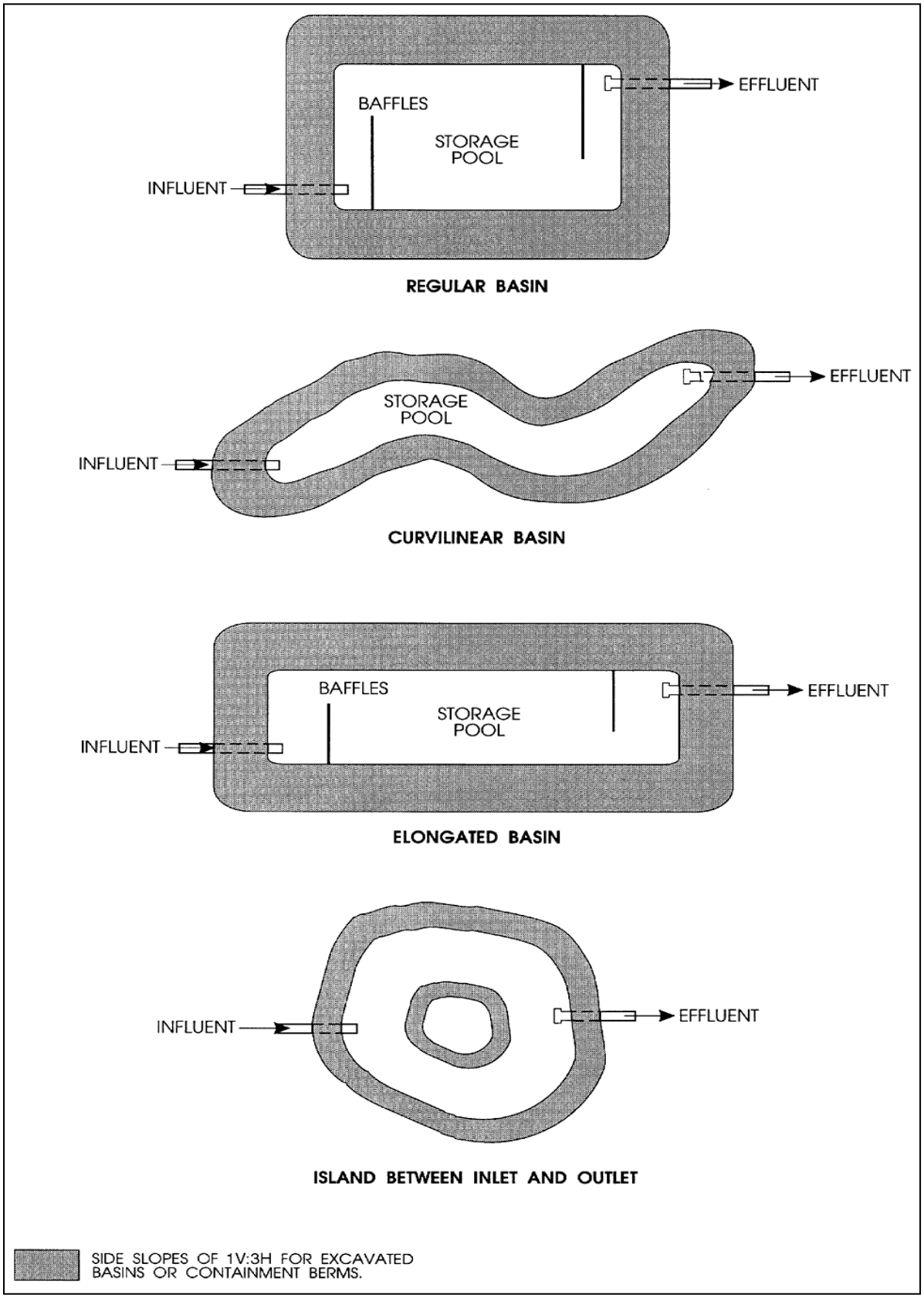
c. Outfall Scour Protection. Scour protection should be provided to prevent scour at the pond outfall, using rock or other means of energy dissipation based upon the maximum rate of discharge through the principal spillway.

d. Weirs and Orifices. The minimum orifice diameter should be no smaller than 75mm (3 in) for clean out and maintenance purposes to prevent the outlet from clogging. Weirs should be designed such that the integrity of the riser structure is not compromised during the life of the outlet structure. To facilitate the construction of these features the dimensions should be set at values which are easily defined and measured, such as whole numbers set in increments of 25 mm (1 in).

e. Trash Racks. Trash racks should be provided as necessary to protect orifices from clogging. Although trash racks are required for the lowest orifice they may also be provided for other orifices if debris is anticipated to be a potential problem. Trash racks should be provided for the large opening in the top of the riser structure to provide for safety and prevent the deposition of debris within outlet control structure, especially if a grate is not provided on the top of the structure and the top of the structure is used to convey the larger events through the pond. The Standards for Roadway Construction, RC 70M, may be referenced for standard drawings concerning trash racks, storage facilities and outlet works.

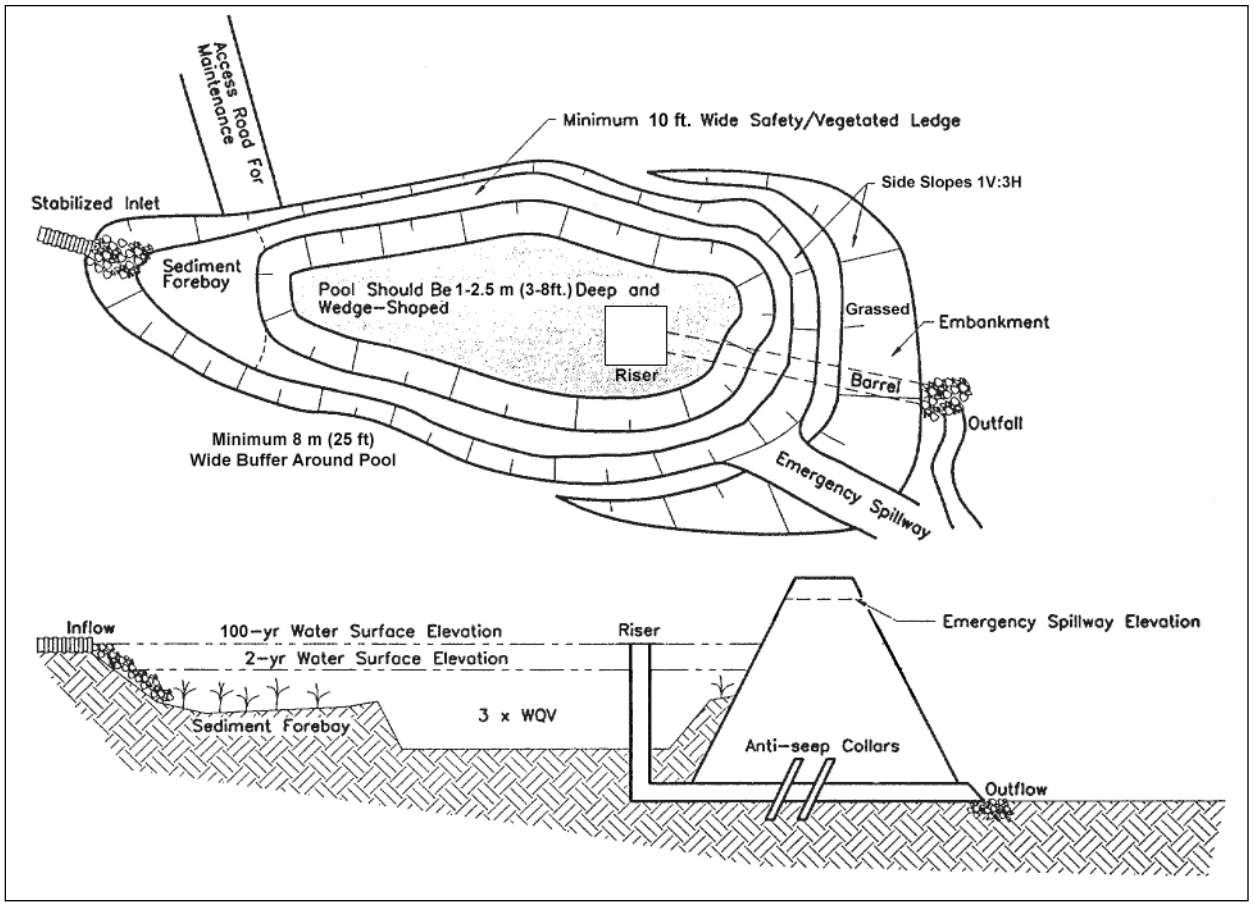
f. Anti-Seep Collars. Provide anti-seep collars along the principal spillway outlet pipe to prevent seepage around the outfall pipe.

Figure 14.31 Example of Methods of Increasing the Length-to-Width Ratio of a Storage Facility



Reference: Schueler, 1987

Figure 14.32 Wet Pond



Reference: Schueler, 1987

B. Design Objectives.

1. Quality. For quality management, the permanent pool should be at least 3 times the WQV for the watershed. The theory behind the function of a wet pond is that incoming runoff displaces old stormwater from the pond, with the new runoff retained until it is displaced by more runoff from the next storm. A permanent pool of 3 times the WQV provides adequate retention time for the removal of non-point source pollutants and maintenance of a healthy aquatic environment. Watershed size, soil conditions and groundwater elevation must be evaluated to ensure the capability of the site to support a permanent wet pond. To enhance pollutant removal, several other BMPs should be added into the flow path of the stormwater before it enters the wet pond to provide pretreatment of the stormwater. At a minimum, all wet ponds should include a sediment forebay at the entrance to the pond, which is separated from the main storage chamber of the pond by an earthen berm or gabion wall to remove sediment before it enters the permanent pool of the pond. The forebay should be sized to accommodate approximately 10 to 15% of the permanent pool volume. Typically, the forebay should be between 1.2 to 1.8 m (4 to 6 ft) deep and fitted with a spillway to convey flow from the forebay into the main chamber of the pond.

Pond depth is essential to maintaining water quality. Ponds that are shallower than the specified limit can allow insect breeding and wind re-suspension of settled particles while those that are deeper may result in thermal stratification and anaerobic conditions in deep water areas. Wet ponds are most effective and are more readily accepted when they contain a curvilinear shape and blend into the existing topography (see Figure 14.31). Other shapes and configurations may be suitable.

2. Quantity. For quantity, the wet pond system should be designed similarly to the dry pond. The pond should be designed to reduce the peak flow for the design event and be able to pass the design flow through the basin without the regular use of the emergency spillway. When performing stormwater management computations to support the design of the pond, the volume of the permanent pool should not be considered part of the available

storage volume as this will inflate the available storage volume and result in erroneous stage storage and outflow data.

14.17 PROTECTIVE TREATMENT

Protective treatment may be required to prevent entry to facilities that present a hazard to children and, to a lesser extent, the public in general. Fences may be required for storage areas where one or more of the following conditions exist:

- Occurrence of rapid-stage increases and the configuration of facility geometrics which make escape difficult, especially near areas where small children frequently visit such as playgrounds.
- Existence of storage depths that either exceed 0.9 m (3 ft) for more than 24 hours or are permanently wet and have side slopes steeper than 1V:4H (4H:1V).
- Presence of a low-flow watercourse or ditch traversing a storage area with a depth greater than 1.5 m (5 ft) or a flow velocity greater than 1.5 m/s (5 ft/s).
- Existence of side slopes equal to or steeper than 1V:1.5H (1.5H:1V).

Guards or grates may be appropriate for other conditions but, in all circumstances, heavy debris should be kept from entering the detention area. In some cases, it may be advisable to fence the watercourse or ditch rather than the entire storage area.

Fencing should be considered for all retention areas with design depths in excess of 0.9 m (3 ft) for 24 hours, unless the area is within a fenced, limited-access facility.

