VERMONT
AGENCY OF TRANSPORTATION

TECHNICAL MANUAL

Hydraulics Manual

May 28, 2015

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Vermont Agency of Transportation

Hydraulics Unit

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About the Manual

The intent of the VTrans Hydraulics Manual is to identify approaches to the analysis of road and highway hydraulics that are approved for use by VTrans. This manual is not intended to be a comprehensive summary of all available techniques for hydrologic and hydraulic analysis, but rather it is meant to provide the designer with (1) guidance on the use of selected methods and (2) references to sources of additional information.

The methods presented in this manual make reference to other guidance documents and engineering manuals, such as those produced by the American Association of State Highway and Transportation Officials (AASHTO), the Federal Highway Administration (FHWA), the U.S. Army Corps of Engineers (USACE), the U.S. Geological Survey (USGS), and the Vermont Agency of Natural Resources (ANR) that provide more comprehensive explanations of the methods along with the scientific basis. VTrans recommends that designers make use of the latest revisions of these manuals when performing calculations.

Computing power has dramatically increased in recent years, so most of the methods that are covered in this manual can be performed using free applications that were developed by federal agencies using the methodologies outlined in their respective guidance documents. The benefit of using software is the ability to perform multiple iterations easily, allowing different design scenarios to be compared quickly and accurately.

As a living document, VTrans intends to provide periodic updates to this manual, which may include replacing individual pages or sections as new information or techniques become available. Periodically check the VTrans website to download manual updates.

Outline

The following outline provides a brief description of each chapter in the manual. The final sections of the manual are devoted to summarizing materials—a list of acronyms, definitions of key terms, references, and an index.

Chapter 1 Hydraulic Design Guidelines presents an overview of the key guidelines that should be considered when preparing a hydraulic design.

Chapter 2 Planning and Location identifies the planning and permitting considerations that must be addressed prior to and throughout the design process.

Chapter 3 Data Collection, Resources, and Tools provides an overview of the data collection requirements for different types of hydraulic structures. This chapter summarizes key resources and tools referenced throughout the manual and incorporates lists to assist the designer and reviewer in confirming that necessary information has been gathered and considered during the design process.

Chapter 4 Hydrology identifies the preferred methods of hydrologic analysis and their proper application. The results from the hydrologic analysis are a critical component of the project and ultimately inform the remainder of the design process.

Chapter 5 Open Channels provides guidance for evaluating flow in open channels and appropriate measures to use in designing stable open-channel drainage systems. Two key FHWA Hydraulic Engineering Circular (HEC) publications are given as the basis for open channel design: HEC-14, *"Hydraulic Design of Energy Dissipation for Culverts and Channels,"* and HEC-15, *"Design of Roadside Channels with Flexible Linings"*.

Chapter 6 Crossing Structures highlights the analysis tools that are most helpful in the design of bridges and culverts. Emphasis is placed on using the USACE's Hydraulic Engineering Center River Analysis System (HEC-RAS) software for bridge analysis and design and the FHWA's HY-8 software for culvert analysis and design.

Chapter 7 Channel Stability and Scour at Bridges provides guidance on the evaluation of flow through and around bridges. Three key FHWA HEC publication are given as the basis for bridge scour analysis and protection: HEC-18, *"Evaluating Scour at Bridges,"* HEC-20, *"Stream Stability at Highway Structures,"* and HEC-23, *"Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance."*

Chapter 8 Storm Drainage Systems is concerned with the specifics of clearing stormwater runoff from pavement to ensure safe travel conditions. Guidance is provided for the design and analysis of drainage infrastructure such as catch basin grates, curb and gutter sections, and closed drainage systems. FHWA's HEC-22, *"Urban Drainage Design Manual,"* is referenced as the basis for stormwater drainage system design. Reference is also made to the most recent version of the ANR *"Vermont Stormwater Management Manual"* (VSMM) for the appropriate design of stormwater treatment facilities (stormwater wetlands, wet ponds, infiltration systems, etc.) for use in Vermont.

Chapter 9 Documentation identifies the backup calculations, maps, reports, and types of other documentation that must be assembled by the designer and provided to VTrans as part of a complete design package.

Appendix A Manning's n Values provides a summary of typical Manning's n values used as part of hydraulic analyses.

Appendix B Field Investigation Forms includes copies of the VTrans Field Investigation Form and the Hydraulic Survey Field Inspection Checklist. The Field Investigation is also available electronically for use with spreadsheet software.

Appendix C Hydraulics Form includes two copies of the VTrans Final Hydraulics Report Form—one version offering guidance for filling out the form, and one version that is blank. The Final Hydraulics Report Form should be used to summarize the hydraulic performance of existing and proposed crossing structures with clear spans of 6 feet or greater. It can also be used for structures with clear spans that are less than 6 feet if site conditions warrant the level of detail. All of the information from this form should be included on project plans. The Final Hydraulics Report Form is available electronically for use with spreadsheet software.

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Chapter 1 Hydraulic Design Guidelines

1.1 Introduction

1.1.1 Overview

Hydraulic design and engineering are some of the most important components of highway design and construction. This chapter presents specific guidelines concerning the hydraulic design of culverts and bridge structures. When followed, these guidelines provide an appropriate level of consideration for the multitude of variables that influence hydraulic design. The factors that influence hydraulic design can be generally classified in one of the following four categories:

- Hydrologic Analysis
- Hydraulic Analysis
- Geomorphic Analysis
- Site Characterization

1.1.2 Criteria, Guidelines, and Guidance

The following sections of this chapter present information concerning hydraulic design of culverts and bridge structures and related guidelines. Some sections are limited to outlining the relevant guidelines (with references indicating where details can be obtained) while other sections state the guidelines and give detailed information.

The rest of the manual presents information in a similar fashion. A wealth of material on the subjects of hydrology and hydraulics is available from federal and state agencies, educational institutions, and other reputable organizations. Through this manual, VTrans presents its most current and best practices for the analysis of road and highway hydraulics, but designers should not rely on this manual as a singular reference material. Designers should use the embedded hyperlinks to navigate to and become familiar with primary and supplementary resources, tools, and data sources.

Throughout the manual, criteria, guidelines, and guidance are presented in active statements directing the designer to take certain steps. More optional steps are indicated as such with qualifying words (e.g. should, can, may, etc.) that leave more opportunity for the designer to use their best judgment given the individualized nature of each project. Key decisions impacting the analysis and design must always be discussed with applicable VTrans personnel. Even the most basic hydraulic projects typically involve iterative and interdisciplinary components, so collaboration and clear communication provide the cornerstone of a successful project.

1.2 Hydrologic Analysis Guidelines

1.2.1 Introduction

The following subsections provide a summary of guidelines that are recommended for completing a hydrologic analysis for open channel design or crossing structures. A detailed discussion of the specific analytical methods used to conduct hydrologic analyses is presented in Chapter 4 *"Hydrology."* Also refer to Chapter 4 for an overview of terminology related to hydrologic analysis, such as design frequency, recurrence interval (RI), and annual exceedance probability (AEP).

1.2.2 Background Information

Complete a review of existing information related to the hydrologic factors that influence the highway design prior to beginning the design process. Typical information to collect and review can include:

- Topography
- Aerial photographs
- Stream flow records
- Regulatory or designated floodplain areas
- Historical high water elevation data
- Flood flow discharges
- Fluvial erosion hazard zones
- Locations of hydraulic features (i.e. reservoirs, dams, pump stations for water withdrawals)
- Locations of regulated water resource areas/wetlands
- Applicable aquatic organism passage (AOP) considerations

As part of the review process, evaluate pertinent spatial data available from the Vermont Center for Geographic Information (VCGI) and from the Vermont Agency of Natural Resources (ANR) Natural Resources Atlas. Collect available studies and reports from federal, state, local, and private sources. Conducting a field investigation is strongly encouraged because site observations often reveal location-specific information that cannot be identified during the literature and desktop review stages. Based on the findings from the background information review, the designer will need to decide whether it is necessary to gather additional data or conduct a study. The scope and complexity of additional analyses should be commensurate with the importance and magnitude of the project and the problems that are encountered. Refer to Chapter 3 *"Data Collection, Resources, and Tools"* for more links to more resources and additional guidance on the data collection process.

Have a clear understanding of the factors that contribute to the volume, velocity, and timing of flood flows. These factors form the basis for hydraulic design. Be cognizant of the presence of floodplain and flood hazard areas in the vicinity of the project, as described below.

1.2.3 Floodplain/Flood Hazard Areas

The hydrologic analysis should consider the flood history of the area and the effect of flooding conditions on existing and proposed structures. If flood hazard areas are present in the project area, the designer should identify potential impacts to construction, long-term safety, design performance, and maintenance. The flood hazard evaluation should include areas subject to inundation as well as areas subject to fluvial erosion hazards. Inundation hazards are addressed through the Federal Emergency Management Agency (FEMA) mapping process and National Flood Insurance Program (NFIP) regulations. Fluvial erosion hazards are addressed through the Vermont Department of Environmental Conservation (DEC) Rivers Program, which is part of the ANR. Fluvial erosion hazards are described in more detail in Section 1.4.

1.2.4 Factors Affecting Flood Flows

For all hydrologic analyses, evaluate the following factors when they are likely to significantly impact the final results:

- Drainage basin characteristics (i.e. size, shape, slope, land use, vegetative cover, geology, soil type, surface infiltration, storage, existing drainage systems, etc.)
- Stream channel characteristics (i.e. geometry and configuration, geomorphology, natural and artificial controls, channel modification, aggradation and degradation, ice and debris, etc.)
- Floodplain characteristics
- Meteorological characteristics (i.e. precipitation amounts and types, storm cell size and distribution characteristics, storm direction, and rate of precipitation)

1.2.5 Hydrologic Methods

Standard engineering practice relies on formulas and models for estimating hydrologic flows based on statistical analyses of rainfall and runoff records. These methods provide statistical estimates of flows with varying degrees of error.

The designer is responsible for selecting appropriate hydrologic methods and obtaining runoff data—where available—in order to evaluate, calibrate, and ultimately quantify predicted flows corresponding to the desired design frequency. Because the predicted flood flows represent the designer's best estimate, there is a chance that the real-world flow experienced during a flood event will be greater or smaller than the predicted value. The expected magnitude of this variation can be determined for some formulas or models as a part of the hydrologic design procedure. Calibrate and test the selected methods to local conditions for accuracy and reliability. Suggested methods for calculating flood flows and evaluating the error associated with each method are presented in Chapter 4 *"Hydrology."* The use of multiple hydrologic methods for estimating flood flows is required.

1.2.6 Design Frequency

Select the design frequency that corresponds to the roadway classification and the potential flood hazard to property. Refer to Chapter 4 *"Hydrology"* for more information about design frequencies by roadway classification.

When developing hydrologic analyses, VTrans recommends that designers evaluate a range of standard AEP storm events in addition to the design event. These additional calculations are typically quick to perform and supply important flood information for use during the hydraulic analysis. Standard AEP events, from smallest storm to largest storm, include:

- 50% AEP (2-year RI)
- 43% AEP (2.33-year RI)
- 20% AEP (5-year RI)
- 10% AEP (10-year RI)
- 4% AEP (25-year RI)
- 2% AEP (50-year RI)
- 1% AEP (100-year RI)
- 0.5% AEP (200-year RI)
- 0.2% AEP (500-year RI)

The 43% AEP event is used to approximate the mean annual flow rate in a channel. Additionally, regardless of the design event, evaluate all proposed culverts and bridges over perennial streams for performance during a 1% AEP event to ensure that there are no unexpected flood hazards.

1.3 Hydraulic Analysis Guidelines

1.3.1 Introduction

The following subsections provide a summary of guidelines that are recommended for completing a hydraulic analysis for open channel design or crossing structures. Refer to Chapters 5 through 7 (*"Open Channels," "Crossing Structures,"* and *"Channel Stability and Scour at Bridges,"* respectively) for more detailed discussions of these guidelines.

1.3.2 Hydraulic Analysis

The hydraulic design process involves analyzing existing conditions and preliminary or trial selections of alternative designs that are judged to meet the site conditions and accommodate the flood flows for the selected design frequency. Perform the hydraulic analysis using appropriate formulas and computer programs to define, calibrate, and validate the performance of the preliminary designs over a range of flows.

1.3.3 Engineering Evaluation

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

- Hydraulic adequacy
- Geomorphic compatibility
- Flood hazards to highway users and neighboring property owners
- Costs
- Constructability
- Physical constraints of the site
- Environmental and social concerns

1.3.4 General Guidelines

The following general guidelines apply to the designer and the design process. The guidelines are categorized for easy reference, but many of them have a complex relationship with the design process.

1.3.4.1 <u>Costs and Constructability</u>

- Adhere to a level of detail that is commensurate with the complexity of the project and with other engineering or environmental factors. Guidance for completing a risk assessment and evaluating the level of documentation required for different types of projects is included in Chapter 9 *"Documentation."*
- Where practicable, evaluate various crossing structure designs during the hydraulic analysis in order to determine the most cost-effective proposal consistent with design constraints.

1.3.4.2 <u>Hydraulic Adequacy</u>

- Provide an adequate hydraulic design based on good engineering practice. Note that the designer is not required to provide a structure that will handle all conceivable flood flows under all possible site conditions.
- Use the overtopping and/or design flood as criteria for evaluating the adequacy of a proposed design. The overtopping flood and the design flood may vary widely depending on the grade, alignment, and classification of the road and the characteristics of the water course and floodplain.
    - o *<u>Overtopping Flood.</u>* The most frequent (least intense) flood that will result in flow over the highway or other watershed boundary.
    - o *<u>Design flood.</u>* The AEP of the flood for which the crossing structure is sized to assure that no traffic interruption or significant damage will result.
- Consider developing a hydraulic performance curve for the proposed design depicting the relationship between floodwater stage (elevation) and flood-flow magnitudes and frequencies. The performance curve should range from the base flow to the 1% AEP flood flow. Performance curves are useful for evaluating the adequacy of a design for a range of flows, as long as the designer remains cognizant of the errors associated with the hydrologic estimating procedure.
- Use larger floods to evaluate cases of extreme flooding or scour on critical structures. Evaluate bridge foundation scour for the incipient overtopping event, the scour design event, and the scour check event. A discussion of this topic is included in Chapter 7 *"Channel Stability and Scour at Bridges."*

1.3.4.3 <u>Floodplains/Flood Hazard Areas</u>

- If the project area is located within a regulatory floodplain, refer to the effective FEMA Flood Insurance Study (FIS) for information about existing flood hazards. The 1% AEP event serves as the present engineering standard for evaluating flood hazards and as the basis for regulating floodplains under the NFIP. Note that the available FIS data is sometimes outdated and does not accurately reflect existing conditions or adhere to up-to-date modeling techniques.

- If there are discrepancies between flows reported in the FIS and existing conditions flows calculated by the designer using the methods presented in this manual, the designer should attempt to reconcile the differences. However, if there is sufficient confidence in the designer's values, these values should be used for design. Attempt to identify the likely causes of the discrepancies.
- VTrans typically recommends that the designer:
    - Complete a hydraulic model using hydrologic and hydraulic data from the FEMA FIS to demonstrate that the proposed structure conforms to FEMA floodplain regulations.
    - Use hydrologic and hydraulic data derived using the methods in this manual to size and design crossing structures and to complete scour analyses.
- Evaluate the proposed design during the 1% AEP event in order to identify predicted flood hazards.
- Where work within the regulatory floodplain is necessary, select a final design that does not significantly alter the flow distribution in the floodplain and avoids encroaching on the floodway within a floodplain.
- Select a final design that does not exceed the maximum allowable backwater under the NFIP, unless exceedance is permitted due to special hydraulic conditions. Consult with the ANR Flood Hazard Management Program for projects that may affect flood elevations.

1.3.4.4 Geomorphic Compatibility

- Estimate potential degradation or aggradation of the channel as well as contraction and local scour that results from the construction of the structure.
- Design the structure foundations to extend below the total scour depth.
- Refer to Section 1.4 for additional guidelines and reference to available resources from the ANR.

1.3.4.5 Environmental and Social Concerns

- Where practicable, provide additional clearance within hydraulic crossing structures to allow for passage of ice and debris. Refer to Chapter 6 *"Crossing Structures"* for more specific guidance.
- For navigable waters, determine the required vertical clearance by using normally expected flows during the navigation season and conforming to federal requirements. Consult with the U.S. Coast Guard (USCG) and the U.S. Army Corps of Engineers (USACE) for projects that affect navigable waters.
- Where practicable, select the "crest-vertical curve profile" for crossings designed to allow embankment overtopping at lower discharges. This type of design helps to protect the structure from damage during overtopping events and may allow the road to be re-opened more quickly following an event.

1.3.5 Auxiliary Openings for Flood flow

The use of auxiliary openings is generally not appropriate in Vermont, except under limited circumstances where site constraints preclude the expansion of the principal waterway opening to pass the entire flood flow. Obtain approval from the VTrans Hydraulics Engineer prior to designing an auxiliary opening for flood relief.

The need for auxiliary waterway openings, or relief openings, as they are commonly termed, arises on streams with wide floodplains. When the stream reaches a certain stage, design auxiliary openings to pass a portion of the flood flow in the floodplain. Auxiliary openings do not provide relief for the principal waterway opening in the same way that an emergency spillway at a dam does, but they do have predictable capacity during flood events. Basic objectives in choosing the location of auxiliary openings include:

- Maintenance of flow distribution and flow patterns
- Accommodation of relatively large flow concentrations on the floodplain
- Avoidance of floodplain flow along the roadway embankment for long distances
- Crossing of significant tributary channels

The most complex factor in designing auxiliary openings is determining the division of flow between two or more structures. If flow is incorrectly proportioned, one or more of the structures may be overtaxed during a flood event. The design of auxiliary openings should usually be generous to guard against that possibility.

1.4 Geomorphic Analysis Guidelines

1.4.1 Introduction

In broad terms, geomorphology is the study of the physical features of the earth's surface and their relation to geologic activity. Fluvial geomorphology focuses more specifically on the function and characteristics of streams and rivers as landscape processes. The successful design of roadways adjacent to or crossing streams depends on an understanding of these processes and their potential to impact roadway infrastructure. The ANR has prepared the *"Vermont Standard River Management Principles and Practices: Guidance for Managing Vermont's Rivers Based on Channel and Floodplain Function,"* (VT SRMPP) which is a valuable resource establishing a consistent set of principles and practices that can be applied to both public infrastructure and private property. Explanations and guidance in the ANR manual support and expand upon the hydraulic design guidance provided here.

1.4.2 Location of Stream Crossing

Although many factors, including nontechnical ones, enter into the final location of a stream crossing system, the hydraulics of the proposed location must have a high priority. Hydraulic considerations for selecting the stream crossing location include:

- Floodplain width and roughness
- Flow distribution and direction
- Stream type (braided, straight, or meandering)
- Stream regime (aggrading, degrading, or equilibrium)
- Nearby stream confluences
- Stream controls

The hydraulics of a proposed location also affect environmental considerations such as aquatic life, wetlands, sedimentation, and stream stability. Additionally, the hydraulics of a particular site determine whether or not certain national objectives such as wise use of floodplains, reduction of flooding losses, and preservation of wetlands can be met.

Identify sites that are located within a stable reach of the stream channel when selecting locations for new crossings. The ideal site would have existing grade controls upstream and downstream of the project site and be located so the project would not impact the vertical or horizontal meander of the reach. Stream alterations that impair the connectivity or equilibrium of the stream channel are not permitted under the ANR *"Stream Alteration General Permit"* (ANR GP).

1.4.3 Stream Morphology and Scour

Stream morphology refers to the form and shape of the stream path created by its erosion and deposition characteristics. A stream can be braided, straight, or meandering, or it can be in the process of changing from one form to another as a result of natural or constructed influences. The following items provide some guidance on taking stream morphology into account during the design process:

- If the stream displays unstable characteristics, a historical study of the stream morphology at a proposed stream crossing site may be necessary. This study should also include an assessment of any long-term trends in aggradation or degradation.
- Whenever possible, avoid locating stream crossing sites over braided streams and alluvial fans.
- The ANR Rivers Program can often provide detailed information about stream morphology at specific locations, along with detailed histories of issues at stream crossing sites.
- During the preliminary design stage, the designer should look at existing scour and scour sensitivity in the field and factor it into design considerations.

The extreme hazard posed by bridge scour failures dictates that a more conservative approach be taken in selecting suitable flood magnitudes to use for scour analysis. Refer to Chapter 7 *"Channel Stability and Scour at Bridges"* for guidance selecting the appropriate AEPs for scour analysis storm events. Additionally, the Federal Highway Administration (FHWA), Natural Resources Conservation Service (NRCS), and the ANR provide guidance on stream stability and designing for scour impacts.

1.4.4 Flood and Fluvial Erosion Hazards

Carefully analyze flood flow characteristics at stream crossing structures to determine their effect on the road and to evaluate the effects of the road on flood flows. Such an evaluation can assist in determining those locations at which construction and maintenance will be unusually expensive or hazardous. Include flood hazard effects to private property both upstream and downstream (i.e. overtopping floodwaters diverted onto previously unaffected property).

In addition, consider the river corridor or fluvial erosion hazard area that is associated with the stream or river and determine if the project can be designed to avoid or minimize impacts in such areas. The fluvial erosion hazard area characterizes the meander belt associated with a natural stream channel. The width of this meander belt varies according to the stream type and setting. The goal of avoiding new infrastructure in such areas is to provide room for the stream to maintain its dynamic equilibrium over time and to prevent costly expenditures to maintain infrastructure that conflicts with the natural character of the stream. The "River Corridors" data layer produced by the ANR Rivers Program incorporates the fluvial erosion hazard concept and can be used to provide preliminary information about stream dynamics in the vicinity of the project site. Contact the Rivers Program for additional information and support.

It is important for the designer to identify the flood and fluvial erosion hazards to determine if the hazard will be increased, decreased, or the same with and without the proposed highway improvement. Where flood or fluvial erosion hazards exist, consider redesigning the road crossing or seek alternative alignments to avoid unnecessary risk associated with these conditions.

1.5 Other Considerations

1.5.1 Environmental Considerations

The preservation of wetlands and protection of aquatic habitat typically require the expertise of a biologist on the design team. Include stormwater and water quality specialists in the design process. Address the following environmental criteria as part of hydraulic designs:

- Wetlands protection
- Aquatic habitat protection
- Water quality

Design stormwater management systems in compliance with the ANR *"Vermont Stormwater Management Manual"* (VSMM) and acquire permits in accordance with the ANR Stormwater Program. Properly treated stormwater runoff will minimize the impact of the impervious area on receiving waters. The DEC Watershed Management Division provides detailed information about Green Stormwater Infrastructure (GSI), which maximizes the use of infiltration, evapotranspiration, and storage to control and treat stormwater runoff.

As a practical matter, the hydraulic design criteria related to scour, degradation, aggradation, flow velocities, and lateral distribution of flow are important criteria for evaluation of environmental impacts as well as the safety of the stream crossing structures.

1.5.2 Future Hydrologic Projections

Regulations and current standard practice typically rely on the statistical patterns in past hydrologic data to estimate peak flows for hydraulic design. However, these methods often do not account for trends in increasing peak flows that are widely attributed to changing climate patterns. The designer should be aware of the limitations in assuming stationarity for design storm estimates and evaluate whether or not that assumption is appropriate on a case-by-case basis. Consult with the VTrans Hydraulics Engineer before factoring climate change into a hydrologic analysis.

1.5.3 Construction/Maintenance

Incorporate measures that reduce construction and maintenance costs in the stream crossing design whenever possible. These measures include:

- Stone fill protection of abutments and embankments
- Embankment overflow at elevations lower than the bridge deck
- Alignment of piers and abutments with the flow
- Where feasible, site grading that facilitates access for inspection and maintenance of stream crossing structures

Design temporary structures and crossings used during construction for a specified risk of failure due to flooding during the construction period. The following steps should be completed:

- Identify and address potential impacts on normal water levels, fish passage, and normal flow distribution.
- Contact the ANR Rivers Program for additional information and support regarding the requirements of temporary structures and other in-stream work.
- As appropriate, participate in the pre-construction/final inspection meetings so that apparent and potential maintenance problems may be noted and possible solutions discussed.

1.5.4 Agency Coordination

Consider the interests of other government agencies (both state and federal) in the evaluation of a proposed stream crossing system. Most projects will require coordination with the following agencies:

- USACE
- ANR Rivers Program
- Vermont Wetlands Program

Depending on the scope of the project, coordinate with:

- DEC Stormwater Management Program
- Vermont's Regional Planning Councils
- District Environmental Commissions
- Local municipalities

Cooperation and coordination with these agencies is essential to the success of a project because of the impact that highway and water resource projects may have on one another. In addition, organizations can share data and experiences to assist in the completion of the hydrologic and hydraulic analysis. The VTrans Project Delivery Bureau Environmental Section assists with this coordination and outreach and is responsible for ensuring compliance with state and federal resource regulatory agencies.

1.5.5 Documentation

The design of hydraulic structures must be adequately documented because it is often necessary to refer to plans and specifications long after the actual construction of the project has been completed. Thus, the designer must fully document the results of the entire hydrologic and hydraulic analysis. Refer to Chapter 9 *"Documentation"* for more information.

Chapter 2 Planning and Location

2.1 Introduction

During the planning stages of a project, the designer will identify the scope and extents of the project, the required components of the analysis, permitting considerations, and potential project constraints that must be accommodated or overcome.

2.1.1 Scope and Complexity

Project scoping is usually initiated between VTrans and the local municipality. Other state and federal agencies may be involved, depending upon the location and complexity of the project. The scope of the project will influence the type and quantity of data that is required to support the engineering analysis and design.

2.1.2 Components of Hydraulic Design Process

2.1.2.1 Project Planning

Once a prospective project has been identified, a project planning phase defines the limits of the project, prospective funding sources, permitting criteria, existing data sources, and project constraints. The project planning process is outlined in more detail later in this chapter.

2.1.2.2 Data Collection

Data collection will typically occur in several phases once the project has been identified. A preliminary data collection phase may occur in conjunction with the project planning phase in order to assist in the project definition and identification of regulatory issues and project constraints. Additional data collection will likely be necessary to gather information required to complete the hydrologic and hydraulic analysis and design. Recommended types and sources of data to collect are described in Chapter 3 *"Data Collection, Resources, and Tools."*

2.1.2.3 Hydrologic Analysis

The hydrologic analysis is critical to the successful design of stream crossing structures and stormwater drainage systems because the design flood flows that are the product of this analysis are used in all subsequent steps of the hydraulic design process. Methods for evaluating the hydrology of contributing drainage areas to a stream crossing structure are presented in Chapter 4 *"Hydrology."*

2.1.2.4 Hydraulic Analysis

The hydraulic analysis evaluates the characteristics of flows through the stream crossing structure and channel in the project area. The results from this analysis are used to identify the required structure size and configuration, as well as channel modifications that are required to protect the structure and adjacent property from damage during high flow events. The hydraulic evaluation of open channels is discussed in Chapter 5 *"Open Channels"* and the hydraulic evaluation of bridges and culverts is presented in Chapter 6 *"Crossing Structures."* Hydraulics for highway stormwater drainage are presented in Chapter 8 *"Storm Drainage Systems."*

2.1.2.5 Channel Stability and Scour Evaluation

The successful design of a stream crossing structure ensures that the structure is able to process the naturally-occurring sediment and debris carried by the stream and avoids impacting (or being impacted by) the dynamic equilibrium of the stream. Evaluate relevant design parameters during the data collection phase, including stream type, valley setting, and substrate. When these parameters indicate that the potential for instability or the site risk and consequences are high, perform additional evaluation according to the methods outlined in Chapter 7 *"Channel Stability and Scour at Bridges."*

2.1.2.6 Stormwater Management Planning

Prior to design, ensure that the planning phase has properly located facilities and adequately addressed local concerns, permitting requirements, legal considerations, and potential problem categories. This chapter provides general guidelines and major considerations for evaluating these factors during the planning and location process.

Communicate with VTrans Maintenance District personnel during the planning phase to determine what existing collection and conveyance systems can possibly be eliminated to reduce operation and maintenance (O&M) and promote Green Stormwater Infrastructure (GSI). Coordinate with the VTrans Stormwater Management Engineer to resolve questions related to stormwater permitting jurisdiction for public linear projects and connections to VTrans infrastructure.

Refer to the latest edition of the Vermont Agency of Natural Resources (ANR) *"Vermont Stormwater Management Manual"* (VSMM) for the stormwater management requirements associated with new development and the redevelopment of existing impervious areas.

Stormwater drainage systems located in the watersheds of [Stormwater Impaired Waters]() may have additional stormwater management requirements that exceed the minimum standards specified in the VSMM.

Specific information regarding the design and assessment of stormwater management within the roadway environment can be found in Chapter 8 *"Storm Drainage Systems."*

2.1.2.7 Construction Management

Many serious construction problems occur because drainage and water-related factors were overlooked or neglected in the planning and location phases of the project. Review the analysis of available data regarding site conditions and resources, and properly schedule work to develop cost-effective solutions and to prevent extended problems due to poor planning, which could include:

- Soil erosion
- Sediment transport and deposition
- Drainage and landslide
- Uncoordinated timing of project stages (construction phase management as it relates to sedimentation and erosion control and resource protection/flood control)
- Insufficient protection of fish habitat (limit in-stream work to July 15 to October 1 whenever possible to avoid disturbing spawning seasons)
- Contamination of pumping and distribution facilities
- Insufficient protection of streams, lakes, and rivers from construction-related sediment and pollutants
- Insufficient protection of wetlands
- Compaction of soils, often from staging and equipment operations, where soil permeability and infiltration are essential to the function of the design

2.1.2.8 Maintenance Problems

Consider potential erosion and sedimentation problems with respect to completed highway construction when conducting planning and location studies. If a particular location will require frequent and expensive maintenance due to the drainage design, consider alternate locations unless the potentially high maintenance costs can be reduced by special design. The best indicator of maintenance problems in a specific area is experience. Conduct interviews with local officials and VTrans Maintenance District personnel for support with identifying potential erosion and sedimentation problems. For additional information, refer to the *"VTrans Project Post-Construction (Operational) Stormwater Protocol"* and the *"VTrans EPSC Protocol."*

Additional investigation—such as review of highway maintenance reports, flood reports, and damage surveys, as well as interviews with local residents—should be conducted at the discretion of the designer.

Always consider the implications to maintenance responsibilities when changing channels and making drainage modifications. Such modifications can lead to erosion and stream channel degradation which can have significant impacts on maintenance procedures. Design for feasibility of maintenance as much as is practicable.

2.1.3 Permitting Considerations

Depending on the location and scope of the project, coverage under different types of permits or approvals may be required. Although not expressly related to the hydraulic design, the project designer must identify constraints that may be imposed by these permits/approvals and consider them in the design process.

Typical examples of permits, by agency, include:

- Vermont ANR
  - *Title 19 Consultation;* required for road repairs or other VTrans construction involving streams and rivers.
  - *Construction General Permit (3-9020);* required for construction activity.
- Vermont Natural Resources Board (NRB)
  - *Act 250 Land Use Permit* (VT District Environmental Commissions); required for projects that alter existing land uses.
- U. S. Army Corps of Engineers (USACE)
  - *Section 10 of the Rivers and Harbors Act* and *Section 404 of the Clean Water Act;* required for projects impacting federal waters, including wetlands, resulting in land disturbance of one acre or more.
  - *General Permit (3-9015*); required for the expansion or redevelopment of impervious areas totaling one acre or more.

- National Environmental Policy Act (NEPA)
  - *Programmatic Categorical Exclusion (CE);* required evaluation for all federal actions/federally funding projects where no significant environmental impacts result
  - *Environmental Assessment (EA)* and *Environmental Impact Statement (EIS);* required evaluation for all federal actions/federally funded projects where significant environmental impacts will result
- Federal Highway Administration (FHWA)
  - *Department of Transportation Act, Section 4(f);* required to assess impacts on public parks, recreation lands, or wildlife and waterfowl refuges.
  - *National Historic Preservation Act, Section 106;* required to assess impacts on historic, cultural, and archaeological resources.

Project designers should contact the VTrans Environmental Specialist assigned to their project for additional information. Additional information regarding environmental permitting may be found in the VTrans Environmental Procedures Manual.

2.1.4 Potential Constraints

Identify potential constraints early in the design to avoid unnecessary redesign of project elements. Appropriate due diligence during the planning phase includes identifying the presence of concerns and constraints such as hazardous materials, soil/geological conditions, and historic, archaeological, cultural, or natural resources.

2.2 Design Considerations

2.2.1 Location Considerations

From a hydraulic perspective, the impact of highway infrastructure encroachment on a floodplain is typically the most important factor driving the selection of crossing structure locations in highway planning. The FHWA's 2001 Hydraulic Design Series (HDS) publication, HDS-6, *"River Engineering for Highway Encroachments,"* presents hydraulic and environmental considerations for highway river crossings and encroachments. HDS-6 provides sixteen hypothetical examples of typical river environments and identifies possible local, upstream, and downstream effects of highway encroachments.

Consider the principal factors in locating a stream crossing that involves encroachment within a floodplain, which are:

- River type—straight or meandering
- River characteristics—stable or unstable
- River geometry and alignment
- Hydrology
- Hydraulics
- Floodplain flow
- Needs of the area
- Economic and environmental concerns

A detailed evaluation of these factors is part of the site-specific hydraulics study. When a suitable crossing location has been selected, determine specific crossing components. When necessary, these include:

- The geometry and length of the approaches to the crossing
- Probable type and approximate location of the abutments or alignment of the culvert
- Probable number and approximate location of the piers
- The location of the longitudinal encroachment in the floodplain
- The amount of allowable longitudinal encroachment into the main channel
- The required river training works to ensure that river flows approach the crossing or the encroachment in a complementary way

Exact information on these components is usually not developed until the final stage, however, a planning-level understanding of the project site and potential project alternatives requires collection of preliminary data.

2.2.2 Interagency Coordination

Coordinate with concerned agencies during the project planning phase to produce a design that is more satisfactory to all. Coordinated planning among the federal, state, and local agencies engaged in water-related activities (such as flood control and water resources planning) can propagate substantial cost savings and other benefits. Interagency cooperation is an essential element for serving the public interests.

2.2.3 Intra-agency Coordination

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

2.3 Initial Data Review

2.3.1 Drainage Surveys

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

Topographic maps, aerial photographs, and stream flow records provide helpful preliminary drainage data, but historic high-water elevations and flood flows are of particular interest in establishing waterway requirements.

Comprehensive hydraulic investigations may be required when route selection involves important hydraulic features such as water-supply wells and reservoirs, flood control dams, water resource projects, and encroachment on floodplains of major streams. Ensure that special studies and investigations—including consideration of the environmental and ecological impact—are commensurate with the importance and magnitude of the project and the complexity of the problems encountered.

2.3.2 Data Collection

Obtain and evaluate several categories of data as part of planning and location studies, including:

- Physical characteristics of drainage basins
- Maps and topographic data, including channel surveys and cross sections
- Runoff quantity data—hydrologic and precipitation data
- Channel and floodplain delineations and related studies
- Flood history and problem inventory
- Existing stormwater management facility characteristics

2.3.3 Next Steps

During the planning and location assessment phases, begin to conceptualize design alternatives that could work with known site constraints. In addition, identify opportunities to incorporate multipurpose designs in appropriate areas.

Additional details associated with data collection, including types of data required, where to obtain data, and how to evaluate different sources of data, are outlined in Chapter 3 *"Data Collection, Resources, and Tools."*

Chapter 3 Data Collection, Resources, and Tools

3.1 Introduction

Data collection is a crucial part of any design project. Before beginning a project design, the design engineer should gather as much relevant material for the project as possible.

The purpose of this chapter is to outline the types of data that are normally required for hydraulic analysis and design and to provide information on several resources and tools to support the data collection process. Identify which types of data are required for a project prior to conducting the engineering analysis. Project data will be a compilation of different types and will need to be tailored to the specific project. Note that not all of the data discussed in this chapter will be needed for every project.

Data collection for a specific project must be tailored to the site conditions, the scope of the engineering analysis, and social, economic and environmental requirements. A well-planned data collection program leads to a more orderly and effective analysis and a design that is commensurate with project cost, project scope, complexity of site hydraulics, and regulatory requirements.

3.2 Required Data

3.2.1 Overview

A complete design requires the compilation of extensive site-specific data. Major types of data that are typically required to complete a design include:

- Watershed characteristics,
- Stream reach data (especially in the vicinity of the highway infrastructure),
- Hydrologic and meteorological data (stream flow and rainfall data related to maximum or historic peak as well as low flows and hydrographs applicable to the site),
- Existing and proposed land use data in the subject drainage area and in the general vicinity of the hydraulic structure,
- Anticipated changes in land use and/or watershed characteristics,
- Floodplain and environmental regulations, and
- Other physical data in the general vicinity of the facility such as utilities and easements.

To forecast flood flows using predictive methods, ensure the hydrologic characteristics of the watershed are accurately identified. Refer to Chapter 4 *"Hydrology"* for guidance about using these characteristics to calculate flood flows. Always conduct a field visit to confirm hydrologic data and to fill any knowledge gaps.

3.2.2 Watershed Characteristics

3.2.2.1 <u>Contributing Size</u>

Determine the size of the contributing drainage area using some or all of the following:

- Field surveys with conventional surveying instruments
- Topographic maps together with field checks, either through a stand-alone geographic information systems (GIS) analysis or by using U.S. Geological Survey (USGS) StreamStats
- Aerial maps, aerial photographs, or orthophotos
- Previous hydraulic studies

Consider areas that have runoff diverted into or out of the drainage area when delineating the contributing area for analysis.

3.2.2.2 <u>Topography</u>

Acquire topographic data for most sites requiring hydraulic studies. This data is needed so that analysis of existing flow conditions, as well as those caused by various design alternatives, can be performed. Locate and document the elevations of significant physical and cultural features in the vicinity of the project. Features such as residences, commercial buildings, schools, churches, farmlands, other roadways and bridges, and utilities can affect and be affected by the design of new hydraulic structures. Sometimes, recent topographic surveys will not be available at this early stage of project development, however aerial photographs, photogrammetric maps, USGS quadrangle sheets, and even old highway plans may be used during the planning and location phases. When better survey data becomes available, these early estimates will need to be revised to correspond with the most recent field information.

At the time of this writing, the Vermont Center for Geographic Information (VCGI) is in the process of developing state-wide Light Detection and Ranging (LiDAR) coverage that provides relatively high resolution topographic data that can be accessed using GIS software. VTrans has surveyed the interstate highway corridors throughout the state using this technology; however the width of the coverage may be too limited to be useful for hydraulic analysis.

Determine the slope of the stream channel, the average slope of the watershed (basin slope), and other characteristics of the terrain. Many of the hydrologic and hydraulic procedures in other chapters of this manual are dependent on watershed slopes and related physical characteristics.

3.2.2.3 Land Use

Define and document the present and expected future land use. Information on existing use and future trends may be obtained from:

- Aerial photographs
- Zoning maps and master plans
- USGS and other maps
- Municipal planning agencies
- Satellite imagery
- District environmental office

Existing land use data for small watersheds can best be determined or verified from a field survey. Use field surveys to update information on maps and aerial photographs, especially in basins that have experienced changes in development since the maps or photos were prepared.

3.2.2.4 Streams, Rivers, Ponds, Lakes, and Wetlands

Identify and acquire any information that is available for all streams, rivers, ponds, lakes, and wetlands that will affect or may be affected by the proposed structure or construction. These data are used to determine the expected hydrology and may be needed for regulatory permits. Examples include:

- Detailed descriptions of any natural or artificial spillway or outlet works, including dimensions, elevations, and operational characteristics.
- Profiles along the tops of any dams and typical cross sections of each dam.
- Notes about existing conditions with respect to turbidity and silt in the water bodies and wetlands.

Obtain the stream profile, horizontal alignment, and cross sections to perform an accurate hydraulic analysis. Data to this detail sometimes is not available during the planning and location phases. The designer must therefore make preliminary analyses based on data such as aerial photographs, USGS maps, and old plans. Additional information is available through the Vermont Agency of Natural Resources (ANR) Natural Resources Atlas,

The River Theme layer in the Natural Resources Atlas provides links to the Stream Geomorphic Assessments (SGAs) for most streams within the state. The final reports are also available through the SGA Data Management System (SGA-DMS). Phase I SGAs include general information about the stream bed material, size, and characteristics. More detailed Phase II SGA assessments are also available for some stream segments.

Photography can be useful in determining channel characteristics such as the type of material in stream beds and banks, type and coverage of vegetation, and evidence of drift, debris, or ice. Field visits made early in the project can include photographing the channel—including upstream and downstream reaches—and the adjoining floodplain. The photographs are usually valuable aids for conducting preliminary studies and documenting existing conditions. The use of Global Positioning System (GPS) enabled digital cameras provides additional information by documenting the location of each photograph, allowing the photographed features to be easily located on a map.

Determine the detail of field survey required at the site during the early phases of project development. This should include the upstream and downstream limits of the survey, the number of or distance between cross sections, and how far to either side of the channel the sections should extend. The number of cross sections that are necessary will vary with the study requirements and the particular stream characteristics. Consider the level of accuracy needed from the survey when determining the degree of hydraulic analysis needed. The U.S. Army Corps of Engineers (USACE) has made a detailed study of survey requirements in *"Accuracy of Computed Water Surface Profiles."*

3.2.2.5 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:

- Information to define the environmental sensitivity of the site location relative to impacted surface waters or wetlands. Some of this information is

available in the water quality standards and criteria published by ANR. Types of relevant information include:
  - o Water use, quality, and standards
  - o Aquatic and riparian wildlife biology
  - o Wetlands information
- Information to define the need for environmental mitigation measures. Fish and fish habitat information is available from ANR and the U.S. Fish and Wildlife Service (USFWS). Types of relevant information include:
  - o Fish characteristics (type, size, migratory habits)
  - o Fish habitat (depth, cover, pool-riffle relationship)
  - o Water use, water quality, and related standards
- Information about historical sensitivity, if applicable.

Wetlands are unique, and data needs can be identified through coordination with ANR, USACE, and USFWS.

3.2.3 Site Characteristics

Good hydraulic design requires a complete understanding of the physical nature of the natural channel or stream reach—particularly at the site of interest. Study any work being performed, proposed, or completed that changes the hydraulic efficiency of a stream reach to determine its effect on the stream flow. Be aware of plans for channel modifications and any other changes that might affect the facility design. The stream may be classified as:

- Rural or urban
- Narrow or wide
- Rapid or sluggish flow
- Stable, transitional, or unstable
- Sinuous, straight, braided, alluvial, or incised

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

- Sediment transport and related data
- Stability of form over time (braided, meandering, etc.)
- Scour history/evidence of scour
- Bed and bank material identification

3.2.3.1 Roughness Coefficients

Estimate roughness coefficients, ordinarily in the form of Manning's n values, for the entire flood limits of the stream. A tabulation of Manning's n values with descriptions of their applications can be found in Appendix A *"Manning's n Values."*

3.2.3.2 Stream Profile

Obtain stream bed profile elevations for the site. Ensure this data extends sufficiently upstream and downstream to determine the average slope and to encompass any proposed construction or aberrations.

If the project complexity warrants the creation of a detailed hydraulic model such as with USACE's Hydrologic Engineering Center's River Analysis System (HEC-RAS), the longitudinal profile should extend far enough upstream and downstream to mitigate any potential rise in water surface elevation (WSE) resulting from the proposed design. This may not be possible in very flat gradient streams, but the attempt should be made whenever possible. At the very least, the longitudinal profile must extend far enough upstream and downstream from the proposed crossing in order to adequately model the expansion and contraction reaches. Additional details on HEC-RAS analysis are provided in Chapter 6 *"Crossing Structures."*

3.2.3.3 Stream Cross Sections

Obtain stream cross-section data that will represent typical conditions at the structure site as well as other locations where stage-discharge and related calculations will be necessary.

Federal regulations addressing ordinary high water (OHW) and bankfull width (BFW) under the Clean Water Act identify the lateral extents of non-tidal streams in 33 CFR 328.3(e) as follows:

> "*The term ordinary high water mark means that line on the shore established by the fluctuations of water and indicated by physical characteristics such as a clear, natural line impressed on the bank, shelving, changes in the character of soil, destruction of terrestrial vegetation, the presence of litter and debris, or other appropriate means that consider the characteristics of the surrounding areas.*"

BFW (the channel width at bankfull stage) may share many of the same physical characteristics as OHW, but it is defined according to the depth of flow associated with the statistically-determined 67% to 50% annual exceedance probability (AEP) events, which correspond to 1.5 to 2-year recurrence interval (RI) events, respectively. The limit of bankfull stage is commonly identified in the field as the flow that just fills the natural channel to the top of its banks and to the point where water begins to overflow onto the active floodplain. Identify BFW at multiple locations within the vicinity of the project site but away from unusual constrictions or impoundments that change the character of the stream.

Whereas OHW is a regulatory threshold that must be considered as part of the permitting process, BFW is an important physical parameter that must be considered throughout the hydraulic design process. As an initial estimate, the designer can approximate BFW using the watershed area and the Vermont Regional Hydraulic Geometry Curves. However, the BFW should subsequently be verified in the field because the curves are not applicable to all drainage areas.

If the project complexity warrants the creation of a detailed hydraulic model, such as with HEC-RAS, extend the cross-section survey laterally and vertically to encompass the largest area of flooding that would be anticipated under the analysis. Additional details on HEC-RAS analysis are provided in Chapter 6 *"Crossing Structures."*

3.2.3.4 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. Note the manner in which existing structures have been functioning with regard to such things as scour, overtopping, debris and ice passage, fish passage, etc. Include span lengths, types of piers, and substructure orientation with bridges. The necessary culvert data includes other things such as size, inlet and outlet geometry, slope, end treatment, culvert material, and flow line profile. Photographs and high-water profiles or marks of flood events at the structure and past scour event data can be valuable in assessing the hydraulic performance of the existing facility.

Construction and as-built plans for many VTrans facilities are available electronically through the VTrans website.

The characteristics of existing structures on the stream under study can be a valuable indicator when selecting the size and type of any new structure. Indications that a structure is too small for a site include scour holes, erosion around the abutments or just upstream or downstream, or abrupt changes in material gradation or type. With knowledge of flood history, the age and overall substructure condition may also aid in determining if the structure is too small. If a structure is relatively new, information may still be available on the structure and why it was replaced.

Although crossings are normally replaced due to poor structural conditions, sometimes other underlying conditions—often hydraulic in nature—also enter into the decision to build a new structure. The durability of the existing structure may indicate how well the proposed structure will fare at this location. Old plans may contain high-water or flood information that can be of use. Collect information about crossing structures located upstream and downstream of the site under study (if such structures exist) for the factors just discussed.

3.2.3.5 Acceptable Flood Levels

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

3.2.3.6 Flood History

The collection of flood data is a basic survey task in performing any hydraulic analysis. These data can be collected both in the office and in the field. Office data collection includes the acquisition of past flood records, stream gaging records, and published accounts. The field collection consists mainly of interviews with residents, maintenance personnel, and local officials who may have recollections or photos of past flood events in the area. In some cases, a stream gaging station may be present on the stream under study in close proximity to the crossing site. If the gage has many years of measurements, it may be the only hydrologic data needed. Analyze these to ensure stream flows have not changed over the time of measurement due to watershed alterations such as the construction of a large storage facility, diversion of flow to another watershed, addition of flow from another watershed, or development that has significantly altered the runoff characteristics of the watershed.

The history of past floods and their effect on existing structures is critical in producing flood hazard evaluation studies and sizing structures. Evaluate changes in channel and watershed conditions since the occurrence of a flood to relate historical floods to present conditions.

Recorded flood data are available from sources such as:

- Federal Emergency Management Agency (FEMA)
- USACE
- USGS
- State libraries (newspapers, town, county, and state histories, historical accounts, etc.)

Sometimes high-water marks provide the only available data about past floods. When this is the case, include the date and elevation of the flood event whenever possible. The cause of the high-water mark should also be noted. Often the mark is caused by unusual debris or ice jams rather than an inadequate structure. Designing roadway or structure grades to such an elevation could lead to an unrealistic, uneconomical design. High-water marks can be identified in several ways. Small debris—such as grass or twigs caught in tree branches, hay or crops matted down, and mud lines on buildings or bridges are all high-water indicators. Beware, however, that grass, bushes, and tree branches bend over during flood flows and spring up after the flow has passed, which may give a false reading of the high-water elevation. Ice will often cut or gouge into the bark of trees, indicating high-water elevations.

3.2.3.7 Debris and Ice

Investigate the quantity and size of debris and ice carried or available for transport by a stream during flood events for use in the design of structures. 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. The USACE Cold Regions Research and Engineering Laboratory (CRREL) conducts multiple ice jam studies across the State of Vermont and is a valuable resource for obtaining ice jam data.

3.2.3.8 Scour Potential

Scour potential is an important consideration relative to the stability of the structure over time. Scour potential is determined by a combination of the stability of the natural materials at the hydraulic structure, tractive shear force exerted by the stream, and sediment transport characteristics of the stream. Bed and bank material data for classifying channel type, stability, and gradations may be required. These data will help to determine a reliable Manning's n as well as a scour estimate. Additional information on scour analysis is presented in Chapter 7 *"Channel Stability and Scour at Bridges."*

3.2.3.9 Controls Affecting Design Criteria

Many controls will affect the criteria applied to the final design of hydraulic structures, including allowable headwater level, allowable flood level, allowable velocities and resulting scour, and other site-specific considerations. Obtain data and information related to such controls from site investigations and federal, state, and local regulatory agencies to determine what natural or man-made controls to consider in the design. In addition, document any downstream and upstream controls as applicable.

- *Downstream Control:* Note any ponds or reservoirs and their spillway elevations and design levels of operation. Their effect on backwater and/or stream bed aggradation may directly influence the proposed structure. Study any downstream confluences of two or more streams to determine the effects of backwater or streambed change resulting from that confluence.
- *Upstream Control:* Note runoff in the watershed. Conservation and/or flood control reservoirs in the watershed may reduce peak discharges at the site and may also retain some of the watershed runoff. Obtain capacities and operational designs for these features if possible. The Natural Resources Conservation Service (NRCS), USACE, consulting engineers, and other reservoir sponsors often have complete reports concerning the operation and design of proposed or existing conservation and/or flood control reservoirs. The redirection of flood waters can significantly affect the hydraulic performance of a site. Some actions that redirect flows are debris jams, ice jams and highways or railroads.

3.3 On-Site Data Collection

Complete and accurate on-site data collection is integral to developing a design that will best serve the requirements of a site. The individual(s) in charge of field inspection and survey must have a general knowledge of drainage design and coordinate thoroughly with the designer. Designers can often interpret published sources of data much more quickly and easily once they have working knowledge of the site.

The amount of on-site data and survey gathered should be commensurate with the importance and cost of the proposed structure and the expected flood hazard. In particular, extend survey cross sections far enough from the top of the bank to allow hydraulic modeling to identify the extent of the floodplain adjacent to the stream and far enough upstream and downstream from the crossing site to encompass the contraction and expansion reaches. Whenever possible, coordinate field inspection and survey to avoid repeat visits.

Proceed with the design of the hydraulic structure only after the collection of required information has been analyzed and a thorough study of the area has been completed. Document all pertinent data and facts gathered through the survey as explained in Chapter 9 *"Documentation."*

3.3.1 Field Inspection

The most complete survey data cannot adequately depict all site conditions or substitute for personal inspection by someone experienced in hydraulic design. Factors that most often need to be confirmed by field inspection are:

- Contributing drainage area characteristics and general ecological information
- Observation of land use and related flood hazards
- Evaluation of apparent flow direction and diversions
- Flow concentration
- High-water marks or profiles
- Selection of roughness coefficients
- Geomorphic relationships
- Drift/debris characteristics

Visit the site where the project will be constructed before undertaking any detailed hydraulic design. The designer may choose to have a joint site visit with others, such as the roadway and structural designers, environmental reviewers, and local officials. In other situations, the designer may choose to visit the site separately in order to collect data.

Prior to the field visit, determine what kind of equipment should be taken, and most importantly, what the critical items to be reviewed at the site are. Plan to take photographs. At a minimum, photos should be taken looking upstream and downstream from the site as well as along the contemplated highway centerline in both directions. Take detailed photographs of the stream bed and banks and the structures in the vicinity both upstream and downstream.

3.3.2 Hydraulic Surveys

Survey requirements for small crossing structures are less extensive than those for major facilities such as bridges. However, the purpose of each survey is to provide an accurate picture of the conditions within the zone of hydraulic influence of the crossing structure. Obtain or verify the following data:

- Stream reach data—cross sections and thalweg profile
- Existing structure(s) at project site, if applicable
- Upstream or downstream structures

Make use of existing data available from ANR and other state agencies.

- SGAs have been completed for many streams and rivers and are available through the ANR Natural Resources Atlas and the SGA Data Management System.
- Vermont Online Bridge and Culvert Inventory
- VTrans Small Culvert Inventory
- VTrans Culvert Inventory

3.3.3 Documentation

Forms that can be used in identifying and cataloging field information can be found in Appendix B *"Field Investigation Forms"* and include:

- Form 1: Field Investigation Form
- Form 2: Hydraulic Survey Field Inspection Checklist

The criteria that are evaluated during the field visit may be required at different points of the hydraulic design process and will ultimately be incorporated in the Final Hydraulics Report and on the Project Information Sheet that is included with the Project Plans. The Final Hydraulics Report Form is provided in Appendix C *"Hydraulics Form."*

3.4 Data Evaluation

3.4.1 Objective

After the necessary data have been collected, compile data into a usable format. The designer must ascertain 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.

3.4.2 Evaluation

Experience, knowledge, and judgment are important parts of data evaluation. In this phase, the designer should separate reliable data from data that is less reliable and combine historical data with data obtained from measurements. Evaluate the data for consistency and to identify any changes from established patterns. Review previous studies, old plans, etc., for types and sources of data, how the data were used, and any indications of accuracy and reliability. Review historical data to determine whether significant changes have occurred in the watershed and whether these data can be used.

Data acquired from the publications of established sources, such as the USGS, can usually be considered valid and accurate. However, the designer should always vet data by reading qualifying statements associated with a publication. Typical qualifying statements pertain to the impacts of regulation on a watershed, the effects of ice, the quality of different resolution data, and the movement of data collection equipment.

Carefully study the data for accuracy and reliability. Evaluate and summarize basic data, such as stream flow data derived from non-published sources, before use. Compare maps, aerial photographs, satellite imagery, and land use studies with one another and with the results of the field survey and resolve any inconsistencies. 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.

3.4.3 Sensitivity

Use sensitivity studies to evaluate data and identify the 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. Sensitivity studies establish the effect of the data on the final design. This is useful in determining what specific data items have major effects on the final design and the importance of possible data errors. Spend more time and effort on more sensitive data items, making sure these data are as accurate as possible.

Use the results of this type of data evaluation to generate a reliable description of the site within the allotted time and using the resources committed to this effort. The effort of data collection and evaluation should be commensurate with the importance and extent of the project and/or facility.

3.5 Resources

Resource materials providing guidance on the subjects of hydrology and hydraulics for transportation design are widely available from federal, state, academic, and private sources. The following list covers just a few of the most applicable publications.

3.5.1 Federal Highway Administration

3.5.1.1 Hydraulic Design Series

The Federal Highway Administration (FHWA) Hydraulic Design Series (HDS) publications provide information about engineering principles and practices related to highway hydrology, including hydraulic design for highway culverts, highway bridges, and highway encroachments. Refer to the complete list of FHWA Current Hydraulic Engineering Publications to find the most applicable HDS publication.

3.5.1.2 Hydraulic Engineering Circular

The FHWA's Hydraulic Engineering Circular (HEC) publications provide information about design principles and engineering techniques related to stream stability, bridge scour, culverts, channels, drainage, and encroachments. Refer to the complete list of FHWA Current Hydraulic Engineering Publications to find the most applicable HEC publication.

3.5.2 American Association of State Highway Transportation Officials

The American Association of State Highway and Transportation Officials (AASHTO) 2007 publication "*Highway Drainage Guidelines*" provides a general overview of hydrology related to highway engineering. This publication is not currently available for free online viewing.

3.5.3 Natural Resources Conservation Service

The NRCS's National Engineering Handbook (NEH) *"Part 654, Stream Restoration Design,"* includes technical descriptions of processes that affect rivers and streams and descriptions of techniques to stabilize systems undergoing change.

3.5.4 Vermont Agency of Natural Resources

3.5.4.1 Vermont Fish and Wildlife Department

The Vermont Fish and Wildlife Department (FWD) publication entitled *"Guidelines for the Design of Stream/Road Crossings for Passage of Aquatic Organisms in Vermont"* provides information about aquatic organism passage (AOP) and stream continuity.

3.5.4.2 Vermont Department of Environmental Conservation

The Vermont Department of Environmental Conservation (DEC) publication entitled *"Vermont Standard River Management Principles and Practices: Guidance for Managing Vermont's Rivers Based on Channel and Floodplain Function"* (VT SRMPP) provides guidance focused on the channel and floodplain considerations.

The ANR's *"Vermont Stream Geomorphic Assessment Handbooks,"* offer separate but interrelated phases for examining and evaluating geomorphic (stream stability) and habitat conditions of a streams. The handbooks include:

- Phase 1, *"Watershed Assessment: Using Maps, Existing Data, and Windshield Surveys"*
- Phase 2, *"Rapid Stream Assessment: Field Protocols"*
- Phase 3, *"Survey Assessment: Field and Data Analysis Protocols"*

Each phase requires progressively more elaborate data collection and analysis. Analyses should progress through the phases until the appropriate level of detail is achieved for a specific crossing design.

3.6 Data Sources

Designers have an array of sources from which to gather necessary data and information. The designer must use experience and judgment when weighing the accuracy and reliability of these sources, but FEMA and VCGI are two of the most commonly used, publically available sources.

3.6.1 Federal Emergency Management Agency

FEMA developed the Map Service Center to provide information about Flood Insurance Studies (FISs) and Flood Insurance Rate Maps (FIRMs), which FEMA produces to summarize estimated flood conditions in communities. Historically, FISs were issued for individual communities. Going forward, FISs will be issued on a county-wide basis.

3.6.2 U.S. Army Corps of Engineers

The USACE provides technical references and support for ice jam analyses via the CRREL.

3.6.3 Vermont Center for Geographic Information

The VCGI is Vermont's clearinghouse for GIS data. They host the following data that may be of use to the designer:

- Geospatial data and imagery
- Regional and local GIS data
- Links to the Interactive Map Viewer
- Dynamic online mapping tools
- Information about geospatial technology

3.7 Data Tools

Designers have an array of tools at their disposal to help interpret and analyze the collected data, some of which are listed below. The designer must use experience and judgment when weighing the accuracy and reliability of these tools.

3.7.1 U.S. Geological Survey

3.7.1.1 Peak FQ

Peak FQ is a tool developed by the USGS for processing USGS stream gage data using methods described in Chapter 4 *"Hydrology."* Identify the stream gage of interest and manually download the annual peak data in the required format from the USGS National Water Information System (NWIS).

3.7.1.2 National Streamflow Statistics

The National Streamflow Statistics (NSS) program, a software package developed and distributed by the USGS, provides a user interface for accessing the regional regression equations for all 50 states (including Vermont) and selected territories. The NSS program provides estimates of streamflow statistics for rural, ungaged basins; flood magnitudes by frequency for sites in urbanized basins; errors associated with the streamflow statistics; maximum floods; hydrographs; improved flood-frequency estimates for gaging stations by weighting estimates with systematic flood records; and improved flood-frequency estimates for ungaged sites by weighting estimates with a flow per unit area from measured sites. The program also allows the user to manipulate the appearance of input and output tables and graphs and save the data in text and graphic files.

3.7.1.3 StreamStats

StreamStats is a web-based GIS tool that facilitates implementation of the USGS hydrologic regression equations. Part of the more general functionality of StreamStats involves a tool for delineating a watershed to a selected point on the map. This tool is a useful aid for many of the hydrologic methods presented in Chapter 4 *"Hydrology."* Once a point is identified, StreamStats uses topographic data from the USGS National Elevation Dataset (NED) at a resolution of 10 meters to calculate and delineate a watershed. The resulting watershed can be modified in StreamStats and exported to a GIS shapefile. Be sure to adhere to the cautionary message below when using StreamStats.

> **Caution!**
>
> Use caution with the StreamStats watershed delineation tool. StreamStats delineates contributing areas using 10-meter topographic data originating from the USGS NED. Verify the watershed delineation independently using the best available data and adjust the watershed boundaries prior to exporting results from StreamStats.

3.7.2 U.S. Army Corps of Engineers

3.7.2.1 HEC-SSP

Hydrologic Engineering Center Statistical Software Package (HEC-SSP) processes stream gage data from the USGS consistent with the hydrologic method described in Bulletin 17B. HEC-SSP can also perform (1) generalized frequency analyses on other types of hydrologic data, (2) volume frequency analyses on high and low flows, (3) duration analyses, (4) coincident frequency analyses, and (5) curve combination analyses.

3.8 Design Tools

Design tools help the designer transform raw data into something that can be interpreted and ultimately constructed to fulfill the goals of a project. The general design platforms described in this section, computer-aided design (CAD) and GIS, are not specific to hydraulics, but their wide range of capabilities make them powerful tools for transferring data between programs for analysis, integrating data and spatially representing analyzed data, and preparing final design products that are polished and visually appealing.

The remainder of the listed design tools are specifically for hydraulic analysis and design. They are organized according to the agency that supports them.

3.8.1 General Design Platforms

3.8.1.1 Computer-Aided Design (CAD)

CAD software has powerful computational capabilities that streamline and expedite data acquisition and analysis that would otherwise have been done by hand. Survey information taken in the field can be easily digitized and modeled in CAD. For recommendations on the use of CAD in hydrologic and hydraulic analysis, see Chapter 5 *"Open Channels."*

3.8.1.2 Geographic Information Systems (GIS)

GIS software can expedite desktop review and analysis of spatial data and provide a platform for integrating data between supported programs. The vast array of publicly-available GIS data has greatly streamlined and simplified the preliminary planning of most projects. Statewide coverages (or data layers) are available for most types of data that are required. These coverages are described in the chapters of the manual to which they pertain, along with the static data sources that they supplement or replace.

The use and manipulation of GIS data assumes a detailed understanding of the strengths and limitations of different data sources. Make sure to correctly determine the projection of any coverage and check that GIS is correctly processing sources of data that are in different projections.

Review the metadata included with data layers provided by others to determine if the data is suitable for the type of analysis being performed. Layers created during the GIS analysis process must be supplied with metadata in order for users and reviewers to be able to identify the source of the information and to determine if it is suitable for the intended use.

3.8.2 Federal Highway Administration

3.8.2.1 Hydraulic Toolbox

The current version of the FHWA Hydraulic Toolbox includes twelve calculators for evaluating systems typical to hydraulic design for highway applications. VTrans recommends that users of the FHWA Hydraulic Toolbox verify their results with manual calculations and engineering judgment to validate the performance of the calculator.

3.8.2.2 HY-8

The HY-8 software was developed by FHWA to perform one-dimensional steady-state culvert hydraulics calculations. The user can enter, edit, and save culvert and channel data for one or more crossings. The program computes the culvert hydraulics for circular, rectangular, elliptical, arch, and user-defined geometry. The output from the program can be printed or exported and incorporated directly into a hydraulic report.

The procedure for using the HY-8 program is similar to that for using other culvert design methods. Hydrologic data for the contributing watershed must be calculated separately and input into the model. The program calculates and compares the headwater elevations for both inlet and outlet control. The program selects the higher of the two elevations as the control elevation and considers the effects of tailwater when calculating these elevations. If the controlling headwater elevation would overtop the roadway embankment, the program performs an overtopping analysis whereby the flow is balanced between the culvert discharge and the surcharge over the roadway.

3.8.3 U.S. Army Corps of Engineers

3.8.3.1 HEC-HMS

Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) is designed to simulate the complete hydrologic processes of dendritic watershed systems.
The process includes many traditional hydrologic analysis procedures such as event infiltration, unit hydrographs, and hydrologic routing. HEC-HMS also has procedures necessary for continuous simulation including evapotranspiration, snowmelt, and soil moisture accounting.

3.8.3.2 HEC-RAS

HEC-RAS is recommended as a computational tool for performing step-backwater analysis. The software was developed specifically to perform one-dimensional steady-state and unsteady-state flow hydraulics calculations for open channels.

The most recent version of the software (version 4.1) includes capabilities to model sediment transport (mobile bed modeling) and water temperature analysis. HEC-RAS also includes capabilities to perform hydraulic modeling at crossing structures (see Chapter 6 *"Crossing Structures"*).

Quick Tip

At the time of writing, HEC-RAS version 5.0 is currently in its beta phase of release. This version of HEC-RAS will reportedly support 2D hydrodynamic flow routing within the unsteady flow-routing analysis. Designers will be able to model 2D flow areas independently or in conjunction with 1D flow areas. Refer to Chapter 6, *"Crossing Structures"* for more information about two-dimensional models.

3.8.3.3 HEC-GeoRAS

HEC-GeoRAS is a set of tools providing the capability to geo-reference a hydraulic model through interface with digital terrain models (DTMs) and GIS software. Geo-referencing HEC-RAS models provides many advantages in model development, use, review, and re-use, including:

- Realistic representation of model inputs in user interface;
- Increased efficiency and accuracy in model geometry development;
- Reduced ambiguity regarding locations of model elements (i.e. river centerline, cross section cut lines tied to a horizontal datum); and
- Facilitated mapping of model results.

The GIS toolset allows the user to create model inputs in a map-based, graphical interface by overlaying 2-dimensional flow paths, cross sections, and banks over 3-dimensional topographic data.

3.8.4 U.S. Department of the Interior

3.8.4.1 SRH-2D

The U.S. Bureau of Reclamation (USBR) has developed a two-dimensional hydraulic, sediment, temperature, and vegetation model for river systems called Sediment and River Hydraulics – Two Dimensional (SRH-2D).

As stated on the USBR website for the model, "SRH-2D is a 2D model, and it is particularly useful for problems where 2D effects are important. Examples include flows with in-stream structures, through bends, with perched rivers, with side channel and agricultural returns, and with braided channel systems. A 2D model may also be needed if one is interested in local flow velocities, eddy patterns, flow recirculation, lateral velocity variation, and flow over banks and levees."

For situations with significant two-dimensional flow in the horizontal plane, relying on a one-dimensional analysis may lead to improper or costly over-design of a hydraulic structure. Obtain approval in writing from the VTrans Hydraulics Engineer before performing two-dimensional modeling. Refer to Chapter 6 *"Crossing Structures"* for more information about two-dimensional models.

3.8.5 U.S. Forest Service

3.8.5.1 FishXing

FishXing is a free software tool that provides assistance with the evaluation and design of culverts for fish passage.

Chapter 4 Hydrology

4.1 Introduction

4.1.1 Overview

Hydrology is the study of the rain cycle and the distribution of water in the atmosphere and on the earth. For the purpose of this manual, a hydrologic study is an approximation of the complex relationship between the precipitation that falls on a drainage basin and the surface water that runs off of the basin. Runoff is typically defined by three characteristics, (1) peak rate, (2) total volume, and (3) distribution over time.

Stormwater conveyance and storage infrastructure are designed to safely and cost-effectively manage runoff. Poor understanding of runoff characteristics can have a significantly adverse impact on the design and performance of highway structures. Under-designed structures often have low initial cost and potentially exorbitant long-term costs. Overdesigned structures may be cost-prohibitive. Proper hydrologic analyses support the design and construction of safe, cost-effective solutions for transportation infrastructure.

Factors that influence the rainfall-runoff response within a watershed include:

- Contributing drainage area
  - Size
  - Shape (rounded, elongated)
  - Drainage pattern (radial, dendritic, parallel, trellis)
  - Orientation (north-south, east-west)
  - Elevation range
- Precipitation characteristics
  - Depth and intensity
  - Type (rain, snow, hail, mixed)
- Ground conditions
  - Land use (pervious, impervious)
  - Antecedent moisture
  - Frozen or thawed
- Soils
- Slopes of terrain and streams
- Storage
  - Ponds, lakes, reservoirs
  - Wetlands
  - Channels
  - Floodplains
- Potential for Watershed Development

The need for and the availability of the data on the watershed characteristics listed here will vary from site to site. The design engineer is responsible for determining what information is available and applicable to a particular analysis. For a comprehensive list of data sources and availability, refer to Chapter 3 *"Data Collection, Resources, and Tools."*

4.1.2 Resources

4.1.2.1 <u>Federal Highway Administration</u>

For more information about engineering principles and practices related to hydrology for transportation and highway infrastructure, refer to the Federal Highway Administration (FHWA) Hydraulic Design Series (HDS) publication, HDS-2, *"Highway Hydrology."*

4.1.2.2 <u>American Association of State Highway Transportation Officials</u>

The American Association of State Highway and Transportation Officials (AASHTO) 2007 publication "*Highway Drainage Guidelines*" provides a general overview of hydrology related to highway engineering. This publication is not currently available for free online viewing.

The FHWA and AASHTO publications also reference research and hydrologic studies that can provide the engineer with more in-depth guidance on specific aspects of hydrologic analysis.

4.1.3 Data Sources

4.1.3.1 <u>Federal Emergency Management Agency</u>

The Federal Emergency Management Agency (FEMA) has produced studies, reports, and web tools that may be used to assess national and local hydrologic conditions. These resources are free to the public and, when used effectively, they can save time and streamline effort.

FEMA's Map Service Center provides information about Flood Insurance Studies (FISs) and Flood Insurance Rate Maps (FIRMs), which FEMA produces to summarize estimated flood conditions in communities. Historically, FISs were issued for individual communities. Going forward, FISs will be issued on a county-wide basis. The studies follow a typical format:

Section 1-	Introduction
Section 2-	Area Studied
Section 3-	Engineering Methods
Section 4-	Flood Plain Management Applications
Section 5-	Insurance Application
Section 6-	FIRMs
Section 7-	Other Studies
Section 8-	Location of Data
Section 9-	Bibliography

Supplemental Data:

Summary of Discharges Table
Floodway Data Tables
Flood Profiles

Be sure to review the FIS applicable to the study area to:

- Determine if the area of interest is within an area studied by approximate or detailed methods.
- Determine the methods used to estimate the hydrologic conditions at the area of interest.
- Determine the date the study was completed.

Perform hydrologic analyses consistent with the methods described in this chapter and compare the results to the information published in the FIS. Keep in mind that the older the FIS report is, the less likely it is to accurately depict existing conditions and adhere to up-to-date modeling techniques. Refer to Chapter 5 *"Open Channels"* and Chapter 6 *"Crossing Structures"* for information about referencing FEMA FIS data during the hydraulic analysis.

4.1.3.2 Vermont Center for Geographic Information

The Vermont Center for Geographic Information (VCGI) is Vermont's clearinghouse for geographic information system (GIS) data. They host the following data that may be of use to the designer:

- Geospatial data and imagery
- Regional and local GIS data
- Links to the Interactive Map Viewer
- Dynamic online mapping tools
- Information about geospatial technology

4.1.4 Data Tools

4.1.4.1 StreamStats

The U.S. Geological Survey (USGS) has developed a web-based GIS tool named StreamStats to facilitate implementation of the USGS Regression Equation Method, discussed in Section 4.7.

Part of the more general functionality of StreamStats involves a tool for delineating a watershed to a selected point on the map. This tool is a useful aid for many of the hydrologic methods presented in this chapter. Once the point is identified, StreamStats uses topographic data from the USGS National Elevation Dataset (NED) at a resolution of 10 meters to calculate and delineate a watershed. The resulting watershed can be modified in StreamStats and exported to a GIS shapefile. Be sure to adhere to the cautionary message below when using StreamStats.

Caution!

Use caution with the StreamStats watershed delineation tool. StreamStats delineates contributing areas using 10-meter topographic data originating from the USGS NED. Verify the watershed delineation independently using the best available data and adjust the watershed boundaries prior to exporting results from StreamStats.

4.2 Design Frequency

4.2.1 Terminology

Hydrologic analysis forms the basis of most hydraulic designs, and statistical probability is a cornerstone of most hydrologic analyses. Statistical concepts can be difficult to understand, and the subject is often further confused by the wide array of terminology that is used within the hydrologic specialty. In order to be as clear and consistent as possible, this section provides an overview of some widely used terms and lays out how they are used in this manual.

Different types of transportation infrastructure are designed to provide different levels of service. For example major highways, often designated as evacuation routes, are designed to be passable during (and following) infrequent, extreme weather events. The "frequency" of an event refers to the average interval of time expected to pass before an event of the same magnitude occurs again. The frequency may also be referred to as the "return period" or the "recurrence interval" (RI). The likelihood that a flood will occur in any given year is the reciprocal of the RI and is referred to as the "annual exceedance probability" (AEP), which is given as a percent.

It is not generally economical to build local roads to the same hydraulic standards as major highways, so each roadway classification has an associated minimum “design frequency,” which refers to an event with a designated AEP that the roadway crossing must be hydraulically capable of conveying without flooding and becoming impassable. Section 4.2.3 specifies the minimum design frequency for roadways of differing classifications. Design frequency is a statistical term, and its physical counterpart is the “design event.” A design event is a representation of natural processes that have the potential to affect the performance and use of an engineered structure. Generally speaking, when the term “design” is used to introduce a compound term in hydrologic discussion within this manual, it ties back to the design frequencies by roadway classification presented in Section 4.2.

In the practice of hydrologic and hydraulic modeling for bridge and culvert design, each design event can be further broken down into the “design storm” (or generically “storm event”), which refers the meteorological components, and the “design flood” (or generically “flood event”), which refers to a river or stream’s response to the storm event.

In hydrologic modeling, the designer defines the depth, duration, and distribution of the precipitation associated with the storm event and estimates the water levels and flow rates produced by the storm that dictate the flood event. The flow rates that are determined in response to the storm event are often generically referred to as “flood flows.”

Additional terminology comes into play in discussions of scour analysis and design. Scour-related concepts are fully presented in Chapter 7 *“Channel Stability and Scour at Bridges.”* In bridge design, flooding of the roadway is not the only concern. Scouring of the bed material can expose foundations and compromise the structural stability of a bridge. Thus, scour analysis warrants a more conservative approach, and “check” terminology is introduced in place of the design terminology. The “check frequency” and corresponding “check event” are analyzed to ensure that foundation components can remain free standing (i.e. able to support the bridge) in the event that the “check storm” comes to pass.

4.2.2 Calculating Annual Exceedance Probabilities and Recurrence Intervals

As discussed in Section 4.2.1, AEP is the reciprocal of RI and is expressed as a percent. In this manual, AEPs will be presented instead of RIs because they are more statistically precise and are increasingly supplanting RIs in newer publications. However, it is still convenient to be able to easily convert between the two, and the equation for performing that conversion is given by:

$$AEP = \frac{1}{RI} * 100$$

Where:
AEP = annual exceedance probability, %
RI = recurrence interval, years

Table 4-1presents the conversions between common AEPs and RIs. AEPs and RIs are not statistical guarantees—nothing precludes a 1% AEP event from occurring multiple times over a 100-year period or even over the course of a single year.

Table 4-1. Annual Exceedance Probabilities and Recurrence Intervals

Annual Exceedance Probability	Recurrence Interval
50%	2 years
10%	10 years
4%	25 years
2%	50 years
1%	100 years
0.5%	200 years
0.2%	500 years

4.2.3 Design Frequencies by Roadway Classification

As discussed in Section 4.2.1, different roadway classifications are designed to convey floods of different frequencies. Table 4-2 presents the design frequencies for various types of VTrans infrastructure, as defined by the *Vermont State Design Standards*. New structures should always meet the minimum design frequency, but existing and/or rehabilitated structures sometimes are not able to meet the minimum design frequency for various reasons. Designs that exceed the minimum criteria—and are therefore designed to withstand events less frequent (more intense) than those presented in Table 4-2—are acceptable provided that the design does not cause unintended negative impacts to the ecology and surrounding property. The VTrans Hydraulics Engineer must review and approve any designs that deviate from the specified design frequency in order to verify that designs are safe and cost-effective.

Table 4-2. Minimum Design Frequency by Roadway Classification

Roadway Classification	AEP (%)
Freeways	1%
Principal arterial roads and streets	2%
Minor arterial roads and streets	2%
Collector roads and streets	2%
Local roads and streets	4%
Limited access roadways	*
Roadside, median, and storm drain systems	**
Railroads	2%

* *At the discretion of the VTrans Hydraulics Engineer.*

** *Refer to Chapter 8 "Storm Drainage Systems."*

Maps illustrating VTrans roadways and their current classifications are available at:

- VTrans Highway Maps
- VCGI: To download a statewide shapefile for use with GIS software, select the Theme "Transportation Networks" and navigate alphabetically through the listed items to TransRoad_FUNCCLASS.

While all structures and conveyances must be designed for the minimum design frequency, they must also be analyzed for acceptable performance during extreme storm event conditions, as well as during frequent, low intensity storm events. VTrans recommends that designers evaluate a range of standard AEP storm events in addition to the design event. These additional calculations are typically quick to perform and supply important flood information for use during the hydraulic analysis. Standard AEP events, from smallest storm to largest storm, include:

- 50% AEP (2-year RI)
- 43% AEP (2.33-year RI)
- 20% AEP (5-year RI)
- 10% AEP (10-year RI)
- 4% AEP (25-year RI)
- 2% AEP (50-year RI)
- 1% AEP (100-year RI)
- 0.5% AEP (200-year RI)
- 0.2% AEP (500-year RI)

The 43% AEP event, discussed in Section 4.2.5, is used to approximate the mean annual flow rate in a channel. Additionally, regardless of the design event, evaluate all proposed culverts and bridges over perennial streams for performance during a 1% AEP event to ensure that there are no unexpected flood hazards.

4.2.4 Extreme Flood Flows: High Intensity (Low Frequency) Events

Hydrologic evaluations of roadway infrastructure at stream crossings should not be limited to the specified design event. A complete structure assessment must include considerations for the following:

- The impact of the proposed structure on the stream.
- The impact of the proposed structure on the regulatory floodplain.
- The upstream and downstream impacts of the proposed structure and any resulting changes to flow patterns.
- The impact of high intensity floods on structural stability.
- The required performance during very large storm events. If deemed appropriate given the acceptable level of risk, include hydrologic evaluations for the 1%, 0.5%, and 0.2% AEP events.

4.2.5 Permit Flows: Low Intensity (High Frequency) Events

VTrans has derived an empirical formula to estimate the peak flow rate corresponding to ordinary high water (OHW) for permitting purposes. This calculation is part of an alternate, less direct approach to estimating the elevation of OHW. The U.S. Army Corps of Engineers (USACE) provides a Regulatory Guidance Letter, dated December 7, 2005, to aid with field identification of the OHW mark. VTrans recommends following current USACE guidance as the preferred method of finding the elevation of OHW.

However, the empirical formula for flow estimations associated with OHW is as follows:

$$Q_{OHW} = \frac{Q_{43}}{2.33}$$

Where:

Q_{OHW} = peak flow rate at OHW, cfs

Q_{43} = peak flow rate associated with the 43% AEP (2.33-year RI) event, cfs

The 43% AEP event is associated with the Gumbel distribution in hydrologic analyses. The Gumbel distribution is a skewed statistical distribution for extreme value analysis, and the Q_{43} is used with the Gumbel distribution to approximate the mean annual flow rate. Refer to Section 4.12 for more information.

4.3 Hydrologic Procedures Selection

Methods used to assess hydrologic characteristics can be divided into two categories: (1) methods based on streamgage measurements and (2) methods based on watershed characteristics. Streamgage measurements are only available at a limited number of locations, so statistical or empirical methods based on watershed characteristics are often used. The statistical and empirical methods may be subdivided into methods that estimate (a) peak runoff rates, (b) peak runoff rates and total runoff volumes, and (c) runoff hydrographs.

4.3.1 Hydrologic Methods

VTrans has determined that the methods presented in Table 4-3 are the most appropriate and accurate methods for assessing hydrologic conditions for highway design in Vermont. Note that although some of these methods may be appropriate for evaluating the hydrology of storm drainage systems, this chapter is not intended for that use. Refer to Chapter 8 *"Storm Drainage Systems"* for more information about that subject.

VTrans recommends that the design engineer use the best method available for a particular site. If the designer feels that the use of more than one method would be of value, they may combine the methods using the process described in Section 4.12. Additionally, the VTrans Hydraulics Engineer may approve the use of other methods not presented in Table 4-3. For example, for sites located adjacent to a state border (e.g. New Hampshire, New York), the designer may consider including estimates that originate from a regression equation method applicable to the adjacent state(s).

Figure 4-1 provides the Hydrologic Data Form, which can be used to summarize the input data required for the hydrologic methods commonly used by VTrans.

After the designer has assessed conditions at the site and estimated peak runoff rates, they should write a brief narrative explaining how the method(s) were selected, summarize the results, select the AEP to be used for the hydraulic design, specify the design flood flows consistent with the selected AEP, and submit the analysis to the VTrans Hydraulics Engineer for review.

4.3.2 Complex Systems

Most of the assessment methods presented in this chapter are suitable for estimating peak runoff rates. Peak runoff rates are generally adequate for designing conveyance systems including storm drains and channels. They should <u>not</u> be used for designing systems under the following circumstances:

- The study area is influenced by storage basins and/or complex conveyance networks.
- A detailed understanding of system performance throughout the course of a precipitation event is required.
- The system involves a crossing structure, and impacts to upstream and downstream flows are a concern (i.e. an existing structure is not being replaced in kind).

Studies of complex systems—such as those listed above—often require flood routing. The fundamentals of procedures for evaluating flood routing are described in Ven Te Chow's *"Handbook of Applied Hydrology*," McGraw-Hill, 1984. Flood routing calculations are most easily performed using computer software.

Table 4-3. Methods for Evaluating Hydrology

Hydrologic Method	Drainage Area Limit	Reference	Data Requirements									
			Streamgage Records	Drainage Area	Watershed Storage	Watershed Elevation	Geographical Location	Topo and Length (to find t_c or SL)	Average Annual Precipitation	Soils	Land Cover	Design Precipitation
Bulletin 17B	Unlimited	U.S. Geological Survey (USGS). 1982. "Guidelines for Determining Flood Flow Frequency, Bulletin # 17B of the Hydrology Subcommittee."	X									
Runoff Curve Number and Unit Hydrograph	Not typically the best method for watersheds > 0.5 mi^2 (320 ac.)	U.S. Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS). 1986. "Urban Hydrology for Small Watersheds," *Technical Release 55 (TR-55).*		X				X		X	X	X
Rational	< 0.3 mi^2 (190 ac.)	Federal Highway Administration (FHWA). 2002. "Highway Hydrology," Hydraulic Design Series No. 2. (HDS-2). Second Edition. Publication No. FHWA-NHI-02-001.		X				X			X	X
USGS Regression	0.2–700 mi^2 (130–448,000 ac.)	U.S. Geological Survey (USGS). 2014. "Estimation of Flood Discharges at Selected Annual Exceedance Probabilities for Unregulated, Rural Streams in Vermont," by Olson, S.A. Scientific Investigations Report 2014-5078.		X	X				X			
NETC Regression	0.2–130 mi^2 (130–83,000 ac.)	University of New Hampshire (UNH). 2010. "Estimating the Magnitude of Peak Flows for Steep Gradient Streams in New England," by Jacobs, J. New England Transportation Consortium (NETC). Project No. NETC 04-3.		X				X	X			
FHWA	< 50 mi^2 (32,000 ac.)	Federal Highway Administration (FHWA).1977. "Runoff Estimates for Small Rural Watersheds and Development of a Sound Design Method," Vol. I, Research Report, by Fletcher, J. E., et al. Publication No. FHWA-RD-77-158.		X	X	X	X					
USGS Urban Hydrograph	Unlimited	U.S. Geological Survey (USGS). 2007. "Chapter 6 of Book 4, Hydrologic Analysis and Interpretation, Section A. Statistical Analysis. Techniques and Methods 4-A6," *The National Streamflow Statistics Program: A Computer Program for Estimating Streamflow Statistics for Ungaged Sites*, by Ries III, K.G.		X	X			X				

Figure 4-1. Hydrologic Data Form

Hydrologic Data Form

Town		**Stream**	
Project Number		**Tributary to**	
Highway Number		**Structure No.**	
By		**Checked By**	
Date		**Date**	

General Drainage Area and Geographic Data

Drainage area (A) = ______ acres ______ square miles

Area of surface water = ______ acres ______ square miles ______ % of A

Area of wetlands = ______ acres ______ square miles ______ % of A

Terrain and land use = ______

Hydrologic Methods

1) Bulletin 17B

USGS Stream Gage: ______

Period of record = ______ years Station Skew = ______ Weighted Skew = ______

Skew preference = ______ MSE Station Skew = ______

2) Runoff Curve Number and Unit Hydrograph (RCN/UH)

Recurrence interval (years)	Precipitation Depth (in)
2	
10	
25	
50	
100	
200	
500	

Duration = ______ hours

Distribution = ______

RCN = ______

TC = ______ minutes

3) Rational Method

C = ______ Land Cover Coefficient

TC = ______ minutes

DF = ______ Duration Factor

RIF = ______ Recurrence Interval Factor

RI = ______ Intensity for 2-year, 30-minute event

I = ______ Rainfall Intensity (inches/hour)

4) USGS Regression Equation (may be replaced by verified StreamStats Output)

Basin-Wide Average Annual Precipitation (P) = ______ inches

Percent of Drainage Area Covered by Wetlands or Open Water, plus 1.0% = ______ %

Figure 4-1. Hydrologic Data Form (Cont.)