14.18 MAINTENANCE

Regular maintenance is an essential part of preserving function of an SCM. Poorly maintained SCMs often function less efficiently and may cause more problems than they were intended to resolve. It is important to consider maintenance issues when selecting, locating, and designing an SCM. Less maintenance intensive SCMs, as indicated by "low" or "medium" in the Routine Maintenance Effort column in Table 14.1, are more desirable because they reduce the long-term burden on maintenance forces. In general, an SCM should always be accessible from the legal right-of-way. Access pathways and gates should be wide enough to accommodate maintenance equipment and vehicles. All features within the SCM, such as the flow control structure, should be accessible during both wet and dry conditions. Finally, the SCM should be easy to identify from the surface. If the SCM is primarily underground, signs or delineators should be installed to alert maintenance. Guidance on specific inspection and maintenance procedures for Department SCMs can be found in Publication 888, *Stormwater Control Measures Maintenance Manual*.

14.19 SOIL CHARACTERIZATION AND INFILTRATION TESTING

A. Introduction.

Managing the net change in runoff volume from pre- to post-construction conditions is one of the primary PCSM objectives. The PCSM Plan preparer is expected to:

- Conduct a pre-development site characterization to identify potential infiltration areas on the site.
- Determine the saturated hydraulic conductivity rate for infiltration SCM design.
- Perform post-construction testing for confirm infiltration capacity.

As noted in Section 13.7.J of Publication 13M (DM-2), pre-development site characterization for identifying potential infiltration areas involves two steps: (1) evaluating existing features and (2) preliminary infiltration testing. These steps must be done early in the design process to identify areas suitable or unsuitable for infiltration SCMs, and it should be conducted by a professional(s) experienced in soil classification, geology, and the design

of infiltration SCMs. The requirements for soil characterization and infiltration testing are described further in this section.

Soil characterization and infiltration testing for final SCM designs should generally be conducted by (or under the direct supervision of) either a Certified Professional Soil Scientist; a professional member of the Pennsylvania Association of Professional Soil Scientists; a licensed Professional Geologist with professional experience and education in soil science; or a professional engineer (PE) with experience in soils and infiltration in Pennsylvania.

B. Soil Characterization.

Soil characterization testing and visual assessment of the soil profiles is needed to identify soil structure and limiting layers. It is highly recommended that soil samples be obtained, and laboratory soil classification should be performed, or at a minimum identified via field methods if deemed appropriate by the professional. Soil characterization testing (and soil sampling) are usually performed in conjunction with in-situ infiltration testing. These tests can be performed at the same time as infiltration testing and can be supplemented with recent boring logs.

Characterization of subsurface soils is needed to provide information on the underlying site conditions and investigate the presence of limiting layers below the infiltration interface. Soil excavations should be performed to depths below the proposed infiltration interface (e.g., SCM bottom elevation). The testing professional must document the presence of any limiting layers, seasonal high water table (SHWT), and soil characteristics. The presence of limiting zones within 2 vertical feet of the infiltration interface generally prohibits the use of infiltration SCMs. Limiting zones are geological features in a soil profile that generally inhibit infiltration. These features include redoximorphic features, compacted fill, fragipans, regularly occurring or perched SHWT, and bedrock. Impermeable soils (hardpan) are not considered a limiting zone if permeable soils exist below them.

For SCMs utilizing the Managed Release Concept (MRC) (see Section 14.5.O), limiting zones should not exist within one vertical foot of the infiltration surface. Use of MRC SCMs should be limited to soils in which the saturated hydraulic conductivity is 0.25 inch/hour or less.

In karst areas, a soil mantle of at least 3 feet below the proposed infiltration interface and above bedrock should be maintained, unless a licensed geotechnical professional or professional geologist with experience in karst geology indicates that less soil poses no risk. The presence of SHWT within 2 vertical feet of the infiltration interface could cause groundwater mounding.

Exploratory test pits (test pits) are the preferred method for soil characterization in the field. Test pits provide a greater understanding of soil structure that can be easily missed using soil borings and they are often needed to conduct infiltration testing. Soil borings can be used when conditions render the excavation of test pits impractical, such as when the proposed SCM footprint is in an area with existing structures or utilities present, or the depth to the infiltration interface does not allow for a benched excavation with existing site constraints.

Key considerations for test pits include the following:

- For each infiltration SCM, at least one test pit should be excavated to a minimum depth of 2 feet below the proposed infiltration interface of the SCM (i.e., the SCM bottom elevation), or until bedrock or fully saturated conditions are encountered if less than 4 feet.
- When conditions prevent the excavation of test pits to the minimum depth, soil borings, in addition to the excavated test pits, should be used to provide soil classification down to the required depths.
- Test pits must be excavated in accordance with Occupational Safety and Health Administration (OSHA) Regulations.

Key considerations for soil borings are as follows:

- Soil borings should be advanced to a depth of 2 feet below the SCM bottom elevation or until auger refusal with continuous split spoon sampling.
- Locate no more than 25 feet away from the infiltration test locations.

- Drilling and sampling procedures should be in accordance with the American Society of Testing and Materials (ASTM) D6151 with a minimum 3-inch inner tube diameter.
- Standard Penetration Tests (SPT), if performed, should be in accordance with ASTM D1586 when prior compaction is a concern.

Soil sampling is necessary to complete soil characterization testing and should be performed in conjunction with infiltration testing. Soil sampling should meet the following minimum criteria.

- At least 1 soil sample should be taken at an elevation within 1 vertical foot of the infiltration interface at the location of an infiltration test.
- All soil samples should undergo visual identification via ASTM D2488. Soil structure for each sample should be identified.
- It is highly recommended that soil samples undergo laboratory particle size analysis to determine USDA textural class.
- Split spoon sampling, if performed, should be completed in accordance with ASTM D1586.

The need for soil testing for density or compaction both before and after site disturbance should be considered by the professional.

C. Infiltration Testing.

Infiltration tests are required for both the preliminary site evaluation for infiltration capabilities and for final design of each infiltrating SCM. Underlying soils with saturated hydraulic conductivity rates less than or equal to 0.2 inch/hour are considered limited (Lee et al., 2016). Often, capped back-up underdrains are recommended for marginal soils, which are generally considered between 0.2 and 0.5 inch/hour. If infiltration is demonstrated to be limited, the PCSM Plan preparer may propose the use of Managed Release Concept (MRC) SCMs. Refer to DEP's guidelines (linked in their Alternative E&S and PCSM BMPs document) on the design of MRC.

Acceptable infiltration testing methods include double ring infiltrometer and single ring infiltrometer tests. The cased borehole test is an acceptable infiltration testing methodology if done in conjunction with soil borings. Percolation tests are not acceptable procedures for measuring saturated hydraulic conductivity.

Hydraulic conductivity and infiltration rate are the two terms most often used to describe soil water movement. Infiltration is the rate at which water enters the soil. Hydraulic conductivity is the rate at which water moves through the soil profile. In general, the final infiltration rate is used for SCM design. In the field, final infiltration is determined to have been reached when the difference between successive infiltration measurements is minimal. Final infiltration rate is approximately equivalent to saturated hydraulic conductivity.

Preliminary

The goal of Preliminary Infiltration Testing is to identify locations suitable for infiltration SCMs. At a minimum, one infiltration test for every 40,000 square feet outside of the roadway cross-section and within the project site boundary should be performed with a minimum of four tests, equally distributed across a site. Infiltration tests must be done at a variety of locations and depths that can be demonstrated as both representative of the existing site and reasonable for the project. Soil profile data or a soil characterization test pits/bore holes within 100 feet of all infiltration tests are needed to demonstrate that infiltration testing was performed in infiltration-conducive soil horizons.

The minimum number of infiltration tests can be reduced by 50% if it can be demonstrated and certified by a licensed professional experienced in soil classification, geology, and the design of infiltration SCMs that subsurface conditions are expected to be uniform with similar infiltration rates. The total number of infiltration tests, and the distribution of the tests across the site can be altered (either reduced or increased) if it can be demonstrated to the satisfaction of DEP that a sound reason exists to change the recommendations for predevelopment infiltration testing.

Final Design

For final design of infiltration SCMs, refer to Table 14.7 for the number of infiltration tests considered adequate for each infiltration testing method.

Table 14.7: Minimum Number of Infiltration Tests for Final Design of Infiltration SCMs

Criteria (use more conservative)	Number of Infiltration Tests Per Test Type		
	Double Ring	Single Ring	Cased Borehole
Each SCM	3	5	5
SCM surface area	1 per 4,350 ft^2	1 per 2,900 ft^2	1 per 2,900 ft^2

The minimum number of infiltration tests specified in Table 14.7 should be performed within 100 feet of the SCM. At least one test must be within 25 feet of the final location of the SCM and within 1 vertical foot of the proposed infiltration interface (SCM bottom elevation).

More infiltration tests may be warranted if the results of the closest three tests vary by an order of magnitude. The same test may be used for two different infiltration SCMs if the test meets the requirements for both SCMs. Follow-up testing may be necessary if the location or elevation of any SCM changes to the degree that the infiltration testing previously performed no longer meets proximity, elevation, and density (i.e., number of tests per area) requirements.

In the case of an infiltrating SCM previously used as a temporary E&S sediment basin or trap, infiltration testing must be performed on the SCM infiltrating surface after sediment removal to confirm saturated hydraulic conductivity rates.

14.20 PCSM PLAN

A PCSM Plan is required for all Department projects that require a Chapter 102 permit and SCMs are implemented. In accordance with DM-3 Chapter 6, the PCSM Plan shall be handled as an "Also Included" Plan. Note that on projects that do not require a Chapter 102 permit, any information for SCMs included in the project may be incorporated into the Construction Plan - a separate PCSM Plan is not required. All standards for a Department PCSM Plan are contained in DM-3 Chapter 6 and Chapter 15.

Each PCSM Plan must be accompanied by supporting project design information. PA DEP's Chapter 102 permit application package includes modules for PCSM, antidegradation, and riparian buffers. Any additional information or calculations that are required should be organized, appropriately labeled, and attached to the permit. Additional information may include design standards and assumptions, design calculations, drainage area maps, and any other relevant design information.

14.21 PCSM ON NON-CHAPTER 102 PERMIT PROJECTS

A PCSM analysis involving quantitative runoff calculations and SCM design is typically only required for projects with a permit pursuant to 25 PA Code Chapter 102 (Chapter 102) permit or Act 167 SMP requirements (refer to 13.7.D of Pub. 13M). However, other PA DEP permits also require compliance with Chapter 102. For example, all water obstruction and encroachment permits authorized under 25 PA Code Chapter 105 require compliance with Chapter 102. As described in Section 13.7.M of Pub. 13M, the types of projects that fall into this category (no Chapter 102 permit but need a Chapter 105 permit) typically result in minimal to no change in stormwater runoff that reaches a surface water. Chapter 102 requirements can be satisfied by including a narrative in the report for the ESPC Plan that describes how the items in Section 102.8.b of Chapter 102 have been considered. Examples of how these items can be addressed are as follows.

1. Preserve the integrity of stream channels and maintain and protect the physical, biological and chemical qualities of the receiving stream. This is the goal of PCSM in Chapter 102, and implementing the other items that follow in this list to the extent practicable is the recommended approach to achieving this goal. As long as the other items are addressed in accordance with this guidance, a statement such as this can be included:

The project is designed to ensure that the integrity of the stream channel is preserved and the physical, biological and chemical qualities are protected.

2. Prevent an increase in the rate of stormwater runoff. If a project does not significantly increase impervious land coverage within the right of way and does not substantially alter drainage patterns, the peak rate of runoff from the project should be unaffected. For example, widening bridge approaches to transition the roadway into a wider bridge does not affect the peak rate of runoff from the project. This item can be addressed by stating:

The peak rate of runoff to the receiving surface water(s) will not increase as a result of the project.