Hydrologic Data Form

Town		**Stream**	
Project Number		**Tributary to**	
Highway Number		**Structure No.**	
By		**Checked By**	
Date		**Date**	

General Drainage Area and Geographic Data

Drainage area (A) =		acres		square miles		
Area of surface water =		acres		square miles		% of A
Area of wetlands =		acres		square miles		% of A
Terrain and land use =						

Hydrologic Methods

5) NETC Regression Equation Method

Basin-Wide Average Annual Precipitation (P) = ______ inches

6) FHWA

Zone (choose one): ☐ Zone 5 ☐ Zone 9

Rainfall Runoff Erosivity Factor (R) =		
Elevation at extreme point of major stream =		feet NAVD88
Elevation at watershed outlet/site =		feet NAVD88
Difference in Height (DH) =		feet
Storage Correction Multiplier (S_c) =		

7) Urban Hydrograph

TC =		minutes	______ hours
L =		miles	(Basin Length)
BDF =			(Basin Development Factor)
SL =		ft/mile	(Main channel slope measured between points which are 10% and 85% of the main channel length upstream from the study site)

4.4 *Bulletin 17B Method to Estimate Peak Flows on Gaged Streams*

4.4.1 Applicability

The Bulletin 17B Method is applicable where:

- The project site is located on stream(s) in the vicinity of an active or historic USGS streamgage(s) with at least 10 years of data. (Using a streamgage with at least 25 years of data is statistically preferable, and if such data is available, the designer will not need to compute a weighted skew.)

This method is preferred above other methods because it is based on long-term measurements in the vicinity of the project site. However, the designer should always vet the data by checking the streamgage station's annual report to determine whether the data has been influenced by upstream regulation.

4.4.2 Introduction

Bulletin 17B was developed in 1976 by the USGS Interagency Advisory Committee on Water Data to provide a consistent approach to determining flood flow frequencies from record data. The method is an extension and an update to Bulletin 15, developed in 1967 by the Water Resources Council. Bulletin 17B describes a procedure for applying a Lognormal Pearson (Log-Pearson) Type III distribution to streamgage measurements of annual peak daily flow to predict flood flow frequencies.

4.4.2.1 Root-Mean-Square Error

The root-mean-square error is used in Section 4.12 to weight predicted flows and obtain appropriately averaged values for the purposes of permitting and design. Root-mean-square error represents the sample standard deviation of the differences between predicted and observed values.

The mean squared error for this method is provided as part of the output from the USACE Hydrologic Engineering Center Statistical Software Package (HEC-SSP) described in detail under Section 4.4.5. Root-mean-square error can be calculated by taking the square root of the mean squared error.

4.4.3 Description

The Log-Pearson Type III distribution is defined by three parameters, (1) the mean, (2) the standard deviation, and (3) the skew of the logarithms of the annual peak flows. Estimates of the mean and standard deviation are straight forward. Estimates of skew are affected by the length of the data record and by the influence of extreme events.

$$\bar{X} = \frac{\sum X_i}{n} \qquad \text{Mean}$$

$$S = \left[\frac{\sum (X_i - \bar{X})^2}{n-1}\right]^{0.5} \qquad \text{Standard Deviation}$$

$$G = \frac{n\sum (X_i - \bar{X})^3}{(n-1)(n-2)S^3} \qquad \text{Station Skew}$$

Where:

$\bar{X}$ = average of the series X_i

X_i = logarithm of the annual peak flows

n = length of the annual peak flow series at the station

S = standard deviation of the series X_i

G = station skew of the series X_i

For stations with short records (10–25 years), the station skew and generalized regional skew coefficients should be combined to form a better, weighted estimate.

The generalized regional skew is a pooled estimate of skew from the records at many nearby stations. In 2014, the USGS published a revised generalized regional skew estimation for Vermont incorporating additional data that has become available since Bulletin 17B was published. The new generalized regional skew is $\bar{G} = 0.44$, and the corresponding mean square error of the generalized skew is $MSE_{\bar{G}} = 0.078$.

$$G_W = \frac{MSE_{\bar{G}}(G) + MSE_G(\bar{G})}{MSE_{\bar{G}} + MSE_G} \qquad \textit{Weighted Skew}$$

Where:

G_W = the weighted skew of the station

$\bar{G}$ = generalized regional skew

G = station skew

$MSE_{\bar{G}}$ = mean squared error of generalized skew

MSE_G = mean squared error of station skew

$$MSE_G = 10^{A-B\left[Log_{10}\left(\frac{n}{10}\right)\right]}$$

Where:

$$A = -0.33 + 0.08|G| \qquad if\ |G| \le 0.90$$

$$A = -0.52 + 0.30|G| \qquad if\ |G| > 0.90$$

$$B = 0.94 - 0.26|G| \qquad if\ |G| \le 1.50$$

$B = 0.55 \qquad\qquad\qquad\qquad if\ |G| > 1.50$
n = length of the annual peak flow series at the station

Bulletin 17B also describes methods for handling extreme high and low flow outliers and for comparing analyses from station to station. These procedures are not described in this manual.

4.4.4 Area Relationship Adjustment

The area-relationship adjustment technique is appropriate to use with this method. Refer to Section 4.10 for more information on the subject.

4.4.5 Tools

The Bulletin 17B calculations can be performed manually; however, the USACE provides a convenient, free software tool that downloads the necessary data, assesses the length of the data record, performs the analysis, and produces a graph and table of the results.

The USACE developed HEC-SSP for processing streamgage data from the USGS consistent with the method described in Bulletin 17B. HEC-SSP can also perform (1) generalized frequency analyses on other types of hydrologic data, (2) volume frequency analyses on high and low flows, (3) duration analyses, (4) coincident frequency analyses, and (5) curve combination analyses.

4.4.6 Procedure

1. Determine if there is an active or historic USGS streamgage on the stream and in the vicinity of the project site. See Figure 4-2 and Table 4-4 below or check the USGS National Water Information System (NWIS).

2. Use HEC-SSP (with an active internet connection) to download streamgage data at the gage of interest.

3. Load the annual peak flow data for the desired streamgage into the tool and set the preferences for the analysis.

4. Set the analytical preferences including:
   a. Skew estimate preference
      i. Station Skew
      ii. Weighted Skew
      iii. Regional Skew
   b. Expected probability curve
      i. To compute
      ii. Not to compute
   c. The plotting positions
      i. Weibull
      ii. Median
      iii. Hazen
      iv. Custom
   d. Confidence Limits
   e. Time window modification
   f. Low outlier threshold
   g. Historic period data
   h. User-specified frequency ordinates

If the station has more than 25 years of annual peak data, it is acceptable to use the station skew, which is calculated internally by the software. If the station does not, then opt to use the weighted station skew and manually enter values for the generalized regional skew and its mean squared error. Other analytical preferences may be left as default.

4.4.7 Alternative Tools

Alternatively, use Peak FQ, a tool developed by the USGS for processing USGS streamgage data using methods consistent with those described in Bulletin 17B. The user must identify the streamgage of interest and manually download the annual peak data in the required format from the USGS NWIS.

4.4.8 Method References

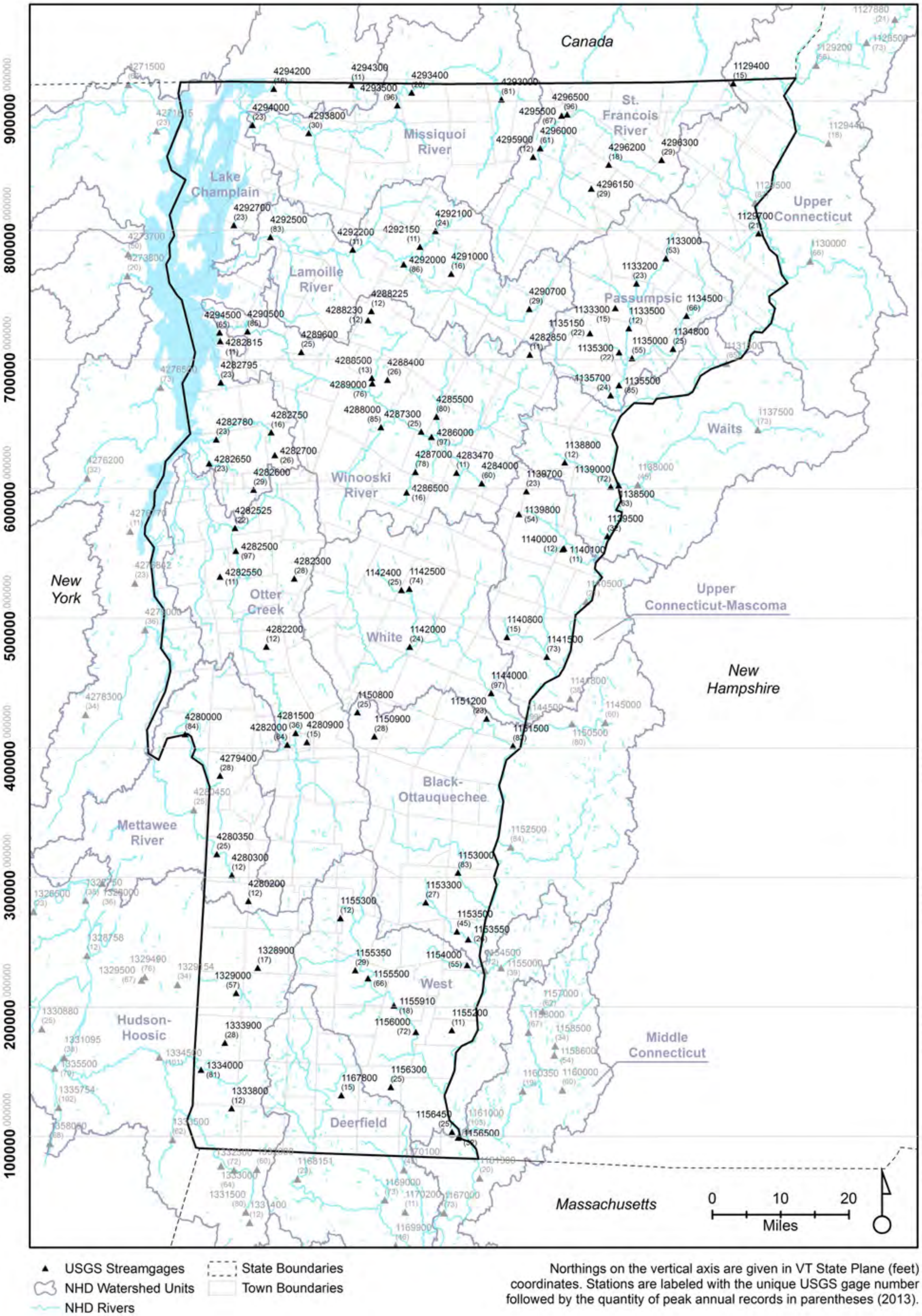
U.S. Army Corps of Engineers (USACE). 2010. *Hydrologic Engineering Center Statistical Software Package (HEC-SSP), Version 2.* http://www.hec.usace.army.mil/software/hec-ssp/

U.S. Geological Survey (USGS). 1982. "Guidelines for Determining Flood Flow Frequency, Bulletin # 17B of the Hydrology Subcommittee." http://water.usgs.gov/osw/bulletin17b/dl_flow.pdf

U.S. Geological Survey (USGS). 2006. "Estimating Magnitude and Frequency of Floods Using the PeakFQ Program: USGS Fact Sheet 2006-3143," by Flynn, K.M., W.H. Kirby, R. Mason, and T.A. Cohn. http://pubs.usgs.gov/fs/2006/3143/

U.S. Geological Survey (USGS). 2014. "Estimation of Flood Discharges at Selected Annual Exceedance Probabilities for Unregulated, Rural Streams in Vermont," by Olson, S.A. Scientific Investigations Report 2014-5078. http://pubs.usgs.gov/sir/2014/5078/pdf/sir2014-5078.pdf

Figure 4-2. Bulletin 17B Method – USGS Streamgages and Northing Index



Northings on the vertical axis are given in VT State Plane (feet) coordinates. Stations are labeled with the unique USGS gage number followed by the quantity of peak annual records in parentheses (2013).

Table 4-4. Table of Streamgages in Vermont and in Adjacent States

Station ID	Station Name
1129400	BLACK BROOK AT AVERILL, VT
1129420	CAPON BROOK AT VT 102, NEAR CANAAN, VT
1129700	PAUL STREAM TRIBUTARY NEAR BRUNSWICK SPRINGS, VT
1133000	EAST BRANCH PASSUMPSIC RIVER NEAR EAST HAVEN, VT
1133200	QUIMBY BROOK NEAR LYNDONVILLE, VT
1133300	COLD HILL BROOK NEAR LYNDON, VT
1133500	PASSUMPSIC R @ PIERCE'S MILLS, NR ST JOHNSBURY, VT
1134500	MOOSE RIVER AT VICTORY, VT
1134800	KIRBY BROOK AT CONCORD, VT
1135000	MOOSE RIVER AT ST. JOHNSBURY, VT
1135150	POPE BROOK (SITE W-3) NEAR NORTH DANVILLE, VT
1135300	SLEEPERS RIVER (SITE W-5) NEAR ST. JOHNSBURY, VT
1135500	PASSUMPSIC RIVER AT PASSUMPSIC, VT
1135700	JOES BROOK TRIBUTARY NEAR EAST BARNET, VT
1136000	SOUTH PEACHAM BROOK AT WEST BARNET, VT
1138500	CONNECTICUT RIVER AT WELLS RIVER, VT
1138800	KEENAN BROOK AT GROTON, VT
1139000	WELLS RIVER AT WELLS RIVER, VT
1139500	CONNECTICUT RIVER AT SOUTH NEWBURY, VT
1139700	WAITS RIVER TRIBUTARY NEAR WEST TOPSHAM, VT
1139800	EAST ORANGE BRANCH AT EAST ORANGE, VT
1139830	PIKE HILL BR AB RICHARDSON RD, NR BRADFORD, VT
1139833	PIKE HILL BR @ PIKE HILL RD W X, NR BRADFORD, VT
1139838	PIKE HILL BROOK @ PIKE HILL ROAD, NR BRADFORD, VT
1140000	SOUTH BRANCH WAITS RIVER NEAR BRADFORD, VT
1140100	SOUTH BRANCH WAITS R TR NEAR BRADFORD CENTER, VT
1140570	LAKE MOREY TRIBUTARY #6 NEAR FAIRLEE, VT
1140575	LAKE MOREY TRIBUTARY #5 NEAR FAIRLEE, VT
1140580	BIG BROOK (LAKE MOREY TRIBUTARY #4) NR FAIRLEE, VT
1140590	GLENN FALLS BK (LAKE MOREY TR #3) NEAR FAIRLEE, VT
1140600	LAKE MOREY OUTLET AT FAIRLEE, VT
1140800	WEST BR OMPOMPANOOSUC R TR AT SOUTH STRAFFORD, VT
1141500	OMPOMPANOOSUC RIVER AT UNION VILLAGE, VT
1142000	WHITE RIVER NEAR BETHEL, VT
1142400	THIRD BRANCH WHITE RIVER TRIBUTARY AT RANDOLPH, VT
1142500	AYERS BROOK AT RANDOLPH, VT
1142550	ADAMS BROOK AT RANDOLPH, VT
1144000	WHITE RIVER AT WEST HARTFORD, VT
1150800	KENT BROOK NEAR KILLINGTON, VT
1150900	OTTAUQUECHEE RIVER NEAR WEST BRIDGEWATER, VT
1151000	OTTAUQUECHEE RIVER AT WOODSTOCK, VT
1151200	OTTAUQUECHEE RIVER TRIBUTARY NEAR QUECHEE, VT
1151500	OTTAUQUECHEE RIVER AT NORTH HARTLAND, VT
1152660	BARKMILL BROOK AT WEATHERSFIELD BOW, VT
1152800	BLACK R AT COVERED BRIDGE, AT WEATHERSFIELD, VT
1152860	NORTH BRANCH BLACK RIVER AT FELCHVILLE, VT
1153000	BLACK RIVER AT NORTH SPRINGFIELD, VT
1153015	GREAT BROOK AT MAIN ST, @ NORTH SPRINGFIELD, VT
1153280	WILLIAMS RIVER AT CHESTER, VT
1153300	MIDDLE BRANCH WILLIAMS RIVER TR AT CHESTER, VT
1153500	WILLIAMS RIVER AT BROCKWAYS MILLS, VT
1153550	WILLIAMS RIVER NEAR ROCKINGHAM VT
1153800	BULL CREEK TRIBUTARY NEAR ATHENS, VT
1153900	TRIB TO SAXTONS R TRIBUTARY NEAR SAXTONS RIVER, VT
1154000	SAXTONS RIVER AT SAXTONS RIVER, VT
1155200	SACKETS BROOK NEAR PUTNEY, VT
1155300	FLOOD BROOK NEAR LONDONDERRY, VT
1155350	TRIB TO WEST RIVER TRIB @ RT 30, NR JAMAICA, VT
1155500	WEST RIVER AT JAMAICA, VT
1155910	WEST RIVER BELOW TOWNSHEND DAM NEAR TOWNSHEND, VT
1156000	WEST RIVER AT NEWFANE, VT
1156300	WHETSTONE BROOK TRIBUTARY NEAR MARLBORO, VT
1156450	CONNECTICUT RIVER TRIBUTARY NEAR VERNON, VT
1156500	CONNECTICUT RIVER AT VERNON, VT
1167800	BEAVER BROOK AT WILMINGTON, VT
1328900	TANNER BROOK NEAR SUNDERLAND, VT
1329000	BATTEN KILL AT ARLINGTON, VT
1333800	SOUTH STREAM NEAR BENNINGTON, VT
1333900	PARAN CREEK NEAR SOUTH SHAFTSBURY, VT
1334000	WALLOOMSAC RIVER NEAR NORTH BENNINGTON, VT
4279400	POULTNEY RIVER TRIBUTARY AT EAST POULTNEY, VT
4279490	LAKE BOMOSEEN AT OUTLET, NEAR FAIR HAVEN, VT
4280000	POULTNEY RIVER BELOW FAIR HAVEN, VT
4280200	METTAWEE RIVER TRIBUTARY NO. 2 AT EAST RUPPERT, VT
4280240	METTAWEE R TR NO. 3 @ VT 30, @ EAST RUPERT, VT
4280300	METTAWEE RIVER TRIBUTARY NEAR PAWLET, VT
4280350	METTAWEE RIVER NEAR PAWLET, VT
4280800	SOUTH FORK NEAR ORWELL, VT
4280900	MOON BROOK AT RUTLAND, VT
4280910	MOON BROOK BELOW MUSSEY BROOK AT RUTLAND, VT
4281500	EAST CREEK AT RUTLAND, VT
4282000	OTTER CREEK AT CENTER RUTLAND, VT
4282200	NESHOBE RIVER AT BRANDON, VT
4282300	BRANDY BROOK AT BREAD LOAF, VT
4282500	OTTER CREEK AT MIDDLEBURY, VT
4282525	NEW HAVEN RIVER @ BROOKSVILLE, NR MIDDLEBURY, VT
4282550	BEAVER BROOK AT CORNWALL, VT
4282600	LITTLE OTTER CREEK TRIBUTARY NEAR BRISTOL, VT
4282650	LITTLE OTTER CREEK AT FERRISBURG, VT.
4282700	LEWIS CREEK TRIBUTARY AT STARKSBORO, VT
4282750	LEWIS CREEK TRIBUTARY NO. 2 NEAR ROCKVILLE, VT
4282780	LEWIS CREEK AT NORTH FERRISBURG, VT.
4282795	LAPLATTE RIVER AT SHELBURNE FALLS, VT.
4282800	MUNROE BROOK AT SHELBURNE, VT
4282805	BARTLETT BROOK AT SOUTH BURLINGTON, VT
4282813	POTASH BR @ QUEEN CITY PARK RD, NR BURLINGTON, VT
4282815	ENGLESBY BROOK AT BURLINGTON, VT
4282850	WINOOSKI RIVER TRIBUTARY NO 2 NEAR CABOT, VT
4283470	STEVENS BRANCH TRIBUTARY AT SOUTH BARRE, VT
4283500	EAST BARRE DETENTION RESERVOIR AT EAST BARRE, VT
4284000	JAIL BRANCH AT EAST BARRE, VT
4285000	WRIGHTSVILLE DETENTION RESERVOIR @ WRIGHTSVILLE VT
4285500	NORTH BRANCH WINOOSKI RIVER AT WRIGHTSVILLE, VT
4286000	WINOOSKI RIVER AT MONTPELIER, VT
4286500	DOG RIVER AT NORTHFIELD, VT
4287000	DOG RIVER AT NORTHFIELD FALLS, VT
4287300	SUNNY BROOK NEAR MONTPELIER, VT
4288000	MAD RIVER NEAR MORETOWN, VT
4288225	W BRANCH LITTLE R ABV BINGHAM FALLS NEAR STOWE, VT
4288230	RANCH BROOK AT RANCH CAMP, NEAR STOWE, VT
4288400	BRYANT BROOK AT WATERBURY CENTER, VT
4288500	WATERBURY RESERVOIR NEAR WATERBURY, VT
4289000	LITTLE RIVER NEAR WATERBURY, VT
4289600	WINOOSKI RIVER TRIBUTARY NEAR RICHMOND, VT
4290335	ALLEN BROOK AT VT 2A, NEAR ESSEX JUNCTION, VT
4290500	WINOOSKI RIVER NEAR ESSEX JUNCTION, VT
4290575	INDIAN BROOK NEAR ESSEX JUNCTION, VT
4290700	BAILEY BROOK AT EAST HARDWICK, VT
4291000	GREEN RIVER AT GARFIELD, VT
4291500	LAMOILLE RIVER AT CADYS FALLS, VT
4292000	LAMOILLE RIVER AT JOHNSON, VT
4292100	STONY BROOK NEAR EDEN, VT
4292150	GIHON RIVER TRIBUTARY NEAR JOHNSON, VT
4292200	LAMOILLE RIVER TRIBUTARY AT JEFFERSONVILLE, VT
4292355	MORGAN BR TRIB @ OLD NO 11 RD, NR WESTFORD, VT
4292500	LAMOILLE RIVER AT EAST GEORGIA, VT
4292700	STONE BRIDGE BROOK NEAR GEORGIA PLAINS, VT
4292750	MILL RIVER AT GEORGIA SHORE RD, NR ST ALBANS, VT
4292770	STEVENS BROOK AT LEMNAH DRIVE, AT ST ALBANS, VT
4292795	STEVENS BROOK AT KELLOGG ROAD, NEAR ST. ALBANS, VT
4292810	JEWETT BROOK AT VT 38, NEAR ST. ALBANS, VT
4293000	MISSISQUOI RIVER NEAR NORTH TROY, VT
4293005	DUNN BROOK AT VT 100, NEAR NEWPORT CENTER, VT
4293200	MUD CREEK AT BEAR MOUNTAIN RD, NEAR NORTH TROY, VT
4293400	WHITTAKER BROOK AT RICHFORD, VT
4293430	NORTH BRANCH ABOVE RIVER STREET, AT RICHFORD, VT
4293500	MISSISQUOI RIVER NEAR EAST BERKSHIRE, VT
4293600	TROUT RIVER AT HOPKINS BR, NR ENOSBURG FALLS, VT

Table 4-4. Table of Streamgages in Vermont and in Adjacent States (Cont.)

Station ID	Station Name
4293700	TYLER BRANCH @ DUFFY HILL RD NR ENOSBURG FALLS, VT
4293795	BLACK CREEK ABOVE BRIDGE STREET, AT SHELDON, VT
4293800	MISSISQUOI RIVER TRIBUTARY AT SHELDON JUNCTION, VT
4293900	HUNGERFORD BR @ HIGHGATE RD NR HIGHGATE CENTER, VT
4294000	MISSISQUOI RIVER AT SWANTON, VT
4294140	ROCK RIVER NEAR HIGHGATE CENTER, VT
4294200	SAXE BROOK NEAR HIGHGATE SPRINGS, VT
4294300	PIKE RIVER AT EAST FRANKLIN, NR ENOSBURG FALLS, VT
4294500	LAKE CHAMPLAIN AT BURLINGTON, VT
4295500	LAKE MEMPHREMAGOG AT NEWPORT, VT
4295900	WARE BROOK NEAR COVENTRY, VT
4296000	BLACK RIVER AT COVENTRY, VT
4296150	LORD BROOK NEAR EVANSVILLE, VT
4296200	BROWNINGTON BRANCH NEAR EVANSVILLE, VT
4296280	BARTON RIVER NEAR COVENTRY, VT
4296300	PHERRINS RIVER TRIBUTARY NEAR ISLAND POND, VT
4296500	CLYDE RIVER AT NEWPORT, VT

Station ID	Station Name
1161300	MILLERS BROOK AT NORTHFIELD, MA
1167000	CONNECTICUT RIVER AT TURNERS FALLS, MA
1168151	DEERFIELD RIVER NEAR ROWE, MA
1169000	NORTH RIVER AT SHATTUCKVILLE, MA
1169900	SOUTH RIVER NEAR CONWAY, MA
1170000	DEERFIELD RIVER NEAR WEST DEERFIELD, MA
1170100	GREEN RIVER NEAR COLRAIN, MA
1170200	ALLEN BROOK NEAR SHELBURNE FALLS, MA
1170900	MILL RIVER NEAR SOUTH DEERFIELD, MA
1171200	SCARBORO BROOK AT DWIGHT, MA
1171300	FORT RIVER NEAR AMHERST, MA
1171500	MILL RIVER AT NORTHAMPTON, MA
1171800	BASSETT BROOK NEAR NORTHAMPTON, MA
1171910	BROAD BROOK NEAR HOLYOKE, MA
1172000	CONNECTICUT RIVER AT HOLYOKE, MA
1172003	CONNECTICUT RIVER BELOW POWER DAM AT HOLYOKE,MA
1331400	DRY BROOK NEAR ADAMS, MA
1331500	HOOSIC RIVER AT ADAMS, MA
1332000	NORTH BRANCH HOOSIC RIVER AT NORTH ADAMS, MA
1332500	HOOSIC RIVER NEAR WILLIAMSTOWN, MA
1333000	GREEN RIVER AT WILLIAMSTOWN, MA

Station ID	Station Name
1052500	DIAMOND RIVER NEAR WENTWORTH LOCATION, NH
1053500	ANDROSCOGGIN RIVER AT ERROL, NH
1053800	ANDROSCOGGIN RIVER AT BERLIN, NH
1054000	ANDROSCOGGIN RIVER NEAR GORHAM, NH
1064300	ELLIS RIVER NEAR JACKSON, NH
1064400	LUCY BROOK NEAR NORTH CONWAY, NH
1064500	SACO RIVER NEAR CONWAY, NH
1064801	BEARCAMP RIVER AT SOUTH TAMWORTH, NH
1065000	OSSIPEE RIVER AT EFFINGHAM FALLS, NH
1072100	SALMON FALLS RIVER AT MILTON, NH
1072800	COCHECO RIVER NEAR ROCHESTER, NH.
1072850	MOHAWK BROOK NEAR CENTER STRAFFORD, NH
1073000	OYSTER RIVER NEAR DURHAM, NH
1073500	LAMPREY RIVER NEAR NEWMARKET, NH
1073587	EXETER RIVER AT HAIGH ROAD, NEAR BRENTWOOD, NH
1073600	DUDLEY BROOK NEAR EXETER, NH
1074500	EAST BRANCH PEMIGEWASSET RIVER NEAR LINCOLN, NH
1074520	EAST BRANCH PEMIGEWASSET RIVER AT LINCOLN, NH
1075000	PEMIGEWASSET RIVER AT WOODSTOCK, NH
1075500	BAKER RIVER AT WENTWORTH, NH
1075800	STEVENS BROOK NEAR WENTWORTH, NH
1076000	BAKER RIVER NEAR RUMNEY, NH
1076500	PEMIGEWASSET RIVER AT PLYMOUTH, NH
1077000	SQUAM RIVER AT ASHLAND, NH
1078000	SMITH RIVER NEAR BRISTOL, NH
1080000	LAKE WINNIPESAUKEE AT WEIRS BEACH, NH
1080500	LAKE WINNIPESAUKEE OUTLET AT LAKEPORT, NH

Station ID	Station Name
1081000	WINNIPESAUKEE RIVER AT TILTON, NH
1081500	MERRIMACK RIVER AT FRANKLIN JUNCTION, NH
1082000	CONTOOCOOK RIVER AT PETERBOROUGH, NH
1083000	NUBANUSIT BK BLW MACDOWELL DAM NR PETERBOROUGH NH
1084000	NORTH BRANCH RIVER NEAR ANTRIM, NH
1084500	BEARD BROOK NEAR HILLSBORO, NH
1085000	CONTOOCOOK RIVER NEAR HENNIKER, NH
1085500	CONTOOCOOK R BL HOPKINTON DAM AT W HOPKINTON, NH
1085800	WEST BRANCH WARNER RIVER NEAR BRADFORD, NH
1086000	WARNER RIVER AT DAVISVILLE, NH
1087000	BLACKWATER RIVER NEAR WEBSTER, NH
1088000	CONTOOCOOK RIVER AT PENACOOK, NH
1088500	MERRIMACK RIVER AT GARVINS FALLS, NH
1089000	SOUCOOK RIVER NEAR CONCORD, NH
1089100	SOUCOOK RIVER, AT PEMBROKE ROAD, NEAR CONCORD, NH
1089500	SUNCOOK RIVER AT NORTH CHICHESTER, NH
1090500	MERRIMACK RIVER AT MANCHESTER, NH
1090800	PISCATAQUOG RIVER BL EVERETT DAM, NR E WEARE, NH
1091000	SOUTH BRANCH PISCATAQUOG RIVER NEAR GOFFSTOWN, NH
1091500	PISCATAQUOG RIVER NEAR GOFFSTOWN, NH
1092000	MERRIMACK R NR GOFFS FALLS, BELOW MANCHESTER, NH
1093000	SUCKER BROOK AT AUBURN, NH
1093800	STONY BROOK TRIBUTARY NEAR TEMPLE, NH
1094000	SOUHEGAN RIVER AT MERRIMACK, NH
10965852	BEAVER BROOK AT NORTH PELHAM, NH
1100505	SPICKET RIVER AT NORTH SALEM, NH
1100561	SPICKET RIVER NEAR METHUEN, MA
1127880	BIG BROOK NEAR PITTSBURG, NH
1128500	CONNECTICUT R AT FIRST CONN LK NR PITTSBURG, NH
1129200	CONNECTICUT R BELOW INDIAN STREAM NR PITTSBURG, NH
1129440	MOHAWK RIVER NEAR COLEBROOK NH
1129500	CONNECTICUT RIVER AT NORTH STRATFORD, NH
1130000	UPPER AMMONOOSUC RIVER NEAR GROVETON, NH
1131500	CONNECTICUT RIVER NEAR DALTON, NH
1137500	AMMONOOSUC RIVER AT BETHLEHEM JUNCTION, NH
1138000	AMMONOOSUC RIVER NEAR BATH, NH
1140500	CONNECTICUT RIVER AT ORFORD, NH
1141800	MINK BROOK NEAR ETNA, NH
1144500	CONNECTICUT RIVER AT WEST LEBANON, NH
1145000	MASCOMA RIVER AT WEST CANAAN, NH
1150500	MASCOMA RIVER AT MASCOMA, NH
1152500	SUGAR RIVER AT WEST CLAREMONT, NH
1154500	CONNECTICUT RIVER AT NORTH WALPOLE, NH
1155000	COLD RIVER AT DREWSVILLE, NH
1157000	ASHUELOT RIVER NEAR GILSUM, NH
1158000	ASHUELOT RIVER BELOW SURRY MT DAM, NEAR KEENE, NH
1158500	OTTER BROOK NEAR KEENE, NH
1158600	OTTER BROOK BELOW OTTER BROOK DAM, NEAR KEENE, NH
1160000	S BR ASHUELOT RIVER AT WEBB, NR MARLBOROUGH, NH
1160350	ASHUELOT RIVER AT WEST SWANZEY, NH
1161000	ASHUELOT RIVER AT HINSDALE, NH

Station ID	Station Name
1326500	HUDSON RIVER AT SPIER FALLS, NY
1327750	HUDSON RIVER AT FORT EDWARD, NY
1328000	BOND CREEK AT DUNHAM BASIN, NY
1328758	PECKS CR AT FORT MILLER, NY
1329154	STEELE BROOK AT SHUSHAN, NY
1329490	BATTEN KILL BELOW MILL AT BATTENVILLE, NY
1329500	BATTEN KILL AT BATTENVILLE, NY
1329780	SESSIONS BROOK AT PORTER CORNERS, NY
1329900	GLOWEGEE CR TRIB AT MOSHERVILLE, NY
1330000	GLOWEGEE CREEK AT WEST MILTON, NY
1330500	KAYADEROSSERAS CREEK NR WEST MILTON, NY
1330880	SARATOGA LAKE TRIB NR BEMIS HEIGHTS, NY
1331095	HUDSON RIVER AT STILLWATER, NY
1333500	LITTLE HOOSIC RIVER AT PETERSBURG, NY
1334500	HOOSIC RIVER NEAR EAGLE BRIDGE, NY

Table 4-4. Table of Streamgages in Vermont and in Adjacent States (Cont.)

Station ID	Station Name
1335500	HUDSON RIVER AT MECHANICVILLE, NY
1335754	HUDSON R ABOVE LOCK 1 NR WATERFORD, NY
1358000	HUDSON RIVER AT GREEN ISLAND, NY
4271500	GREAT CHAZY RIVER AT PERRY MILLS, NY
4271815	LITTLE CHAZY RIVER NEAR CHAZY, NY
4273700	SALMON RIVER AT SOUTH PLATTSBURGH, NY
4273800	LITTLE AUSABLE RIVER NEAR VALCOUR, NY
4276200	BOUQUET RIVER AT NEW RUSSIA, NY
4276500	BOUQUET RIVER AT WILLSBORO, NY
4276770	MILL BROOK AT PORT HENRY, NY
4276842	PUTNAM CREEK EAST OF CROWN POINT CENTER, NY
4278300	NORTHWEST BAY BROOK NEAR BOLTON LANDING, NY
4279000	LA CHUTE AT TICONDEROGA, NY
4280450	METTAWEE RIVER NEAR MIDDLE GRANVILLE, NY

4.5 Runoff Curve Number and Unit Hydrograph (RCN/UH) Method

4.5.1 Applicability

The Runoff Curve Number and Unit Hydrograph (RCN/UH) Method is applicable for sites where:

- Runoff flow is unconfined over land and in channels.
- The runoff curve number is greater than 40.
- Runoff is not constricted or controlled in culverts or storage areas. This limitation can be overcome if a flow routing method is used in conjunction with RCN/UH.

This method is not limited by drainage area. However, large, non-homogeneous watersheds should be subdivided into homogeneous areas, and each subwatershed should be characterized individually. This is not typically the best method for watersheds that are larger than 0.5 square miles (320 acres). For larger watersheds, use of regional regression equations is preferable.

4.5.2 Introduction

The RCN/UH Method is described in the Technical Release 55 (TR-55), *"Urban Hydrology for Small Watersheds,"* published by the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) in 1986.

"*The [RCN/UH] model described in TR-55 begins with a rainfall amount uniformly imposed on the watershed over a specified time distribution. Mass rainfall is converted to mass runoff by using a runoff curve number (RCN). RCN is based on soils, plant cover, amount of impervious areas, interception, and surface storage. Runoff is then transformed into a hydrograph by using unit hydrograph theory and routing procedures that depend on runoff travel time through segments of the watershed."* (NRCS. 1986).

The RCN/UH Method uses watershed characteristics to estimate peak runoff rates, total runoff volumes, and runoff hydrographs. A watershed is characterized by its drainage area, runoff curve number, and time of concentration. The time of concentration is the time it takes for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed.

4.5.2.1 Root-Mean-Square Error

The root-mean-square error is used in Section 4.12 to weight predicted flows and obtain appropriately averaged values for the purposes of permitting and design. Assume a root-mean-square error of 0.25 for this method.

4.5.3 Data Requirements and Resources

Data requirements for implementing this method include:

1. Rainfall Characteristics
   a. Depth
   b. Duration
   c. Distribution
2. Watershed Characteristics
   a. Soils
   b. Land cover
   c. Topography

4.5.3.1 Rainfall Characteristics

Rainfall depths, durations, and distributions for extreme events occurring within the state of Vermont are available via the Northeast Regional Climate Center (NRCC) Extreme Precipitation in New York and New England.

4.5.3.2 Watershed Characteristics

The RCN/UH Method requires the characterization of the contributing watershed by the hydrologic soil group (HSG) of the underlying soils. Soil data, including the HSG, is available for download in spatial and tabular form using the NRCS Web Soil Survey (WSS).

The RCN/UH Method also requires the characterization of the contributing watershed by land cover. Several large-area spatial land cover datasets are available for download at the VCGI website for use with GIS. Statewide datasets include the USGS 2001 Impervious Land Cover dataset (LandLandcov_IMPERV2001) and the USGS 2001 National Land Cover dataset (LandLandcov_NLCD2001). To download these shapefiles, select the Theme "Earth Surface Characteristics" and navigate alphabetically through the listed items to find the file names in parenthesis above.

Other large datasets localized to the Lake Champlain Basin and environs include the 1992 VCGI dataset, the 1992 University of Vermont (UVM) dataset, the 2001 UVM dataset. County-wide and smaller datasets may be available via local regional commissions. If land cover data is not available for the area of interest, aerial photographs may be used to approximate the land cover within the watershed. Currently, VCGI provides orthophotographs taken in 2013 at resolutions ranging from 15 to 50 centimeters per pixel.

Statewide coverage of low-resolution (30 meter) topography data is available from the USGS NED and can be viewed and downloaded from the National Map Viewer. Local coverage of higher resolution data (10 meter) is available for Burlington and St. Albans and the environs. Local coverage of counties and smaller areas may be available from local regional commissions. The user should use the best topography data available to delineate drainage basins and estimate flow paths. High-resolution, site-specific data should supersede or supplement low resolution datasets. Additionally, at the time of this writing, VCGI is in the process of developing state-wide Light Detection and Ranging (LiDAR) coverage that provides relatively high resolution topographic data that can be accessed using GIS software.

4.5.4 Runoff Curve Number Analysis

The runoff curve number is dependent upon the HSG, the antecedent moisture condition, and the land cover of the contributing watershed. GIS software can simplify the runoff curve number calculations with the following procedure:

1. Use GIS software to overlay the watershed boundaries, land covers, and soil types to create a table of areas with unique watershed, land cover, and HSG combinations.

2. Assign a runoff curve number to each unique land cover and soil pair.

3. Calculate the area-weighted curve number for each sub watershed.

Factors influencing the runoff curve number may include:

- Soil types
- Land cover type
- Land cover treatment
- Hydrologic condition
- Antecedent runoff condition (ARC)
- Directly connected impervious areas

4.5.4.1 Soil Types

The NRCS hosts and distributes statewide datasets for soil surveys for Vermont. Data is available online via the NRCS WSS. The soil survey contains information about the soil HSG, which may be used to characterize the propensity of certain soils to attenuate runoff. Soils are classified as follows:

A- Sand, loamy sand, or sandy loam with a low runoff potential and high infiltration rate, even when wet. Typical infiltration rate greater than 0.3 inches per hour.

B- Silt loam or loam with a moderate infiltration rate, even when wet. Typical infiltration rate ranges between 0.15 and 0.3 inches per hour.

C- Sandy clay loam with a low infiltration rate. Typical infiltration rate between 0.05 and 0.15 inches per hour.

D- Clay loam, silty clay loam, sandy clay, silty clay, or clay with a high runoff potential and very low infiltration rate. Typical infiltration rate less than 0.05 inches per hour.

4.5.4.2 Antecedent Runoff Condition

Runoff from areas will vary depending on the initial soil moisture conditions. The RCNs presented in TR-55 tables assume the average ARC. The National Engineering Handbook (NEH) provides more information on the ARC in Part 630, Chapter 10, "*Estimation of Direct Runoff from Storm Rainfall*."

4.5.4.3 Directly Connected Impervious Areas

Watershed delineations should consider directly connected impervious areas as separate subwatersheds. Impervious areas are considered directly connected if their runoff flows directly into the drainage system or flows as shallow-concentrated flow over pervious areas and then directly into the drainage system. Standard practice is to calculate runoff from directly connected impervious areas separately and to avoid combining non-homogeneous areas into a single watershed. Separating areas based on these criteria yields more representative runoff volumes.

4.5.4.4 Other Considerations

In some locations, infiltrated precipitation can discharge to surface flow, especially in areas with large sources of subsurface flow, high relief, ledge cuts, or high groundwater levels. Often, these areas are located in forested areas with HSG Type A soils or in areas of with high bedrock elevations, where infiltration is minimal at the surface and flow runs along the rock surface until it can break out above ground. Adjust the RCNs as necessary to account for the re-emergence of surface flows, but note that such flows are only likely to have a noticeable effect during smaller storms.

4.5.5 Time of Concentration Analysis

Use the methods described in NEH Part 630, Chapter 15, "*Time of Concentration,*" to perform time of concentration analyses. Time of concentration is based on the length and travel time for runoff between the most hydraulically remote portion of the watershed and the discharge point of the watershed. GIS software can simplify time of concentration calculations with the following procedure:

1. Draw the time of concentration flow path and divide the flow path into segments classified by flow type, including:
   a. Sheet flow
   b. Shallow concentrated flow
   c. Open channel flow

2. Sum the flow times for each segment to determine the total travel time along the time of concentration flow path for the subwatershed. If the calculated sum is less than 10 minutes, use 10 minutes as the minimum acceptable time of concentration.

3. For sites where the contributing runoff originates from various sources (onsite, offsite) and originates from various surfaces (pervious, impervious), estimate the time of concentration for several flow paths and use the most conservative (the shortest) travel time, still maintaining 10 minutes as the minimum acceptable time of concentration.

4.5.6 Calculating Total Runoff Volumes and Runoff Hydrographs

The designer can choose to calculate the runoff characteristics by hand using the methods described in TR-55 or use free-ware or proprietary software. VTrans approves the use of the Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS), which is freeware produced by the USACE.

To calculate runoff depths and volumes manually, use the following equations, presented originally in TR-55.

4.5.6.1 Runoff Depth

$$D_R = \frac{(P - I_a)^2}{(P - I_a) + S}$$

Where:
D_R = runoff depth, in
P = rainfall, in
S = potential maximum retention after runoff begins, in
I_a = initial abstraction, in

And where:

$$I_a = 0.2S$$

$$S = \frac{1000}{RCN} - 10$$

4.5.6.2 Runoff Volume

$$V_R = \frac{D_R}{12} * A$$

Where:
V_R = runoff volume, ft^3
D_R = runoff depth, in
A = contributing drainage area, ft^2

4.5.6.3 Runoff Hydrographs

VTrans recommends the use of software such as HEC-HMS to calculate rainfall and runoff hydrographs. To use manual methods, refer to TR-55 or NEH Part 630, Chapter 16, "*Hydrographs*."

4.5.7 Method References

U.S. Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS). 1986. "Urban Hydrology for Small Watersheds," *Technical Release 55 (TR-55)*. http://www.cpesc.org/reference/tr55.pdf

U.S. Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS). 2004. "Part 630, Hydrology: Chapter 10, Estimation of Direct Runoff from Storm Rainfall," *National Engineering Handbook*. ftp://ftp.wcc.nrcs.usda.gov/wntsc/H&H/NEHhydrology/ch10.pdf

U.S. Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS). 2007. "Part 630, Hydrology: Chapter 16, Hydrographs," *National Engineering Handbook*. ftp://ftp.wcc.nrcs.usda.gov/wntsc/H&H/NEHhydrology/ch16.pdf

U.S. Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS). 2010. "Part 630, Hydrology: Chapter 15, Time of Concentration," *National Engineering Handbook*. ftp://ftp.wcc.nrcs.usda.gov/wntsc/H&H/NEHhydrology/ch15.pdf

U.S. Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS). 2012. "Part 630, Hydrology: Chapter 18, Selected Statistical Methods," *National Engineering Handbook*. ftp://ftp.wcc.nrcs.usda.gov/wntsc/H&H/NEHhydrology/ch18.pdf

4.6 *Rational Method*

4.6.1 Applicability

The Rational Method is applicable for sites where:

- The drainage area to the site is less than 0.3 square miles (190 acres).
- The system does not have significant stormwater detention.
- The contributing drainage area does not include restrictions to natural flow, such as highway crossings and dams.

The Rational Method is one of the simplest methods for estimating runoff. This method is typically used to estimate instantaneous peak runoff rates to very small storm drainage networks and on-site roadway drainage facilities meeting the applicability criteria listed above.

4.6.2 Introduction

Use of the Rational Method can be traced back to the mid-nineteenth century. This method has significant limitations that have been addressed by more up-to-date methods. It remains popular because it is simple.

The Rational Method assumes that precipitation and outflow occur in steady-state during the peak intensity of the desired design storm. The steady-inflow/outflow assumption depends on the relationship between storm intensity and the time it takes for runoff from the most hydraulically remote portion of the watershed to reach the design point. The method assumes the storm continues at peak intensity for that length of time. If the watershed is very large, this assumption becomes unrealistic.

Watershed storage is not accounted for in the Rational Method. The method assumes that ponds, channels, or floodplains—natural or engineered—are completely full and discharging under steady-state conditions (i.e. with no outlet control, so inflow is equal to outflow).

The Rational Method is best applied to drainage areas with one (or only a few) clearly defined types of land cover. The subjective task of selecting runoff coefficients is accomplished more consistently in areas with homogenous land cover, slope, and soil type.

4.6.2.1 Root-Mean-Square Error

The root-mean-square error is used in Section 4.12 to weight predicted flows and obtain appropriately averaged values for the purposes of permitting and design. Assume a root-mean-square error of 0.36 for this method.

4.6.3 Description

The Rational Method formula is used to generate peak runoff rates given the contributing basin area and the desired storm AEP.

$$Q = C * I * A$$

Where:

Q = the peak runoff rate, cfs

C = the dimensionless runoff coefficient for the underlying land cover

I = the average rainfall intensity for a duration equal to the time of concentration for a selected AEP, in/hr

A = the area of the contributing drainage basin, acres

Typical values of C are presented in Table 4-5. Select C values with careful consideration to land use as well as soil group and average land slope.

Table 4-5. Rational Method – Runoff Coefficients

Surface	C
Concrete or sheet asphalt, pavement	0.8-0.9
Asphalt macadam pavement	0.6-0.8
Gravel roadways and shoulders	0.4-0.6
Bare earth	0.2-0.9
Steep grass areas (2:1)	0.5-0.7
Turf meadow	0.1-0.4
Forested area	0.1-0.3
Cultivated field	0.2-0.4
Flat residential 30% impervious	0.4
Flat residential 60% impervious	0.55
Moderately steep residential, 50% impervious	0.65
Moderately steep residential, 70% impervious	0.8
Flat commercial, 90% impervious	0.8

In some cases, the contributing drainage area may be composed of subcatchments that exhibit vastly different land cover characteristics. This is particularly common in more urbanized areas. When this occurs, a composite version of the Rational Method formula should be used to account for the different surface characteristics.

$$Q = I * \sum_{j=1}^{m} C_j * A_j$$

Where:

Q = the peak runoff rate, cfs

I = the average rainfall intensity for a duration equal to the time of concentration for a selected AEP, in/hr

C = the dimensionless runoff coefficient for the underlying land cover

A = the area of the contributing drainage basin, acres

m = the number of subareas

j = numerical subscript identifier for catchment data used in summation

The time of concentration of the contributing watershed is determined using the methods described in TR-55 or NEH Part 630, Chapter 15, "*Time of Concentration*". The Rational Method is often used to calculate peak runoff rates to storm drainage networks comprised of directly connected impervious area and offsite pervious areas. Note that runoff rates estimated using the time of concentration from highly impervious portions of drainage areas may result in a greater peak flow than runoff rates estimated using the time of concentration from the entire watershed. In cases where this may occur, it is prudent to estimate the time of concentration for several flow paths and to use the most conservative time of concentration.

Set the duration of the rainfall equal to the time of concentration and estimate the rainfall intensity. Because the Rational Method is restricted to sites with small drainage areas, in some cases, the calculated time of concentration may be less than 10 minutes. When this occurs, a minimum time of concentration of 10 minutes should be used to determine the rainfall intensity. Rainfall intensity, I, is determined by the following equation:

$$I = DF * RIF * RI$$

Where:

I = the average rainfall intensity for a duration equal to the time of concentration for a selected AEP, in/hr

DF = the duration factor (see Table 4-6 and Table 4-8)

RIF = the recurrence interval factor (see Table 4-7)

RI = the rainfall intensity for the 50% AEP (2-year RI), 30 minute event (refer to NRCC or other appropriate source for data)

Table 4-6. Rational Method – Duration Factors

Duration (minutes)	Factor
5	2.22
10	1.71
15	1.44
20	1.26
30	1.00
40	0.80
50	0.70
60	0.60
80	0.50
120	0.40

Table 4-7. Rational Method – Recurrence Interval Factors

Recurrence Interval (years)	Annual Exceedence Probability (%)	Factor
1	100%	0.75
2	50%	1.00
5	20%	1.30
10	10%	1.60
25	4%	1.90
50	2%	2.20

4.6.4 Procedure

1. Delineate the drainage area to the project location.

2. Calculate the time of concentration to the project location.

3. Calculate the rainfall intensity by setting the time of concentration equal to the duration of rainfall and using the tables and figures provided.

4. Identify the area(s) and corresponding runoff coefficient(s) of the drainage basin that exhibit unique land cover characteristics.

5. Calculate the peak runoff rate to the project location.

4.6.5 Method References

U.S. Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS). 1986. "Urban Hydrology for Small Watersheds," *Technical Release 55 (TR-55).* http://www.cpesc.org/reference/tr55.pdf

U.S. Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS). 2004. "Part 630, Hydrology: Chapter 10, Estimation of Direct Runoff from Storm Rainfall," *National Engineering Handbook*. ftp://ftp.wcc.nrcs.usda.gov/wntsc/H&H/NEHhydrology/ch10.pdf

U.S. Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS). 2010. "Part 630, Hydrology: Chapter 15, Time of Concentration," *National Engineering Handbook*. ftp://ftp.wcc.nrcs.usda.gov/wntsc/H&H/NEHhydrology/ch15.pdf

U.S. Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS). 2012. "Part 630, Hydrology: Chapter 18, Selected Statistical Methods," *National Engineering Handbook*. ftp://ftp.wcc.nrcs.usda.gov/wntsc/H&H/NEHhydrology/ch18.pdf

Table 4-8. Rational Method – Duration Factors (Smaller Increments)

	Tenths									
Minutes	**0.0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**
5	2.22	2.21	2.20	2.18	2.17	2.16	2.14	2.13	2.11	2.10
6	2.09	2.08	2.06	2.05	2.04	2.03	2.02	2.01	2.00	1.99
7	1.97	1.96	1.95	1.94	1.93	1.92	1.91	1.90	1.89	1.88
8	1.87	1.86	1.85	1.85	1.84	1.83	1.82	1.82	1.81	1.80
9	1.79	1.78	1.77	1.77	1.76	1.75	1.75	1.74	1.73	1.72
10	1.71	1.71	1.70	1.69	1.68	1.68	1.67	1.67	1.66	1.65
11	1.65	1.64	1.63	1.63	1.62	1.62	1.61	1.60	1.60	1.59
12	1.58	1.58	1.57	1.57	1.56	1.56	1.55	1.54	1.54	1.53
13	1.53	1.52	1.52	1.51	1.51	1.50	1.50	1.49	1.49	1.48
14	1.48	1.47	1.47	1.47	1.46	1.46	1.45	1.45	1.44	1.44
15	1.44	1.43	1.43	1.42	1.42	1.41	1.41	1.40	1.40	1.40
16	1.40	1.39	1.39	1.39	1.38	1.38	1.37	1.37	1.37	1.36
17	1.36	1.35	1.35	1.35	1.34	1.34	1.33	1.33	1.32	1.32
18	1.32	1.32	1.31	1.31	1.31	1.30	1.30	1.29	1.29	1.29
19	1.28	1.28	1.28	1.27	1.27	1.27	1.26	1.26	1.26	1.25
20	1.25	1.24	1.24	1.24	1.23	1.23	1.23	1.22	1.22	1.22
21	1.21	1.21	1.21	1.20	1.20	1.20	1.20	1.19	1.19	1.19
22	1.18	1.18	1.18	1.17	1.17	1.17	1.16	1.16	1.16	1.16
23	1.15	1.15	1.15	1.15	1.14	1.14	1.14	1.13	1.13	1.13
24	1.13	1.13	1.12	1.12	1.12	1.12	1.12	1.11	1.11	1.11
25	1.11	1.10	1.10	1.10	1.10	1.09	1.09	1.09	1.08	1.08
26	1.08	1.08	1.07	1.07	1.07	1.07	1.06	1.06	1.06	1.06
27	1.06	1.05	1.05	1.05	1.05	1.04	1.04	1.04	1.04	1.03
28	1.03	1.03	1.03	1.03	1.02	1.02	1.02	1.02	1.02	1.02
29	1.01	1.01	1.01	1.01	1.01	1.00	1.00	1.00	1.00	1.00
30	1.00	1.00	1.00	0.99	0.99	0.99	0.98	0.98	0.98	0.98
31	0.97	0.97	0.97	0.97	0.97	0.96	0.96	0.96	0.96	0.95
32	0.95	0.95	0.95	0.94	0.94	0.94	0.94	0.93	0.93	0.93
33	0.93	0.93	0.92	0.92	0.92	0.92	0.91	0.91	0.91	0.91
34	0.90	0.90	0.90	0.90	0.90	0.89	0.89	0.89	0.89	0.89
35	0.88	0.88	0.88	0.88	0.87	0.87	0.87	0.87	0.87	0.86

4.7 U.S. Geological Survey (USGS) Regression Equation Method for Ungaged Sites

4.7.1 Applicability

The USGS Regression Equation Method is applicable for sites where:

- The drainage area to the site is between 0.2 and 700 square miles (130 – 448,000 acres).
- Less than 20% of the drainage area to the site is covered by wetlands or open water.
- Average annual precipitation across the drainage area to the site is between 34 and 70 inches.

Note that if the site lies on a stream that is monitored as part of the USGS streamgage network, the Bulletin 17B Method is preferable.

4.7.2 Introduction

The USGS Regression Equation Method estimates the magnitude and frequency of streamflow along ungaged and unregulated rural streams. This method was developed by the USGS in cooperation with FEMA to provide an economical way to estimate flows to support culvert, bridge, and other near-stream structure design.

The USGS Regression Equation Method includes procedures for estimating the magnitude of peak flows at AEPs of 50%, 20%, 10%, 4%, 2%, 1%, 0.5%, and 0.2%. The regression equations developed by the USGS are shown on Table 4-9, the Method Summary Sheet.

4.7.2.1 Root-Mean-Square Error

The root-mean-square error is used in Section 4.12 to weight predicted flows and obtain appropriately averaged values for the purposes of permitting and design. Root-mean-square error represents the sample standard deviation of the differences between predicted and observed values.

For this method, use the root-mean-square errors presented in Table 4-9a and Table 4-9b, or the error reported in the output for the StreamStats or National Streamflow Statistics (NSS) tools described in Section 4.7.4.

4.7.3 Procedure

The USGS Regression Equation Method requires the following data:

1. *Drainage Area.* Delineate the drainage area using the best available topographic data. Low-resolution statewide topographic data sources include USGS Quadrangles and the USGS NED, discussed in Section 4.1.4. High-resolution regional and site-specific datasets may also be available.

2. *Percentage of drainage area with land cover categorized as wetlands or open water.* Determine the percentage of watershed area occupied by wetlands or open water using the National Land Cover Database.

3. *Basin-wide mean of average annual precipitation.* The USGS regression equations were developed using average annual precipitation data for Vermont from the Northwest Alliance for Computational Science and Engineering (NACSE) Parameter-elevation Regressions on Independent Slopes Model (PRISM) dataset. The GIS spatial dataset for the 30-year normal from 1981–2010, which was used to develop the equations, is available through Oregon State University. In order to obtain the regression equation input parameter, the designer must calculate the mean average annual precipitation across the drainage area to the site. Figure 4-3 presents the average annual precipitation in Vermont using the PRISM dataset from 1981-2010.

4.7.4 Tools

Items 1 through 4 may be estimated independently in a GIS or computer-aided design (CAD) environment; however, the web-based StreamStats software, developed and hosted by the USGS, automatically calculates the required information consistent with the USGS Regression Equation Method. The NSS software package, developed and distributed by the USACE, is also capable of performing the calculations.

4.7.4.1 USGS StreamStats

The USGS developed a web-based GIS tool called StreamStats to facilitate implementation of the USGS Regression Equation Method. The tool is hosted on the web by the USGS and provides users with access to an assortment of analytical tools useful for water resources planning and management. The user selects the location where flow estimates are desired and the tool delineates the drainage area, calculates the required watershed characteristics, and estimates the peak flows for a variety of AEPs. The tool also includes output reporting capabilities. Output reports include an illustration of the contributing watershed and summaries of the model parameters and results.

Caution!

Use caution with the StreamStats watershed delineation tool. StreamStats delineates contributing areas using 10-meter topographic data originating from the USGS NED. Verify the watershed delineation independently using the best available data and adjust the watershed boundaries prior to exporting results from StreamStats.

4.7.4.2 National Streamflow Statistics

The NSS program, a software package developed and distributed by the USGS, provides a user interface for accessing the regional regression equations for all 50 states (including Vermont) and selected territories. The NSS program provides estimates for streamflow statistics for rural, ungaged basins; flood frequencies for sites in urbanized basins; errors associated with the streamflow statistics; maximum floods; hydrographs; improved flood-frequency estimates for gaging stations by weighting estimates with systematic flood records; and improved flood-frequency estimates for ungaged sites by weighting estimates with a flow per unit area from measured sites. The program also allows the user to manipulate the appearance of input and output tables and graphs and save the data in text and graphic files.

4.7.5 Method References

Oregon State University (OSU), PRISM Climate Group. 2004. http://prism.oregonstate.edu/. Data is available for download at: http://nationalatlas.gov/mld/prism0p.html

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U.S. Geological Survey (USGS). 2007. "The National Streamflow Statistics Program: Estimating High and Low Streamflow Statistics for Ungaged Sites: USGS Fact Sheet 2007-3010," by Turnipseed, P. and K.G. Ries III. http://pubs.usgs.gov/fs/2007/3010/

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Table 4-9. USGS Regression Equations Method – Summary Sheet

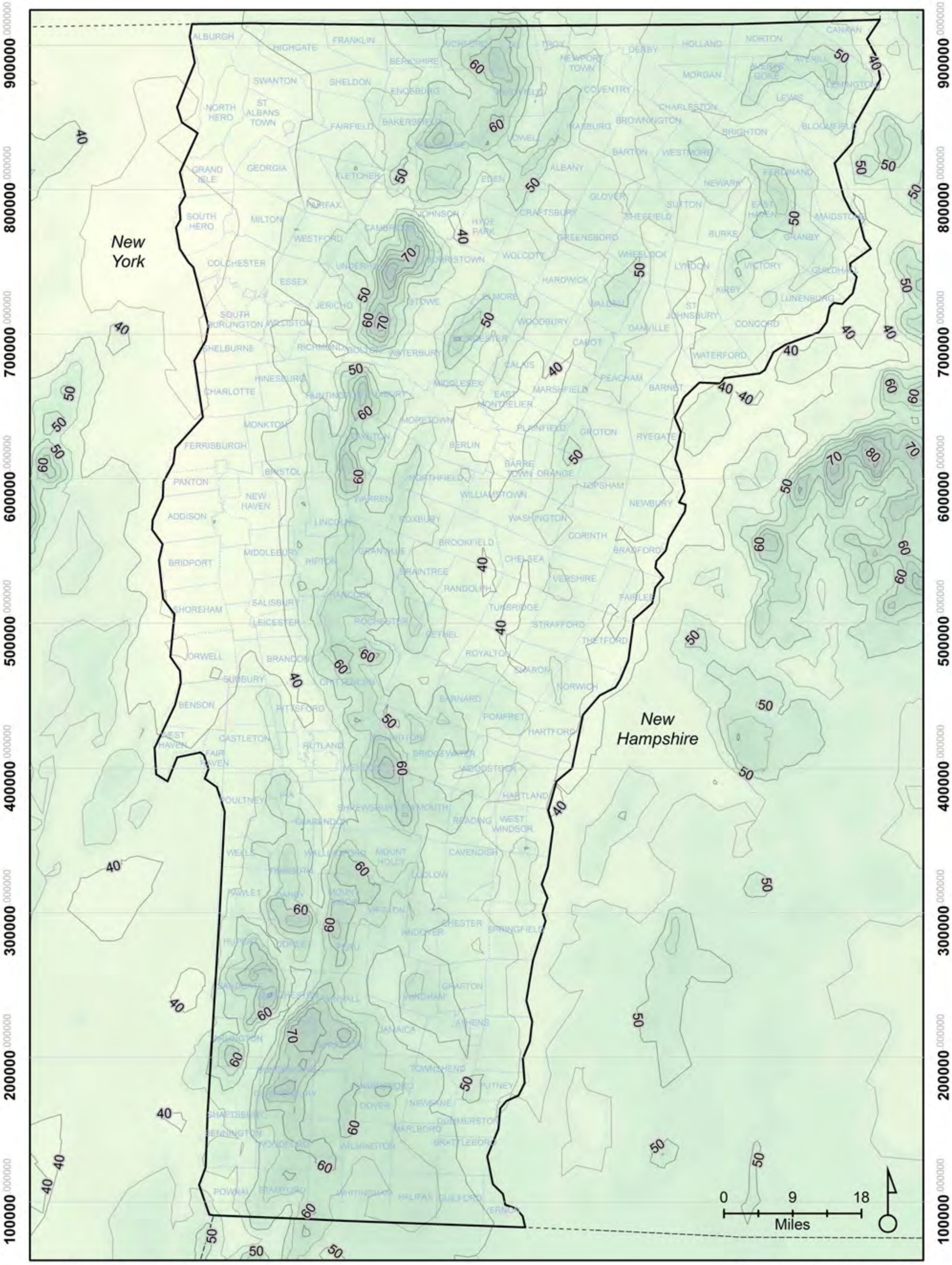
Table 4-9a. Model Parameters

Variable	Description	Units
Q_P	The estimated peak flow corresponding to the *P*% annual exceedance probability.	cfs
A	The drainage area of the basin.	mi^2
W	The percentage of the drainage basin with land cover categorized as wetlands or open water, plus 1.0%.	%
P	The basin-wide mean of the average annual precipitation.	in

Table 4-9b. USGS Regression Equations for Peak Flows

Annual Exceedance Probability (%)	Function	Root-Mean-Square Error (log units)
50%	$Q_{50} = 0.145\, A^{0.900}\, W^{-0.274}\, P^{1.569}$	0.147
20%	$Q_{20} = 0.179\, A^{0.884}\, W^{-0.277}\, P^{1.642}$	0.152
10%	$Q_{10} = 0.199\, A^{0.875}\, W^{-0.280}\, P^{1.685}$	0.162
4%	$Q_{4} = 0.219\, A^{0.866}\, W^{-0.286}\, P^{1.740}$	0.177
2%	$Q_{2} = 0.237\, A^{0.860}\, W^{-0.291}\, P^{1.774}$	0.186
1%	$Q_{1} = 0.251\, A^{0.854}\, W^{-0.297}\, P^{1.809}$	0.195
0.5%	$Q_{0.5} = 0.266\, A^{0.849}\, W^{-0.301}\, P^{1.840}$	0.208
0.2%	$Q_{0.2} = 0.289\, A^{0.844}\, W^{-0.309}\, P^{1.876}$	0.224

Figure 4-3. USGS Regression Equations Method – Average Annual Precipitation



Mean Annual Precipitation (inches)
State Boundaries
Town Boundaries

Mean annual precipitation data (1981-2010) from PRISM Climate Group, Oregon State University.

4.8 New England Transportation Consortium (NETC) Regression Equation Method for Ungaged Sites in Steep Watersheds

4.8.1 Applicability

The New England Transportation Consortium (NETC) Regression Equation Method is applicable for sites where:

- The drainage area to the site is between 0.2 and 130 square miles (130 and 83,000 acres).
- The site is located in an area considered to be 'steep' with an average main channel slope of at least 50 feet per mile.
- Average annual precipitation across the drainage area to the site is between 35 and 74 inches.

Note that if the site lies on a stream that is monitored as part of the USGS streamgage network, the Bulletin 17B Method is preferable.

4.8.2 Introduction

The NETC Regression Equation Method permits the user to estimate the magnitude and frequency of streamflow at ungaged and unregulated rural streams. The method was developed by the NETC in cooperation with the FHWA and the departments of transportation from New England states including Maine, Connecticut, New Hampshire, Rhode Island, Massachusetts, and Vermont. This method is intended to provide an economical way to estimate flows in steep gradient areas to support culvert, bridge, and other near-stream structure design.

The NETC Regression Equation Method includes procedures for estimating the magnitude of peak flows at AEPs of 50%, 20%, 10%, 4%, 2%, 1%, and 0.2%. The regression equations developed from the NETC study are shown on Table 4-10, the Method Summary Sheet.

4.8.2.1 Root-Mean-Square Error

The root-mean-square error is used in Section 4.12 to weight predicted flows and obtain appropriately averaged values for the purposes of permitting and design. Root-mean-square error represents the sample standard deviation of the differences between predicted and observed values.

For this method, use the root-mean-square errors presented in Table 4-10b. They have been calculated from the average prediction error percentages reported in the original NETC report.

4.8.3 Procedure

The NETC Regression Equation Method requires the following information:

1. *Drainage area.* Delineate the drainage area using the best available topographic data. Low-resolution statewide topographic data sources include USGS Quadrangles and the USGS NED, discussed in Section 4.1.4. High-resolution regional and site-specific datasets may also be available.

2. *Average Main Channel Slope.* In order for the NETC regression equations to be used appropriately, the average main channel slope, SL, through the drainage area must be at least 50 feet per mile. The main channel slope is the difference in elevation at points 10% and 85% of the distance along the main channel (measured from the location of the desired peak flow to the drainage divide) divided by the distance between the points.

3. *Basin-wide mean of average annual precipitation.* The NETC regression equations were developed using average annual precipitation data in Vermont from the NACSE PRISM dataset for the 30-year normal from 1961–1990. In order to be as true to the original equations as possible, the designer should use the same data when calculating the mean average annual precipitation across the drainage area to the site. The 1961–1990 data is displayed in Figure 5 of the NETC Regression Equation Method report, entitled *"Estimating the Magnitude of Peak Flows for Steep Gradient Streams in New England,"* or it can be found at the PRISM website. Figure 4-3 displays the average annual precipitation across Vermont for the most recent 30-year normal, which is from 1981–2010.

4.8.4 Tools

The drainage area and average main channel slope (Items 1 and 2, respectively) can be estimated independently in a CAD or GIS environment. The web-based StreamStats software, developed and hosted by the USGS, automatically calculates the required information consistent with the USGS Regression Equation Method, including drainage area. Refer to the Section 4.1.4 for more information.

4.8.5 Method References

Oregon State University (OSU), PRISM Climate Group. 2004. http://prism.oregonstate.edu/. Data is available for download at: http://nationalatlas.gov/mld/prism0p.html

University of New Hampshire (UNH). 2010. "Estimating the Magnitude of Peak Flows for Steep Gradient Streams in New England," by Jacobs, J. New England Transportation Consortium (NETC). Project No. NETC 04-3. http://www.ct.gov/dot/LIB/dot/documents/dresearch/NETCR81_04-3.pdf

Table 4-10. NETC Regression Equations Method – Summary Sheet

Table 4-10a. Model Parameters

Variable	Description	Units
Q_P	The estimated peak flow corresponding to the *P*% annual exceedance probability.	cfs
A	The drainage area of the basin.	mi^2
P	The basin-wide mean of the average annual precipitation.	in

Table 4-10b. Regression Equations for Peak Flows

Annual Exceedance Probability (%)	Function	Root-Mean-Square Error (log units)
50%	$Q_{50} = 0.01601\, A^{0.889} P^{2.12}$	0.171
20%	$Q_{20} = 0.01965\, A^{0.889} P^{2.19}$	0.165
10%	$Q_{10} = 0.02430\, A^{0.891} P^{2.21}$	0.169
4%	$Q_{4} = 0.03387\, A^{0.893} P^{2.20}$	0.180
2%	$Q_{2} = 0.04372\, A^{0.895} P^{2.18}$	0.193
1%	$Q_{1} = 0.05765\, A^{0.897} P^{2.15}$	0.206
0.2%	$Q_{0.2} = 0.111\, A^{0.903} P^{2.08}$	0.243

4.9 Federal Highway Administration (FHWA) Method

4.9.1 Applicability

The FHWA Method is applicable for sites where:

- The drainage area to the site is less than 50 square miles (32,000 acres).

Note that if the site lies on a stream that is monitored as part of the USGS streamgage network, the Bulletin 17B Method is preferable.

4.9.2 Introduction

The FHWA Method is a revision and expansion of the methods described by W.D. Potter under the Bureau of Public Roads (1960), predecessor to the FHWA. Results obtained using the FHWA Method are validated and correlate well with estimates based on streamgage measurements.

4.9.2.1 Root-Mean-Square Error

The root-mean-square error is used in Section 4.12 to weight predicted flows and obtain appropriately averaged values for the purposes of permitting and design. The route-mean-square error for the FHWA Method varies based on the hydrophysiographic zone in which the drainage area lies.

For this method, use the root-mean-square errors presented in Table 4-11. They have been calculated from the standard error of estimate percentages reported in the original FHWA report.

4.9.3 Procedure

The FHWA Method requires the following information:

1. *Drainage area.* Delineate the drainage area using the best available topographic data. Low-resolution statewide topographic data sources include USGS Quadrangles and the USGS NED, discussed in Section 4.1.4. High-resolution regional and site-specific datasets may also be available.

2. *Zone 5/Zone 9.* Use Figure 4-5 to determine the appropriate hydrophysiographic zone for the drainage area.

3. *Rainfall-Runoff Erosivity Factor (R).* Use Figure 4-5 to estimate the rainfall-runoff erosivity factor (R), or calculate R using the FHWA publication, "*Runoff Estimates for Small Rural Watersheds and Development of a Sound Design Method*," vol. 1.

4. *Difference in Height (DH).* Determine the difference in elevation between the extreme point of a major stream and the outlet of the watershed. The extreme point of a major stream can be the beginning of a stream as shown on the USGS quadrangle. The outlet of the watershed can be the elevation of the point of investigation.

5. *Storage Correction Multiplier.* Determine the amount of storage within the contributing watershed. This step can be performed within a CAD or GIS environment, or by using the USGS StreamStats as described in Section 4.1.4. Identify all areas within the contributing watershed covered by swamps, lakes, reservoirs, or valleys. The amount of storage should be converted to a percentage of the total drainage area then used with Figure 4-4 to determine the corresponding storage correction multiplier (S_c). If Figure 4-4 indicates that the percent of surface water storage is less than 4%, set S_c equal to 1.0.

6. *Calculate Total Runoff Volumes.* Calculate the Q_{10} for the appropriate geographic zone using the equations given in Table 4-11b. Use the Q_{10} value in the subsequent Table 4-11c equations to estimate the peak flows corresponding to the 43%, 2%, and 1% AEP events.

4.9.4 Tools

The drainage area (Item 1) can be estimated independently in a CAD or GIS environment; however, the web-based StreamStats software, developed and hosted by the USGS, automatically calculates the required information consistent with the USGS Regression Equation Method. Refer to Section 4.1.4 for more information.

4.9.5 Method References

Federal Highway Administration (FHWA). 1977. "Runoff Estimates for Small Rural Watersheds and Development of a Sound Design Method," *Vol. I, Research Report,* by Fletcher, J. E., et al. Publication No. FHWA-RD-77-158. http://digitalcommons.usu.edu/cgi/viewcontent.cgi?article=1475&context=water_rep

Figure 4-4. FHWA 3-Parameter Method – Storage Corrective Curve

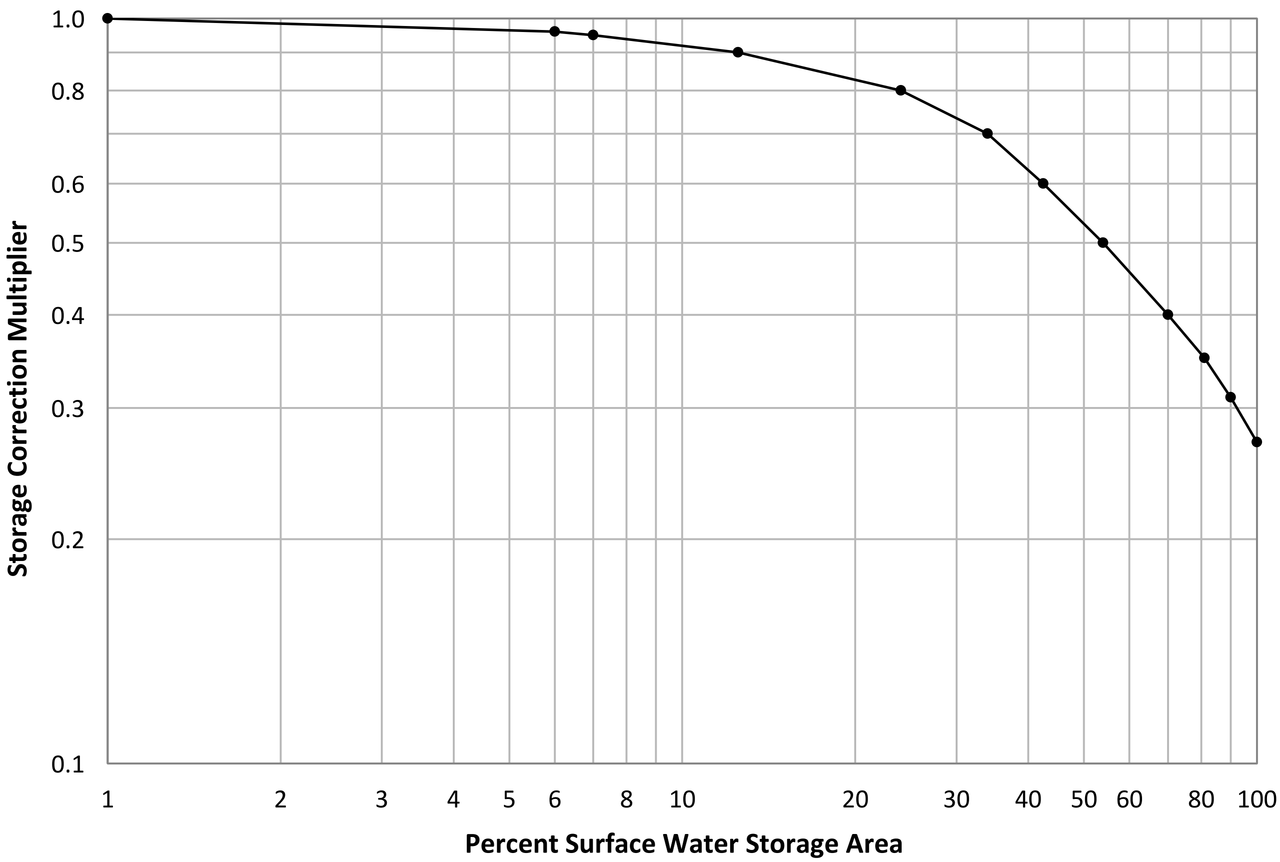


Table 4-11. FHWA Method – Summary Sheet

Table 4-11a. Model Parameters

Variable	Description	Units
Q_P	The estimated peak flow corresponding to the *P*% annual exceedance probability.	cfs
A	The drainage area of the basin.	mi^2
R	The rainfall-runoff erosivity factor.	--
DH	The difference in height.	ft
S_c	The storage correction multiplier.	--

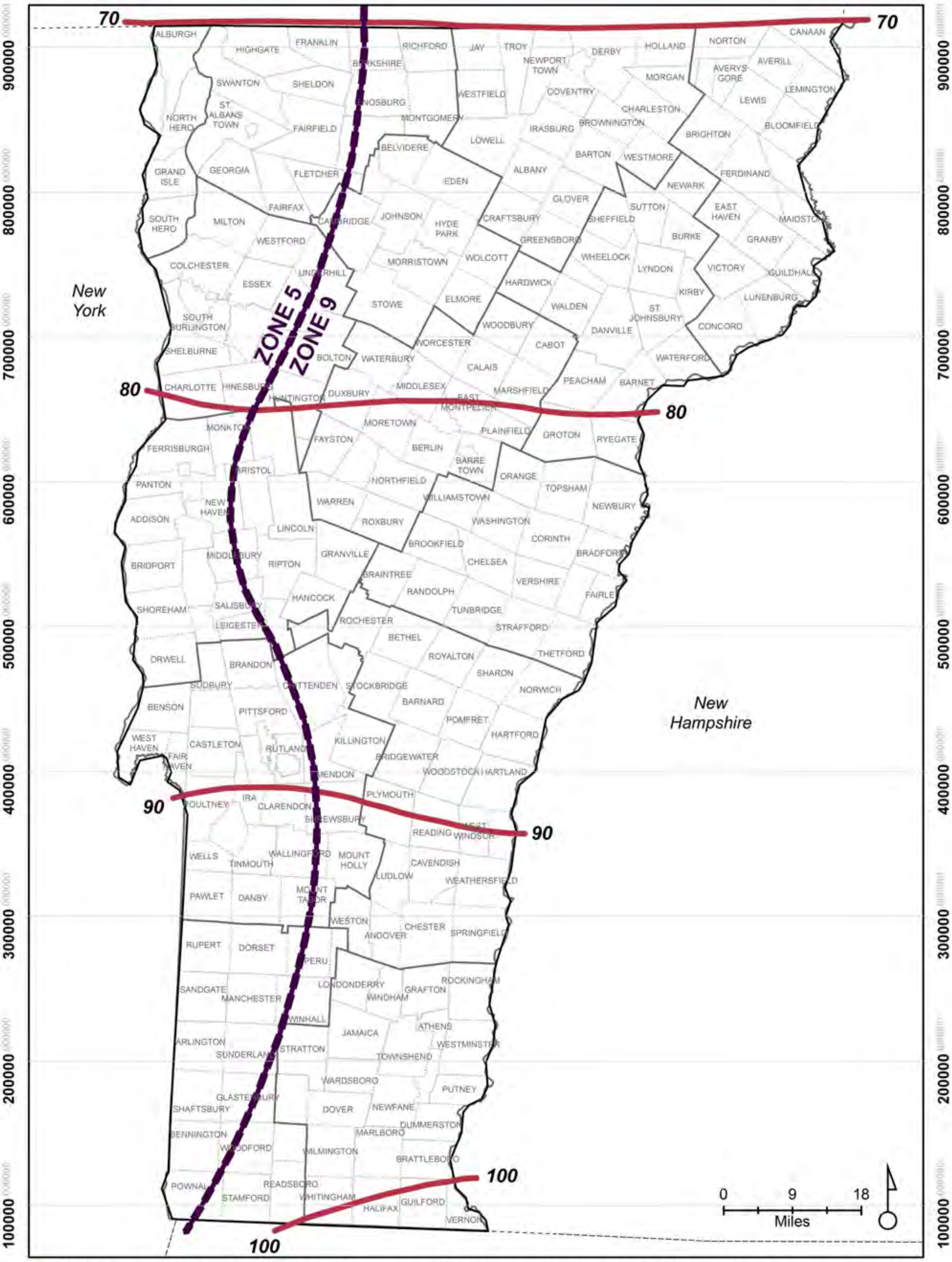
Table 4-11b. Equations for Calculating Q_{10} Based on Geographical Zone

Geographical Zone	Function	Root-Mean-Square Error (log units)	
		Zone 5	Zone 9
5	$Q_{10} = 1.14069\, A^{0.81060} R^{0.81127} DH^{0.16225} S_c$	0.16	0.27
9	$Q_{10} = 0.50051\, A^{0.69229} R^{0.74166} DH^{0.39729}\, S_c$	0.16	0.27

Table 4-11c. Equations for Peak Flows

Annual Exceedance Probability (%)	Function	Root-Mean-Square Error (log units)	
		Zone 5	Zone 9
43%	$Q_{43} = 0.46921\, {Q_{10}}^{1.00243}$	0.16	0.27
2%	$Q_2 = 1.45962\, {Q_{10}}^{1.02342}$	0.16	0.27
1%	$Q_1 = 1.64380\, {Q_{10}}^{1.02918}$	0.16	0.27

Figure 4-5. FHWA 3-Parameter Method – Hydrophysiographic Zones and Rainfall-Runoff Erosivity Factors



Hydrophysiographic Zones
Rainfall-Runoff Erosivity Factors
State Boundaries
County Boundaries
Town Boundaries

Hydrophysiographic zones and rainfall-runoff erosivity factors were digitized from: FHWA.1977. "Runoff Estimates for Small Rural Watersheds and Development of a Sound Design Method," by Fletcher, J. E., et al. Publication No. FHWA-RD-77-158.

4.10 Area-Relationship Adjustment Techniques

4.10.1 Applicability

The Area-Relationship Adjustment Techniques are applicable for sites where:

- Flow rates have been determined at a nearby streamgage located on the same stream, or
- Flow rates have been determined using statistical methods including the Bulletin 17B method.
- The drainage area to the location of unknown flows is 0.5 to 1.5 times the drainage area to the location of known flows.
- There are no significant tributaries between the streamage and the area of interest, especially if they originate in different types of terrain or if they are subject to dam and reservoir controls.

4.10.2 Introduction

In general, these methods enable the designer to scale known peak flow rates from one location to another using an area-weighted correction factor.

4.10.3 Methods

4.10.3.1 Method 1

The Area-Relationship Adjustment Technique presented here is described in Book 4, Section A, Chapter 6 of the USGS publication, *"Techniques of Water-Resources Investigations Reports."* Chapter 6 is entitled, *"The National Streamflow Statistics Program: A Computer Program for Estimating Streamflow Statistics for Ungaged Sites"* and presents the following area-relationship adjustment:

$$Q_{AEP_1} = \left[\frac{A_1}{A_2}\right]^b Q_{AEP_2}$$

Where:

Q_{AEP_1} = the peak runoff rate (at the selected AEP) for the watershed of unknown flows, cfs

Q_{AEP_2} = the peak runoff rate (at the selected AEP) for the watershed of known flows, cfs

A_1 = the drainage area to the location where flow is unknown, mi^2

A_2 = the drainage area to the location where flow is known, mi^2

b = the exponent to the drainage area term (A) from the appropriate USGS Regression Equation in Table 4-9b, which accompanies Section 4.7.