3. Minimize any increase in stormwater runoff volume. Certain construction activities and roadway features will generally result in an increase in runoff volume from the improved areas (compared to the pre-project conditions). Examples are a net increase in impervious area, removal of trees and mature vegetation, conversion of sheet flow over vegetated areas to concentrated flow, soil compaction, and topsoil stripping. In small amounts, these can be assumed to have minimal effects on runoff volume, and a general statement similar to the following can be included in the narrative:

The proposed project activities, including temporary construction activities and permanent roadway improvements, cumulatively result in minimal or no net increase in runoff volume from disturbed areas.

If this conclusion cannot be made, SCMs should be incorporated into the design to the extent that is necessary to minimize or eliminate increases in runoff volume. In this case, a statement should be included:

The net increase in stormwater runoff volume has been minimized and managed through the implementation of SCMs, specifically: [insert SCMs].

Examples of SCMs that are well-suited for small reductions in runoff volume along highways are: vegetated swale with check dams, vegetated filter strip with soil restoration, conversion of channel flow to sheet flow, and retentive grading.

4. Minimize impervious areas. PennDOT policies dictate minimum roadway and bridge widths, which are based on driver and pedestrian safety. Widening of roadway and bridge cross sections commonly occurs on reconstruction projects, as highway facilities are brought up to current safety standards and capacity is added to accommodate population growth. As a matter of practice, PennDOT minimizes impervious area creation by not arbitrarily over-widening roads and bridges. Additionally, abandoned paved areas should be restored to vegetative cover. In the PCSM narrative, any impervious area that is converted into pervious area should be identified, and a statement similar to the following should be included:

Roadway and bridge widths are consistent with PennDOT's design standards, which balance the public health, safety and welfare needs of the traveling public with the desire to protect the Commonwealth's environmental resources.

5. Maximize the protection of existing drainage features and existing vegetation. Natural drainageways that are not surface waters (e.g., ephemeral channels) should be protected to the extent practicable when present. Where engineered vegetated drainage features are present, such as roadside swales and diversion ditches, they should continue to be utilized in lieu of paved or piped conveyances to the extent practicable. Scour protection and/or energy dissipation should be provided at storm sewer outfalls, as necessary, to prevent channel erosion. The following statement and a description of any protected drainageways and preserved vegetated drainage features should be included in the narrative:

Natural drainageways and existing vegetation have been protected to the extent practicable.

6. Minimize land clearing and grading. Clearing of trees and mature vegetation should be done only to the extent needed to construct the proposed highway improvements. Excess clearing can be prevented by delineating the limits of disturbance on the ESPC Plan and instructing the contractor to mark the limits with flagging prior to earthmoving activities. Trees located within the limit of disturbance that do not interfere with construction activities should be preserved and protected during construction. Grading operations should also be contained within the

limits of disturbance, and temporary grading should be restored and revegetated prior to project completion. In general, a statement to this effect can be included in the narrative:

Land clearing and grading has been minimized to the extent practicable.

7. Minimize soil compaction. Unintentional (or incidental) soil compaction occurs in areas where construction vehicles and materials are staged or used for access. Soil compaction can be minimized by identifying the areas that are likely to be compacted and specifying soil treatment as part of the final site stabilization sequence. These areas should be included within the limits of disturbance on the ESPC Plan. The following may be stated in the narrative:

Unpaved areas within the project limits that experience frequent loading (vehicles or materials) will be treated in the final site stabilization process. Soils will be loosened and the appropriate treatment, seed formula, and protection will be applied in dry conditions.

8. Utilize other structural or nonstructural BMPs that prevent or minimize changes in stormwater runoff. Other SCMs may be considered after the previously noted SCMs have been applied to the extent practicable and there remains a concern about potential impacts to the receiving surface water. If a project does require SCMs to manage the post-construction runoff, then a separate PCSM Plan must be prepared (102.8.d). The PCSM Plan shall be consistent with Pub. 14M (DM-3) and Pub. 13M (DM-2), as amended and updated.

14.22 DEFINITIONS

1. Antidegradation Best Available Combination of Technologies (ABACTs). Defined in 25 PA Code Chapter 102. Environmentally sound and cost effective treatment, land disposal, pollution prevention and stormwater reuse BMPs that individually or collectively manage the difference in the net change in stormwater volume, rate, and quality for storm events up to and including the 2-year/24-hour storm when compared to the stormwater rate, volume and quality prior to the earth disturbance activities to maintain and protect the existing quality of receiving surface waters.

2. Best Management Practices (BMPs). Defined in 25 PA Code Chapter 102. Activities, facilities, measures, planning or procedures used to minimize accelerated erosion and sedimentation and manage stormwater to protect, maintain, reclaim, and restore the quality of waters and the existing and designated uses of waters within this Commonwealth before, during, and after earth disturbance activities.

3. Combined Sewer Systems (CSSs). Defined by 25 PA Code Chapter 92a. A sewer system that has been designed to serve as both a sanitary sewer and a storm sewer.

4. Degradation. For HQ and EV watersheds, degradation is an adverse effect that results in a negative change in the existing water quality of the receiving surface water. For non-HQ and non-EV watersheds, it is a negative change in the existing or designated in-steam water use or the level of water quality necessary to protect the use.

5. Disturbed area. Defined in 25 PA Code Chapter 102. Unstabilized land area where an earth disturbance activity is occurring or has occurred.

6. Ephemeral Stream. Defined in 25 PA Code Chapter 89. A water conveyance which lacks substrates associated with flowing waters and flows only in direct response to precipitation in the immediate watershed or in response to melting snowpack and which is always above the local water table.

7. Evapotranspiration (ET). The sum of evaporation and plant transpiration of water. Evapotranspiration accounts for a significant portion of the rainfall that is lost (not returned to streams via surface runoff) in Pennsylvania watersheds. The amount of evapotranspiration is influenced mostly by the types of vegetation and land use in a watershed. Because water transpired through leaves comes from the roots, plants with deep reaching roots can more constantly transpire water. Thus, herbaceous plants transpire less than woody plants because herbaceous plants usually lack a deep taproot. Also, woody plants keep their structure over long winters while herbaceous plants must grow up from seed in the spring in seasonal climates, and will contribute almost nothing to evapotranspiration in the spring.

8. Impaired Stream. A surface water that does not meet the water quality criteria for its designated or existing use under 25 PA Code Chapter 93. Streams are examined for four main uses: water supply, aquatic life, recreation, and fish consumption. PA DEP monitors surface waters for biology, chemistry, and physical habitat using methods outlined in the approved 2015 Assessment Methodology. Impaired waters are listed in Categories 4 and 5 of Pennsylvania's Integrated Water Quality Monitoring and Assessment Report.

9. Impervious Area. A surface that is compacted or covered with a layer of material so that it is resistant to infiltration by water, including but not limited to structures and buildings; non-porous asphalt and concrete; gravel roads, driveways and parking areas; land areas covered with significantly compacted and unvegetated soil such as dirt roads, driveways and parking/storage areas; and other surfaces routinely used for vehicle parking and movement.

10. Infiltration. The process by which surface water penetrates through the ground surface into the soil. The soil texture and structure, vegetation types and cover, water content of the soil, soil temperature, and rainfall intensity all play a role in controlling infiltration rate and capacity. For example, coarse-grained sandy soils have large spaces between each grain and allow water to infiltrate quickly. Vegetation creates more porous soils by both protecting the soil from pounding rainfall, which can close natural gaps between soil particles, and loosening soil through root action. This is why forested areas have the highest infiltration rates of any vegetative types.

11. Long Term Control Plan. A plan developed by municipalities and/or municipal authorities designed to mitigate the impact of combined sewer system discharges and meet water quality standards.

12. Municipal Separate Storm Sewer System (MS4). Defined by 25 PA Code Chapter 92a (and paraphrased here). A separate storm sewer (including roads with drainage systems, municipal streets, catch basins, curbs, gutters, ditches, manmade channels or storm drains) which is owned or operated by a public body (created by or under State law) having jurisdiction over disposal of sewage, industrial wastes, stormwater or other wastes; that discharges to surface waters of this Commonwealth; is designed or used for collecting or conveying stormwater; is not a combined sewer; and is not part of a publicly owned treatment works.

13. Net Change. Refers to the change from pre-development to post-development conditions.

14. Non-discharge Alternative. Defined in 25 PA Code Chapter 102. Environmentally sound and cost-effective BMPs that individually or collectively eliminate the net change in stormwater volume, rate and quality for storm events up to and including the 2-year/24-hour storm when compared to the stormwater rate, volume and quality prior to the earth disturbance activities to maintain and protect the existing quality of the receiving surface waters of this Commonwealth.

15. Pre-development. Refers to runoff condition that exists onsite immediately before the planned project occurs. Pre-development is not intended to be interpreted as the period before any human-induced land disturbance activity has occurred.

16. Post Construction Stormwater Management (PCSM). Defined in 25 PA Code Chapter 102. Management of stormwater associated with a project site after the earth disturbance activity has been completed and the project site is permanently stabilized. The term "post-construction" is used to differentiate PCSM from discharges during construction. Erosion and sediment pollution control (ESPC) plans are required for most construction projects to show that runoff from disturbed areas during construction is properly managed. PCSM deals with runoff from the project after the earth disturbance is completed and the site has been stabilized.

17. Post-development. Refers to runoff condition that exists onsite after the earth disturbance activity has been completed and the project site is permanently stabilized.

18. Riparian Buffer. A vegetated area that plays a key role in maintaining water quality and providing nutrients in associated streams, rivers and lakes, thus providing economic and environmental benefits. It is simply defined as "A BMP that is an area of permanent vegetation along surface waters," in 25 PA Code Chapter 102.

19. Riparian Forest Buffer. Defined in 25 PA Code Chapter 102. A type of riparian buffer that consists of permanent vegetation that is predominantly native trees, shrubs and forbs along surface waters that is maintained

in a natural state or sustainably managed to protect and enhance water quality, stabilize stream channels and banks, and separate land use activities from surface waters.

20. Road Maintenance Activity (RMA). Earth disturbance activities within the existing road cross-section, as defined in 25 PA Code Chapter 102. Guidance on the types of work that meet the definition can be found in Pub. 584, Chapter 12, Appendix E.

21. Special Protection Waters. Antidegradation requirements in Pennsylvania provide protection to waterbodies in discrete levels or tiers based on their existing uses, level of water quality, and environmental characteristics. The standard level of protection given to all waterbodies is to protect and maintain their designated and existing uses. A special level of protection is afforded to waterbodies with High Quality and Exceptional Value existing or designated uses, which is to protect existing water quality. These waterbodies are often referred to as Special Protection Waters.

22. Stormwater Control Measure (SCM). Stormwater Control Measure (SCM). A technique, measure, or structural control that is used for a given set of conditions to manage stormwater runoff quantity, minimize erosive conditions, and improve effluent quality in a cost-effective manner.

23. Surface Waters. Defined in 25 PA Code Chapter 102. Perennial and intermittent streams, rivers, lakes, reservoirs, ponds, wetlands, springs, natural seeps, and estuaries, excluding water at facilities approved for wastewater treatment such as wastewater treatment impoundments, cooling water ponds, and constructed wetlands used as part of a wastewater treatment process.

24. Thermal Impact. Defined in 25 PA Code Chapter 96. The threshold for a thermal impact to the water quality of a receiving surface water is more than two degrees Fahrenheit. change during a one-hour period in mean water temperature. The water quality criteria do not preclude the allowance of a reasonable mixing zone if there is no significant effect on the ambient temperature of the stream outside the mixing zone.

25. Total Maximum Daily Load (TMDL). Defined by 25 PA Code Chapter 96. The sum of individual waste load allocations for point sources, load allocations for nonpoint sources and natural quality and a margin of safety expressed in terms of mass per time, toxicity or other appropriate measures. It is the amount of pollutant loading that a waterbody can assimilate and meet water quality standards. The TMDL process is a planning tool to develop pollution reduction goals that will improve impaired waters to meet water quality standards.