4.10.3.2 Method 2

The 2014 report detailing the USGS Regression Equation Method, entitled *"Estimation of Flood Discharges at Selected Annual Exceedance Probabilities for Unregulated, Rural Streams in Vermont,"* presents a weighting technique for estimating peak flow rates to ungaged sites that are near streamgages on the same unregulated stream. This method combines the USGS Regression Equation Method with the Bulletin 17B method. Refer to pages 17–18 of the source report for more information.

4.10.4 Technique References

U.S. Geological Survey (USGS). 2007. "Chapter 6 of Book 4, Hydrologic Analysis and Interpretation, Section A. Statistical Analysis. Techniques and Methods 4-A6," *The National Streamflow Statistics Program: A Computer Program for Estimating Streamflow Statistics for Ungaged Sites,* by Ries III, K.G. http://pubs.usgs.gov/tm/2006/tm4a6/pdf/tm4a6.pdf

U.S. Geological Survey (USGS). 2007. "The National Streamflow Statistics Program: Estimating High and Low Streamflow Statistics for Ungaged Sites: USGS Fact Sheet 2007-3010," by Turnipseed, P. and K.G. Ries III. http://pubs.usgs.gov/fs/2007/3010/

U.S. Geological Survey (USGS). 2014. "Estimation of Flood Discharges at Selected Annual Exceedance Probabilities for Unregulated, Rural Streams in Vermont," by Olson, S.A. Scientific Investigations Report 2014-5078. http://pubs.usgs.gov/sir/2014/5078/pdf/sir2014-5078.pdf

4.11 U.S. Geological Survey (USGS) Urban Hydrograph Technique

4.11.1 Applicability

The USGS Urban Hydrograph Method is applicable for sites where:

- The contributing drainage basin is urban with a large proportion of impervious area.
- The drainage area includes storage features, (e.g. ponds and/or detention basins) and runoff hydrographs are necessary.
- Peak runoff rates have already been calculated using other methods.

4.11.2 Introduction

The USGS Urban Hydrograph Method is a modification of the Clark Method for developing synthetic, dimensionless hydrographs that can be used to estimate flood hydrographs in ungaged, urban watersheds. Following the method described in this section will yield the average flood hydrograph for a given peak flow based on the USGS Dimensionless Hydrograph Coordinates.

The USGS NSS software package, described in Section 4.7.4.2, includes analytical tools for producing urban hydrographs.

4.11.3 Procedure

1. *Estimate peak runoff rates.* Use other methods described in this manual to estimate peak runoff rates for the AEP of interest.

2. *Estimate basin lag time.* Basin lag time may be co-related to the basin time of concentration by the following:

$$Lag = 0.6\,x\;\; t_c$$

Alternatively, the basin lag time may be calculated using the following:

$$Lag = 0.85\left(\frac{L}{\sqrt{SL}}\right)^{0.62}(13 - BDF)^{0.47}$$

Where:

Lag = lag time, hours

t_c = time of concentration, hours

L = basin length measured on a topographic map along the main channel from the gaging station to the basin divide, mi

SL = the main channel slope measured between points which are 10% and 85% of the main channel length upstream from the study site, ft/mile. (For sites where SL is greater than 70 ft/mile, use 70 ft/mile in the equation)

BDF = Basin Development Factor, as determined using methods described by Sauer. It ranges between 0 and 12.

3. *Estimate the Design Flood Hydrograph.* Use Table 4-12 to develop the dimensionless unit hydrograph. Multiply the lag time by each Time Factor and the Peak Runoff Rate by each Flow Factor to develop the dimensionless USGS urban hydrograph.

Table 4-12. Ratios for Dimensionless USGS Urban Hydrograph

Time Factor	Flow Factor	Time Factor	Flow Factor
0.0	0.00	1.3	0.65
0.1	0.04	1.4	0.54
0.2	0.08	1.5	0.44
0.3	0.14	1.6	0.36
0.4	0.21	1.7	0.60
0.5	0.37	1.8	0.25
0.6	0.56	1.9	0.21
0.7	0.76	2.0	0.17
0.8	0.92	2.1	0.13
0.9	1.00	2.2	0.10
1.0	0.98	2.3	0.06
1.1	0.90	2.4	0.03
1.2	0.78	2.5	0.00

4.11.4 Technique References

U.S. Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS). 1992. "Computer Program for Project Formulation Hydrology," *Technical Release 20*. ftp://ftp.wcc.nrcs.usda.gov/wntsc/H&H/other/tr20userManual.pdf

U.S. Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS). 2007. "Part 630, Hydrology: Chapter 16, Hydrographs," *National Engineering Handbook*. ftp://ftp.wcc.nrcs.usda.gov/wntsc/H&H/NEHhydrology/ch16.pdf

U.S. Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS). 2010. "Part 630, Hydrology: Chapter 15, Time of Concentration," *National Engineering Handbook*. ftp://ftp.wcc.nrcs.usda.gov/wntsc/H&H/NEHhydrology/ch15.pdf

U.S. Geological Survey (USGS). 1983. "Flood Characteristics of Urban Watersheds in the United States," by Sauer, V.B., W.O. Thomas, Jr., V.A. Stricker, and K.V. Wilson. Publication No. WSP2207. http://pubs.usgs.gov/wsp/2207/report.pdf

U.S. Geological Survey (USGS). 1989. "Dimensionless Hydrograph Method of Simulating Food Hydrographs," by Sauer, V.B.

U.S. Geological Survey (USGS). 2007. "Chapter 6 of Book 4, Hydrologic Analysis and Interpretation, Section A. Statistical Analysis. Techniques and Methods 4-A6," *The National Streamflow Statistics Program: A Computer Program for Estimating Streamflow Statistics for Ungaged Sites,* by Ries III, K.G. http://pubs.usgs.gov/tm/2006/tm4a6/pdf/tm4a6.pdf

4.12 Summarize Results for Methods Used

VTrans recommends using the best hydrologic method for a particular site. The design engineer should support this decision with a narrative and/or supplemental calculations. VTrans will not accept results from a single hydrologic method without justification.

If the design engineer elects to use more than one method, the results of the hydrologic analyses for each AEP should be summarized, and the following statistics for the peak flow rate estimates should be calculated (at a minimum):

- Mean
- Median
- Standard deviation
- Inverse weighted mean

Justification for the use of multiple methods should also be provided.

4.12.1 Calculate Inverse Weighted Mean

Calculate the inverse weighted mean for each AEP using the root-mean-square error for each analytical method as follows.

$$Q_w = \frac{\sum \frac{Q_i}{RMSE_i}}{\sum \frac{1}{RMSE_i}}$$

Where:

Q_w = the inverse weighted peak flow rate at the AEP of interest, cfs

Q_i = the peak flow rate for respective method i, cfs

$RMSE_i$ = the root-mean-square error for method i

For most cases, VTrans recommends selecting design flows based on the estimates of the inverse weighted mean (Q_w). Follow the guidance provided in Chapter 9 *"Documentation"* to document the methods used to arrive at the flow rates that will be carried forward into the hydraulic design.

4.12.2 Interpolate and Extrapolate Intermediate and Extreme AEP Events

Perform a regression analysis using the Gumbel probability distribution to obtain runoff rates for intermediate and extreme AEP events. Use graphical or computational techniques.

4.12.2.1 Graphical Regression Analysis

Figure 4-6 provides a blank piece of Gumbel distribution probability paper for manual graphical interpolations and extrapolations. Plot the RI, in years, corresponding to the selected AEP along the horizontal axis and the respective flow rate (Q_{AEP}) along the vertical axis and draw a straight line of best fit. Use the line of best fit to interpolate or extrapolate the peak flow rate at the RI/AEP of interest.

4.12.2.2 Computational Regression Analysis

Specialized statistical software is not necessary to perform a regression analysis based on the Gumbel distribution. Use common spreadsheet software (e.g. MS Excel) to plot flow rate (Q_{AEP}) vs. the Gumbel reduced variate (y_{AEP}).

$$y_{AEP} = -\ln[-\ln(q_{AEP})]$$

$$q_{AEP} = \frac{AEP}{100}$$

Where:

Q_{AEP} = the peak flow rate at annual exceedance probability AEP, cfs

y_{AEP} = Gumbel reduced variate

q_{AEP} = probability

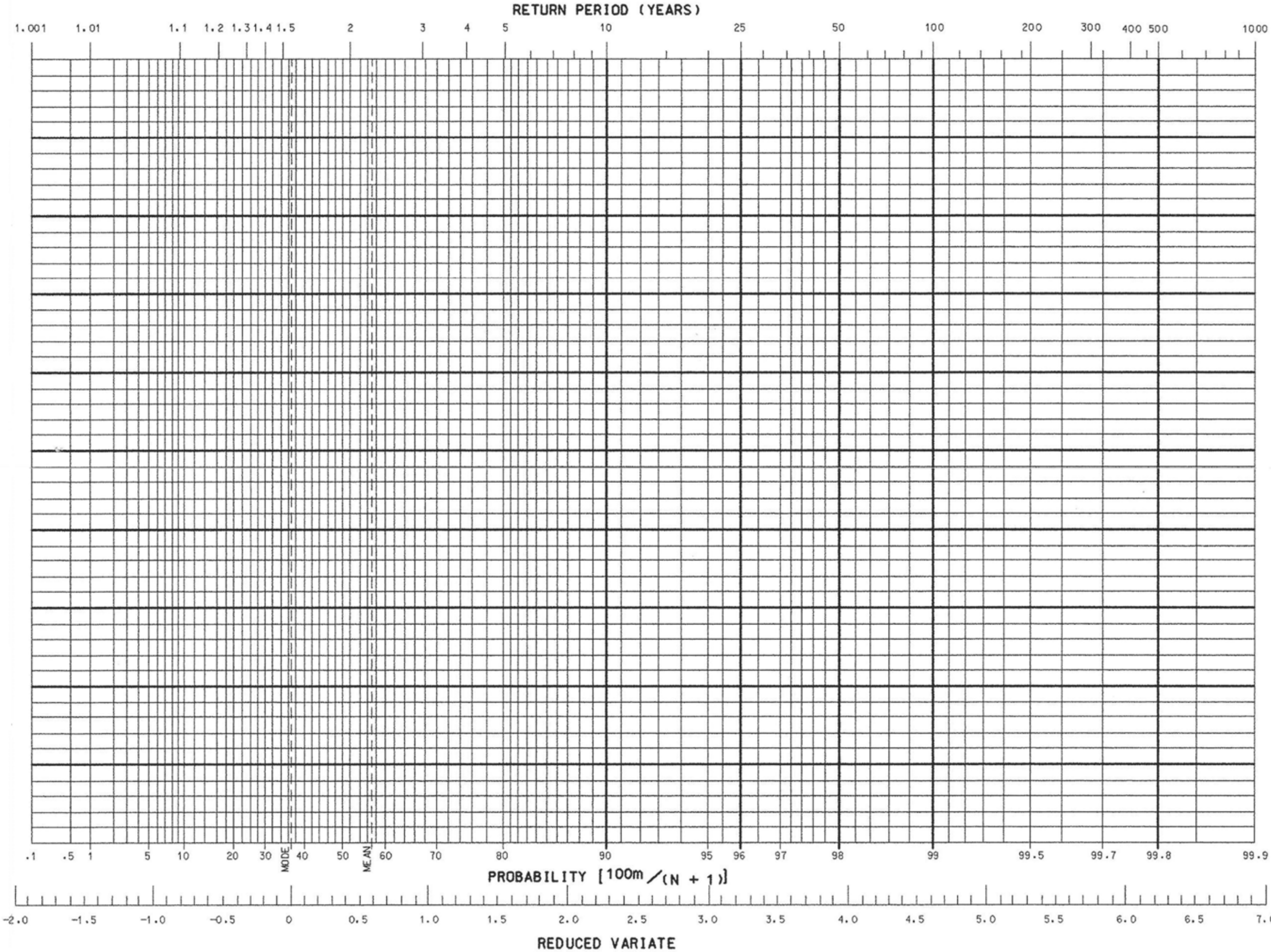
AEP = annual exceedance probability, %

Apply a linear trendline to the plot and use the properties of the trend line (i.e. slope, intercept) to determine the flow rate for the AEP(s) of interest.

4.12.3 Accuracy and Precision

Avoid misrepresenting the accuracy and precision of hydrologic calculations. While the designer may choose to provide input data at a greater precision and to carry a greater number of significant figures through intermediate calculations, VTrans recommends reporting peak flow rates to no more than two significant figures.

Figure 4-6. Gumbel Probability Paper



Chapter 5 Open Channels

5.1 Introduction

5.1.1 Overview

Open channels are defined as natural or artificial conveyances for water where the water surface is exposed to the atmosphere and the motion of the water is driven by gravity.

The various types of open channels encountered by the designer of transportation facilities include:

- Stream channel
- Roadside channel or ditch
- Irrigation channel
- Drainage ditch

Stream channels are typically:

- Natural channels where the size and shape are determined by natural forces.
- Compound in cross section with a main channel for conveying low flows and a floodplain for conveying flood flows.
- Shaped geomorphologically by the long-term history of sediment load and flow that they experience.

Artificial channels include roadside channels, irrigation channels and drainage ditches which are:

- Man-made channels with regular geometric cross sections.
- Unlined, or lined with artificial or natural material to protect against erosion.

While the principles of open channel flow are the same regardless of the channel type, stream channels and artificial channels will be treated separately in this chapter as needed. References to artificial channels will primarily focus on roadside channels.

5.1.2 Analysis and Design

Channel analysis is necessary for the design of transportation drainage systems in order to ensure that the proposed design:

- Addresses potential flooding caused by changes in water surface profiles.
- Avoids disturbance of the areas upstream and downstream of the highway right-of-way.
- Accounts for changes in lateral flow distributions, velocity, or direction of flow.
- Provides adequate conveyance of flows and disposes of excess runoff.
- Incorporates channel improvements such as linings, check dams, or other measures as necessary to prevent erosion.
- Maintains the vertical and horizontal connectivity of natural streams.
- Protects natural resources including benthic and aquatic organisms, stream and shoreline habitat, and equilibrium channel conditions.
- Provides an adequate level of protection for transportation investments.

Hydraulic design of natural and artificial channels involves the selection and evaluation of alternatives according to established criteria. VTrans has established standards that serve as these criteria to ensure that a highway facility meets its intended purpose without endangering the structural integrity of the facility itself and without undue adverse effects on the environment or the public welfare.

5.1.3 Resources

5.1.3.1 Federal Highway Administration

For more information about engineering principles and practices related to hydraulics for transportation and highway infrastructure, refer to the following Federal Highway Administration (FHWA) Hydraulic Design Series (HDS) publication:

- HDS-4, *"Introduction to Highway Hydraulics"*
- HDS-6, *"River Engineering for Highway Encroachments"*

For supplementary information about design principles and engineering techniques related to channel design, refer to the following FHWA Hydraulic Engineering Circular (HEC) publications:

- HEC-14, *"Hydraulic Design of Energy Dissipators for Culverts and Channels"*
- HEC-15, *"Design of Roadside Channels with Flexible Linings"*
- HEC-22, *"Urban Drainage Design Manual."*

5.1.3.2 Other Agencies

General permit and other state requirements concerning channel design and construction can be found in the following locations:

- Vermont Agency of Natural Resources (ANR), *"Stream Alteration General Permit"* (ANR GP)
- U.S. Army Corps of Engineers (USACE), USACE Vermont General Permit
- VTrans *"Vermont State Design Standards"*

Supplementary resources are available at the following agency website:

- ANR Rivers Program

5.1.4 Data Sources

5.1.4.1 Federal Emergency Management Agency

The Federal Emergency Management Agency (FEMA) has produced studies, reports, and web tools that can be useful aids in developing hydraulic analyses. These resources are free to the public and, when used effectively, they can save time and streamline effort.

FEMA's Map Service Center provides information about Flood Insurance Studies (FISs) and Flood Insurance Rate Maps (FIRMs), which FEMA produces to summarize estimated flood conditions in communities. Historically, FISs were issued for individual communities. Going forward, FISs will be issued on a county-wide basis. The studies follow a typical format:

Section 1- Introduction
Section 2- Area Studied
Section 3- Engineering Methods
Section 4- Flood Plain Management Applications
Section 5- Insurance Application
Section 6- FIRMs
Section 7- Other Studies
Section 8- Location of Data
Section 9- Bibliography

Supplemental Data:
Summary of Discharges Table
Floodway Data Tables
Flood Profiles

Be sure to review the FIS applicable to the study area to:

- Determine if the area of interest is within an area studied by approximate or detailed methods.
- Determine the methods used to estimate the hydrologic and hydraulic conditions at the area of interest.
- Determine the date the study was completed. Keep in mind that the older the FIS report is, the less likely it is to accurately depict existing conditions and adhere to up-to-date modeling techniques.
- Review the Summary of Discharges table to determine if the FIS includes peak flow rates at or near the area of interest. The FIS typically includes stream flow rates for the 10%, 2%, 1%, and 0.2% annual exceedance probability (AEP) events for areas studied using detailed methods.
- Review the FIS to determine if the area of interest is within a regulatory floodway. If so, the FIS's Floodway Data Table will include flood elevations for the 1% AEP event at representative locations along the river or stream.
- If the area of interest is not in the vicinity of a representative cross section, review the supplemental flood profiles to determine flood elevations at the area of interest for the 10%, 2%, 1%, and 0.2% AEP events.

Perform hydraulic analyses consistent with the methods described in this chapter and compare the results to the information published in the FIS. If there are discrepancies between flows reported in the FIS and existing conditions flows calculated by the designer using the methods presented in this manual, the designer should attempt to reconcile the differences. However, if there is sufficient confidence in the designer's values, these values should be used for design. Attempt to identify the likely causes of the discrepancies.

VTrans typically recommends that the designer:

- Complete a hydraulic model using hydrologic and hydraulic data from the FEMA FIS to demonstrate that the proposed channel work conforms to FEMA floodplain regulations.
- Use hydrologic and hydraulic data derived using the methods in this manual to design channel components.

5.1.4.2 Vermont Center for Geographic Information

The Vermont Center for Geographic Information (VCGI) is Vermont's clearinghouse for geographic information system

(GIS) data. They host the following data that may be of use to the designer:

- Geospatial data and imagery
- Regional and local GIS data
- Links to the Interactive Map Viewer
- Dynamic online mapping tools
- Information about geospatial technology

5.1.5 Design Tools

Design tools help the designer transform raw data into something that can be interpreted and ultimately constructed to fulfill the goals of a project. FHWA and USACE offer design tools that may be used to support evaluations of open channel hydraulics.

5.1.5.1 Hydraulic Toolbox

The current version of the FHWA Hydraulic Toolbox includes twelve calculators for evaluating systems typical to hydraulic design for highway applications. Two of the calculators relevant to this chapter include channel capacity and channel lining. VTrans recommends that users of the FHWA Hydraulic Toolbox verify their results with manual calculations and engineering judgment to validate the performance of the calculator.

5.1.5.2 HEC-RAS

The USACE Hydrologic Engineering Centers River Analysis System (HEC-RAS) software is recommended as a computational tool for performing step-backwater analysis. The software was developed specifically to perform one-dimensional steady and unsteady flow hydraulics calculations for open channels. The HEC-RAS User's Manual serves as a useful source for more detailed information about using the program.

The most recent version of the software (version 4.1) includes capabilities to model sediment transport (mobile bed modeling) and water temperature analysis. HEC-RAS also includes capabilities to perform hydraulic modeling at crossing structures (see Chapter 6 *"Crossing Structures"*).

5.1.5.3 HEC-GeoRAS

HEC-GeoRAS provides the capability of geo-referencing a hydraulic model through interface with digital terrain models (DTMs) and GIS software. Geo-referencing HEC-RAS models provides many advantages in model development, use, review, and re-use, including:

- Realistic representation of model inputs in user interface;
- Increased efficiency and accuracy in model geometry development;
- Reduced ambiguity regarding locations of model elements (i.e., river centerline, cross section cut lines tied to a horizontal datum); and
- Facilitated mapping of model results.

The GIS toolset allows the user to create model inputs in a map-based, graphical interface by overlaying 2-dimensional flow paths, cross sections, and banks over 3-dimensional topographic data.

5.2 Guidelines

5.2.1 General

Guidelines provide a set of goals that establish a course or method of action to determine present and future decisions (see Chapter 1 *"Hydraulic* Design Guidelines"). The following subsections summarize some of the applicable guidelines from different regulatory agencies.

5.2.2 Federal Guidelines

If the project involves federal funding, channel designs and/or designs of highway facilities that impact channels must:

- Meet the policies of the FHWA applicable to floodplain management.
- Satisfy FEMA floodplain and floodway regulations and policies and USACE restrictions for permits.

5.2.3 VTrans Guidelines

The following guidelines are specific to channels:

- Select the channel design based on the roadway classification, consequences of traffic interruption, flood hazard risks, and local site conditions.
- Coordinate with other Federal, State, and local agencies concerned with water resources planning. Input from these agencies will be given high priority in the planning of highway facilities.
- Assess environmental impacts of channel modifications, including disturbance of fish habitat, wetlands, and streambank stability.
- Design artificial drainage channels and other facilities with consideration to the frequency and type of maintenance expected and to allow for access of maintenance equipment.

- Apply natural channel design principles (fluvial geomorphology) where practicable. Application of these design principles will minimize the amount of maintenance required and will provide additional environmental benefits.

5.2.4 VT Department of Environmental Conservation Guidelines

Channel modifications and other activities within perennial streams must meet the statutory criteria and performance standards outlined in the ANR GP.

Key Terminology

A perennial stream is defined by the ANR as "*a watercourse or portion, segment, or reach of a watercourse, generally exceeding 0.5 square miles in watershed size, in which surface flows are not frequently or consistently interrupted during normal seasonal low flow periods. Perennial streams that begin flowing subsurface during low flow periods, due to natural geologic conditions, remain defined as perennial. All other streams, or stream segments of significant length, shall be termed intermittent. A perennial stream shall not include the standing waters in wetlands, lakes, and ponds.*"

The two primary requirements of the ANR GP are as follows:

1. *Equilibrium Standard*—Avoid vertical stream bed adjustments due to disruptions of sediment transport through the reach.
2. *Connectivity Standard*—Avoid abrupt changes to or disconnects within the horizontal alignment of stream banks or vertical profile of the stream bed.

The ANR GP and supporting documentation available from the ANR Rivers Program describe these standards in more detail.

5.2.5 U.S. Army Corps of Engineers Guidelines

Section 404 of the Clean Water Act regulates work within navigable waters of the United States, specifically work below ordinary high water (OHW) that involves dredge or fill materials.

The USACE provides information regarding the specific requirements for work under the USACE Vermont General Permit.

5.3 *Design Criteria*

5.3.1 General

Design criteria establish "good engineering practice" for open channel design. They form the basis for the selection of the final design configuration. Criteria are subject to change when conditions dictate, as approved by VTrans.

5.3.2 Stream Channels

The following design criteria apply to stream channels and may be revised as approved by the VTrans Hydraulics Engineer:

- Evaluate the hydraulic effects of floodplain encroachments for a range of event frequencies, including the 43%, 10%, 4%, 2%, and 1% AEP events. Hydraulic analysis should be based on peak flow rates estimated using the techniques described in Chapter 4 *"Hydrology."* Refer back to Section 5.1.4.1 for information about evaluating floodplain impacts using FEMA FIS data.
- Provide stabilization measures as necessary to protect the stream bank against erosion following disturbance or encroachment. Apply required treatments to the disturbed site as well as adjacent banks upstream and downstream of the disturbance. Consider potential impacts to vegetated wetlands, aquatic organism passage, and riparian habitat when deciding where stone fill should be placed.
- Use the methods presented in HEC-14, *"Hydraulic Design of Energy Dissipators for Culverts and Channels"* and HEC-15, *"Design of Roadside Channels with Flexible Linings"* to design channel protection. At a minimum, the recommended average channel velocity ranges for using stone fill erosion protection are provided in Table 5-1. The listed values are intended to be used for quick and conservative reference, but channel protection design should always be based on appropriate calculations. Tools such as the FHWA Hydraulic Toolbox can aid with channel lining calculations. Refer back to Section 5.1.5.1 for more information.

The different types of stone fill are classified as follows:

- *Type I.* The longest dimension of the stone should vary from 1–12 inches, and the median particle diameter (D_{50}) of the stone should be 4 inches.
- *Type II.* The longest dimension of the stone should vary from 2–36 inches, and the D_{50} of the stone should be 12 inches.

- *Type III.* The longest dimension of the stone should vary from 3–48 inches, and the D_{50} of the stone should be 16 inches.
- *Type IV.* The longest dimension of the stone should vary from 3–60 inches, and the D_{50} of the stone should be 20 inches.

Table 5-1. Stream Channel Protection with Stone Fill

Velocity (ft/s)	Stone Fill Type
≤ 5	Type I
5 – 9	Type II
9 – 10	Type III
10 – 11	Type IV
≥ 11	Complete detailed design using HEC-14 & HEC-15

Note: *Use the average computed velocity within the main channel.*

The velocity ranges in Table 5-1 are based on the Ishbash equation for the movement of stone in flowing water (USACE, 1977). For the purposes of stream channel protection, flow was assumed to be high turbulence to be slightly conservative, and the selected stone was assumed to have a specific weight of 165 pounds per cubic foot. The Ishbash equation can be written as:

$$V = C\left[2g\left(\frac{\gamma_S - \gamma_W}{\gamma_W}\right)\right]^{0.5}(D_{50})^{0.5}$$

Where:

V = velocity, ft/s

C = Ishbash constant (0.86 for high turbulence flow and 1.20 for low turbulence flow)

γ_S = specific weight of stone, lb/ft^3

γ_W = specific weight of water (62.5 lb/ft^3)

g = gravitational acceleration, ft/s^2

D_{50} = median particle diameter, ft

The thickness of stone fill should normally be based on the type of stone fill used, as shown in Table 5-2. The values in this table are based on the depth of fill necessary to apply a consistent layer of material with a thickness that is two to three times the D_{50} of each type of stone fill. These thicknesses apply to all locations where stone fill is used, including stream channels, ditches, culvert outlets, and slopes. A greater thickness of stone fill than is shown in Table 5-2 may be used in special locations as deemed appropriate by the designer.

Table 5-2. Stone Fill Thickness

Stone Fill Type	D_{50} (in)	Thickness (ft)
Type I	4	1
Type II	12	2
Type III	16	3
Type IV	20	4

An underlying filter material will often be required beneath stone fill in order to prevent loss of finer soil particles through the coarser material (i.e., piping). Filter material may consist of a geotextile filter fabric, sized with the correct Apparent Opening Size (AOS) or a granular filter blanket comprised of particles sizes that are intermediate between the native soils and the stone fill. Check with the appropriate laboratory or structures personnel to identify the best material for each location.

HEC-15 provides additional guidance on the correct selection and sizing of bedding material filter media. Table 5-3 indicates the recommended maximum longitudinal slope for selecting different channel lining materials. The listed values are intended to be used for quick reference, but channel protection design should always be based on the appropriate calculations identified in HEC-15.

Table 5-3. Channel Protection by Longitudinal Slope

Slope (%)	Lining
< 1%	Grass/Erosion Matting
1 – 2.5%	Stone Fill Type I
2.5 – 10%	Stone Fill Type II
>10%	Refer to HEC-15

Note: *If flow exceeds 20 cfs, Stone Fill Type II or larger is required regardless of the slope. Defer to Table 5-1 to select the appropriate type of stone fill based on the average velocity in the main channel.*

If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions as much as is practical. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated. Channel modification must be designed and permitted in conjunction with the ANR GP and the USACE Vermont General Permit and the performance standards contained in each.

> **Caution!**
> Sizing of stone fill should account for the slope of the embankment or channel, as well as the velocity of flows in the channel.

Slope stabilization—including placement of stone fill, geotextile, and grubbing material—should generally be placed as shown on Figure 5-1 and in accordance with the bulleted items below. For additional guidance on the design and installation of channel protection, see HEC-15.

- Construct a keyway along the toe of slope when placing stone fill on stream banks to prevent undermining. Size the width of the keyway to be two times the thickness of stone fill.
- Place grubbing material (to a minimum depth of 1.0-foot) over stone fill above the OHW elevation. The use of grubbing material minimizes the visual and environmental impact associated with stone fill and allows for the stream bank to develop vegetation more easily.

5.3.3 Roadside Channels

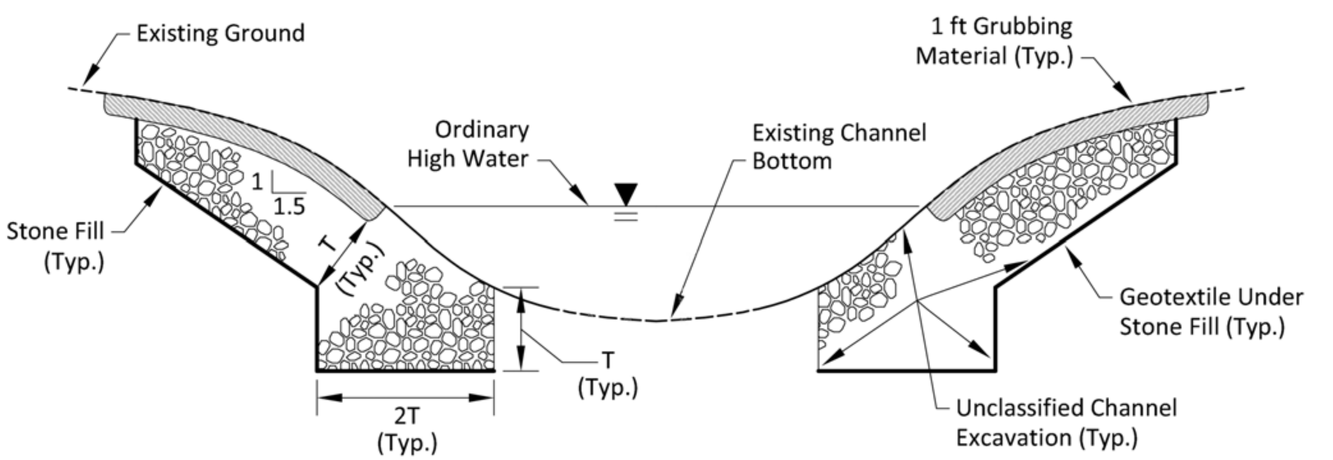
The following design criteria should be applied to roadside channels and may be revised as approved by the VTrans Hydraulics Engineer or Stormwater Engineer.

- Ensure that channel side slopes do not exceed the angle of repose of the soil and/or lining and will be 1.5:1 or flatter in the case of stone fill lining.
- Design flexible linings according to the method of allowable tractive force.
- Design permanent roadside ditch linings to accommodate peak flows from the 10% AEP storm event.
- Provide at least 1.0 foot of channel freeboard.
- Design erosion protection for roadside ditches and channels according to HEC-15, *"Design of Roadside Channels with Flexible Linings."*

HEC-15 provides a series of design procedures that were developed to aid the engineer in evaluating various types of flexible linings used to stabilize roadside channels, including vegetative linings; manufactured linings (i.e. rolled erosion control products, RECPs); riprap, cobble, and gravel linings; and gabion mattress linings.

The primary references for the design of rigid channels are HDS-4, "*Introduction to Highway Hydraulics*" and HEC-22, *"Urban Drainage Design Manual."* For channels which require other protection measures, the design of energy dissipaters and grade-control structures can be found in HEC-14, *"Hydraulic Design of Energy Dissipators for Culverts and Channels."*

Figure 5-1. Typical Stream Channel Cross Section



T= Stone fill thickness as shown in Table 5-2.

5.4 *Hydraulic Design of Channels*

5.4.1 General

Hydraulic design for roadway drainage systems relies in large part on the analysis of open channel flow through natural and artificial channels. Open channel flow principles apply the basic equations of fluid mechanics (continuity, momentum, and energy) to determine the energy grade line and the position of the free surface. The determination of these unknowns is central to open channel flow analysis and depends largely on the calculation of flow resistance.

For a general textbook discussion of open channel flow, refer to Chow (2009), Brater (1996) or other basic hydraulics manuals. HDS-4, "*Introduction to Highway Hydraulics*" and HDS-6, *"River Engineering for Highway Encroachments"* also provide good information on open channel drainage systems in the highway environment. The following section provides general definitions of the types and characteristics of flow in open channels and the terms required to evaluate them.

5.4.2 Concepts

5.4.2.1 Specific Energy

Specific energy is defined as the energy head relative to the channel bottom. See Figure 5-2 for a plot of the specific energy diagram. If the channel is not too steep (slope less than 10%) and the streamlines are nearly straight and parallel (so that the hydrostatic assumption holds), the specific energy becomes the sum of the depth and velocity head:

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

Where:

E = specific energy, ft

y = depth, ft

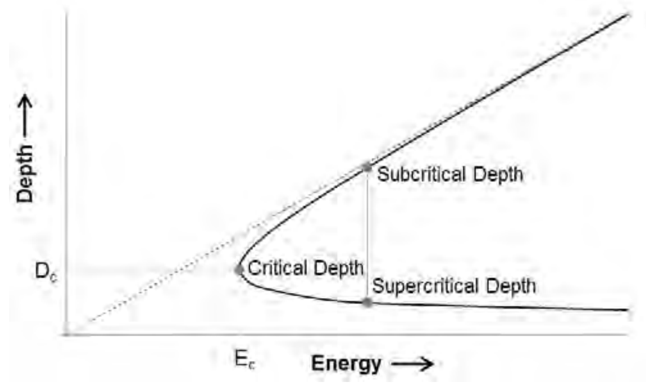
V = mean velocity, ft/s

g = gravitational acceleration, ft/s^2

5.4.2.2 Total Energy Head

The total energy head is the specific energy head plus the elevation of the channel bottom with respect to a datum. The change in energy head from one cross section to the next defines the energy grade line.

Figure 5-2. Specific Energy Curve



5.4.2.3 Steady and Unsteady Flow

Steady flow occurs when the discharge passing a given cross section is constant with respect to time. The maintenance of steady flow in any reach requires that the rates of inflow and outflow be constant and equal. When the discharge varies with time, the flow is unsteady.

5.4.2.4 Uniform Flow and Non-uniform Flow

Steady flow can be further classified as uniform or non-uniform. Uniform flow can only occur in a channel with a constant cross section, roughness, and slope in the flow direction. A non-uniform flow is one in which the velocity and depth vary in the direction of motion, and it can occur either in a prismatic channel or in a natural channel with variable properties.

5.4.2.5 Gradually Varied and Rapidly Varied Flow

Steady-state non-uniform flow can be further classified as gradually varied, in which the depth and velocity change gradually enough in the flow direction that vertical accelerations can be neglected, or as rapidly varied, in which there is a pronounced curvature of the streamlines and the assumption of hydrostatic pressure is no longer valid.

5.4.2.6 Froude Number

The Froude number is an important dimensionless parameter in open channel flow. It represents the ratio of inertia forces to gravity forces and is defined by:

$$Fr = \frac{V}{(gd)^{0.5}}$$

Where:

Fr = Froude number, dimensionless

V = mean velocity (Q/A), in ft/s

g = gravitational acceleration, ft/s^2

d = hydraulic depth (A/B), ft
Q = mean flow rate, cfs
A = cross-sectional area of flow, ft^2
B = channel top width at the water surface, ft

This expression for the Froude number applies to any single-section channel of non-rectangular shape. The value of the Froude number identifies the type of flow in the channel as subcritical, critical, or supercritical according to the following ranges:

Fr < 1.0 – Subcritical Flow
Fr = 1.0 – Critical Flow
Fr > 1.0 – Supercritical Flow

5.4.2.7 Critical Flow

Critical flow occurs when the Froude number has a value of 1.0, indicating that the inertial forces and gravitational forces are equal. The depth of critical flow is referred to as critical depth and represents the minimum specific energy for a given flow rate. Critical depth is thus the depth of maximum discharge when the specific energy is held constant.

5.4.2.8 Subcritical Flow

Subcritical flow occurs when the Froude number has a value that is less than 1.0, indicating that the gravitational forces are greater than the inertial forces. Subcritical flows correspond to depths greater than critical depth and can be described as tranquil or streaming—characteristics of deep slow moving flows. In this state of flow, small water surface disturbances can travel both upstream and downstream, and the location of the flow control is always located downstream.

5.4.2.9 Supercritical Flow

Supercritical flow occurs when the Froude number has a value that is greater than 1.0, indicating that the inertial forces are greater than the gravitational forces. Supercritical flows correspond to depths less than critical depth and can be described as rapid, shooting, or torrential—characteristics of shallow flows with higher velocities often associated with steep slopes. Small water surface disturbances are always swept downstream, and the location of the flow control is always upstream.

5.4.2.10 Hydraulic Jump

A hydraulic jump will occur at an abrupt transition from supercritical to subcritical flow in the flow direction. Depth and velocity change significantly in the jump, and energy is dissipated. For this reason, the hydraulic jump is sometimes employed to dissipate energy and control erosion at highway drainage structures.

5.4.3 Equations

The following equations are those most commonly used to analyze open channel flow. The use of these equations in analyzing open channel hydraulics is discussed in Section 5.6.

5.4.3.1 Continuity Equation

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

$$Q = A_1 V_1 = A_2 V_2$$

Where:
Q = mean flow rate, cfs
A = cross-sectional area of flow, ft^2
V = mean cross-sectional velocity taken perpendicular to the cross section, ft/s

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

5.4.3.2 Energy Equation

The energy equation expresses conservation of energy in open channel flow as energy per unit weight of fluid, which has dimensions of length and is therefore called energy head. The energy head is composed of potential energy head (elevation head), pressure head, and kinetic energy head (velocity head). The sum of these energy heads gives the total energy head at any cross section. Written between an upstream open channel cross section designated as 1 and a downstream cross section designated as 2, the energy equation is:

$$h_1 + \left(\frac{V_1^2}{2g}\right) = h_2 + \left(\frac{V_2^2}{2g}\right) + h_L$$

Where:
h_1 = upstream stage, ft
h_2 = downstream stage, ft
V_1 = mean velocity upstream, ft/s
V_1 = mean velocity downstream, ft/s
h_L = head loss from expansion, contraction, and friction, ft

The stage at a given location can be expressed as:

$$h = z + y$$

Where:
h = the stage of water above the channel bottom, ft
z = the elevation at the channel bottom, ft
y = the pressure head, or depth of flow, ft

The terms in the energy equation are illustrated graphically in Figure 5-3. The energy equation states that the total energy head at an upstream cross section is equal to the energy head at a downstream section plus the intermediate energy head loss. The energy equation can only be applied between two cross sections at which the streamlines are nearly straight and parallel so that vertical accelerations can be neglected.

5.4.3.3 Manning's Equation

A fundamental component of the design and analysis of open channels is the determination of average flow velocity. Manning's Equation is an empirically-derived formula commonly used to analyze average flow velocity and incorporate elements representing channel roughness, geometry, and slope. Assuming uniform and turbulent flow, Manning's Equation takes the following general form:

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

Where:
V = velocity, ft/s
n = Manning's roughness coefficient, dimensionless
R = hydraulic radius, (A/P), ft
S = channel slope, ft/ft
A = cross-sectional area of flow, ft^2
P = wetted perimeter, ft

The principle of roughness (or friction) is central to the evaluation of the flow capacity in a channel or conduit. For a given channel configuration (cross section and slope), increasing the friction of the channel lining will decrease the velocity of flow through the channel, and decreasing the friction will increase the velocity of flow.

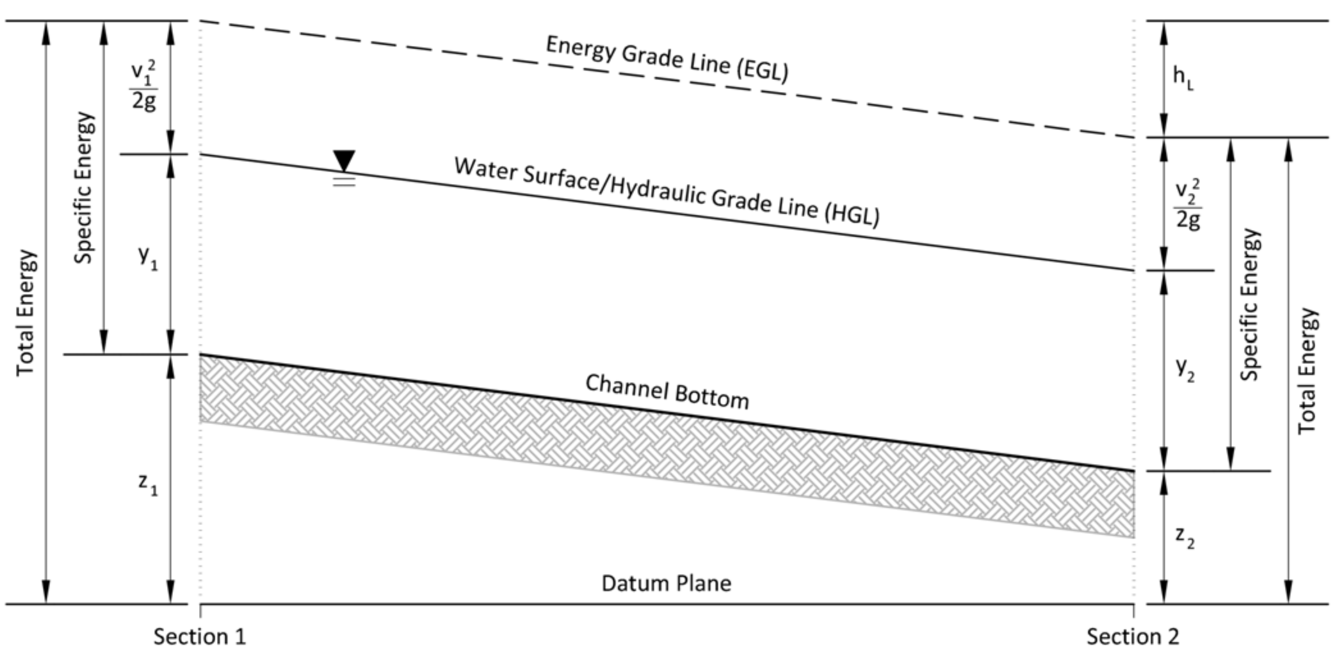
For a given channel geometry, slope, roughness, and a specified flow rate, a unique value of depth occurs in steady uniform flow. It is called the normal depth. The normal depth is used to design artificial channels in steady, uniform flow and is computed from Manning's Equation. If the normal depth computed from Manning's Equation is greater than critical depth, the slope is classified as a mild slope. On a steep slope, the normal depth is less than critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

In channel analysis, it is often convenient to group the channel properties into a single term called the channel conveyance.

$$K = \left[\left(\frac{1.486}{n}\right) A R^{2/3}\right]$$

Where:
K = channel conveyance, cfs
All other variables are as last defined

Figure 5-3. Terms in the Energy Equation



Manning's Equation can then be written as:

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

The conveyance represents the carrying capacity of a stream cross section based upon its geometry and roughness characteristics alone and thus is independent of the streambed slope. In some cases, the design engineer may want to analyze conveyance across different portions of a cross section (i.e. for the overbank areas and the main channel). Assuming that the energy grade line slope is the same across the cross section, the separate conveyances can be calculated using the same formulas and added together to yield the total conveyance across the cross section.

5.4.3.4 Manning's n Value Selection

Many factors affect Manning's n, so selecting an appropriate value for natural channels depends heavily on engineering experience. Pictures of channels and floodplains for which the discharge has been measured and Manning's n has been calculated are very useful. U.S. Geological Survey (USGS) Water Supply Paper 1849, *"Roughness Characteristics of Natural Channels"* and USGS Water Supply Paper 2339, *"Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains"* are good references. Whenever possible, attempt to verify that the Manning's n values used in the analysis reproduce the water surface elevations (WSEs) of the high water marks or gaged flows. Appendix A *"Manning's n Values"* provides tables of n values that are applicable for open channel design in the highway environment.

5.5 Methodologies

5.5.1 General

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

Two, one-dimensional methods are commonly used in hydraulic analysis of open channels: the step-backwater method and the single-section method. Refer to Chapter 6 *"Crossing Structures"* for more information about two-dimensional methods.

5.5.2 Step-Backwater Method

The step-backwater method (also known as the standard step method) is the preferred one-dimensional method for performing the following hydraulic analyses:

- Computing the complete water surface profile in a stream reach.
- Evaluating the unrestricted WSEs for the hydraulic design of crossing structures.
- Analyzing other gradually varied flow problems in streams.

Step-backwater analysis is useful for determining unrestricted water surface profiles where a highway crossing is planned and analyzing how far upstream the WSEs are affected by a culvert or bridge. Because the calculations involved in this analysis are tedious and repetitive, VTrans recommends using a computer program such as the USACE's HEC-RAS. More information about HEC-RAS is available in Sections 5.1.5.2 and 5.1.5.3.

HEC-RAS is widely used for calculating water surface profiles for steady or gradually varied flow in natural or artificial channels. The software is also capable of evaluating how bridges, culverts, weirs, and other structures in the floodplain affect the flow regime and water surface profile. The program is regularly applied in floodplain management and FISs.

HEC-RAS computes water surface profiles using the step-backwater method in which the stream reach of interest is divided into a number of sub-reaches by cross sections spaced such that the flow is gradually varied in each sub-reach. The energy equation is then solved in a stepwise fashion for the stage at one cross section based on the stage at the previous cross section. These equations are solved numerically in a step-by-step procedure from one cross section to the next.

The method requires definition of the geometry and roughness of each cross section, discussed in greater detail later in this chapter in Section 5.6.1. Manning's n values can vary both horizontally across the section as well as vertically. The user can also specify expansion and contraction head loss coefficients, variable main channel and overbank flow lengths, and the method of averaging the slope of the energy grade line.

The application of HEC-RAS to analyze bridge openings requires the definition of specific cross sections upstream and downstream of the existing or proposed structures. Refer to

Chapter 6 *"Crossing Structures"* for further explanation of these requirements and the use of HEC-RAS software.

5.5.3 Single-Section Method

The single-section method (also known as the slope-area method) involves calculating a simple solution of Manning's Equation for the normal depth of flow given the discharge and cross-sectional properties of the channel. The cross-sectional properties include geometry, slope, and roughness. The method implicitly assumes that flow through the channel is steady and uniform, which is rarely the case for stream or roadside channels. As a result, the single-section method will generally yield less dependable results because it relies on simplifications of channel characteristics and does not account for potential interactions between varying cross sections.

Nevertheless, the single-section method is often used for the following applications:

- Designing artificial channels for uniform flow as a first approximation.
- Developing stage-discharge rating curves for determination of tailwater conditions at a culvert or storm drain outlet.
- Determining flow velocities for evaluating channel lining materials.
- Analyzing other situations in which uniform or nearly uniform flow conditions exist.

The single-section method is not an accurate method for determining high water elevations in a bridge opening because of these simplifications. There are situations, however, where use of the single-section method is justified—for example, roadway ditches, minor culverts, and storm drain outfalls.

Consider the following guidelines when evaluating channel designs using the single-section analysis method:

- Select the typical cross section at or near the location where the results are needed.
- Because uniform flow is assumed, the average slope of the streambed can be used.
- Supercritical flow conditions (Fr > 1.0) and corresponding high velocities suggest that additional analysis may be warranted.
- Flow velocities higher than 16 feet per second are not typically acceptable in the highway environment due to excessive scour and potential risk to infrastructure. Further evaluation, including refinement of the analysis and design, may be required.

> **Caution!**
> Use single-section analysis only for stormwater outfalls and drainage ditches. It is not appropriate for analysis of bridges or other complex structures. A step-backwater model is required in these situations.

The FHWA Hydraulic Toolbox software includes calculators developed specifically for performing a single-section analysis. More information about FHWA's Hydraulic Toolbox is available in Section 5.1.5.1. However, the best approach for analyzing channel hydraulics—especially in stream channels—is to use a computer program that performs step-backwater calculations, such as HEC-RAS.

5.6 *Hydraulic Analysis*

5.6.1 Cross Section Development

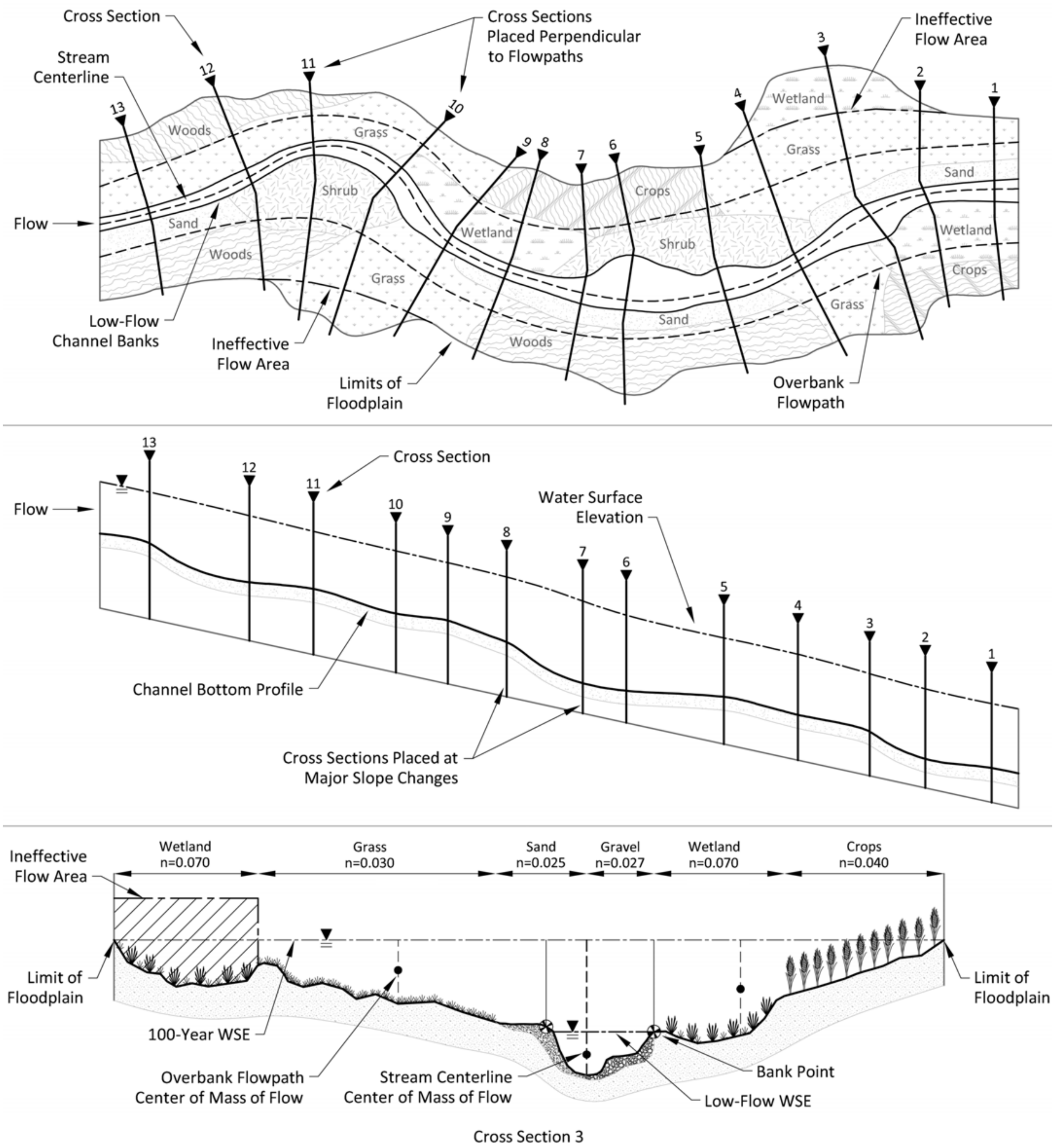
Application of both the step-backwater and the single-section methods relies on the development of the cross-sectional geometry of the channel being modeled.

The locations to be analyzed with either method must be selected carefully for the results to be meaningful. Figure 5-4 illustrates the process of characterizing the stream reach with the placement of cross sections perpendicular to the flow path and in locations where changes in the stream geometry and/or flow paths are present.

Select cross section locations so that they are representative of the sub-reaches between them. Stream locations with major breaks in bed profile, abrupt changes in roughness or shape, control sections such as free overfalls, bends and contractions, or other abrupt changes in channel slope or conveyance will require cross sections taken at shorter intervals in order to better model the change in conveyance. Chapter 6 *"Crossing Structures,"* includes additional discussion about cross section placement when using HEC-RAS to evaluate bridge openings.

The computational power of GIS and computer-aided design (CAD) tools allows rapid development of cross sections based on ground survey data and digital terrain models (DTMs). These cross sections can then be used within HEC-RAS or other step-backwater software packages to model the reach of interest. HEC-GeoRAS provides a convenient user interface within GIS for geo-referencing HEC-RAS models. More information about HEC-GeoRAS is available in Section 5.1.5.3.

Figure 5-4. Hypothetical Cross Section Development and Selection of Manning's n Values



Alternatively, ASCII text files generated by GIS or CAD software can be imported to HEC-RAS to convert digital elevation data into geo-referenced cross sections. Routines are available or can be developed to automate the import and conversion process.

Account for the variability of Manning's n across a channel in the definition of each cross section. As an example, this may involve incorporating the decreased roughness of concrete or sand-bottomed channels and the increased roughness provided by dense vegetation or large boulders.

Whether entered manually or through an automated process, the general principles of cross section selection remain the same. Cross sections must extend sufficiently into the upland areas to capture the changes in topography that define the overbank and floodplain areas. Cross sections should be taken normal to the flow direction along a single straight line where possible. In wide floodplains or along meander bends it may be necessary to use a section along intersecting straight lines—that is, a "dogleg" section. It is important to make a plot of the project area showing the cross section locations so that the characteristics of these features can be reviewed.

5.6.2 Calibration

Calibrate hydraulic models to ensure that they accurately represent local channel conditions. Use the following parameters, in order of preference, for calibration: Manning's n, slope, flow rate, and cross section. Proper calibration is the key to obtaining accurate results.

In stream channels, the transverse variation of velocity in any cross section is a function of subsection geometry and roughness and may vary considerably from one stage and flow rate to another. It is important to know this variation for purposes of designing erosion control measures and locating relief openings in highway fills, for example. The best method for establishing transverse velocity variations is by current meter measurements. If obtaining current meter measurements is not feasible, the single-section method can be used by dividing the cross section into subsections of relatively uniform roughness and geometry. Assuming that the energy grade line slope is the same across the cross section, all of the subsection conveyances can be added together to yield the total conveyance, K_T, given by:

$$K_T = \sum_{i=1}^{n} K_i$$

Where:
K_i = conveyance in each cross-sectional subsection, cfs
n = total number of subsections

The total discharge is then:

$$Q_T = K_T S^{1/2}$$

Where:
Q_T = total discharge within a cross-section, cfs
K_i = conveyance in each cross-sectional subsection, cfs
n = total number of subsections

The velocity in each subsection is obtained from the continuity equation:

$$V_i = \frac{Q_i}{A_i}$$

Where:
V_i = velocity in each cross-sectional subsection, ft/s
Q_i = discharge in each cross-sectional subsection, cfs
A_i = cross-sectional area of each subsection, ft^2

In practice, flow variations across a cross section are readily modeled in HEC-RAS.

Alluvial channels present a more difficult problem when establishing stage-discharge relations using the single-section method because the bed itself is deformable and may generate bed forms such as ripples and dunes in lower regime flows. These bed forms are highly variable with the addition of form resistance, and selection of a value of Manning's n is not straightforward. Instead, several methods outlined in Vanoni (2006) have been developed for this case (Einstein-Barbarossa; Kennedy-Alam-Lovera; and Engelund) and should be followed unless it is possible to obtain a measured stage-discharge relation.

There may be locations where a stage-discharge relationship has already been measured in a channel. The USGS typically develops stage-discharge relationships to accompany data collected at the stream gaging stations that they maintain. Measured stage-discharge curves will generally yield more accurate estimates of WSE and should take precedence over the analytical methods described above.

5.6.3 Profile Computation

Water surface profile computation requires a beginning value of elevation or depth (the boundary condition) and proceeds upstream for subcritical flow and downstream for supercritical flow. In the case of supercritical flow, critical depth is often the boundary condition at the control section, but in subcritical flow, uniform flow and normal depth may be the boundary conditions. The starting depth in this case can either be found using the single-section method or by computing the water surface profile upstream to the desired location for several starting depths and the same discharge. These profiles should converge toward the desired normal depth at the control section to establish one point in the stage-discharge relationship.

If several profiles do not converge, a combination of adjustments can be made:

- Extend the stream reach downstream.
- Shorten the distance between cross sections.
- Adjust the range of starting water-surface elevations.

A plot of the convergence profiles can be a very useful tool in such an analysis (see Figure 5-5). Given a long enough stream reach, the water surface profile computed by step-backwater will converge to normal depth at some point upstream for subcritical flow. The design engineer must establish the upstream and downstream boundaries of the stream reach in order to define the limits of data collection and subsequent analysis.

Begin calculations far enough downstream to ensure accurate results at the structure site, and continue calculations a sufficient distance upstream to accurately determine the impact of the structure on upstream water surface profiles (see Figure 5-6).

In the 1986 publication, *"Accuracy of Computed Water Surface Profiles."* The USACE developed regression equations for determining the required upstream and downstream reach lengths as follows:

$$L_{DC} = 6{,}600 * H_D / S$$

$$L_{DN} = 8{,}000 * {H_D}^{0.8} / S$$

$$L_U = 10{,}000 * {H_D}^{0.6} * {H_L}^{0.5} / S$$

Where:

L_{DC} = downstream study length (along main channel) for critical depth starting conditions, ft

L_{DN} = downstream study length (along main channel) for normal depth starting conditions, ft

L_U = estimated upstream study length (along main channel) required for convergence of the modified profile to within 0.1 feet of the base profile, ft

H_D = average hydraulic depth (1% AEP flood flow area divided by the top width), ft

S = average reach slope, ft/mile

H_L = head loss ranging between 0.5–5.0 feet at the channel crossing structure for the 1% AEP flood flow, ft

Figure 5-5. Profile Convergence Pattern for Backwater Computations

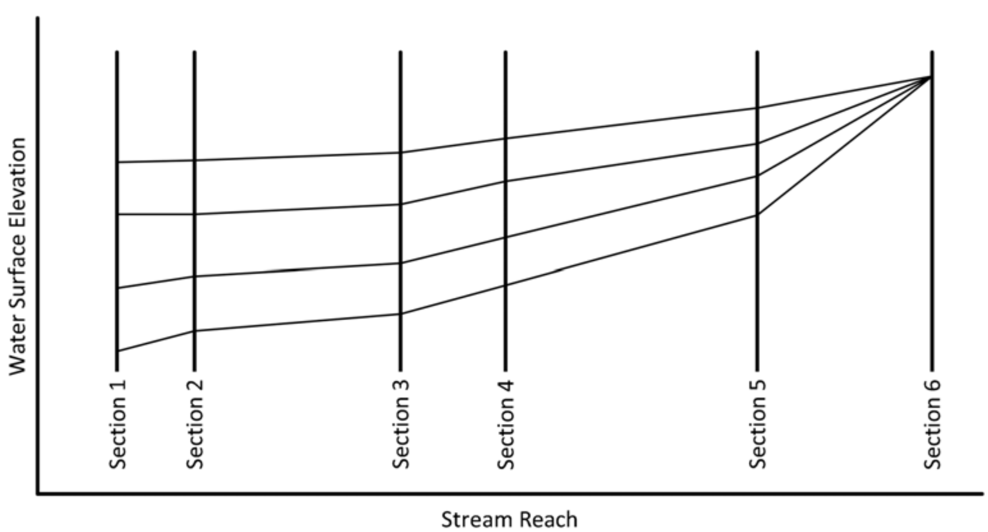
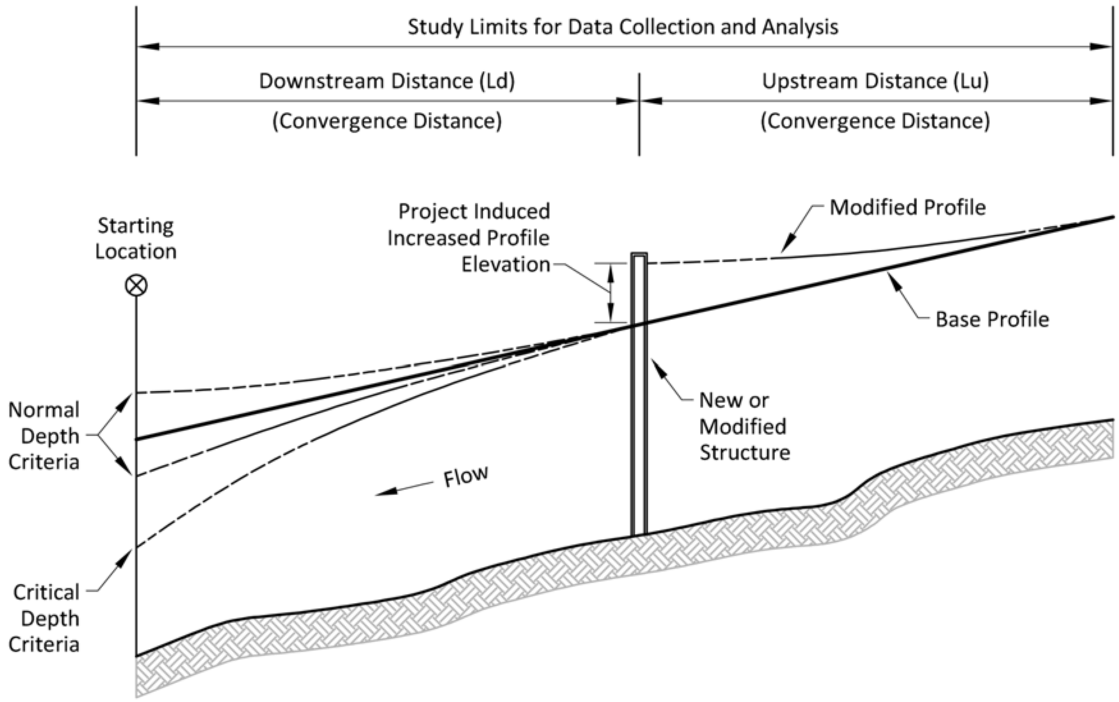


Figure 5-6. Profile Study Limits



References such as the 1984 USGS publication, "*Computation of Water-Surface Profiles in Open Channels*," and the USACE's "*Accuracy of Computed Water Surface Profiles*" are very valuable sources of additional guidance on the practical application of the step-backwater method to highway drainage problems involving open-channels. These references contain more specific guidance on stream reach determination and on cross section determination, location, and spacing.

5.7 Design Procedure

5.7.1 General

The design procedures for all types of channels include some common elements as well as some substantial differences. This section will outline a process for assessing a natural stream channel and a more specific design procedure for roadside channels.

5.7.2 Stream Channel Design Procedure

Stream-channel analysis is often conducted 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 damage the highway, stream, or adjacent property. An assessment of the existing channel is usually necessary to determine the potential for problems that might result from a proposed action. The detail of associated studies should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream and adjoining floodplain (see Chapter 6 *"Crossing Structures"* and Chapter 7 *"Channel Stability and Scour at Bridges"*).

Refer to Section 5.7.3, on the following page, for a step-by-step procedure. Although the procedure may not be appropriate for all possible applications, it does outline a process which will usually apply.

5.7.3 General Steps for Stream Channel Design

1. *Assemble site data and project file*
   a. Collect the following data, as available (see Chapter 3 *"Data Collection, Resources, and Tools"*).
      i. Topographic, site, and location maps
      ii. Roadway profile
      iii. Photographs
      iv. Field reviews
      v. Design data at nearby structures
      vi. Gaging records
   b. Find applicable studies by other agencies.
      i. FEMA FISs
      ii. Floodplain studies
      iii. Watershed studies
   c. Evaluate environmental constraints.
      i. Floodplain encroachment
      ii. Floodway designation
      iii. Stream crossing guidelines
      iv. Fish habitat
      v. Rare, threatened, or endangered species
      vi. Commitments in review documents
   d. Review the Design Criteria Section.

2. *Determine the project scope*
   a. Determine the level of assessment needed based on the following criteria:
      i. Stability of existing channel
      ii. Potential for damage
      iii. Sensitivity of the stream
   b. Determine the type of hydraulic analysis needed, selecting from the following:
      i. Qualitative assessment
      ii. Single-section analysis
      iii. Step-backwater analysis
   c. Determine what survey data is available and what additional survey may be needed, keeping in mind the following items:
      i. Extent of streambed profiles
      ii. Locations of cross sections
      iii. Elevations of flood-prone property
      iv. Details of existing structures
      v. Properties of bed and bank materials

3. *Evaluate hydrologic variables*
   a. Compute flow rates for selected AEPs.
   b. Consult Chapter 4 *"Hydrology."*

4. *Perform hydraulic analysis*
   a. For the single-section method (if appropriate):
      i. Select representative cross section.
      ii. Select appropriate Manning's n values (see Appendix A *"Manning's n Values"*).
      iii. Examine the stage-discharge relationship and confirm that the channel capacity is sufficient for the anticipated flows.
   b. For the step-backwater method (if appropriate):
      i. Select the study reach and identify upstream and downstream extents required to achieve convergence.
      ii. Identify appropriate cross section locations.
      iii. Select appropriate Manning's n values (see Appendix A *"Manning's n Values"*), including additional definition of bank and overbank areas.
   c. Calibrate with known high water elevations or previous studies.

5. *Perform stability analysis*
   a. Evaluate channel stability with respect to the following:
      i. Geomorphic factors
      ii. Hydraulic factors
      iii. Stream response to change
   b. See Chapter 7 *"Channel Stability and Scour at Bridges."*

6. *Design counter-measures*
   a. Use the following criteria for selecting counter-measures:
      i. Erosion mechanism
      ii. Stream characteristics
      iii. Construction and maintenance requirements
      iv. Vandalism considerations
      v. Cost
   b. Select from the following types of countermeasures:
      i. Meander migration countermeasures
      ii. Bank stabilization
      iii. Bend control countermeasures
      iv. Channel braiding countermeasures
      v. Degradation countermeasures
      vi. Aggradation countermeasures

5.7.4 Roadside Channel Design Procedure

A roadside channel is defined as an open channel usually paralleling the highway embankment and within the limits of the highway right-of-way. It is normally trapezoidal or V-shaped in cross section and lined with grass or a special protective lining.

The primary function of roadside channels is to collect surface runoff from the highway and areas that drain to the right-of-way and convey the accumulated runoff to acceptable outlet points. A secondary function of a roadside channel is to drain subsurface water from the base of the roadway to prevent saturation and loss of support for the pavement or to provide a positive outlet for subsurface drainage systems, such as pipe underdrains.

The alignment, cross section, and grade of roadside channels is usually constrained to a large extent by the geometric and safety standards applicable to the project. These channels should accommodate the design runoff in a manner that assures the safety of motorists and minimizes future maintenance, damage to adjacent properties, and adverse environmental or aesthetic effects.

Refer to section 5.7.7, on the following page, for a step-by-step procedure. Each project is unique, but the five basic design steps outlined in the procedure are normally applicable.

5.7.5 Design Considerations

In order to obtain the optimal roadside channel system design, the design engineer may need to iterate through the previous procedure several times before a final design is achieved.

More details on channel lining design may be found in HEC-15, *"Design of Roadside Channels with Flexible Linings,"* including consideration of channel bends, steep slopes, and composite linings.

5.7.6 Temporary Works

Throughout the design process, keep in mind that temporary works may be required in order to complete the construction of permanent works. Examples of temporary works include channel linings, bypass channels, cofferdams, stone fill causeways, or other structures. Temporary crossing structures are discussed separately in Chapter 6 *"Crossing Structures."*

Although temporary works do not need to meet the same standards as the permanent structures, consideration must be given to the anticipated flows that will be experienced during the period that temporary works will be deployed. At a minimum, temporary works must be designed to withstand the 50% AEP event, and their performance must be evaluated during the 10% AEP event.

5.7.7 General Steps for Roadside Channel Design

1. *Establish a roadside plan*
   a. Collect available site data.
   b. Obtain or prepare existing and proposed plan-profile layout including highway, culverts, bridges, etc.

2. *Obtain or establish cross section data*
   a. Provide channel depth adequate to drain the subbase and minimize freeze-thaw effects.
   b. Choose channel side slopes based on geometric design criteria.
   c. Establish the bottom width of trapezoidal channel (if applicable).
   d. Identify features which may restrict cross section design, such as:
      i. Right-of-way limits
      ii. Trees or environmentally sensitive areas
      iii. Utilities
      iv. Existing drainage facilities

3. *Determine initial channel grades*
   a. Plot initial grades on the roadway plan and profile layout. Slopes in roadside ditch in-cuts are usually controlled by highway grades.
   b. Provide minimum grade of 0.3% to minimize ponding and sediment accumulation.
   c. Where possible, avoid features that may influence or restrict grade, such as utility locations.

4. *Check flow capacities and adjust as necessary*
   a. Compute the design flow rate at the downstream end of channel segment (see Chapter 4 *"Hydrology"*).
      i. Break long channel segments into smaller reaches.
      ii. Compute the design flow rate for each segment in order to avoid over-sizing the upstream reaches of the channel.
   b. Set preliminary values of channel size, roughness coefficient, and slope.
   c. Determine the maximum allowable depth of the channel, including freeboard.
   d. Check flow capacity using Manning's Equation and the single-section method.
      i. Confirm that the downstream point of the analysis is not a constriction or subject to tailwater conditions.
      ii. Step-backwater analysis is required if tailwater conditions are predicted to be present during the design event.
   e. If the channel capacity is inadequate, possible adjustments are as follows:
      i. Increase bottom width
      ii. Make channel side slopes flatter
      iii. Make channel slope steeper
      iv. Provide smoother channel lining
      v. Install drop inlets and a parallel storm drain pipe beneath the channel to supplement channel capacity
   f. Check the design of channel lining once the required channel capacity has been achieved. Note that increases in capacity made by increasing the channel slope or decreasing the channel roughness may result in erosive flow velocities.
   g. Design the channel lining to accommodate anticipated flow velocities.
   h. Provide smooth transitions at changes in channel cross sections.
   i. Provide extra channel storage where needed to replace floodplain storage and/or to reduce peak flow rate.

5. *Analyze outlet points and downstream effects*
   a. Identify any adverse impacts to downstream properties which may result from one or more of the following at the channel outlet:
      i. An increase or decrease in flow rate
      ii. An increase in velocity
      iii. Confinement of sheet flow
      iv. Change in outlet water quality
      v. Diversion of flow from another watershed
   b. Mitigate any adverse impacts identified in 5a. Intermediate structures may be required to mitigate for these impacts and protect the channel and receiving waterbody from excess stormwater runoff. Possibilities include:
      i. Enlarge outlet channel and/or install control structures to provide detention of increased runoff in channel.

General Steps for Roadside Channel Design (Cont.)

ii. Install velocity control structures such as stone check dams or rock weirs.
iii. Increase capacity and/or improve lining of downstream channel.
iv. Install sedimentation/infiltration basins. Such basins may also be incorporated into the stormwater management plan and provide additional benefits to the receiving water.
v. Provide runoff disconnection where practicable through the use of level spreaders, infiltration strips, and other low impact development (LID) practices that promote infiltration and reduce concentrated channel flow.
vi. Install sophisticated weirs or other outlet devices to redistribute concentrated channel flow.
vii. Eliminate diversions that result in downstream damage and which cannot be mitigated in a less expensive fashion.

Chapter 6 Crossing Structures

6.1 *Introduction*

6.1.1 Overview

Crossing structures (i.e. bridges and culverts) provide conveyance of surface waters such as streams and stormwater flows across the highway right-of-way. The design of each bridge or culvert involves a structural component to ensure that it can support the anticipated loading from the roadway, a hydraulic component to ensure that it is sized in accordance with the requirements of the roadway classification, and a geomorphic component to ensure that it does not disrupt the dynamic equilibrium of the reach or impair the biological connectivity between upstream and downstream segments. This chapter presents hydraulic analysis methods and geomorphic principles used to design crossing structures. Refer to the hydrologic evaluation techniques described in Chapter 4 *"Hydrology"* to determine the design flow rate that must be accommodated for each structure before starting the hydraulic analysis.

This chapter has three primary purposes:

1. Provide guidance in the hydraulic design of a stream crossing system through the following:
    a. Appropriate policy and design criteria.
    b. Technical aspects of hydraulic design.
2. Incorporate geomorphic design principles that reduce the risk of damage to the structure and result in environmental benefits to the stream.
3. Present a design procedure that emphasizes hydraulic analysis using software tools such as the Federal Highway Administration (FHWA) HY-8 and the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center's River Analysis System (HEC-RAS). See Section 6.6 for more information about these programs.

6.1.2 Terminology

A culvert is defined as the following:

- A structure used to convey surface runoff through embankments.
- A structure, as distinguished from bridges, that is usually covered with embankment and is composed of structural material around the entire perimeter (Note that some are supported on a spread footing with the streambed serving as the bottom of the culvert).
- A structure that is usually hydraulically designed to take advantage of submergence to increase hydraulic capacity.

A bridge is defined as the following:

- A structure that transports roadways over waterways or other obstructions.
- Part of a stream crossing system that includes the approach roadway over the floodplain, relief openings, and the bridge structure.
- A structure, as distinguished from culverts, that is not composed of structural material around the entire perimeter and is not hydraulically designed for submergence.

6.1.3 Analysis and Design

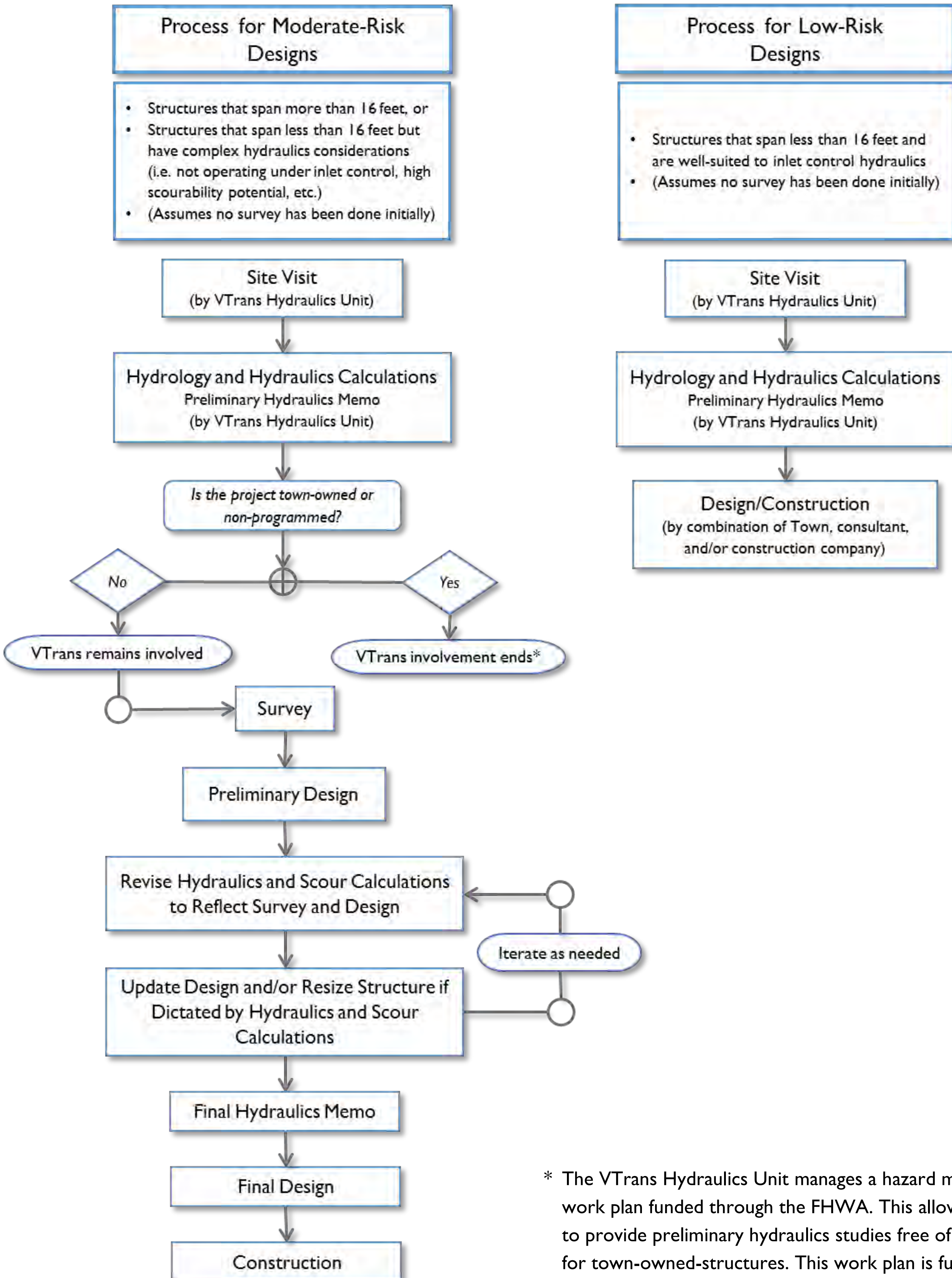
Proper hydraulic analysis and design is as vital as the structural design. In order for the hydraulic analysis to be accurate, the design engineer must first calculate the design flow rate that must be accommodated for each structure using the methods described in Chapter 4 *"Hydrology."*

Design stream crossing structures to meet multiple objectives:

- Accomplish transportation, social, economic, and environmental goals.
- Provide the desired level of hydraulic performance up to an acceptable design frequency or risk level.
- Avoid or mitigate impacts to the natural environment.
- Minimize cost while meeting the design criteria.

Provide a degree of analysis and design detail that is commensurate with the complexity of the project. Figure 6-1 illustrates the general flow of work associated with crossing structure projects of varied risk. The flow charts are simplified and intended to specifically detail the suggested level of effort for each type of project.

Figure 6-1. Crossing Structure Project Flow Charts



* The VTrans Hydraulics Unit manages a hazard mitigation work plan funded through the FHWA. This allows VTrans to provide preliminary hydraulics studies free of charge for town-owned-structures. This work plan is funded and approved on an annual basis and is not guaranteed.

6.1.4 Resources

6.1.4.1 Federal Highway Administration

For more information on hydraulic principles and engineering techniques of culvert and bridge design, refer to the following FHWA Hydraulic Design Series (HDS) publications:

- HDS-5, *"Hydraulic Design of Highway Culverts"*
- HDS-7, *"Hydraulic Design of Safe Bridges"*

For supplementary information about design principles and engineering techniques related to culvert and bridge design, refer to the following FHWA Hydraulic Engineering Circular (HEC) publications:

- HEC-9, *"Debris-Control Structures"*
- HEC-14, *"Hydraulic Design of Energy Dissipators for Culverts and Channels"*
- HEC-15, *"Design of Roadside Channels with Flexible Linings"*
- HEC-18, *"Evaluating Scour at Bridges"*

6.1.4.2 American Association of State Highway Transportation Officials

Key aspects of culvert design and a review of the subject are also discussed in the American Association of State Highway and Transportation Officials (AASHTO) 2007 publication, "*Highway Drainage Guidelines*," and the AASHTO 2011 publication, *"Roadside Design Guide."* These publications are not currently available for free online viewing, but can be purchased.

The FHWA and AASHTO publications also reference research and hydrologic studies that can provide the engineer with more in-depth guidance on specific aspects of hydrologic analysis.

6.1.4.3 Other Agencies

General permit and other state requirements concerning crossing structure design and construction can be found in the following locations:

- Vermont Agency of Natural Resources (ANR), *"Stream Alteration General Permit"* (ANR GP)
- USACE Vermont General Permit
- VTrans, *"Vermont State Design Standards"*
- Vermont ANR Department of Environmental Conservation (DEC), *"The Vermont Standards and Specifications for Erosion Prevention and Sediment Control"*

Additional information on geomorphic design principles can be found in the following Vermont state publications:

- Vermont ANR Fish and Wildlife Department (FWD), *"Guidelines for the Design of Stream/Road Crossings for Passage of Aquatic Organisms in Vermont"* (Bates, 2009)
- DEC, "*Vermont Standard River Management Principles and Practices: Guidance for Managing Vermont's Rivers Based on Channel and Floodplain Function"* (VT SRMPP)

Supplementary resources are available at the following agency website:

- ANR Rivers Program

6.1.5 Data Sources

6.1.5.1 Federal Emergency Management Agency

The Federal Emergency Management Agency (FEMA) has produced studies, reports, and web tools that can be useful aids in developing hydraulic analyses. These resources are free to the public and, when used effectively, they can save time and streamline effort.

FEMA's Map Service Center provides information about Flood Insurance Studies (FISs) and Flood Insurance Rate Maps (FIRMs), which FEMA produces to summarize estimated flood conditions in communities. Historically, FISs were issued for individual communities. Going forward, FISs will be issued on a county-wide basis. The studies follow a typical format:

Section 1- Introduction
Section 2- Area Studied
Section 3- Engineering Methods
Section 4- Flood Plain Management Applications
Section 5- Insurance Application
Section 6- FIRMs
Section 7- Other Studies
Section 8- Location of Data
Section 9- Bibliography

Supplemental Data:
Summary of Discharges Table
Floodway Data Tables
Flood Profiles

Be sure to review the FIS applicable to the study area to:

- Determine if the area of interest is within an area studied by approximate or detailed methods.

- Determine the methods used to estimate the hydrologic and hydraulic conditions at the area of interest.
- Determine the date the study was completed. Keep in mind that the older the FIS report is, the less likely it is to accurately depict existing conditions and adhere to up-to-date modeling techniques.
- Review the Summary of Discharges table to determine if the FIS includes peak flow rates at or near the area of interest. The FIS typically includes stream flow rates for the 10%, 2%, 1%, and 0.2% annual exceedance probability (AEP) events for areas studied using detailed methods.
- Review the FIS to determine if the area of interest is within a regulatory floodway. If so, the FIS's Floodway Data Table will include flood elevations for the 1% AEP event at representative locations along the river or stream.
- If the area of interest is not in the vicinity of a representative cross section, review the supplemental flood profiles to determine flood elevations at the area of interest for the 10%, 2%, 1%, and 0.2% AEP events.

Perform hydraulic analyses consistent with the methods described in this chapter and compare the results to the information published in the FIS. If there are discrepancies between flows reported in the FIS and existing conditions flows calculated by the designer using the methods presented in this manual, the designer should attempt to reconcile the differences. However, if there is sufficient confidence in the designer's values, these values should be used for design. Attempt to identify the likely causes of the discrepancies.

VTrans typically recommends that the designer:

- Complete a hydraulic model using hydrologic and hydraulic data from the FEMA FIS to demonstrate that the proposed structure conforms to FEMA floodplain regulations.
- Use hydrologic and hydraulic data derived using the methods in this manual to size and design crossing structures and to complete scour analyses.

6.1.5.2 Vermont Center for Geographic Information

The Vermont Center for Geographic Information (VCGI) is Vermont's clearinghouse for geographic information system (GIS) data. They host the following data that may be of use to the designer:

- Geospatial data and imagery
- Regional and local GIS data
- Links to the Interactive Map Viewer
- Dynamic online mapping tools
- Information about geospatial technology

6.1.6 Design Tools

Design tools help the designer transform raw data into something that can be interpreted and ultimately constructed to fulfill the goals of a project. VTrans recommends three primary tools for performing hydraulic analyses for culverts and bridges. All are explained in greater detail in their own subsections within Section 6.6:

- FHWA's HY-8 for culvert analysis
- USACE's HEC-RAS for bridge or culvert analysis
- U.S. Bureau of Reclamation (USBR) Sediment and River Hydraulics – Two Dimensional (SRH-2D)

FishXing is a free software tool that can be obtained from the U.S. Forest Service. This software provides assistance with the evaluation and design of culverts for fish passage.