14.23 CHAPTER 14 NOMENCLATURE

Symbol	Definition	Units
A	Cross-sectional area or surface area	m^2 or ft^2
$A_{1,2}$	Surface area at Elevations 1 and 2 respectively for a stage-storage curve	m^2 or ft^2
C	Weir coefficient	dimensionless
C	Discharge coefficient	dimensionless
d	Change in water surface elevation	m or ft
d	Depth of infiltration basin	m or ft
D	Depth of basin	m or ft
D	Pipe diameter	m or ft (cm or in)
f	Infiltration rate	mm/hr or in/hr
g	Acceleration due to gravity	m/s^2 or ft/s^2
H	Head on a structure	m or ft
H	Head above weir crest	m or ft
H_1	Upstream head above crest	m or ft
H_2	Downstream head above crest	m or ft
H_c	Height of weir crest above channel bottom	m or ft
I_i, I_{i+1}	Inflow rate at beginning, end of time interval	m^3/s or cfs
L	Length	m or ft
L	Horizontal weir length	m or ft

O_i, O_{i+1}	Outflow rate at beginning, end of time interval	m^3/s or cfs
Q	Discharge, flow	m^3/s or cfs
Q_f	Free flow	m^3/s or cfs
Q_i	Peak inflow rate	m^3/s or cfs
Q_o	Peak outflow rate	m^3/s or cfs
Q_s	Submergence flow	m^3/s or cfs
S_a	Surface area	m^2 or ft^2
S_i, S_{i+1}	Storage volume at beginning, end of time interval	m^3 or ft^3
T_i	Duration of basin inflow	seconds
T_s	Storage time or detention time	hours
t_b	Time base of the inflow hydrograph	hours
t_p	Time to peak of the inflow hydrograph	hours
V	Volume of basin	m^3 or ft^3
Vol_s	Storage volume of infiltration basin	m^3 or ft^3
V_r	Void ratio	dimensionless
V_S	Storage volume	m^3 or ft^3
V_1, V_2	Storage volume between Elevations 1 and 2 for a stage-storage curve	m^3 or ft^3
W	Width	m or ft
Z	Side slope factor, ratio of horizontal to vertical	dimensionless
θ	Angle of V-notch weir	degrees
Δt	Routing time period for basin routing	seconds

14.24 REFERENCES

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CHAPTER 19

DESIGN-RELATED CONSTRUCTION CONSIDERATIONS

19.0 OVERVIEW

In the completion of PennDOT's transportation improvement projects, different personnel perform the design and construction functions. To successfully and efficiently complete a project, adequate communication between the two segments of the project team is essential. Design personnel should be aware that there are construction-related design considerations, and construction personnel should be aware that there are design-related construction considerations. Thus, it is very important to have good communication between design and construction personnel.

When problems arise, it is crucial that the issues be discussed in a timely manner, and a workable solution decided upon. To facilitate open and unhindered communication between the design and construction portions of the project, a forum for design-construction related discussion sessions should be provided on a regular basis. Such a forum helps to reduce or eliminate redundancies and adherence to archaic methodologies by developing the most viable solution(s) based upon the expertise of both types of personnel. There are different avenues and forums for the design and construction personnel to discuss design and construction issues. Depending on the type of project these sessions should begin at the pre-bid stage or at the latest the preconstruction stage and continue through the final inspection conference. Regardless of when and how often meetings occur between the design team and construction personnel, the familiarity of design personnel with PennDOT publications such as Publication 13M, Design Manual, Part 2, *Highway Design*; Publication 15M, Design Manual, Part 4, *Structures*; Publication 72M, *Roadway Construction Standards*; Publication 408, *Specifications*; and conformance with PennDOT's drainage policies can help reduce the number and complexity of questions during construction.

Open communication between design and construction personnel is essential and eventually results in good working relationships. Ultimately these relationships help to improve designs and ensure that projects are constructed as envisioned without major problems or field revisions.

19.1 DRAINAGE DESIGN AND CONSTRUCTION

A. Relationship of Construction to Design. Construction-related hydrologic and hydraulic considerations are a necessary part of the planning and design phases of a project. Factors that may affect construction timing and methods need to be considered as project development proceeds and need to be supported by appropriate design computations. A few common drainage-related factors that may be encountered and affect construction, for consideration during design include:

- Seasonal fluctuations in flow.
- Variable water surface elevations.
- Groundwater elevations.
- Environmental constraints (i.e., seasonal limitations on in-stream construction due to fish migration).
- Construction staging.
- Availability of materials.
- Construction easements.
- Construction access.
- Maintenance and protection of traffic.
- Constructability.

Any special or unique construction requirements should be communicated to the designer prior to the final design phase of the project. Correspondingly, special instructions to the contractor are to be included in one of the project's Special Provisions. The designer should be prepared to explain important aspects of the design during the preconstruction conference and meetings with the construction team. Before designing or specifying any unique and complex component of a project, the designer should consult with the District's Maintenance Unit to ascertain if there are any special considerations that need to be accommodated for in the design in order to maintain the proposed improvement. It is important that the designer obtain feedback from individuals familiar with typical

construction practices and methods in order to understand the impact of any special provision or complex component of the project upon the project construction schedule. Department personnel responsible for contract administration and construction may need to coordinate their scheduling and construction procedures with the designer to achieve the intended results.

B. Cost Considerations. Cost is an important consideration in any design. A large portion of the initial project costs are related to materials and construction. However, future maintenance and replacement costs are a very important aspect of the overall project cost. For a project to be effective, an appropriate balance between material, construction, maintenance, and replacement costs is necessary.

Ordinarily, material costs are optimized by using readily available materials in a consistent manner, recycling materials, researching programs to identify new construction materials and how they may be utilized efficiently, using reasonable safety factors in design and encouraging and allowing the use of alternatives wherever possible. In some cases, the least expensive material may not always provide the best value, especially when considering the project costs over the service life of the project. This is because even though certain materials and installation costs may be higher than their less expensive alternatives, certain materials may offer a cost advantage in the areas of future maintenance and replacement that override the initial material cost advantage. For example, geosynthetics may be an advantageous alternative to granular filter materials in some situations.

Typically, construction costs are affected by the following items:

- Relative difficulty of construction.
- Laws, rules and/or regulations governing construction procedures.
- Degree of competition among contractors.
- Construction latitude allowed by the specifications.
- Degree of use of standardized construction details and specifications.
- Quality of the construction plans.
- Degree of supervision and inspection provided by PennDOT and regulatory agencies.

The choice of certain construction procedures may be of more value than less complicated and less expensive counterparts if they allow for the use of more economical materials, minimize maintenance costs and eliminate or reduce the need for replacements.

C. Environmental Considerations. The designer should work with other disciplines as necessary to devise and construct mitigation measures that protect and reduce the adverse effects of the project upon sensitive environmental features near the project. The designer is to consider, evaluate and propose locations and sizes for hydraulic facilities (e.g., culverts, bridges, channels), spoil disposal areas, geometry, construction staging and various construction alternatives for all elements of the design. Each element is to be developed in an effective context sensitive solution that also adequately addresses the project's environmental needs and commitments. The designer may also assist in developing programs for protecting surface waters during construction which include but are not limited to:

- Levees and ponds to collect various types and quantities of pollutants including those from construction equipment or which are accidentally spilled.
- Methods to reduce erosion and sedimentation.
- Means of replicating the natural hydrologic regime and hydraulic response of surface waters that are affected by construction.

More information on these programs can be found in Chapter 3, *Policy*, Chapter 11, *Surface Water Environment*; Chapter 12, *Erosion and Sediment Pollution Control;* and Chapter 14, *Post-Construction Stormwater Management*.

D. Water Quality. Water quality of streams and lakes is a very sensitive issue. Construction of drainage systems and non-drainage appurtenances alike may deliver such things as sediment and chemical constituents associated with construction activities to streams, rivers and lakes unless precautions are taken. Precipitation gauges and streamflow data indicate that very low stream flows occur periodically throughout the year. During these periods, even small amounts of additional sediment or chemicals entering the stream from construction areas could be detrimental because of the low dilution effect provided by the receiving waters. Although it is important to control

the inadvertent transport of material from the construction site during normal conditions, it is crucial to control the inadvertent transport of sediment and chemicals from the construction site during times of very low flow. Generally, the more environmentally sensitive a site or receiving water body is the more safety measures and precautions are needed to control the inadvertent transport of chemicals and materials from the site. The effects of sediment or chemicals from highway construction during the low-flow periods should be investigated for sensitive areas (e.g., where stream flow is used for municipal water supply, areas with threatened or endangered species). This investigation may include periods of water quality monitoring and testing. If the investigation concludes that the amount of sediment or chemicals may exceed an acceptable threshold value, the construction periods may have to be adjusted or additional mitigation measures employed to protect the sensitive features. In some extreme cases, water quality testing or monitoring may be necessary to ensure the protection of such sites, especially if the groundwater or water from the stream is being removed for construction purposes.

Normally any work in or around surface waters requires regulatory permits before the work can be performed. Resource management agencies, such as the Pennsylvania Department of Environmental Protection, Pennsylvania Fish and Boat Commission, Pennsylvania Department of Conservation and Natural Resources, Pennsylvania Game Commission, U.S. Army Corps of Engineers and the Federal Emergency Management Agency should be consulted early and often in the design phase of a project whenever work is proposed within a floodplain or channel. PennDOT has developed a series of publications related to various aspects of environmental approvals and clearances. For further information and guidance on water quality and the environmental clearance process, refer to these publications:

- Publication 10B, Design Manual, Part 1B, Chapter 3, *Categorical Exclusion Evaluations*;
- Publication 10B, Design Manual, Part 1B, Chapter 4, *Environment Assessments*;
- Publication 10B, Design Manual, Part 1B, Chapter 5, *Environmental Impact Statements*;
- Publication 13M, Design Manual, Part 2, *Highway Design*;
- Publication 325, *Wetland Resources Handbook*; and
- Publication 349, *Section 4(f) Handbook, Volumes 1 & 2*.
- Publication 689, *Transportation Project Development Process: Cultural Resource Handbook*.

E. Effects of Changes. Problems may be avoided during construction when important drainage or other water-related factors are adequately considered during the Engineering and Environmental Scoping Phase of the project. To better understanding a proposed project's potential scope of work it is often desirable and helpful to include input from county maintenance personnel in the scoping phase of a project as they may have first-hand knowledge of existing drainage and environmental issues associated with a particular site. Whenever possible, known problem areas should first be avoided and only if avoidance is not possible then minimization or mitigation measures should be considered. A site may be considered a problem location for any number of reasons including adverse geological features, environmental concerns, sensitive existing facilities or other issues that might conflict with the proposed project. By avoiding the problem the project designer can reduce the need for any future changes to the proposed or temporary condition when things change and field parameters are different than those used in the design.

The design process is an iterative process that requires input from many different engineering disciplines to efficiently and effectively develop an appropriate drainage or hydrologic and hydraulic design that is acceptable to the regulatory agencies issuing permits for projects. Throughout the design process, the project design should be periodically checked to verify that site conditions are consistent with actual field conditions. Channel meander, migration, bank caving, aggradation, headcutting or other natural or man-induced changes in channels can occur rapidly and can significantly affect the final design of hydraulic structures. Development within a watershed, grading changes and the alteration of land cover and existing drainage infrastructure can all impact stormwater and cause a needed change in a proposed design. If an extended period of time occurs between the project survey and the design, or the design and construction, the designer may need to reconsider design decisions that were based on field conditions that no longer exist. Being aware of the potential for change and the location of potential changes can help the designer develop a project that is easier to construct and which requires less field changes to achieve the intended objective of the drainage design.

19.2 DESIGN PROCEDURES / PROCESSES RELATED TO CONSTRUCTION

A. Construction Plans. The designer is to be aware of the relationship of the drainage design to other aspects of the project in order to fully understand how these elements affect construction costs and to effectively evaluate their intended performance and impact upon the design. Construction plans prepared by the designer should reflect these considerations by containing:

- Construction sequences that consider construction costs, environmental considerations and public convenience.
- Subsurface soil borings.
- Detour plans.
- Complete descriptions of utilities.
- Consistent plan format that enhances the contractor's ability to assimilate and understand PennDOT's plans and contract documents.

Despite the most careful efforts, construction changes are occasionally needed that deviate from the information shown on the construction plans. Regardless of the size or location of these modifications, the contractor needs to consult with the PennDOT Project Manager, and if deemed necessary by the Project Manager, the designer, before these changes are implemented as they may affect the proper functioning of the drainage facilities or hydraulic structures.

B. Constructability Review. To reduce the possibility of developing a design that cannot be constructed, it is important that all elements of the drainage design and design of hydraulic structures undergo a constructability review. A constructability review is a necessary part of the drainage design as drainage issues often impact or are impacted by many other facets of a project. The focus of such a review is to provide confidence that the various elements included in the design can be constructed and function as intended. Some items that may complicate or impact the constructability of a drainage design or hydraulic structure include:

- Environmental permitting.
- Safety issues.
- Material availability.
- Construction access.
- Right-of way requirements.

The time necessary to complete such a review varies depending on the complexity of the design. To obtain a good assessment of the constructability of the drainage design, the review should be completed by an experienced individual or team of individuals that are familiar with drainage, knowledgeable of several different related engineering disciplines (i.e. traffic, right-of-way, structures, highway safety, environmental) and aware of current construction materials, methods and issues. To minimize the amount of rework needed during the construction phase of a project, several reviews may be warranted throughout the design process on large or complex projects. The level of effort associated with a constructability review should be commensurate with the overall cost and complexity of the project.

C. Shop Drawings. Design drawings and their supporting documentation often do not include sufficient detail necessary to manufacture very specialized components of a drainage system or structures when non-standard components or materials are specified in the design documents. In such a case, shop drawings may be needed to direct the manufacturer how to fabricate a particular structural element of the drainage design based upon its intended use. Shop drawings are only needed for very specialized elements of the drainage design that are not fully addressed in Publication 72M, *Roadway Construction Standards*, Publication 219M, *Standards for Bridge Construction*, or Publication 408, *Specifications*. Very specialized elements of the drainage design may include certain concrete culvert configurations, irregular inlet boxes or manholes, outlet control structures, or end treatments for culverts. In such cases, shop drawings may be warranted to provide specific details on how elements of the drainage design are to be manufactured and to identify appropriate materials to use so that these elements of the drainage design would be structurally sound and fully capable of functioning as intended throughout the project's service life.

Design professionals involved in the drainage design or hydraulic analysis occasionally get involved with the review of shop drawings. To minimize the need for shop drawings, designers are encouraged to specify the use of standard pre-approved PennDOT materials and products wherever feasible. See Publication 10C, Design Manual, Part 1C, *Transportation Engineering Procedures,* for more information on this topic.

D. Value Engineering. The purpose of value engineering is to eliminate, modify or combine any project elements, including elements of the drainage design or hydraulic structures that add cost to a project but do not add value. Since these unnecessary elements do not add value to the project, they may be altered or removed from the project as needed, with approval from PennDOT, if it can be demonstrated that the alteration or removal of such elements do not affect the function or overall performance of the project. Establishing the value of certain elements is not always readily apparent since the overall value of each element is assessed based on its value not just at the time of construction but for the overall service life of the project. To accurately assess the value of a specific project element, it is important to consider its function and cost for the entire project service life and not just the construction portion of the project alone. In other instances, even when an element of the design appears as if it does not add value, the feature may not be readily modified or eliminated. This may be the case when certain project features are incorporated into a design based upon regulatory requirements (e.g. stormwater BMPs). Often, alteration of these features requires a permit modification to implement the value engineering.

Typically, these analyses require insight from many different disciplines to accurately assess the value of a particular element. Although drainage and hydraulic design are only a small component of most transportation projects, there are few areas of the project that are not affected by drainage. For example, it may be of more value to install curbing to control stormwater on a given segment of roadway, to separate project drainage from offsite drainage, than it would be to allow both offsite drainage and project drainage to combine. Combining the stormwater could require a larger stormwater management facility to control stormwater for the project. In such a case, a single drainage issue could potentially impact roadway geometrics, right-of-way requirements, roadway safety, maintenance requirements, environmental approvals and other elements of the roadway's design.

Value engineering is normally reserved for larger more complex projects where the cost to complete the analysis can be offset by project savings. Although value engineering is not exclusively related to the drainage design, such an analysis cannot be completed without considering the impact of drainage upon the project. See Publication 10C, Design Manual, Part 1C, *Transportation Engineering Procedures,* for more information on value engineering.

E. Project Partnering. Project partnering is a process that emphasizes a multi-discipline team approach to completing projects on-time, within budget and without disputes. The process is typically employed on large, complex, highly visible projects that involve major stakeholders which may be subject to critical time constraints and/or involve highly sensitive environmental features. Drainage design and the construction of hydraulic structures are often integrally related to sensitive environmental features such as Pennsylvania Code Chapter 93 designated high quality or exceptional value streams and wetlands. Although the drainage design and eventual construction may not be of sole focus in such sessions, the commitments that are made in the partnering process to preserve and restore various aspects of a project can have a significant impact upon a project's drainage design and construction.

The partnering process is meant to foster cooperation between PennDOT, its designer, regulatory agencies, and major stakeholders involved in the project. The objective of this partnering is to develop an improved final design by proactively addressing major issues before they become a problem and identifying essential elements of the project's design that are to be maintained to preserve the project's commitment to sensitive environmental features associated with a project. Several different resource agencies may be involved in project partnering process. However, input from resource agencies are typically limited to only those areas associated with their purview. Resource agencies not authorized to complete regulatory approvals may be involved in the project partnering process but are limited to provide only guidance or recommendations. During the partnering process effective communication is essential to allow the team to respond quickly to unforeseen issues. Thus, by implementing a partnering approach and using the expertise of the various project partners, it is the intent that many or most of the construction issues associated with a project can be resolved in the design phase of the project instead of during construction. See Publication 10C, Design Manual, Part 1C, *Transportation Engineering Procedures,* for more information on this topic.

F. Design-Build Projects. A design-build project is different than a design-bid-build project and may offer more opportunities for cooperation between the designer and the successful contractor to build the project. With a design-bid-build project, the design process is completed under a separate contract and by a different entity than the

organization contracted to construct the project. An important component of the drainage design is the environmental approval process for it is this process that establishes where and when drainage features and hydraulic structures may be constructed and how they are to function. Normally, in the design-bid-build project, the designer is responsible for obtaining all regulatory clearances and environmental approvals for a project before a contractor is selected to build the project. Therefore, the design-bid-build process offers the contractor no input into the drainage design and commitments to regulatory agencies on how the project is going to be constructed. Only after the design is complete and the contractor's bid is accepted and awarded is the contractor able to comment on the design. As the permits and regulatory approvals have already been obtained as part of the design process prior to awarding the construction contract, the contractor is unable to make adjustments to the location, size, function of drainage features or methods of access to construct in-stream hydraulic structures without amending the permit and obtaining approval from the regulatory agencies. The process of amending the permits can be lengthy and create construction delays or can result in fines and additional delays if the contractor makes adjustments without obtaining the necessary regulatory approvals.

With the design-build process, both design and construction functions are completed under a single contract. In this type of project, the contractor is included on the project team at the beginning of the design phase. The advantage to the construction contractor with the design-build process is that the contractor can provide insight into the design process upfront instead of addressing construction related issues associated with the drainage design and hydraulic features of a project after the job is awarded to the contractor. Not all design-build contracts involve the same design components. Typically PennDOT obtains the environmental clearance for this type of project and depending on the size and scope of the project they may obtain the necessary regulatory permits as well. In this type of situation the design-build team is often left with fewer design options as PennDOT has already accepted certain commitments as part of obtaining the environmental clearance and regulatory permits.

For projects where PennDOT does not obtain the regulatory permits this task is often assigned to the design-build team that is awarded the contract. As the design and construction is completed under a single contract in a design build project, the contractor and their design partners are responsible to obtain all regulatory permits. The advantage of this type of project to the design-build team is that the construction contractor is able to provide input on how various elements of the design are going to be constructed and correspondingly to the environmental obligations committed to as part of obtaining the regulatory permits. This is a logical approach for many projects as the contractor will ultimately be responsible for constructing the various elements of the design that are needed to obtain regulatory approvals. Design-build projects typically have shorter schedules than their design-bid-build counterparts and are best used for projects that have well defined goals and objectives which can benefit from a compressed schedule. See Publication 448, *Innovative Bidding Toolkit,* for more information on the design-build process.

19.3 TECHNICAL ANALYSES FOR CONSTRUCTION OPERATIONS

A. Introduction. Construction and maintenance of highways may require knowledge of low-flow discharge characteristics (e.g., discharges, flow stages, flow durations) to construct portions of a project in or near an existing stream, water body or floodplain. For example, the construction of a culvert or a bridge may require knowledge of seasonal fluctuations in flows to determine when stream elevations are below certain stages or below certain magnitudes. This knowledge may be useful in scheduling construction or designing temporary construction facilities to facilitate work within a channel or floodplain. For most facilities, completing construction is difficult if the work area is unavailable for prolonged periods of time due to the presence of water. Therefore, it is often necessary to facilitate construction by providing a means of temporary conveyance for the low flows through a project area. The methods of creating the temporary conveyance of low flows during construction operations can be completed in many different manners. However, scheduling such work to avoid flood seasons is especially important whenever any type of obstruction is needed in a floodplain or temporary reduction in stream conveyance is necessary to construct a bridge, culvert, drainage system or channel realignment.

B. Timing and Risk. USGS publishes water resources data for gauged streams throughout Pennsylvania and the United States, tabulating mean daily discharges and other streamflow data based on actual stream measurements at stream gauges. These records can be very useful in determining the type and extent of temporary measures needed to construct proposed features in and around surface waters. Based on these daily records, a low-flow analysis can be used to determine an acceptable discharge for the hydraulic design of temporary conveyance features to facilitate the construction of proposed facilities within a stream or floodplain.

Typically, a rigorous flood frequency analysis is not required to determine the type of controls necessary to accommodate low flows through a construction zone. Using the aforementioned USGS data, mean daily flow discharges may be quickly examined to determine a range of typical daily stream discharges the contractor may anticipate encountering based upon the time of the year the work is projected to be completed. Normally, the larger the period of record (years of data) the more confidence can be placed in the analysis, and depending on the construction timing and the type of work, an appropriate low flow can be selected based upon an acceptable degree of risk. Data for the monthly mean discharges for stream gauges in Pennsylvania may be obtained from the USGS via the National Water Information System: Web Interface on the internet. For an explanation of how to transpose stream gauge data from a gauge to a construction area see Chapter 7, *Hydrology*.

Transposed mean daily flows from stream gauges and rating tables based on the channel and floodplain geometry at the crossing site are useful for the design of temporary methods of conveyance, the proper scheduling of the time of year to complete the work and in selecting the location of work and material storage areas. In the event a stream gauge is not located near the stream crossing site, records from upstream or downstream gauges may be used with care to provide an indication of the usual magnitude, duration and timing flooding events. In the absence of stream gauge data, Chapter 7, *Hydrology*, contains a thorough presentation of various hydrologic methods that may be used as part of the hydrologic analysis for streams and waterways. Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 10 also contains information on different methods of establishing hydrology for a project.