6.2 *Guidelines*

6.2.1 General

Guidelines provide a set of goals that establish a course or method of action to determine present and future decisions (see Chapter 1 *"Hydraulic* Design Guidelines"). The following subsections summarize some of the applicable guidelines from different regulatory agencies.

6.2.2 Federal Guidelines

If the project involves federal funding, culvert and bridge designs must:

- Meet the policies of the FHWA applicable to floodplain management.
- Satisfy FEMA floodplain and floodway regulations and policies and USACE restrictions for permits.

6.2.3 VTrans Culvert Guidelines

The following guidelines are specific to culverts:

- Design culverts to satisfy the hydraulic performance criteria required for the roadway classification. Refer to Chapter 4 *"Hydrology"* to find the design frequency for each roadway classification. Refer to

Section 6.4.2 for information about hydraulic criteria for culvert design, which include:
  - Do not exceed the recommended allowable headwater.
  - Prevent roadway overtopping during the design storm.
- Select the overtopping storm event to be consistent with the roadway classification.
- For sites not covered by the FEMA National Flood Insurance Program (NFIP), backwater increases from culvert crossings should not exceed 1.0 foot during the passage of the 1% AEP storm event. Sites that are covered by the NFIP should satisfy FEMA floodplain and floodway regulations for backwater.
- Provide a degree of design detail that is commensurate with the complexity of the project.
- Include topographic features, channel characteristics, existing structures, and other related site-specific information where appropriate.
- Locate culverts in both plan and profile to reduce and avoid sediment build-up in culvert barrels.
- Evaluate erosion protection at culvert inlets and outlets.
- Design culverts to accommodate and reduce debris to the extent possible.
- Consider durability, including resistance to abrasion and corrosion, when selecting materials.
- Make provisions for aquatic organism passage (AOP).
- Weigh the cost savings of providing for multiple uses (utility accommodation, stock and wildlife passage, land access, etc.) against the advantages of separate facilities to meet site needs and regulatory requirements.
- Design culverts to minimize hazard to traffic and people.
- Provide accommodations for personnel and equipment access to facilitate maintenance.
- Assemble design data and calculations in an orderly fashion and retain for future reference.

Culverts, as discussed in this chapter, are generally straight structures that carry flows from one side of the roadway right-of-way to the other and do not substantially alter the timing or volume of such flows. These structures are distinct from more complex closed drainage systems that collect and manage stormwater flows from the right-of-way or from off-site areas and direct it to a stormwater management practice or other type of structure. For more information regarding closed drainage systems, refer to Chapter 8 *"Storm Drainage Systems"* and the AASHTO "*Highway Drainage Guidelines."*

6.2.4 VTrans Bridge Guidelines

The following guidelines are specific to bridges:

- Design bridges to satisfy the hydraulic performance criteria required for the roadway classification. Refer to Chapter 4 *"Hydrology"* to find the design frequency for each roadway classification. Refer to Section 6.5.1 for information about hydraulic criteria for bridge design, which include:
  - Allow the minimum freeboard above the water surface during the design storm.
  - Prevent roadway overtopping during the design storm.
- Use a water surface profile program, such as HEC-RAS, to evaluate bridge performance and demonstrate compliance with VTrans guidelines and design criteria.
- For sites not covered by the FEMA NFIP, backwater increases from bridge crossings should not exceed 1.0 foot during the passage of the 1% AEP storm event. Sites that are covered by the NFIP should satisfy FEMA floodplain and floodway regulations for backwater.
- Evaluate bridge foundation scour for the incipient overtopping event, the scour design event, and the scour check event. Refer to Chapter 7 *"Channel Stability and Scour at Bridges"* for help selecting the AEPs for each event and performing the required calculations. Detailed instructions on the calculation of Bridge Scour can be found in HEC-18, *"Evaluating Scour at Bridges."*

6.2.5 VT Department of Environmental Conservation Guidelines

Crossings and other activities within perennial streams must meet the statutory criteria and performance standards outlined in the ANR GP. Perennial streams are generally those with contributing watershed areas greater than 0.5 square miles, but streams with smaller drainage areas may also be characterized as perennial.

The two primary requirements of the ANR GP are as follows:

1. *Equilibrium Standard*—Avoid vertical stream bed adjustments due to disruptions of sediment transport through the reach.

2. *Connectivity Standard*—Avoid abrupt changes to or disconnects within the horizontal alignment of stream banks or vertical profile of the stream bed.

The ANR GP and supporting documentation available from the ANR Rivers Program describe these standards in more detail.

6.2.6 U.S. Army Corps of Engineers Guidelines

Section 404 of the Clean Water Act regulates work within navigable waters of the United States, specifically work below ordinary high water (OHW) that involves dredge or fill materials.

The USACE provides information regarding the specific requirements for work under the USACE Vermont General Permit.

6.3 Site Design Criteria

Design criteria are the tangible means for placing accepted policies into action, and they become the basis for the selection of the final design configuration of the stream-crossing system. Criteria are subject to change when conditions dictate, as approved by VTrans.

6.3.1 Structure Type Selection

Closed-bottom culverts are typically used:

- Where bridges are not hydraulically required.
- Where debris and ice are tolerable.
- Where more economical than bridges.

Open-Bottom Culverts are typically used:

- Where bridges are not hydraulically required but where closed-bottom culverts do not meet the ANR GP criteria.
- Where bedrock or other subsurface conditions preclude use of embedded closed-bottom culverts.
- Where more economical than embedding a similarly-sized closed-bottom culvert or constructing a bridge.

Bridges are typically used:

- Where culverts cannot be used.
- Where more economical than culverts.
- To satisfy land use requirements.
- To mitigate environmental harm caused by culverts.
- To avoid floodway encroachments.
- To accommodate ice and large debris.

6.3.2 Configuration and Placement

Select the structure that best accommodates existing topography while minimizing structure length and meeting roadway requirements. To the extent practicable, take the following steps:

- Select a site that permits the structure to be constructed and maintained while limiting the impact to the stream or adjacent wetlands.
- Align the structure with the orientation of the existing stream channel and avoid sharp meander bends that are prone to failure or require extensive armoring.
- Position and design the structure entrance to direct flow into the structure.
- Match the slope through the structure to the slope of the upstream and downstream channel reaches in order to provide sediment continuity and minimize excess sediment accumulation or degradation.
- Conduct additional geomorphic design if the structure is located at a change in the channel slope or has resulted in a large discontinuity between the upstream and downstream reaches. Provide a smooth transition through the structure that can accommodate future changes to the channel profile. Refer to the DEC's VT SRMPP and the FWD's "*Guidelines for the Design of Stream/Road Crossings for Passage of Aquatic Organisms in Vermont*" for additional guidance on designing effective transitions between reaches and installing grade control structures to protect these transitions.

6.3.3 Scour Sensitivity

During the preliminary design stage, the designer should look at existing scour and scour sensitivity in the field and factor it into design considerations.

6.3.4 Ice Build-up

Evaluate the likelihood of ice build-up that would result in property flooding or damage to the structure. Certain sites are more prone to ice build-up due to their channel configuration and are best evaluated from historic knowledge of the site. Obtain this information during the project planning phase from the municipality, the District Maintenance office, or other sources such as the USACE Cold Regions Research and Engineering Laboratory (CRREL). The CRREL conducts multiple ice jam studies across the State of Vermont and is a valuable resource for obtaining ice jam data.

6.3.5 Debris Control

Evaluate the likelihood of debris build-up that would result in property flooding or damage to the structure, and design debris control countermeasures per HEC-9, *"Debris Control Structures"* in the following circumstances:

- For structures prone to damage or failure due to clogging by debris.
- For culverts located in mountainous or steep regions.
- For culverts under high fills (greater than 15 feet).
- Where experience or physical evidence indicates the watercourse will transport a heavy volume of debris.
- For sites with limited clean-out access. Access must be provided for equipment and personnel to clean out the debris control countermeasure.

Quick Tip

Designers should not revert to debris control structures as the preferred method for controlling debris. Debris control structures can be difficult to acquire permits for and require regular maintenance. Structures that span bankfull width (BFW) stand a much lower risk of clogging from debris.

6.3.6 Design Frequency

It is not generally economical to build local roads to the same hydraulic standards as major highways, so each roadway classification has an associated minimum design frequency, which refers to an event with a designated AEP that the roadway crossing must be hydraulically capable of conveying without flooding and becoming impassable. Refer to Chapter 4 *"Hydrology"* for more information about design frequencies by roadway classification. Table 4-2, which was originally presented in Chapter 4 with additional supporting information, is reprinted here for quick reference.

For all structures over perennial streams, consider the potential effects of the 1% AEP storm event on upstream property, the environment, hazards to human life, and floodplain management criteria.

Table 6-1. Minimum Design Frequency by Roadway Classification

Roadway Classification	AEP (%)
Freeways	1%
Principal arterial roads and streets	2%
Minor arterial roads and streets	2%
Collector roads and streets	2%
Local roads and streets	4%
Limited access roadways	*
Roadside, median, and storm drain systems	**
Railroads	2%

* *At the discretion of the VTrans Hydraulics Engineer.*

** *Refer to Chapter 8 "Storm Drainage Systems."*

6.3.7 Erosion and Sediment Control

Include temporary erosion and sediment control measures in the construction plans. Commonly used measures include temporary and permanent mulch, silt fence, rolled erosion control products (RECPs), cofferdams, and stone check dams.

Where applicable, determine if flumes, pumps or other forms of bypass will be necessary to manage stream and storm flows during the construction process. Include details for these measures on the construction plans.

Refer to Chapter 4 of the DEC publication, "*The Vermont Standards and Specifications for Erosion Prevention and Sediment Control*," for more information.

6.3.8 Temporary Crossing Structures

Design temporary crossings to meet the following criteria presented in the USACE Vermont General Permit under the section for Waterway/Wetland Work and Crossings:

- All temporary and permanent crossings of waterbodies shall be suitably culverted, bridged, or otherwise designed to withstand and to prevent the restriction of high flows, to maintain existing low flows, and to not obstruct the movement of aquatic life indigenous to the waterbody beyond the actual duration of construction.
- Temporary culverts must be embedded unless they're installed during low flow (Jul. 15 – Oct. 1) and it's placed on geotextile fabric laid on the stream bed to ensure restoration to the original grade.
- The following apply to temporary crossings in place for more than 90 days:
  - o No activity may substantially disrupt the necessary life-cycle movements of those species

of aquatic life indigenous to the waterbody, including those species that normally migrate through the area, unless the activity's primary purpose is to impound water.

Additionally, design temporary bridges to meet the following hydraulic criteria:

- Provide a clear span that is as large as the existing structure (if one is/was present).
- Accommodate peak flows corresponding to the expected construction duration, as specified in Table 6-2.
- Consider the following factors:
  - Effects on surrounding property and buildings
  - Velocities that would cause excessive scour
  - Damage or inconvenience due to failure of temporary structure
  - ANR concerns

Table 6-2. Minimum Temporary Bridge Capacity

Construction Duration	Design Storm AEP (%)
up to 3 months	20%
3 months to 6 months	10%
6 months to 1 year	4%
1 year to 2 years	2%

Note: Temporary bridge design frequency does not need to be more conservative than the design frequency for the final design.

The hydraulic capacity and durability of temporary crossing structures must take into consideration the length of time that the structure is anticipated to be required. Structures that are expected to remain in place through one or more winter seasons are more likely to experience high flows, debris, or ice conditions that could compromise less robust structures.

6.4 Culvert Design Criteria

The following criteria are specific to culverts and should be considered in addition to the site design criteria covered in Section 6.3,

6.4.1 Analysis

A complete theoretical analysis of culvert flow is extremely complex because the following steps are required:

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

Detailed explanations of the analytical solutions to these problems are given in HDS-5, *"Hydraulic Design of Highway Culverts."* Software tools such as FHWA's HY-8 culvert analysis tool simplify the iteration process and minimize computational errors; however, the design engineer should have a thorough understanding of the underlying theoretical calculations prior to running a computer model.

Culvert Design Tip

The analysis described herein is focused predominantly on clear-water hydraulics. The ANR GP incorporates additional requirements for passage of sediment and aquatic organisms when the culvert carries a perennial stream. These requirements generally result in designs that provide additional hydraulic capacity above that of a pure "clear-water" hydraulic model. Begin design of new structures with the ANR GP requirements and use the clear-water hydraulic analysis to confirm that the structure also meets the hydraulic criteria presented in this manual.

6.4.2 Culvert Design Limitations

6.4.2.1 Allowable Headwater and Backwater

Allowable headwater is the depth of water that can be ponded upstream of a culvert or bridge. Headwater refers to the depth measured from the flow line (invert) of the culvert inlet to the water surface elevation (WSE).

Backwater is the increase in the upstream water surface level resulting from an obstruction to flow, such as a roadway fill with a bridge or culvert opening placed on the floodplain. Backwater should be evaluated two model cross sections upstream of the proposed structure (when using HEC-RAS) or just upstream of the inlet drawdown effects, and it is the maximum difference between the normal WSE and the WSE resulting from the obstruction to flow.

The allowable headwater and backwater for a given crossing structure are governed by a number of site criteria and design requirements, which include but are not limited to the following:

- The headwater should be no higher than the roadway shoulder during the design event.
- The headwater corresponding to the design event should not adversely affect property to a greater degree than what occurs under existing conditions. (In general, damage occurs when floodwaters enter the interior of buildings or other structures.)
- For sites not covered by the FEMA NFIP, backwater increases from culvert crossings should not exceed 1.0 foot during the passage of the 1% AEP storm event. Sites that are covered by the NFIP should satisfy FEMA floodplain and floodway regulations for backwater.
- From a hydraulic perspective, allowable headwater is dictated by the ratio of headwater depth to culvert diameter (or rise), which is HW/D, where HW is headwater depth in feet, and D is the rise or diameter of the culvert in feet. Table 6-3 presents hydraulic criteria for allowable headwater.

Table 6-3. Hydraulic Criteria for Allowable Headwater at Culverts

Culvert Rise	Allowable Headwater
≤ 36 inches	≤ 1.5 times the culvert rise at the design frequency ($HW/D \leq 1.5$)
> 36 inches	≤ 1.2 times the culvert rise at the design frequency ($HW/D \leq 1.2$)
> 60 inches	≤ 1.2 times the culvert rise at the design frequency ($HW/D \leq 1.2$). In addition, check that $HW/D \leq 1.5$ during the 1% AEP storm event regardless of performance at other frequencies

The ANR GP identifies additional sizing and configuration requirements that apply to perennial stream crossings. Refer to Section 6.4.3.1 for more information about how these requirements pertain to culverts. In most cases, the ANR GP requirements will drive the designer to select a larger culvert than the hydraulic headwater requirements.

Surcharging of culverts ($HW/D > 1.0$) at sites with high sediment and debris loads contributes to debris jams and increases the risk of failure. Figure 6-2 illustrates examples of the increased plugging hazard caused by debris blockage in undersized culverts. Evaluate the potential input of large woody debris and sediment at sites with known history of clogging and provide a factor of safety (i.e. $HW/D \leq 0.8$) to allow passage of debris during large storm events.

6.4.2.2 Tailwater Relationship—Channel

Evaluate the tailwater conditions associated with the downstream channel through one or more of the following:

- Evaluate the hydraulic conditions of the downstream channel to determine tailwater depths for a range of flood flows.
- Perform a backwater analysis using HEC-RAS or another step-backwater model in order to determine the tailwater elevation to be used in HY-8 if the culvert tailwater is affected by downstream controls such as natural stream constrictions, irregular downstream cross sections, obstructions, impoundments, or backwater from another stream or body of water.
- Use the critical depth and equivalent hydraulic grade line (HGL) if the culvert outlet is operating with a free outfall.
- Use the headwater elevation of any nearby, downstream culvert if it is greater than the channel depth.

6.4.2.3 Tailwater Relationship—Confluence

Tailwater relationships at confluences can be very complex and require evaluating the probability of peak discharges coinciding for the study river and the receiving river. See Table 6-6 within Section 6.6.4.2 for more information.

6.4.2.4 Storage—Temporary or Permanent

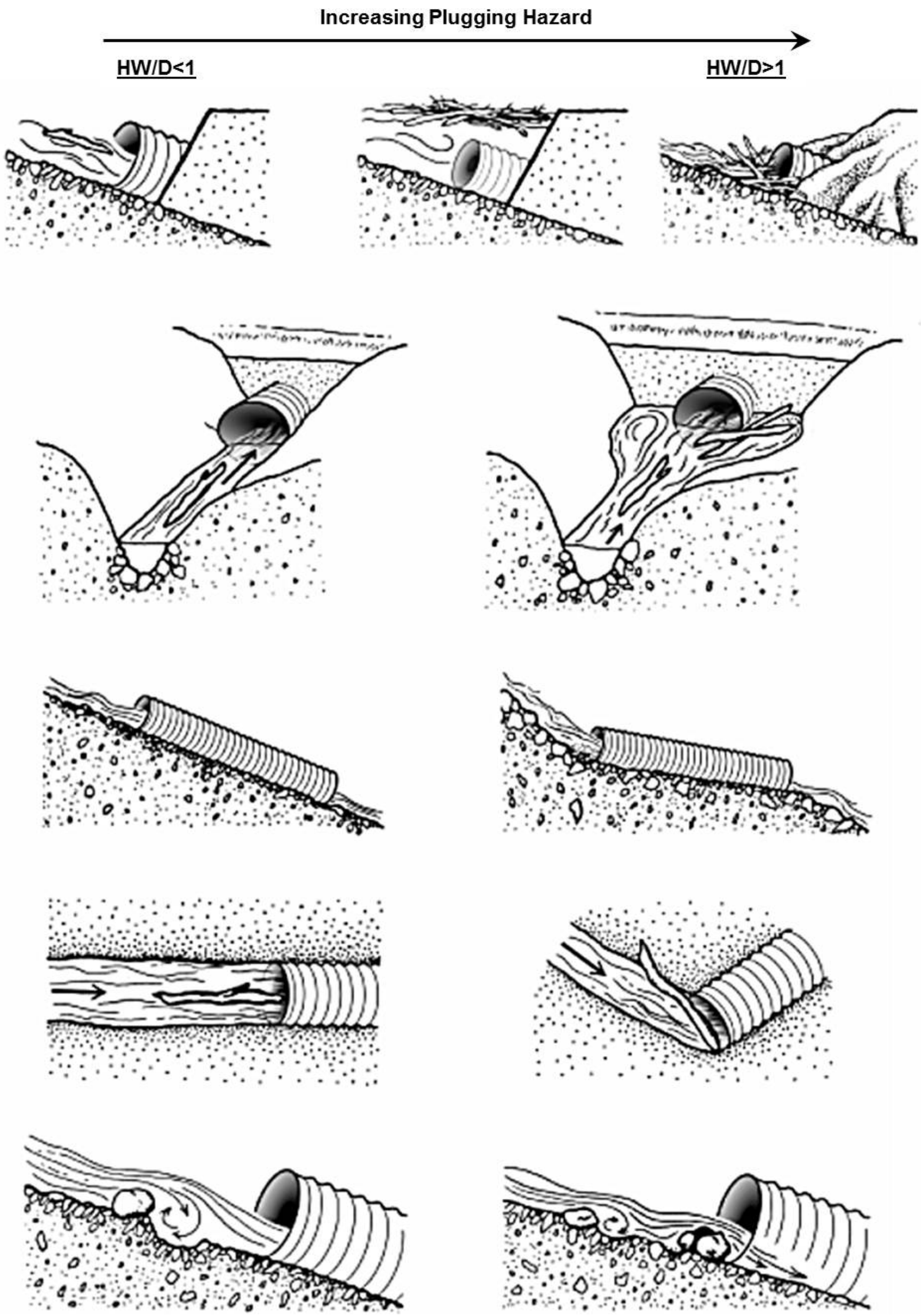
If storage is being assumed upstream of the culvert:

- Consider the total area of flooding and confirm that no structures or transportation facilities will be negatively affected.
- Limit the average time that bankfull stage is exceeded for the design event to 48 hours in rural areas or 6 hours in urban areas.

6.4.2.5 Maximum Velocity

Ensure that the maximum velocity at the culvert exit is consistent with the velocity in the natural channel, or provide appropriate mitigation through channel stabilization or energy dissipation. See HEC-14, *"Hydraulic Design of Energy Dissipators for Culverts and Channels"* and HEC-15, *"Design of Roadside Channels with Flexible Linings"* for appropriate design methods.

Figure 6-2. Examples of Culvert Plugging Hazard



6.4.2.6 Minimum Velocity

Ensure that the minimum velocity in the culvert barrel results in a shear stress (τ) greater than critical shear stress (τ_{cr}) of the transported streambed material at low flow rates. This will help to minimize unwanted sediment accumulation within the culvert.

$$\tau = \gamma D S$$

Where:

τ = shear stress, lb/ft^2

γ = specific weight of water, lb/ft^3

D = mean depth, ft

S = slope of the water surface, ft/ft

Use a minimum velocity of 2.5 feet per second when the size of streambed material size is unknown. If clogging is probable, consider installing a sediment trap, or size the culvert to facilitate cleaning.

6.4.2.7 Aquatic Organism Passage

In addition to meeting structural and hydraulic requirements, new or replacement structures must also make provisions for AOP. These accommodations allow the passage of fish and other aquatic organisms through structures located on natural stream channels. Such accommodations provide ecological connectivity and sediment continuity between the upstream and downstream reaches and reduce the environmental impact of the structure.

Consider culvert length, size, material, and the need for baffles or buried inverts early in the design process. Obtain input from resource agencies regarding AOP considerations, which are incorporated into the ANR GP and the USACE Vermont General Permit. For more information on stream crossing requirements, contact the ANR Rivers Program.

Additional guidance on designing crossings to support AOP through crossing structures can be found in the FWD's "*Guidelines for the Design of Stream/Road Crossings for Passage of Aquatic Organisms in Vermont*."

FishXing is a free software tool that can be obtained from the U.S. Forest Service. This software provides assistance with the evaluation and design of culverts for fish passage.

6.4.3 Culvert Design Features

6.4.3.1 Sizes and Shapes

Select culvert size and shape based on engineering and economic criteria related to site conditions. The following minimum sizes are recommended to avoid maintenance problems and clogging:

- 18 inches for all highways
- 15 inches for drives and bike/pedestrian paths

For authorization under the ANR GP, structures spanning perennial streams must have an opening that is at least equal to the width of the bankfull channel and must have an opening height that is at least 4 times the depth of the average bankfull depth. Smaller structures may be allowable on a case-by-case basis following consultation with an ANR River Management Engineer.

6.4.3.2 Multiple Barrels

The use of multiple barrel culverts is discouraged due to their increased susceptibility to clogging and failure to comply with the ANR GP. If the use of multiple barrels is unavoidable, they should fit within the naturally dominant channel with minor channel widening so as to avoid sediment deposition in any of the barrels and the reduced conveyance that would result. Evaluate the need for a debris control structure to reduce the potential for clogging.

Avoid multiple barrels in situations where:

- Approach flow is high velocity, and particularly if approach flow is supercritical (these sites require either a single barrel or special inlet treatment to avoid adverse hydraulic jump effects).
- Fish passage is required (unless a special provision is made to ensure adequate conveyance of low flows, commonly accomplished by lowering one barrel).

6.4.3.3 Material Selection

Consider hydraulic performance, replacement cost, and difficulty of construction when selecting culvert materials.

In general, choose culverts with corrugated rather than smooth-lined interiors for situations where the culvert outlets directly to the ground. Hydraulic roughness (*n*) has a significant effect on velocities and performance if the culvert is in outlet control. Tables of Manning's *n* values for different materials are provided in Appendix A *"Manning's n Values."*

Select the material by comparing the total cost for the design life of the structure. Do not make the selection using the initial construction cost as the only criteria. The design life of the structure depends on:

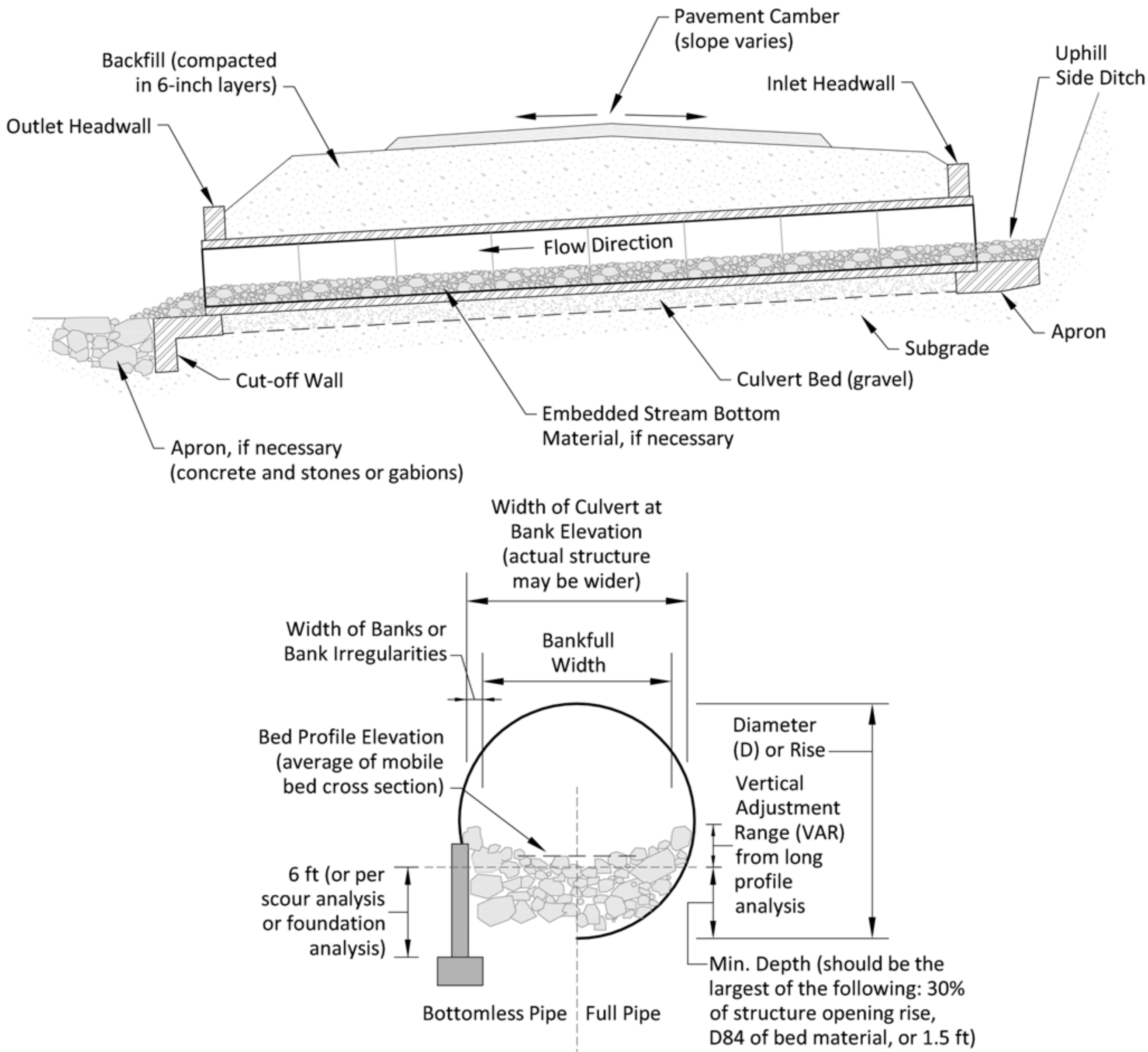
- Material durability
- Structural strength (minimum and maximum cover depths per manufacturer recommendations)
- Bedding conditions
- Abrasion and corrosion resistance
- Degree of water tightness (to minimize risk of soil piping)

6.4.3.4 Embedded Materials

The ANR GP calls for closed bottom structures to be embedded to 30% of the height of the opening. Deeper embedding is required at sites dominated by boulder-sized bed material. Less embedding is permitted at sites with channel slopes of 0.5% or less. Consult with an ANR River Management Engineer to confirm that the design will meet the equilibrium and connectivity performance standards of the ANR GP.

Figure 6-3 shows a typical culvert section—either open bottomed or a closed culvert—with embedded materials.

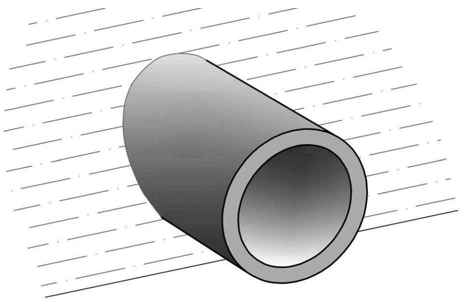
Figure 6-3. Typical Culvert Section Showing Embedded Materials



6.4.3.5 End Treatment (Inlet or Outlet)

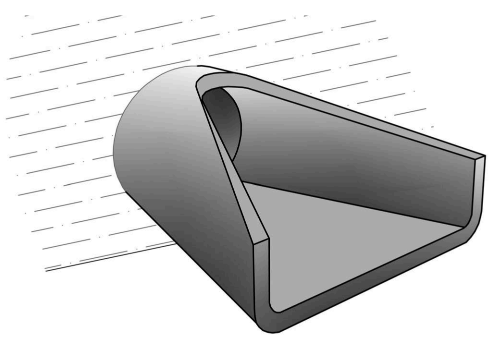
Select the culvert inlet type from the following list based on the considerations given and the inlet coefficient (K_E). Give appropriate consideration to safety requirements because some end treatments can be hazardous to errant vehicles.

1. *Projecting Inlets or Outlets:*



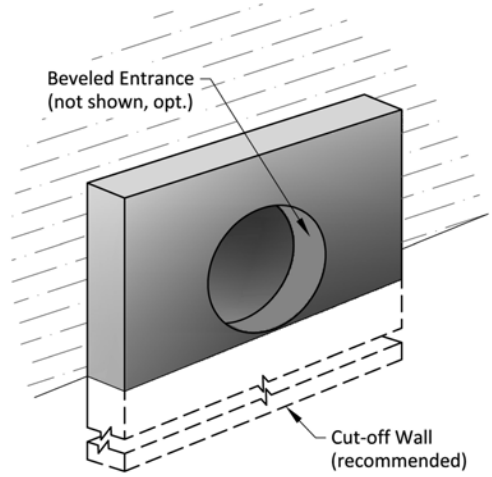
- Extend beyond the embankment of the roadway.
- Have low construction cost.
- Are susceptible to damage during roadway maintenance and automobile accidents.
- Have poor hydraulic efficiency for thin materials.
- Are used predominantly with metal pipe.

2. *Commercial End Sections:*



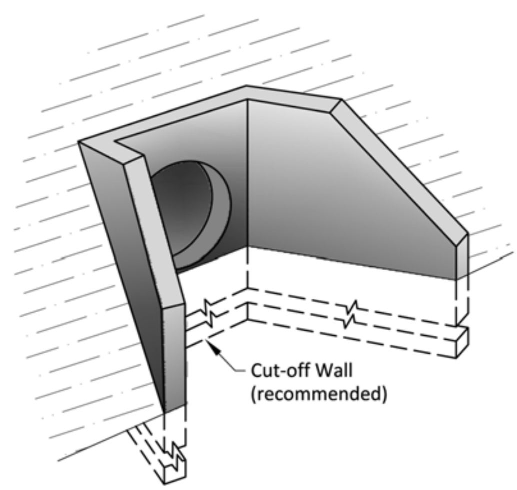
- Are available for metal, concrete, and plastic pipes.
- Retard embankment erosion and reduce damage from maintenance.
- Improve projecting metal pipe entrances by increasing hydraulic efficiency, reducing the accident hazard, and improving their appearance.
- Are hydraulically equal to headwalls and can also be hydraulically equal to beveled or side-tapered entrances if a flared, enclosed transition takes place before the barrel.

3. *Headwalls and Headwalls with Bevels:*



- Increase the efficiency of pipes.
- Provide embankment stability and erosion protection.
- Provide protection from buoyancy.
- Shorten the required structure length.
- Reduce damage from maintenance.
- Should incorporate a cut-off wall to provide protection from undercutting. The cut-off wall should extend to a depth equal to the culvert rise, up to 4 feet.

4. *Wingwalls:*

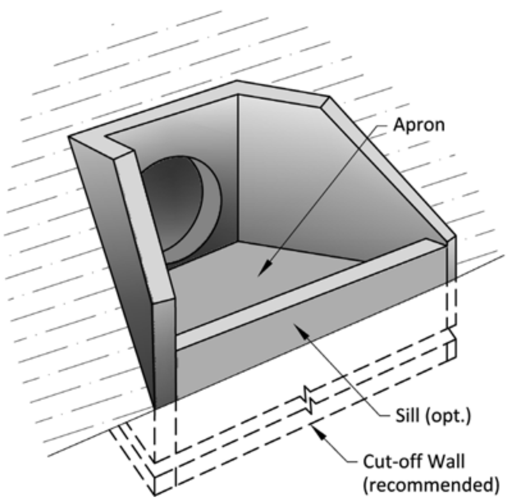


- Help to retain the roadway embankment.
- Are used if the side slopes of the channel are unstable.
- Are used if the culvert is skewed to the normal channel flow.
- Can improve hydraulic efficiency and reduce turbulence if the flare angle is between 30 degrees

and 60 degrees, as measured from the face of the culvert.

- Should incorporate cut-off walls to provide protection from undercutting. Cut-off walls should extend to a depth equal to the culvert rise, up to 4 feet.
- Wingwalls may not require cut-off walls if the headwall already has one. Perform a force analysis prior to making that determination. As an alternative, wingwalls may be cantilevered.

5. *Aprons:*



- Reduce scour at the inlet or outlet from high headwater depths and rapid approach velocities.
- Should extend at least a distance equal to the culvert rise upstream or downstream.
- Should incorporate cut-off walls to provide protection from undercutting. Cut-off walls should extend to a depth equal to the culvert rise, up to 4 feet.
- Wingwalls may not require cut-off walls if the headwall already has one. Perform a force analysis prior to making that determination. As an alternative, wingwalls may be cantilevered.

6.4.3.6 Safety Considerations

Culvert ends present a hazard to errant traffic. Minimize this hazard by specifying end sections for small culverts (30 inches in diameter or less) or incorporating one of the following treatments for culverts greater than 30 inches in diameter:

- Extend culvert to the appropriate "clear zone" distance per the AASHTO *"Roadside Design Guide*."
- Incorporate a grate into the design if the consequences of clogging and causing a potential flooding hazard are less than the hazard of vehicles impacting an unprotected end. If a grate is used, ensure the open area between the bars is equivalent to 1.5 to 3.0 times the area of the culvert entrance.
- Shield the culvert with a traffic barrier if the structure is very large, cannot be extended, has a channel which cannot be safely traversed by a vehicle, or is at a significant risk of flooding with a grate.

6.4.3.7 Weep Holes

Design weep holes in a manner similar to underdrain systems if they are needed to relieve uplift pressure.

6.4.3.8 Overtopping Analysis

Evaluate the stage-discharge curve for culverts with backwater conditions that threaten to overtop the roadway or affect nearby property. This evaluation will demonstrate the potential consequence of high flow rates at the site and provide a basis for evaluating flood hazards.

6.4.4 Culvert-Related Designs

6.4.4.1 Buoyancy Protection

Consider incorporating headwalls, endwalls, slope paving, and other means of anchoring to provide buoyancy protection for all flexible culverts. Buoyancy concerns become more serious as the steepness of the culvert slope, the depth of the potential headwater, the flatness of the upstream fill slope, the height of the fill, and the skew of the culvert increase. Keep in mind that debris blockage may increase the depth of the potential headwater.

6.4.4.2 Outlet Protection

Specify stone fill or another form of outlet protection and energy dissipation in accordance with the design guidelines published in FHWA's HEC-14, *"Hydraulic Design of Energy Dissipators for Culverts and Channels"*. At a minimum, place stone fill as follows in Table 6-4.

In lieu of calculating a specific stone size, use stone pads at the outlets of culverts as follows in Table 6-4 and Table 6-5. Additional information on erosion protection may be found in Chapter 5 *"Open Channels."* Outlet velocity and stone size must be calculated if:

- The culvert span is greater than 5 feet, or
- Flow through the culvert is supercritical, regardless of size.

Table 6-4. Stone Fill Type Required at Culvert Outlets

Culvert Rise (ft)	Design Discharge (cfs)														
	10	20	30	40	50	60	70	80	90	100	120	140	160	180	200
2	I	II	II	IV	—	—	—	—	—	—	—	—	—	—	—
2.5	I	I	II	II	III	IV	—	—	—	—	—	—	—	—	—
3	I	I	II	II	II	II	III	III	IV	—	—	—	—	—	—
3.5	I	I	I	II	II	II	II	II	III	III	IV	—	—	—	—
4	I	I	I	I	II	II	II	II	II	II	III	IV	—	—	—
4.5	I	I	I	I	I	II	II	II	II	II	II	III	III	IV	—
5	I	I	I	I	I	I	II	II	II	II	II	II	III	III	IV

Note: Roman numerals denote stone fill type. Complete a detailed design using HEC-14 if any of the following conditions are true: (1) Culvert span is greater than 5 feet, (2) Flow through culvert is supercritical (3) The table value corresponding to the selected design is denoted with "—".

The different types of stone fill are classified as follows:

- *Type I.* The longest dimension of the stone should vary from 1–12 inches, and the median particle diameter (D_{50}) of the stone should be 4 inches.
- *Type II.* The longest dimension of the stone should vary from 2–36 inches, and the D_{50} of the stone should be 12 inches.
- *Type III.* The longest dimension of the stone should vary from 3–48 inches, and the D_{50} of the stone should be 16 inches.
- *Type IV.* The longest dimension of the stone should vary from 3–60 inches, and the D_{50} of the stone should be 20 inches.

Outlet velocities greater than 16 feet per second are not desirable due to the effects of scour and the potential for damage to the structure and receiving channel. For these situations, consider alternate designs that will create lower velocities. If an alternate design is not practicable, incorporate an energy dissipator into the design.

Riprap aprons are the most commonly used outlet protection for culverts with rises and spans that are 5 feet or smaller. They serve to spread flow and provide a transition to the natural drainage way. Table 6-4 was developed using the following equation published by Fletcher and Grace (1972) for circular culverts. For the purposes of the table, the selected stone was assumed to have a specific weight of 165 pounds per cubic foot, the tailwater was assumed to be 0.4 D, and flow was assumed to be subcritical. The tailwater depth for the equation should be limited to between 0.4 D and 1.0 D.

$$D_{50} = 0.2\mathrm{D}\left(\frac{\mathrm{Q}}{D^{2.5}\sqrt{g}}\right)^{4/3}\left(\frac{D}{TW}\right)$$

Where:

D_{50} = median particle diameter, ft
D = culvert diameter (circular), ft
Q = design discharge, cfs
TW = tailwater depth, ft
γ_W = specific weight of water (62.5 lb/ft^3)
g = gravitational acceleration, ft/s^2

If flow through the culvert is supercritical, the culvert diameter must be adjusted as follows:

$$D' = \frac{D + y_n}{2}$$

Where:

D' = adjusted culvert rise, ft
y_n = normal (supercritical) depth in culvert, ft

Aprons must be properly dimensioned in order to be effective; otherwise erosion will occur downstream of the riprap. Table 6-5 provides guidance on the apron length and depth based on the stone fill type. For culverts without commercial end sections, add an additional 3 feet onto the indicated apron length.

Table 6-5. Stone Fill Apron Dimensions at Culvert Outlets with Commercial End Sections

Stone Fill Type	D_{50} (in)	Apron Length	Apron Depth (ft)
Type I	4	4 X Culvert Span	1.2
Type II	12	5.5 X Culvert Span	2.3
Type III	16	6.5 X Culvert Span	2.8
Type IV	20	7 X Culvert Span	3.3

Note: For culverts without commercial end sections, add an additional 3 feet onto the apron length.

The starting width of the apron should be three times the culvert span, and it should widen at a 1:3 slope (W:L) on either side of the culvert for the full apron length.

6.4.4.3 Relief Opening

Provide stabilization measures to prevent erosion if multiple-use culverts or culverts serving as relief openings have their outlet set above the normal stream flow line.

6.4.4.4 Scour Holes and Energy Dissipators

Scour holes at culvert outlets are useful as efficient energy dissipators. However, outlet protection for the design event should be provided if the outlet scour hole depth analysis indicates that the scour hole:

- Will undermine the culvert outlet;
- May cause costly property damage;
- Causes a nuisance effect;
- Blocks fish passage; or
- Will restrict land-use requirements.

Evaluate existing scour holes in conjunction with culvert replacement projects. Consider partially filling or reconfiguring the scour hole if the scour analysis indicates that the hole is larger than necessary to provide energy dissipation for the proposed replacement structure.

Design energy dissipators, where required, in accordance with FHWA's HEC-14, *"Hydraulic Design of Energy Dissipators for Culverts and Channels"*.

6.4.4.5 Repair and Rehabilitation

Existing culverts that are nearing the end of their useful design life may be repaired or rehabilitated using minimally-disruptive construction techniques such as slip-lining. Such techniques may extend the service life of the structure by several to many years, avoiding a road closure that would otherwise be required to completely replace the structure. Although a slip-lined structure may result in a similar hydraulic capacity from the standpoint of clear-water hydraulics, it may reduce the ability of the structure to pass aquatic organisms between the downstream and upstream reaches. In fact, the resulting higher flow velocities may serve to preclude the passage of slower-swimming organisms. Under the USACE Vermont General Permit, slip-lining a culvert is not permitted as a Category 1 activity. Obtaining a Category 2 or Individual Permit requires a more intensive application and reporting process. See the USACE Vermont General Permit for a complete explanation of which activities fall under which permitting category.