If hydrographs or other hydrologic data are furnished to the contractor or included in the plans for the contractor's use in planning and scheduling operations, the contract documents should indicate that the plots or data are for information only, and PennDOT assumes no responsibility for the conclusions or interpretations made from the records.

C. Temporary Conveyance. Construction of temporary causeways, stream crossings and cofferdams necessary for the construction of bridges, culverts and stream realignments is the responsibility of the contractor. However, the responsibility for design and permitting of these structures varies depending on the type of contract. PennDOT is ultimately responsible for a project's environmental clearance and satisfactorily achieving and maintaining the commitments made to regulatory agencies as part of obtaining the project's permits. However, in many design-build projects, the design-build team is primary agent responsible for carrying out the design, permitting and implementation of the temporary staging on behalf of PennDOT. Conversely, with design-bid-build projects, PennDOT is responsible for the design and obtainment of all permits for temporary causeways and crossings. Ordinarily, these temporary features are designed to minimize or mitigate the adverse effects of the work conducted in the stream on the stream environment. The design of such features is necessary to secure permits from regulatory agencies and reduce the risk assumed by the contractor and thereby reduce construction costs. The primary regulatory agency responsible for reviewing and issuing waterway permits for temporary causeways and crossings is the Pennsylvania Department of Environmental Protection. Other agencies with regulatory authority over temporary activities in the waterway include the United States Army Corps of Engineers, United States Coast Guard, Federal Emergency Management Agency, and Pennsylvania Fish and Boat Commission.

Stream crossings, causeways, and cofferdams to complete work within a channel or floodplain are normally designed to much lesser standards than permanent crossings. The criteria used for the hydraulic design of stream temporary conveyance features should be based on risk factors associated with the probability of flood exceedance during the anticipated service life (construction period); the risk to life and property; and traffic service requirements. Design standards for temporary crossings are discussed in Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 18 and Publication 15M, Design Manual, Part 4, *Structures*, Chapter 5.

Detour stream crossings should accommodate floods larger than the event for which they are designed to avoid undue liability for damages from excessive backwater and to reduce the probability of losing the detour stream crossing structure during a larger flood. In most instances, the conveyance of floods larger than the detour design flood is provided for by a low roadway profile that allows overflow without creating excessive velocities or backwater.

> Design Example for Risk of Exceedance: A two-span bridge is scheduled to be replaced and a causeway is necessary to provide access for the contractor to remove the existing pier and construct the new pier. If the in-stream work is anticipated to take four months (0.333 years) to complete and the causeway is designed to be just above the seasonal normal water surface elevation but below the 2-year water surface elevation, what is the probability that a 2-year event will occur during the construction when the channel is partially obstructed?

The probability (p) of a 2-year event occurring in any given year is 1/return period (F) or 1/2=0.5. Thus, there is a 50% chance of the 2-year event occurring in any given year.

The probability of an (F) year event in (n) years is equal to $P=1-(1-1/F)^n$ or $1-(1-0.5)^{0.333}=0.21$ Therefore, there is a 21% chance of a 2-year event occurring during the construction period.

More information on risk and exceedance is provided in Chapter 7, *Hydrology*.

D. Erosion Control. The concerns of erosion and sediment pollution, where they might occur and how to control them, should be considered, at least in broad terms, during the early phases of the project scoping. All projects involving earth disturbance require erosion and sediment pollution controls. For small projects, this may be as simple as a section of silt fence or inlet protection. However, for larger more complicated projects, this may involve complex sequencing of operations and any number of a variety of erosion and sediment pollution controls. Chapter 12, *Erosion and Sediment Pollution Control,* presents an array of erosion and sediment pollution control best management practices available for use during the construction to control sediment transport and erosion. For simple projects, the information contained in Publication 72M, *Roadway Construction Standards,* and in Publication 408, *Specifications,* provides sufficient description of the controls to enable the contractor to construct them. However, for more complex projects, detailed plans, specifications and special provisions are necessary to indicate to the contractor how to construct the controls and what materials may be used for the construction of the temporary controls.

As part of the design process, all erosion and sediment control considerations need to be thoroughly documented on the plans and in the specifications and special provisions provided to the contractor. The erosion and sediment control considerations specified on the plans and in the specifications are considered the minimum controls that are reasonably necessary to construct the proposed improvements. Although additional work or provisions may be needed to complete the construction based upon unforeseen conditions encountered in the field, the information contained on the plans and the specifications is to consider, evaluate and address all probable events the contractor may likely experience during construction.

PennDOT's representative overseeing construction and his inspection staff use these documents to become knowledgeable about potential sources of erosion and sediment pollution in sensitive areas of the project and to become familiar with the control measures intended to minimize erosion, control sediment transport, and protect the environment during construction. It is very important that the information contained in the construction documents be discussed with the contractor before sensitive operations begin to be certain the contractor understands the function and importance of the controls and that the specified controls are properly included in the formulation of the work plan.

The contract documents are also used by the project's inspection staff to conduct periodic field observations and inspections of the proposed controls to assess their effectiveness, correct deficiencies and improve control procedures throughout the construction phase. Regular inspection of installed controls during construction is essential for effective erosion and sediment control. Regulatory agencies issuing environmental clearance for projects periodically inspect the construction and use the plans to determine if the contractor is conforming to the stipulations of the permit by constructing the controls specified in the permit. If deficiencies in the design or performance of control measures are discovered, the contractor can be directed by the regulatory agencies to take immediate steps to correct the problem. When observed deficiencies in the erosion and sediment pollution control plan are considered complex or have the potential to create significant consequences, the contractor is to contact PennDOT's Project Manager, to determine if consultation with the designer is warranted to correct the potential problem. Therefore, thorough documentation of the erosion and sediment pollution controls on the plans and other construction documents is essential.

E. Sequence of Operations. The time of the year and total construction time should be considered when developing a sequence of operations for a project whenever work is proposed in close proximity to a water resource. Certain elements such as construction of embankments along a stream or working in a floodplain should be completed before the anticipated flood season. In some areas, work cannot be performed in the streams during certain times of the year such as times when fish spawning occurs or migratory fish populations are present. In other areas, the stream or surface water body may have other functional values that need protection. Other functional values of water resources include water supply, irrigation, recreational use, flood control, wildlife habitat,

groundwater recharge and commercial use. During certain periods of the year, flows to certain water resources may be more sensitive to alterations in flow, and careful consideration should be given whenever work is proposed in a stream, waterway or floodplain.

As noted in Chapter 12, *Erosion and Sediment Pollution Control*, PA Code Title 25, Chapter 102 requires all projects requiring an NPDES permit to develop a narrative as part of the project's erosion and sediment pollution control plan. The plan is to contain a sequence of operations, prepared by the design engineer, to control erosion and minimize sediment pollution during construction. Typically, the contractor uses the construction sequence prepared by the design engineer to form the basis of their construction operations. This sequence allows the contractor to plan ahead and control erosion and sediment transport before it becomes a problem rather than adding measures after damages have occurred. If the contractor determines a change in the approved sequence of operations is warranted after the design sequence is completed and the NPDES permit is issued for the project, the contractor is responsible for notifying the PA DEP and County Conservation District of the changes and if necessary, obtaining the appropriate amendments to the permit. If the contractor changes the sequence of operations, all erosion and sedimentation controls required for a given sequence are to be in place prior to commencing earth disturbance activities.

There are many construction-related hydraulic complications related to construction scheduling. Although these problems are studied in detail during the design phase, they should be initially considered, at least in a preliminary manner, as early as possible. Commitments regarding water resource related items made in the Environmental Documentation (i.e. Environmental Impact Statement (EIS), Categorical Exclusion Evaluation (CEE), Environmental Assessment (EA)), are to be made known to the personnel who are involved in the actual construction. Some commitments that appear reasonable upon cursory examination of the issues associated with a project may not be feasible to build. In other cases, construction occurs so long after the Environmental Documentation has been prepared that those commitments are forgotten or not included in the plans or contract documents. PennDOT has developed the Environmental Compliance Tracking and Management Systems (ECTMS) to track compliance with commitments made to the regulatory agency through the design, construction and maintenance phases of the project to ensure the commitments made to the agencies are being followed through in subsequent phases of the project. For more information pertaining to the tracking of environmental commitments for PennDOT projects, refer to Publication 10X, Design Manual, Part 1X, *Appendices to Design Manuals 1, 1A, 1B, and 1C*, Appendix T.

F. Other. Construction activities adjacent to large lakes or reservoirs, areas subject to tidal fluctuations, and waterways where wave action may be a concern (i.e. navigable waterways) all present unique problems that need to be carefully considered on an individual basis during the design. Frequent or large fluctuations in the water levels may require special construction techniques such as dewatering or underwater work in these areas to complete construction and maintain the use of the water resource during construction operations. In other locations unique geologic conditions may exist that require special considerations to safely and responsibly complete construction activities without causing damage to the environment (i.e. areas containing karst geology (areas susceptible to sinkhole formation because of the existing natural carbonate geology), mine areas, and acid rock environments, etc.). Other issues such as access to the site, on-site storage of construction materials, time of year restrictions and sequence of constructions may also warrant special considerations at these sites.

19.4 DESIGN OF BRIDGES

A. Bridge Considerations. Any special hydraulic considerations or other special provisions related to the phasing of bridge construction need to be specified on the plans, such as special erosion and sediment pollution controls or unique considerations needed to manage exiting streamflow or stormwater drainage during construction. Regardless of which entity (PennDOT or the contractor) is responsible for the development of the temporary phasing and obtainment of the permits to facilitate construction, hydraulic considerations during construction usually differ from the design considerations for the completed facility. For construction staging, it is important to provide access to the contractor so that the various elements of the bridge design can be constructed while maintaining conveyance in the stream and floodplain.

Typically, most bridge work requires some activities either in the stream or at the very least in the floodplain. Although there is some flexibility with the selection of an appropriate design storm to facilitate temporary conveyance through the work zone during construction, in most cases the design event is typically no less than the

2.33-year event. In some cases such as large streams and rivers, the 2.33-year event is somewhat large and cannot easily be accommodated when large watersheds are involved. In these cases, it may be acceptable to use an event that is less than the 2.33-year event to design temporary means of conveyance if it can be supported by stream gauge data, time of year restrictions and an appropriate level of risk for the situation.

When a temporary bridge is required for a project to keep the crossing open to the public, the standard design storm is the 10-year storm. However, the design storm may be decreased to a lesser event if the situation warrants a smaller flow, but the design flow is not to be less than the 2.33-year event if the bridge is to remain open to traffic. For further discussion on the topic of temporary bridges, see Publication 13M, Design Manual, Part 2, *Highway Design*, Chapter 18, and Publication 15M, Design Manual, Part 4, *Structures*, Chapter 5.

B. Foundation and Scour. As part of the design and counter measures proposed and analyzed to protect the structures, evaluate any site conditions that may impact the foundation design or create unusual scour problems. Consideration should be given not only to the effects of scour on the permanent structure but also the effects of the temporary staging used to construct the proposed improvements may have upon scour. Cofferdams, falsework and occasionally contractor's equipment (e.g., barges) constrict the stream channel more than the completed substructure and consequently have greater potential for causing scour, bank caving and debris collection.

If specific elements of the bridge design are dependent upon special foundation or scour considerations, it is imperative that the contract documents be prepared so that individuals responsible for constructing and inspecting the proposed improvements are sufficiently informed of conditions used in the design and analysis. This is important so that individuals completing the construction can identify possible problems when conditions are different than shown on the contract documents and evaluated in the design. For additional information on scour and foundation design, refer to Chapter 10, *Bridges*, Publication 15M, Design Manual, Part 4, *Structures*, Chapter 7 or Hydraulic Engineering Circular No. 18.

C. Environmental Aspects. Minimal disturbance of the banks and bed of a stream during the construction period typically reduces erosion damage to the banks and sedimentation in the stream thus reducing the risk of harm to fish and wildlife. Embankments in or along streams should be constructed of erosion-resistant material and/or protected against erosion to avoid adverse sediment concentrations that contribute to the turbidity of the stream and potentially affect the normal depth of water in the stream.

In-stream operations associated with bridge construction, such as the construction of a pier or abutment or use of cofferdams, crossings and causeways that could potentially cause increases in turbidity during the spawning season should be avoided to prevent potential adverse environmental effects from the proposed work. Detours and construction roads are other potential sources of turbidity that should be either constructed at a time that fishery activity is low, or when special protection measures are proposed to control any harmful effects of the construction associated with erosion. Silts and clays generally flush out of the substrate over a period of time, but sands tend to become embedded. Gravel and rock, similar to the gradations found in the existing substrate typically do the least damage to the aquatic habitat. The Pennsylvania Fish and Boat Commission should be contacted to obtain information on the presence of fish at a specific location and any seasonal restrictions to work in or around streams.

Pumping of water from inside cofferdams and other dewatering operations may have a discharge of unacceptable quality to the receiving stream. Mitigation measures, such as settling basins, may be necessary if the ecosystem of the stream could potentially be impaired by the temporary degradation of water quality.

D. Stream Restoration. Bridge construction projects often include extensive work in the stream including construction of temporary roadways, temporary bridges, access roads, crane pads and channel relocations. As part of the design process, consider if any special precautions are needed by the contractor to protect the stream from pollution caused by fuels, oils, bitumen, calcium chloride or other harmful materials during construction operations. Upon completion of the construction work in areas around the bridges that are not stabilized with scour protection measures or other bridge appurtenances, the contractor is to be directed on how to repair all portions of the stream that received damage or were affected by the construction activities. The stream should be restored to its pre-construction condition as reasonably practicable, restoring scour holes and removing all foreign material from the stream used in the construction process.

19.5 DESIGN OF CULVERTS

A. Preparation. It is essential that the design of any culvert takes into consideration how the construction of a culvert impacts the existing conveyance at a site during construction, and the contract documents developed to accurately describe the existing, proposed and temporary conditions the contractor is anticipated to experience during the construction of the culvert. Prior to construction of any culvert or hydraulic structure, the plans, specifications and other construction documents developed by the designer are to be thoroughly reviewed by the contractor and inspection staff to be certain that the design accurately considers and accommodates current site conditions. If the field conditions are not consistent with the conditions used for the design, the contractor is to contact PennDOT and PennDOT may require a revised culvert design.

B. Installation. Fabrication, bedding, assembly, backfill and scour protection are as important to culvert service as the hydraulic and structural design. Without proper construction, both the performance and the service life of the culvert can be seriously lessened. Therefore, development of proper documentation describing how to construct all facets of a culvert is essential in the design phase of the project.

As the pipe performance is contingent upon site conditions it is important to select the appropriate pipe based upon the specific constraints of the installation site. Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, Appendix B contains valuable information to aid in the selection of the appropriate pipe material such as, service life, standard pipe sizes and minimum and maximum acceptable fill heights. Depending on the type of use PennDOT may require alternate pipe designs. Publication 13M, Design Manual, Part 2, *Highway Design,* Chapter 10, contains valuable information describing the process and factors to be considered when selecting the appropriate alternate pipe designs.

Precast concrete box culverts can be constructed to virtually any dimension needed with the size limited only by what can be physically transported to the site. It is important to note that many local roads and state roads with four-digit road numbers cannot support heavy loads and therefore may not be accessible for very large precast pipes. Regardless of their location, all applicable box culverts are to be designed in accordance with Appendix 9A, *Joint Agency Guidance for the Analysis of Environmental Impacts and Other Issues for Short Span Structures*. Unlike concrete box culverts both concrete elliptical pipes and concrete arches come in standard pipe sizes. For standard installation details such as pipe bedding, backfill requirements and end treatments refer to Publication 72M, *Roadway Construction Standards*.

Regardless of their location, material or geometrics culverts should be protected from damage during construction operations and should be inspected throughout the construction phase of the project. A particularly critical time for inspection is after the installation of the culvert and prior to backfilling. Another critical time for inspection is upon completion of grading operations and prior to the start of surfacing operations. The need for proper inspection is not contingent upon culvert location. It is as important to inspect culverts that are not under the roadway as it is for those structures that are under the roadway. Prior to the acceptance of the installation, all culverts should be inspected and cleaned as necessary.

C. Stream Restoration. The installation, replacement or extension of a culvert typically requires minor amounts of channel work. The contract documents are to direct the contractor to take sufficient measures to minimize damage to the stream and only work in portions of the stream that are specified and permitted by the regulatory approval process. Upon completion of the construction activities, the stream or channel is to be restored to its pre-construction condition as reasonably practicable. Any area of the stream outside of the construction limits that may have inadvertently been damaged either directly or indirectly by construction activities is to be restored or repaired by the contractor.

19.6 DESIGN OF OPEN CHANNELS

A. Introduction. Many of the construction considerations for open channels are the same as for culverts and bridges. Thus, Sections 19.5 and 19.6 should be reviewed as they relate to open channel construction. The following sections concentrate on those construction considerations that are unique to open channels.

In designing a permanent or temporary modification of a stream/channel, the designer needs to consider the following:

- Existing stream characteristics over a range of flood magnitudes.
- Class and type of facility involved.
- Environmental issues such as stream classification, fish, and wildlife habitat.

Whenever conditions at the construction site demand a change to the permitted channel design, PennDOT is to be consulted to determine the magnitude and extent of the changes, and if necessary authorize a design consultant to coordinate obtaining approval from the appropriate regulatory agencies. This is why it is important to consider the performance of a permanent or temporary channel over a range of magnitudes. The designer needs to also consider the above factors for any temporary modification of the channel (such as toe of trench construction for rip-rap around bridge abutments or piers). Typically, no special permit from the regulatory agency(ies) is required for temporary modification of a channel to construct proposed modifications as this is usually approved with the permit for the permanent channel modifications by the regulatory agency(ies). More details on channel modification are available in Chapter 8, *Open Channels*.

B. Bank Stabilization. Bank stabilization is an important aspect of open channel construction both for permanent and temporary channels. Without adequate stabilization measures, proposed improvements to the channel may not be sustainable and may potentially create maintenance problems and/or safety concerns. In some cases, a considerable length of a stream or channel may be disturbed by construction which could potentially create significant problems for a project. Immediately prior to the commencement of any channel modification or construction of bank stabilization measures, PennDOT's representative overseeing construction is to use the contract documents to inspect the site to ensure that measures proposed are appropriate based on the site conditions. It is conceivable that if there is a long period of time between the design and construction phase of a project, natural changes to the channel alignment and profile may require modification to a proposed channel design.

Depending on the location and conditions at a site, it is possible for an entire reach of stream to be in need of stabilization well beyond the immediate project limits. Stabilizing the entire reach is not the responsibility of PennDOT. Only those sections of a reach that are directly impacted by a transportation improvement project and necessary to sustain the improvements proposed by PennDOT, as outlined in the project's permits are to be stabilized as part of the project.

C. Excavation. Channel excavation work on some projects may be completed several months before total project completion. The time between completion of channel excavation and total project completion is usually longer when grading and structural improvements are separated from the contract for paving or stabilization. During this period, stabilization control measures are not established and frequent maintenance to preserve the newly constructed channels is important to achieve the intended proposed channel geometry. Chapter 12, *Erosion and Sediment Pollution Control*, provides more information on stabilization methods for channels. The designer should make sure that contract documents require the contractor to complete all maintenance activities during the term of the contract. When there is to be an extended period of time between grading and final stabilization of channels, the contractor should be required to provide interim protective measures and/or advance its own maintenance schedule to assure that minor problems do not develop into major complications that require costly repairs to complete the project and/or maintain it.

D. Access. Damaged channels can be both expensive to repair and hazardous to traffic. To facilitate repair and maintenance, channels should be designed recognizing that periodic maintenance, inspection and repair are required. Where possible, access to the facility should be incorporated into the design for personnel and equipment during the construction period and afterward. Consideration should be given to the size and type of equipment that may be required to complete maintenance, inspections and repairs when assessing the need and location of access easements, entrance ramps, gates, right-of-way, and right-of-way fences.

E. Temporary Stockpiling of Materials in Regulatory Floodplains. The stockpiling of construction materials in a regulatory floodway or floodplain is not encouraged, even if it is only on a temporary basis and has been approved in the project permits by the regulatory agencies. Construction materials should be stored outside a regulated floodway or floodplain when possible to reduce the potential that the stockpiled materials may be swept away by the floodwaters and cause an obstruction, downstream damage, or safety issues.

Section 60.3(d) of the Code of Federal Regulations (44 CFR, Parts 59-79) of the National Flood Insurance Program (NFIP) prohibits encroachments within the FEMA-adopted regulatory floodway unless it is demonstrated through

hydrology and hydraulic analyses that the proposed encroachment does not cause any increase (up to two decimals, 0.00 ft) in the base flood elevation within the community during a base flood (100-year) discharge. In certain instances, the designer may satisfy the above requirements and may obtain FEMA's permission to temporarily stockpile materials in the floodway. In such a case, the stockpiling of materials in the floodway should be specified only during the non-flood season for safety reasons.

19.7 DESIGN OF STORM DRAINS

A. Introduction. Many storm sewer construction considerations are similar to those encountered in culvert and open channel construction. Thus, Sections 19.5 and 19.6 should be reviewed as they relate to storm sewer construction. One common problem with storm drains is leakage through damaged or poorly constructed pipe joints. Therefore, it is important that pipe joints for storm sewer installations be thoroughly inspected prior to backfilling to be certain that there is no leakage from the system that could cause subsequent problems. It is also common to encounter unexpected drainage discharges along the proposed alignment of storm sewer and storm drains during construction which may require special consideration to adequately convey the discharge through the system. Careful observation of existing conditions, including field observations by the design team, are important during the design phase to reduce or eliminate as many of the unforeseen complications as possible before the construction phase. The following section concentrates on those considerations that are unique to storm drainage systems.

B. Drain Locations. Plans for storm sewers are not typically as detailed as those for culverts or bridges. The discovery of groundwater issues that can impact the storm sewer construction are not always readily evident during the design of the storm sewer. During dry seasons, or following a long dry cycle, indications of groundwater problems may be missed entirely. However, with the return of a wet season, serious problems may occur if drains are needed and not installed. The designer should be cognizant of the location of swamps, bogs, springs, areas of lush growth and other possible indicators of excess groundwater when designing and prescribing the location of subsurface drains. Ravines and draws are especially suspect areas where additional drainage appurtenances may be needed to adequately manage stormwater runoff and fill foundation areas should be inspected for additional drainage needs.

During the clearing and grading operations, groundwater problems often become evident. As excavation progresses, perched water or various aquifers may be encountered in the area of slopes or at grade. The location of subsurface drains and other elements of the drainage design may need to be changed to move these facilities to stable areas and low points or other locations where the drainage can be intercepted and allowed to efficiently enter the storm drain system. It is seldom necessary to decrease the number of planned subsurface drains; the opposite effect is usually the case. When proposed drain locations do not adequately address conditions encountered in the field during construction, the designer may be contacted to modify the location of the drains specified in the design. Therefore, careful consideration of field conditions is essential during the design of all components for the drainage system.

19.8 PRE-CONSTRUCTION CONFERENCE

A. Introduction. To minimize changes to the design of a project during the construction phase, a pre-construction conference is required prior to the commencement of all construction activities. Typically, this conference occurs after the design phase of the project has ended and the contract has been awarded; and before construction begins. The purpose of the meeting is to discuss the design and construction aspects of the project, thus affording all parties a common understanding of the proposed work, anticipated problems, and possible solutions that may be expected to occur during the construction.

It is important for the designer to be present at the pre-construction conference to explain special features of the design and planned construction phasing, where these considerations are necessary for proper function of the design and conformance with the regulatory approvals. It is also advisable and necessary at this venue to specify important time limits or special instructions on how the work is to be accomplished. For the pre-construction conference to be effective, the design team should be prepared to answer any construction-design questions that the contractor's construction personnel have identified and endeavor to obtain answers for all outstanding questions as soon as reasonably possible. It may be useful for the contractor to provide a list of questions to PennDOT's Project Manager

and the project designer, prior to the pre-construction conference, to help make sure all of the questions are adequately addressed at the conference.

During this conference, the designer should be prepared to review all important elements of the job with key personnel including PennDOT's Project Manager, contractor's agent, right-of-way agent, traffic engineers, materials engineer, maintenance superintendent, surveyors and others who may have a direct interest in the project as determined by PennDOT's Project Manager. Such a review at this time aids in clarifying the reasons for certain design features such as project work limits, right-of-way requirements; signing needs, maintenance and protection of traffic issues; materials sites; stockpile areas, selected material; foundation treatment; potential slides; environmental commitments; and potential drainage and maintenance problems, including erosion control and water pollution.

For projects requiring an NPDES permit (either a general permit or individual permit), PA DEP and/or the County Conservation District are to be invited to the preconstruction meeting. Licensed professionals or designees responsible for the earth disturbance activity including implementation of erosion and sedimentation and post-construction stormwater control plans are to attend the preconstruction meeting. At least seven (7) days notice of the preconstruction meeting is to be provided.

Depending upon the amount of time that has elapsed between completion of the design plans and the beginning of construction, changes in land cover can significantly affect the validity of many design considerations. Land-cover changes in a watershed can modify the hydrology and impact the hydraulics specified in the design making some of the proposed improvements no longer valid or applicable. There are numerous changes that can occur in a watershed that may impact the design:

- New development along a project can potentially increase the amount or location of stormwater runoff or increase damage risk considerations if situated below a project.
- Commercial mining of materials for construction is a common practice that can change flow velocities, volume of runoff, level of ground water, character of bedload, and distribution of flow at a site (i.e. dewatering activities at a mine site can reduce the groundwater levels in an area, alter baseflow in a stream or potentially increase flows to a stream if an abandoned mine area is reclaimed).
- Land clearing for agricultural purposes may affect the size of waterway openings and the need for spur dikes.
- Changes in stream alignment and profile can result in different flow conditions than those used in the outfall or cross drain design.
- Drainage area changes due to diversions or site grading can affect inlet and outlet locations, type of storm drain, inlet type and configuration of roadside ditches.
- Utilities added after the survey may require extensive redesign of storm drain systems to avoid conflicts; this reinforces the need for good utility surveys prior to design to forestall costly redesigns and delays.

Discovery of potential discrepancies between the design considerations and existing conditions are best identified from field observations by design, construction and maintenance personnel either immediately before or after the pre-construction conference. Additional objectives of the inspection are to assure location survey accuracy and to ascertain if the designer has properly visualized existing situations and designed accordingly.

Table 19.1 contains a list of items, which can be detected during a pre-construction conference field observation, held prior to the beginning of construction, which may help identify the potential for needed changes to the plans. During the pre-construction conference, the designer should be notified of the potential need to modify the design at any drainage facility or means of conveyance that has changed significantly from the conditions that the design was based upon.

Table 19.1. Changes that May Affect Drainage Designs and are Detectable during Field Observation

Possible Errors or Omissions
• Incorrect existing structure and/or invert elevations. • Incorrect drainage area size. • Channel alignment, profile. • Unreported utility. • Existing structure condition as related to service life, outlet scour, siltation. • Local flooding not documented. • Existing slope erosion not reported. • Sensitive receiving waters not reported. • Unreported debris problems. • Unreported ice jams. • Nuisance problems (i.e. odor or insect problems caused by standing water).
Possible Changes to Topography
• Increased development. • Channel improvements. • Channel diversion. • Site grading changes. • New utility. • Loss of outfall due to development.
Miscellaneous
• Incorrect typical section choice and/or incorrect grade.

In some cases, a considerable amount of time may elapse between design and construction phases of a project in which changes in the site conditions may necessitate a change to the project plans, specification and special provisions. Alterations in the watershed (both manmade and natural) that occur between design and construction may require changes to end treatment, streambank stabilization, energy dissipation, river control works, pier locations and orientation, spans, or other modifications of the design to accommodate the changes that have occurred in site conditions.

In other cases, designs may require alteration based on changes in the scope of work or purpose of the project. Some of these changes could significantly affect either hydrology or the hydraulic performance of the drainage feature designed for the site. Therefore, careful consideration should be exercised before altering one of the proposed elements of the design.

Regardless of the cause for the alteration in the drainage design, any changes in the design should be reflected in the plans, specifications and estimates for the project. If questions regarding the drainage design arise after the pre-construction conference or during construction, construction personnel are to consult with PennDOT's Project Manager, and if deemed necessary by the Project Manager, PennDOT may engage the designers of the project's improvements to determine the extent of the alterations required and make the appropriate adjustments to the plans and analyses completed for the project.

B. Permit Review. Another important aspect of the pre-construction conference is the review of the environmental clearances for the project. Typically, many of the regulatory permits have performance standards associated with them which influence the number, type, location, size, elevation, discharge and velocity, construction sequence, and length of time to construct the various features (both proposed and temporary), specified by the construction documents. Some of these specifications are especially critical and can significantly alter the performance of the project and affect permit approvals if not constructed exactly as set forth on the construction documents. To avoid any confusion, it is important that all sensitive aspects of the project, especially those that affect regulatory approvals, be reviewed with the contactor during the pre-construction conference.

C. Other Concerns. Several other issues should be discussed at the pre-construction conference including proper maintenance of drainage facilities during construction, water pollution, erosion control, and permit requirements and penalties. Drainage work on some projects may be completed several months before total project completion. During this period, vegetative erosion control measures are not well established and maintenance to correct erosion and sediment deposition in the newly constructed channels, pipes, and storage areas is important to maintaining conveyance capacity and storage volumes in the various elements of the drainage system. During the term of the contract, all maintenance activities are the responsibility of the contractor. Therefore, the contractor is responsible for all interim protective measures, adhering to the designer's construction sequence and maintenance schedule and/or advancing its own maintenance schedule as determined in the field to assure that minor problems do not develop into major damage that may require costly repairs or replacement when PennDOT assumes the permanent maintenance responsibility. During the pre-construction conference, provisions of the contract relating to pollution control should be reviewed with the contractor to be confident that the contractor understands the minimum requirements necessary to control pollution during the project construction. In addition, remind the contractor that any changes to the erosion and sediment pollution control plan made during construction may require coordination with the County Conservation District to ensure that the proposed changes do not violate the approved erosion and sediment pollution control plan. Chapter 12, *Erosion and Sediment Pollution Control*, describes issues associated with erosion and sediment pollution and presents best management practices available to control erosion during construction.

19.9 CONSTRUCTION

A. Introduction. Most designers do not have an opportunity to participate in the construction of the works that they have designed. For this reason, designs that could be improved for construction purposes tend to be perpetuated simply because the designer is not informed of the common construction problems or design deficiencies. Designers are encouraged to visit construction sites to discuss problems with contractors concerning their designs and possible improvements in future designs. This is especially important for major projects like bridge construction and new roadway alignments.

B. Design and Construction. PennDOT encourages the design community to be knowledgeable about the preferred means and methods commonly used by contractors to construct proposed improvements in order to make appropriate engineering decisions during the design development. However, unless the design consultant is hired by PennDOT to provide construction consultation services or part of the design-build project team, the design consultant is not directly involved in the construction of projects it designs, and may not be aware of complications with a particular element of the design. In most cases, the design consultant is hired by PennDOT to provide construction consultation services or is part of the design-build team and is available for consultation during construction. In such cases, frequent meetings between the design consultant and the contractor constructing the improvements may or may not occur on a regularly scheduled basis; however, the designer is typically available to answer the contractor's questions during construction by issuing a request for information through PennDOT. If PennDOT determines that it needs clarification or direction from the design consultant, PennDOT may forward the request on to the design consultant to investigate the contractor's question and provide an answer.

Some of the questions presented during construction by the contractor may require adjustments to the design to adequately address the issues presented by the contractor. In such a case, and when directed by PennDOT to revise the design, the consultant will make the necessary revisions based on the information supplied by PennDOT. As soon as the engineering work and necessary revisions are completed, the design team is to furnish PennDOT a revised plan, including all geometric information, special provisions and specifications.

C. As-Built Plans. As-Built plans are a set of plans that document what was constructed in the field including any deviations from the contract documents. They serve many functions related to the design and construction process including documentation of:

- Final sizes and location of all elements of the drainage system and related facilities.
- Changes that were made in the design during the construction process (e.g., size of facilities, elevations, materials used, addition or elimination of facilities).
- Variation between the original plans and specifications and the final installed facilities.

The data contained on these plans provides valuable information concerning new drainage facilities that may be used as reference information for entry into PennDOT's Roadway Management System. This System serves as an inventory of PennDOT's facilities located within the right-of-way. As-Built plans are to be prepared on all PennDOT projects in which a set of construction plans were prepared.

The completion of accurate and complete As-Built plans can be invaluable in documenting changes that can be incorporated in future designs and to facilitate future investigations of the project if problems are encountered or there is some need to analyze the facility's performance. The plans also serve as an invaluable reference for potential future legal action pertaining to PennDOT's facilities.

The following records should be kept for each installation:

- Special features, such as:
  - Fishways.
  - Improved inlets.
  - Debris protection.
  - Energy dissipaters.
- Inspection tags.
- Location and layout including:
  - Station.
  - Skew(s).
  - Location of inlets, outlets, junctions.
  - Camber.
  - Alignment.
  - Grade.
  - Geometry (i.e. span and rise).
  - Materials (i.e. CMP, RCP, RCPE).
  - Size and location of scour protection.

For more information on As-Built Plans, see Publication 10C, Design Manual, Part 1C, *Transportation Engineering Procedures*, Chapter 5.

D. Final Inspection. The final inspection following the completion of the project should document any deviations in the drainage design or hydraulic structures from the original plans and provide an initial assessment of the hydraulic performance of newly constructed project elements. Construction personnel should be encouraged to inform the designer of any design-related difficulties that were encountered and provide suggestions to improve future designs. All changes to the construction plans should be incorporated into "As-Built" plans for future reference.

E. Construction Feedback. One of the primary avenues available for PennDOT Project Managers to discuss construction projects and the performance of various design elements included in the project is the After Action Review. This review is set up by the Design Project Manager with key members of the design team before project construction closeout. Upon completion of a project, an After Action Review conducted jointly by designers and field personnel can be a very useful learning experience for all participants. The review is used to evaluate overall effectiveness of the constructed design in achieving the project objectives with the ultimate purpose of the review to help the designer improve the design process for future projects. This review should be an open critique and discussion of difficulties encountered in the construction and possible design changes to prevent such difficulties in the future. This meeting also gives the designer an opportunity to present why some difficulties in construction are necessary because of specific design considerations. During this review, long-term maintenance items may also be discussed as additional future design work may be required to correct any recurring problems. Maintenance personnel should be informed of these potential problems and keep a log of any persistent problems so that an analysis can be made to determine if any alteration of design or construction practices is warranted to reduce maintenance problems.

Another vehicle available to provide feedback to the designer about the design is the Contractor's design evaluation report. This report is prepared at the time of the final inspection at which time the contractor can use the report to identify problem areas or corrective action and provide input to improve the quality of future designs. Different ideas and procedures that have been successfully employed by the contractor can be cited in the report for potential

evaluation by the designer to determine if they warrant consideration for widespread use in future projects. Also of importance for discussion is possible modification to standard design items that would facilitate their construction and/or perhaps reduce their cost.