6.5 Bridge Design Criteria

The following criteria are specific to bridges and should be considered in addition to the site design criteria covered in Section 6.3,

6.5.1 Bridge Design Limitations

6.5.1.1 General Criteria

The following are the AASHTO general criteria related to the hydraulic analysis for the location and design of bridges as stated in the *"Highway Drainage Guidelines."*

- Design the stream crossing so that backwater does not significantly increase flood damage to property upstream of the crossing. For sites not covered by the FEMA NFIP, backwater increases from bridge crossings should not exceed 1.0 foot during the passage of the 1% AEP storm event. Sites that are covered by the NFIP should satisfy FEMA floodplain and floodway regulations for backwater.
- Design the stream crossing so that velocities through the structure(s) do not damage the highway facility or increase damages to adjacent property.
- Maintain the existing flow distribution to the extent practicable.
- Design piers and abutments to minimize flow disruption and potential scour by selecting appropriate spacing and orientation.
- Design foundations to an appropriate depth to avoid failure by scour during scour design and scour check events identified in Chapter 7 *"Channel Stability and Scour at Bridges."*
- Meet performance criteria at structure(s) designed to pass anticipated debris and ice.
- Determine the acceptable risk of damage and implement appropriate measures to anticipate potential changes in geomorphology and other stream characteristics.
- Minimize disruption of ecosystems and values unique to the floodplain and stream.
- Provide a level of traffic service compatible with the class of highway and projected traffic volumes.
- Make design decisions that consider costs for construction, maintenance, and operation, including probable repair and reconstruction and potential liability.

6.5.1.2 Freeboard

Freeboard is the vertical distance between the WSE and the low chord of a bridge. The minimum freeboard required is 1.0 foot at the design frequency appropriate for the roadway classification. For structures where the low chord is sloped between banks and/or across the deck cross section, use the elevation of the lowest point on the bottom of the bridge deck as the low chord elevation. See Figure 6-4 for a diagram depicting this situation.

> **Quick Tip**
>
> When determining the required structure freeboard based on model results, use the predicted WSE two model cross sections upstream of the structure. Standard practice is often to find the predicted upstream WSE at a distance equal to the bridge span, but the key is to get outside of the area contracted by the bridge, which could be more or less than one bridge span upstream.

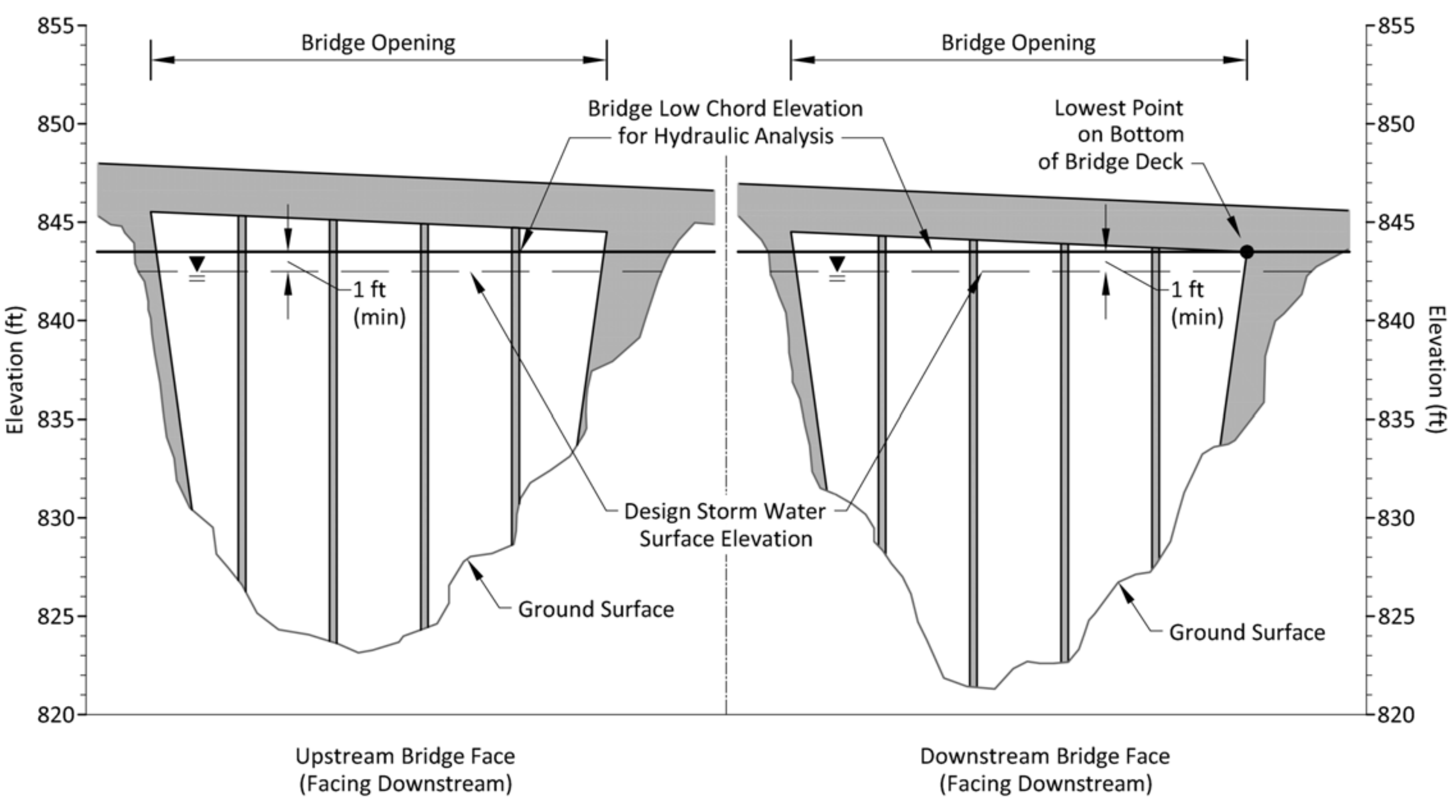
In areas where ice and debris are anticipated to be problematic, the designer should use engineering judgment to establish the amount of additional freeboard needed. For reference, incorporating an additional 1.0 foot of freeboard is typical to allow for moderate to heavy ice and debris passage.

6.5.1.3 Tailwater Relationship—Channel

Evaluate the tailwater conditions associated with the downstream channel through one or more of the following:

- Evaluate the hydraulic conditions of the downstream channel to determine tailwater depths for a range of flood flows.
- Perform a backwater analysis using HEC-RAS or another step-backwater model in order to determine the tailwater elevation if the tailwater is affected by downstream controls such as natural stream constrictions, irregular downstream cross sections, obstructions, impoundments, or backwater from another stream or body of water.
- Use the headwater elevation of any nearby, downstream bridge or culvert if it is greater than the channel depth.

Figure 6-4. Low Chord and Freeboard Determination



6.5.1.1 Tailwater Relationship—Confluence

Tailwater relationships at confluences can be very complex and require evaluating the probability of peak discharges coinciding for the study river and the receiving river. See Table 6-6 within Section 6.6.4.2 for more information.

6.5.1.2 Scour

Evaluate bridge foundation scour for the incipient overtopping event, the scour design event, and the scour check event. Refer to Chapter 7 *"Channel Stability and Scour at Bridges"* for help selecting the AEPs for each event and performing the required calculations. Detailed instructions on the calculation of Bridge Scour can be found in HEC-18, *"Evaluating Scour at Bridges."*

6.5.1.3 Aquatic Organism Passage

Bridges, like culverts, must be designed to maintain ecological connectivity and sediment continuity between the upstream and downstream reaches. Obtain input from resource agencies regarding AOP considerations, which are incorporated into the ANR GP and the USACE Vermont General Permit. For more information on stream crossing requirements, contact the ANR Rivers Program.

6.5.2 Hydraulic Performance of Bridges

Flow through a stream-crossing structure is typically characterized as free-surface flow, but if the WSE rises above the low chord of the bridge, pressure flow can occur. Road embankment overtopping is also possible during storm events. Analyze the hydraulic performance of bridge structures using a computer program such as HEC-RAS unless the VTrans Hydraulics Engineer indicates otherwise. Although alternative methods of analysis for bridge hydraulics may be used in certain conditions, the preferred method is HEC-RAS.

Figure 6-5 illustrates the hydraulic flow types through a stream crossing structure. The flow types are described as follows:

- Type I flow consists of subcritical flow throughout the approach, bridge, and exit cross sections and is the most common condition encountered in practice.
- Type IIA and IIB flow both represent subcritical approach flows that have been choked by the contraction so that critical depth occurs in the bridge opening. In Type IIA, the critical WSE in the bridge opening is lower than the undisturbed normal WSE. In Type IIB, the critical WSE in the bridge opening is higher than the normal WSE, so a weak hydraulic jump could occur immediately downstream of the bridge contraction.
- Type III flow is supercritical approach flow that remains supercritical through the bridge contraction. This flow condition is not subject to backwater unless it chokes and forces the occurrence of a hydraulic jump upstream of the contraction.

When modeling hydraulics to size a bridge opening, the designer should typically assume that the flow regime through the bridge is subcritical. However, the designer should always check these results by testing a mixed flow regime through the system. Refer to Section 6.6 for additional information about how to set up a hydraulic model.

6.6 Methodologies

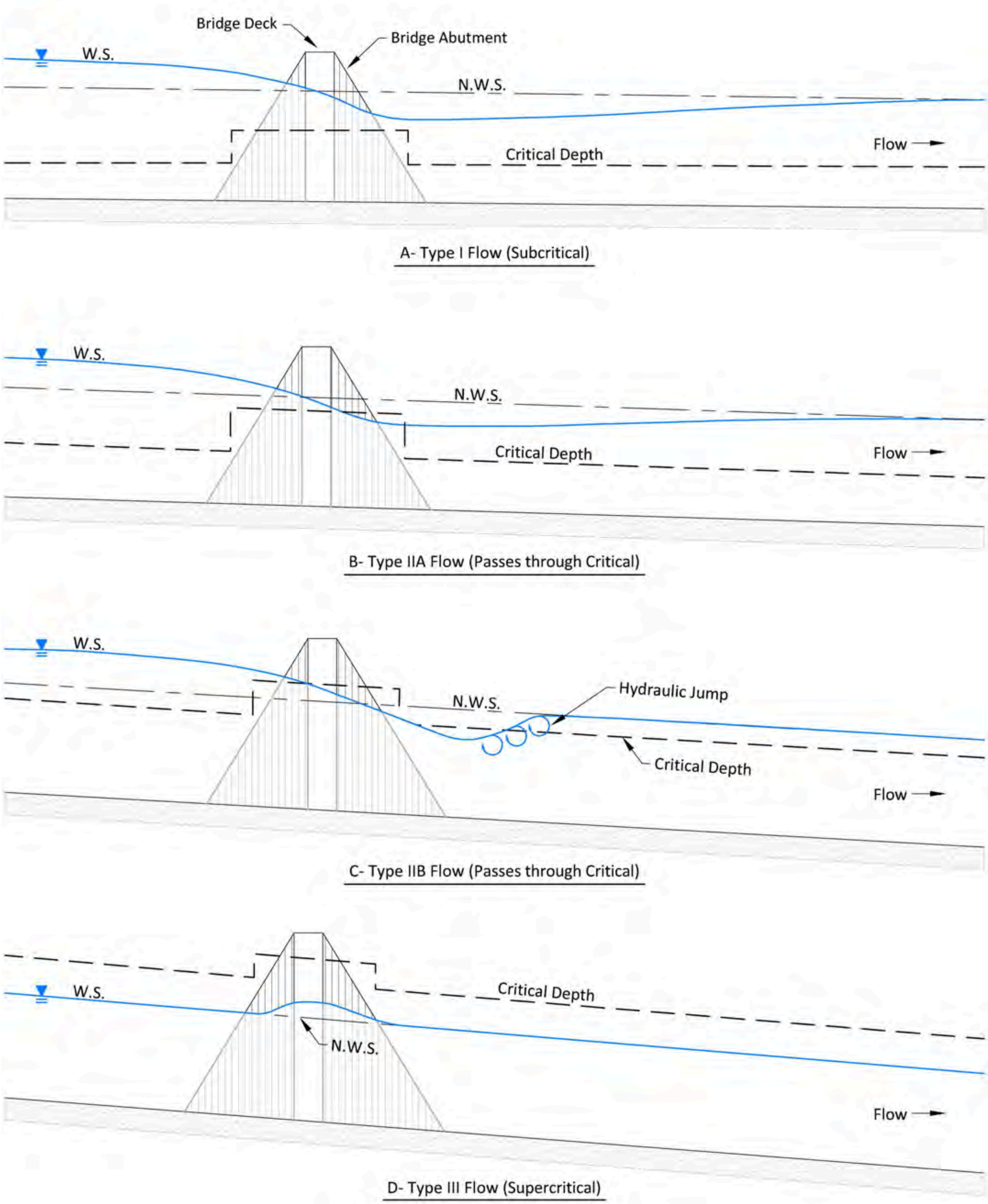
One-dimensional models provide sufficient detail for most analyses of culverts and bridges. FHWA's HY-8 software is the recommended tool for modeling culverts in one dimension, and USACE's HEC-RAS software is the recommended tool for modeling bridges in one dimension. For limited circumstances where two-dimensional modeling is warranted, USBR's SRH-2D is the recommended tool.

6.6.1 HY-8

The HY-8 software was developed by FHWA to perform one-dimensional steady-state culvert hydraulics calculations. The user can enter, edit, and save culvert and channel data for one or more crossings. The program computes the culvert hydraulics for circular, rectangular, elliptical, arch, and user-defined geometry. The output from the program can be printed or exported and incorporated directly into a hydraulic report.

The procedure for using the HY-8 program is similar to that for using other culvert design methods. The designer must calculate hydrologic data for the contributing watershed separately and input it into the model. The program then calculates and compares the headwater elevations for both inlet and outlet control. The program selects the higher of the two elevations as the control elevation.

Figure 6-5. Bridge Flow Types



The effects of tailwater are taken into consideration by the program when calculating these elevations. If the controlling headwater elevation would overtop the roadway embankment, the program performs an overtopping analysis whereby the flow is balanced between the culvert discharge and the surcharge over the roadway.

There are three main groups of data to enter into the program. These groups are:

1. Initial culvert data
   a. Culvert geometry
   b. Length
   c. Inlet configuration
   d. Roughness
   e. Embeddedness
   f. Design flows to evaluate
2. Downstream channel data
   a. Channel geometry
   b. Slope
   c. Side slopes
   d. Roughness
3. Roadway data
   a. Embankment height
   b. Width
   c. Materials

The program sequentially leads the user from one group to the next. The user can then iteratively evaluate different culvert sizes and configurations to meet the requirements of the site.

6.6.2 HEC-RAS

The USACE HEC-RAS software allows the user to perform one-dimensional steady state and unsteady flow river hydraulics calculations. The software can perform steady flow water surface profile computations, unsteady flow computations, and moveable boundary sediment transport computations. HEC-RAS is the preferred methodology for computing hydraulic performance of bridge structures. The HEC-RAS User's Manual serves as a useful source for more detailed information about using the program.

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 bridge backwater. The cross sections that are necessary for the energy analysis through the bridge opening are shown in Figure 6-6.

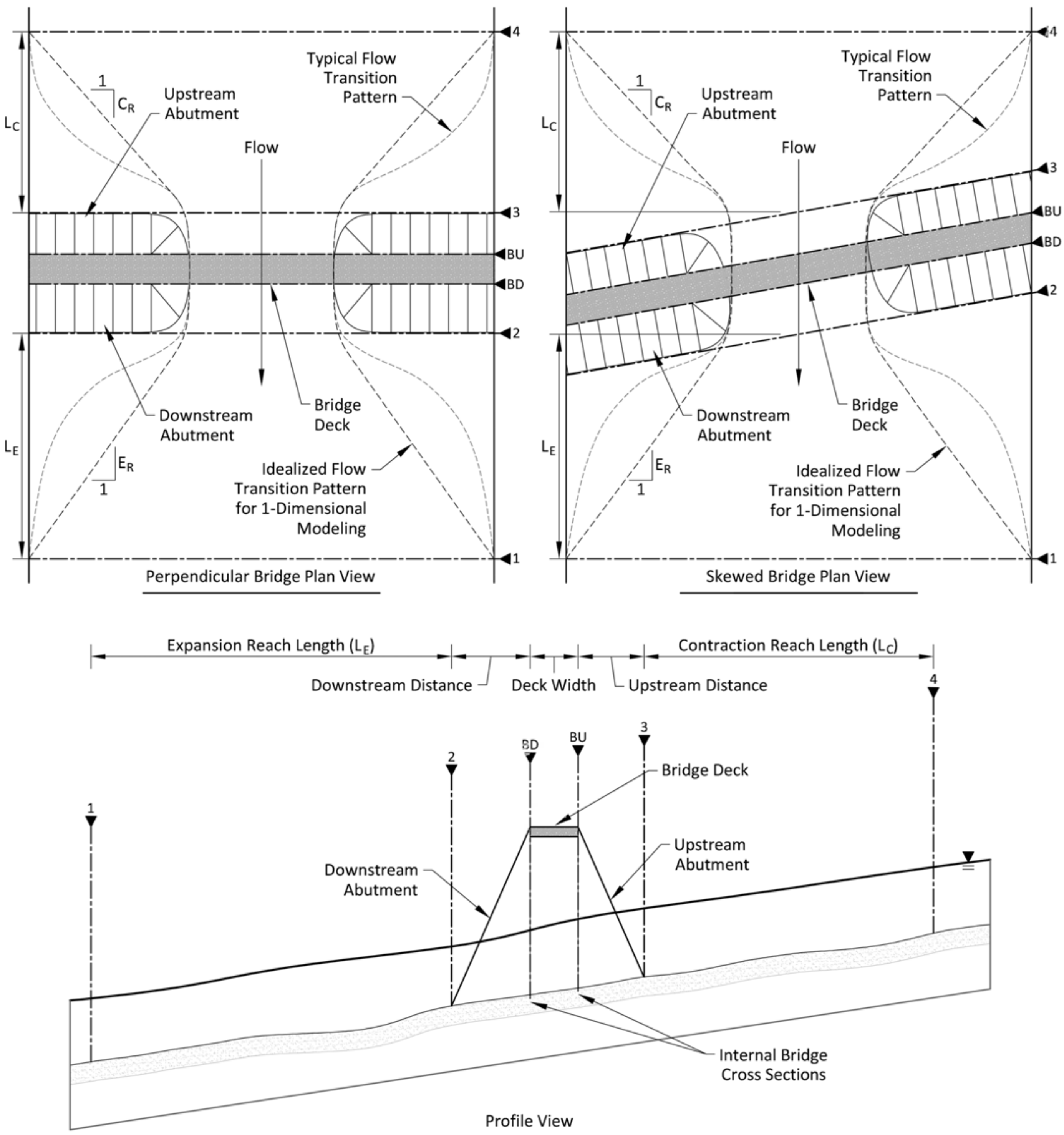
Flood studies and hydraulic modeling of existing structures may have been completed using the earlier HEC-2, Water Surface Profile software, which was released in 1968. HEC-2 bridge cards from previous studies can be imported into HEC-RAS in order to replicate those earlier results. There are minor differences between the two models due to the specific momentum method that is used within the HEC-2 bridge routine to analyze bridge piers. The bridge analysis routine in HEC-RAS results in a more refined estimate of actual hydraulic performance and is the preferred methodology.

HEC-RAS combines step-backwater analysis with bridge backwater calculations. This method allows for pressure flow through the bridge, embankment overtopping, and flow through multiple openings and culverts. The bridge hydraulics rely on the energy principle, as well as an improved technique for determining approach flow lengths and the introduction of an expansion loss coefficient. The flow-length improvement was found necessary when approach flows occur on very wide, heavily vegetated floodplains. The program also greatly facilitates the hydraulic analysis required to determine the lowest-cost alternative.

HEC-RAS is recommended for both preliminary and final analysis of bridge hydraulics. The input data propagation features of HEC-RAS make it easy to develop a model and acquire comprehensive output even if only a single surveyed cross section is available. This simplification must be used with caution if differing boundary conditions apply upstream or downstream from the project site.

Part of what makes HEC-RAS a powerful modeling tool is its ability to pull together different modeling files (i.e. geometry, flow) to create a single "plan" representing a set of conditions. This feature allows the designer to run different combinations of scenarios easily and without much additional work. However, having many model files associated with a HEC-RAS project can make it difficult to keep track of what conditions each file represents. Establish a file naming convention for geometry, flow, and plan files that is descriptive and consistent so that both the designer and others can easily distinguish what is represented.

Figure 6-6. Cross-Section Locations in the Vicinity of Bridges



6.6.3 HEC-RAS Geometric Data

6.6.3.1 Skewed Bridges

For bridges with a skew angle greater than 10 degrees, draw cross sections parallel to the bridge deck and assign the skew angle within the HEC-RAS Bridge/Culvert Data Editor in order to accurately account for the internal dynamics of flow through the crossing. Refer back to Figure 6-6 to see how to place cross sections for skewed bridges. For bridges with a skew angle less than 10 degrees, the flow efficiency is not adversely affected and no compensation is required.

6.6.3.2 HEC-GeoRAS

HEC-GeoRAS provides the capability of geo-referencing a hydraulic model through interface with DTMs and GIS software. Georeferencing HEC-RAS models provides many advantages in model development, use, review, and re-use, including:

- Realistic representation of model inputs in user interface.
- Increased efficiency and accuracy in model geometry development.
- Reduced ambiguity regarding locations of model elements (i.e. river centerline, cross section cut lines tied to a horizontal datum).
- Facilitated mapping of model results.

The GIS toolset allows the user to create model inputs in a map-based, graphical interface by overlaying 2-dimensional flow paths, cross sections, and banks over 3-dimensional topographic data. See Figure 6-7 for an example of a HEC-GeoRAS model. Develop HEC-RAS models for VTrans in a geo-referenced format.

6.6.4 HEC-RAS Flow Data and Boundary Conditions

6.6.4.1 Downstream Boundary Condition

Identification of the downstream boundary condition is critical to accurate modeling of stream crossing structures.

6.6.4.2 Tailwater Conditions

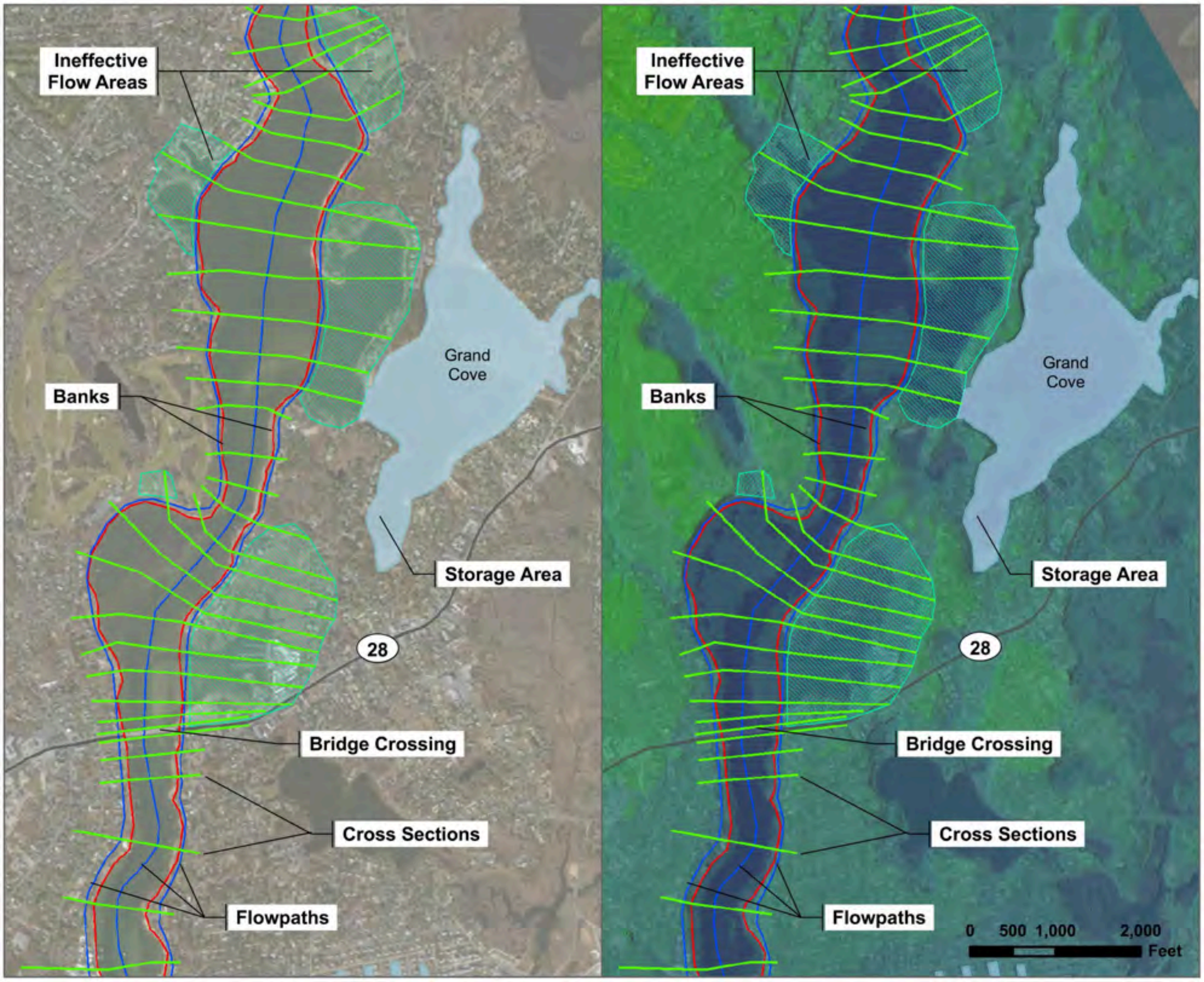
Tailwater conditions can significantly impact hydraulic model results, particularly for flatter stream reaches. Their impact on model results can influence design decisions for crossing structures regarding opening size, low chord elevations and scour protection. Always consider the impact of tailwater boundary conditions on the results of the analysis. For example, a conservatively high tailwater boundary condition may drive up low chord elevations to provide the required freeboard. However, a conservatively high tailwater boundary condition could also underestimate the velocities through the structure and thereby underestimate potential scour.

When evaluating tailwater conditions for a crossing structure, look beyond the local condition and understand downstream conditions and their impact on the structure. For stream reaches, a normal depth boundary condition may be appropriate, but in flatter river reaches, backwater conditions associated with downstream structures can extend for many miles upstream. FEMA FISs and other regional hydraulic studies can be an invaluable resource to help evaluate hydraulic boundary conditions, but always consider the age and level of detail of the information before using it. Refer back to Section 6.1.5.1 for more information on FEMA data sources.

Evaluating backwater conditions associated with a downstream river confluence can be particularly challenging. In situations where a smaller river discharges to a larger river with a broad floodplain, the larger river can backwater flood the smaller river. Usually, the smaller river reaches peak flows well before the larger river. Therefore, using the backwater stage from the larger stream as the design tailwater may misrepresent actual conditions when the smaller river is experiencing peak flows. Incorrectly applying backwater conditions can result in unintended consequences for the structure design.

Situations where the study river and the receiving river are more similar in size can produce even more complex backwater issues. For example, the study river may peak when the receiving river is rising but not yet at full flood stage. In this case, it would not make sense to use a free discharge condition to evaluate freeboard at the structure. However, for evaluation of scour, the free discharge condition could still be an appropriate and conservative method.

Figure 6-7. Georeferenced HEC-RAS Model



(a) Shown with aerial background (b) Shown with digital terrain background

To evaluate tailwater conditions for structures located within the influence of a downstream river confluence:

- If flood events on the tributary and main river occur concurrently as statistically dependent events, use the high water elevation for the same frequency as the design storm (statistically dependent).
- If flood events on the tributary occur as statistically independent events, evaluate the joint probability of flood magnitudes based on Table 6-6 and use the combination resulting in the greater tailwater depth. For example, in order to evaluate a 2% AEP storm event for a main stream and a tributary stream with a watershed area ratio (A_R) of 100:1:

1. Evaluate a 2% AEP storm event to the tributary stream using a tailwater equivalent to the highest WSE predicted in the main stream during the 10% AEP storm event. This is the design combination.
2. Use the reverse combination and evaluate a 10% AEP storm event to the tributary stream using a tailwater equivalent to the highest WSE predicted in the main stream during the 2% AEP storm event. This is the check combination.

Regardless of which case produces the higher headwater elevation, use a no-backwater tailwater condition to evaluate scour.

Table 6-6. Joint Probability Analysis

Area Ratio $A_R = A_M/A_T$	Frequencies for Coincidental Occurrence							
	10% AEP Design		4% AEP Design		2% AEP Design		1% AEP Design	
	Main Stream	Tributary	Main Stream	Tributary	Main Stream	Tributary	Main Stream	Tributary
$A_R \geq 10{,}000$	50%	10%	50%	4%	50%	2%	50%	1%
	10%	50%	4%	50%	2%	50%	1%	50%
$1{,}000 \leq A_R < 10{,}000$	50%	10%	50%	4%	20%	2%	10%	1%
	10%	50%	4%	50%	2%	20%	1%	10%
$100 \leq A_R < 1{,}000$	20%	10%	20%	4%	10%	2%	4%	1%
	10%	20%	4%	20%	2%	10%	1%	4%
$10 \leq A_R < 100$	10%	10%	10%	4%	4%	2%	2%	1%
	10%	10%	4%	10%	2%	4%	1%	2%
$1 \leq A_R < 10$	10%	10%	4%	4%	2%	2%	1%	1%
	10%	10%	4%	4%	2%	2%	1%	1%

Note: *Shaded values denote design combination for coincidental frequency occurrence. Non-shaded values denote check combination for coincidental frequency occurrence.*

6.6.5 HEC-RAS Parameters

When evaluating VTrans structures, use the following HEC-RAS parameters for steady-state modeling:

- Flow regime: Use "Subcritical" for evaluating vertical clearance. Use "Mixed Flow" for evaluating peak velocities, scour, and stone sizing.

- Expansion/contraction coefficients: Use 0.3 (expansion) and 0.1 (contraction) for all model cross sections with the exception of the two cross sections immediately upstream and the one cross section immediately downstream of the bridge structure. For these cross sections, use 0.4 (expansion) and 0.2 (contraction) for most structures unless the contraction is significant. In the case of a significant contraction, use 0.5 (expansion) and 0.3 (contraction).

6.6.6 Extracting HEC-RAS Results

When evaluating VTrans structures, use the following HEC-RAS results for design:

- Determining Structure Freeboard: Use the predicted WSE two model cross sections upstream of the structure to determine structure freeboard.

- Determining Peak Velocity: Use the highest predicted average channel velocity from the six bridge cross sections, or "Six XS Bridge" (includes the two cross sections upstream of the crossing, the two cross sections downstream of the crossing, and the two internal cross sections) for stone sizing or scour protection calculations.

- Model Alterations to Accurately Calculate Scour: Be sure to adjust the bank stations at internal and external bridge cross sections to represent channel flows through the opening during scour events. If the subject reach is typically affected by a confluence downstream, be sure to assume a low tailwater condition to arrive at a conservative estimate of scour. Refer to Chapter 7 *"Channel Stability and Scour at Bridges"* for more information.

6.6.7 Two-Dimensional Models

One-dimensional methods, such as the standard step method found in HEC-RAS, are typically used to predict the water surface profile and velocities in a river reach. While one-dimensional methods are adequate for most applications, these methods cannot provide a detailed determination of the cross-stream WSEs, flow velocities, or flow distribution.

Two-dimensional models are more complex and require more time to set up and calibrate. They require essentially the same field data as a one-dimensional model and, depending on complexity, may require more computer time. Obtain approval in writing from the VTrans Hydraulics Engineer before performing two-dimensional modeling.

Table 4.1 of HDS-7, *"Hydraulic Design of Safe Bridges"* provides a useful visual for helping designers to determine when to select a two-dimensional model over a one-dimensional model. Bridge hydraulic conditions where two-dimensional models are well suited and may be preferred include:

- Wide floodplains
- Highly variable floodplain roughness
- Highly sinuous channels
- Multiple embankment openings
- Multiple channels
- Skewed roadway alignments
- Large tidal waterways and wind-influenced conditions
- Upstream controls
- Countermeasure design
- Detailed analysis needed of bends, confluences, and angle of attack
- Detailed flow distribution needed at bridges
- Significant roadway overtopping

The USBR has developed a two-dimensional hydraulic, sediment, temperature, and vegetation model for river systems called SRH-2D.

As stated on the USBR website for the model, "SRH-2D is a 2D model, and it is particularly useful for problems where 2D effects are important. Examples include flows with in-stream structures, through bends, with perched rivers, with side channel and agricultural returns, and with braided channel systems. A 2D model may also be needed if one is interested in local flow velocities, eddy patterns, flow recirculation, lateral velocity variation, and flow over banks and levees."

Quick Tip

At the time of writing, HEC-RAS version 5.0 is currently in its beta phase of release. This version of HEC-RAS will reportedly support 2D hydrodynamic flow routing within the unsteady flow-routing analysis. Designers will be able to model 2D flow areas independently or in conjunction with 1D flow areas.

6.7 Culvert Design Procedure

6.7.1 General

The following design procedure provides a convenient and organized method for designing culverts for a peak flow rate while considering inlet and outlet control. The procedure does not address the effect of storage. Use this procedure in conjunction with Figure 6-1, which depicts a flow chart simplifying the steps outlined below.

The designer should be familiar with the equations in Chapter 3 of HDS-5, *"Hydraulic Design of Highway Culverts"* before using these procedures. Reference Figure 2.4 within the DEC's VT SRMPP for Stream Crossing Guiding Design Principles. Following the design method without an understanding of culvert hydraulics can result in an inadequate, unsafe, or costly structure.

6.7.2 Culvert Design Procedure

1. *Assemble site data and project file*
   a. Refer to Chapter 3 *"Data Collection, Resources, and Tools"* for guidelines, and investigate the availability of the following resources:
      i. U.S. Geological Survey (USGS), site, and location maps
      ii. Topography data
      iii. Embankment cross section
      iv. Channel dimensions
      v. Geology data
      vi. Roadway profile
      vii. High water marks
      viii. Photographs
      ix. Design data and review of hydraulic performance at nearby structures
      x. History of debris accumulation, ice, and scour
      xi. Rainfall and stream gage records
   b. Collect applicable studies by other agencies.
      i. Small dams—Natural Resources Conservation Service (NRCS), USACE
      ii. FEMA FIS reports
      iii. Floodplain—NRCS, USACE, FEMA, USGS, historical sources
      iv. State and local river corridor and floodplain studies
      v. Hydraulic performance of existing upstream and downstream structures
      vi. Storm drain—local or private
   c. Consider the following influences on hydraulic performance:
      i. Other streams, reservoirs, or water intakes
      ii. Structures upstream or downstream
      iii. Natural features of stream and floodplain
      iv. Location within the valley setting
      v. Channel modifications upstream or downstream
      vi. Floodplain encroachments
      vii. Sediment types and bed forms
   d. Review environmental constraints.
      i. Existing bed or bank instability
      ii. Floodplain land use and flow distribution
      iii. Environmentally sensitive areas (fisheries, wetlands, etc.)
      iv. Historic and archeological areas
      v. Site-specific design criteria
      vi. Application of agency criteria
   e. Review design criteria—refer to Section 6.4.
   f. Conduct a field visit, paying attention to the presence of and details about the following:
      i. Sediment
      ii. Debris
      iii. Available storage
      iv. Land cover

2. *Analyze hydrology*
   a. Refer Chapter 4 *"Hydrology."*
   b. Develop the following minimum data:
      i. Drainage area map
      ii. Discharge-frequency plot
   c. Determine design frequency and Q_d.

3. *Analyze existing conditions*
   a. Consider natural conditions (without any structure), including the following factors:
      i. Stream type—equilibrium conditions and conductivity as driven by the sediment regime
      ii. Stream cross section dimensions
      iii. Geomorphic condition—ongoing channel adjustments, and channel evolution (e.g. aggradation, degradation)
   b. Analyze existing structure (if any) at design discharge.
   c. Consider replacement in kind.

.

Culvert Design Procedure (Cont.)

4. *Evaluate aquatic organism passage*
   a. Evaluate limiting factors (flow depth and velocity) with respect to species of concern
   b. Refer to the ANR GP for additional requirements.
   c. Adjust culvert design accordingly.

5. *Select design alternatives*
   a. Refer to Section 6.4.3 of this chapter.
   b. Choose shape, size, entrance type, and appropriate roughness.

6. *Analyze hydraulics*
   a. Use HY-8 or a similar program to calculate:
      i. Headwater depth
      ii. Outlet velocity and depth
   b. Create performance curves for a range of peak flow rates.

7. *Review results*
   a. Compare design alternatives with consideration to constraints and assumptions. Repeat steps 5 and 6 if any of the following conditions occur:
      i. The proposed culvert barrel does not have adequate cover.
      ii. The proposed culvert length is not close to the approximate length.
      iii. The proposed headwalls and wingwalls do not fit the site.
      iv. The proposed culvert exceeds the allowable headwater.
      v. The proposed culvert exceeds the allowable overtopping event frequency.
      vi. Velocity through the proposed culvert is too high.

8. *Related Designs*
   a. Consider the following options:
      i. Incorporate full headwalls, full headwalls with bevels, or improved inlets if the culvert is in inlet control and has limited available headwater.
      ii. Incorporate energy dissipators if the mean velocity at the culvert outlet (V_0) is larger than the mean velocity in the downstream channel (V_d).
      iii. Provide fish passage as needed. Refer to Section 6.4.2.7.

9. *Select final design*
   a. Base the final selection on the following:
      i. Compliance with established hydraulic criteria.
      ii. Consideration of environmental, social, and economic criteria.
      iii. The need for related designs.

10. *Document results*
    a. Complete project records, permit applications, etc.
    b. Complete the Final Hydraulics Report Form (see Appendix C *"Hydraulics Form"*).
    c. Develop final hydraulic (and potentially scour) report(s).
    d. Refer to Chapter 9 *"Documentation"* for additional guidelines.

6.8 Bridge Design Procedure

6.8.1 General

The following design procedure provides a convenient and organized method for designing bridges for a peak flow rate. Although the scope of the project and individual site characteristics make each design unique, this outline addresses most fundamental issues. Use this procedure in conjunction with Figure 6-1, which depicts a flow chart simplifying the steps outlined below.

The designer should be familiar with the equations in Chapter 3 of HDS-7, *"Hydraulic Design of Safe Bridges"* before using these procedures. Reference Figure 2.4 within the DEC's VT SRMPP for Stream Crossing Guiding Design Principles. Following the design method without an understanding of bridge hydraulics can result in an inadequate, unsafe, or costly structure.

6.8.2 Bridge Design Procedure

1. *Assemble site data and project file*
   a. Refer to Chapter 3, *"Data Collection,"* for guidelines, and investigate the availability of the following resources:
      i. USGS, site, and location maps
      ii. Topography data
      iii. Embankment cross section
      iv. Geology data
      v. Roadway profile
      vi. High water marks
      vii. Photographs
      viii. Design data and review of hydraulic performance at nearby structures
      ix. History of debris accumulation, ice, and scour
      x. Rainfall and stream gage records
   b. Collect applicable studies by other agencies.
      i. Small dams—Natural Resources Conservation Service (NRCS), USACE
      ii. FEMA FIS reports
      iii. Floodplain—NRCS, USACE, FEMA, USGS, historical sources
      iv. State and local river corridor and floodplain studies
      v. Hydraulic performance of existing upstream and downstream structures
      vi. Storm drain—local or private
   c. Consider the following influences on hydraulic performance:
      i. Other streams, reservoirs, or water intakes
      ii. Structures upstream or downstream
      iii. Natural features of stream and floodplain
      iv. Location within the valley setting
      v. Channel modifications upstream or downstream
      vi. Floodplain encroachments
      vii. Sediment types and bed forms
   d. Review environmental constraints.
      i. Existing bed or bank instability
      ii. Floodplain land use and flow distribution
      iii. Environmentally sensitive areas (fisheries, wetlands, etc.)
      iv. Historic and archeological areas
      v. Site-specific design criteria
      vi. Application of agency criteria
   e. Review design criteria—refer to Section 6.5.
   f. Conduct a field visit, paying attention to the presence of and details about the following:
      i. Sediment
      ii. Debris
      iii. Available storage
      iv. Land cover

2. *Analyze hydrology*
   a. Refer Chapter 4 *"Hydrology."*
   b. Develop the following minimum data:
      i. Drainage area map
      ii. Discharge-frequency plot
   c. Determine design frequency and Q_d.

3. *Analyze existing conditions*
   a. Consider natural conditions (without any structure), including the following factors:
      i. Stream type—equilibrium conditions and conductivity as driven by the sediment regime.
      ii. Stream cross-section dimensions
      iii. Geomorphic condition—ongoing channel adjustments and channel evolution
   b. Analyze existing structure (if any) at design discharge.

Bridge Design Procedure (Cont.)

4. *Evaluate aquatic organism passage*
    a. Evaluate limiting factors (flow depth and velocity) with respect to species of concern.
    b. Refer to the ANR GP for additional requirements.
    c. Adjust bridge design accordingly.

5. *Select design alternatives*
    a. Refer to Section 6.5.1 of this chapter.
    b. Choose span, low chord, abutment configuration, and pier configuration (if necessary)

6. *Analyze hydraulics and scour*
    a. Develop computer model. Perform calibration and verification.
    b. Evaluate hydraulic performance for existing conditions.
    c. Evaluate hydraulic performance for proposed designs.
    d. Develop water surface profiles.
    e. Perform scour calculations for proposed designs.

7. *Review results*
    a. Compare design alternatives with consideration to constraints and assumptions. Repeat steps 5 and 6 if any of the following conditions occur:
        i. The bridge does not have adequate freeboard during the design event
        ii. Peak velocities and/or flow rates are unacceptable.
        iii. Calculated scour depths are too high.

8. *Select final design*
    a. Base the final selection on the following:
        i. Compliance with established hydraulic criteria.
        ii. Consideration of environmental, social, and economic criteria.
    b. Design details such as stone fill, scour abatement, and river training.

9. *Document results*
    a. Complete project records, permit applications, etc.
    b. Complete the Final Hydraulics Report Form (see Appendix C *"Hydraulics Form"*).
    c. Develop final hydraulic and scour report(s).
    d. Refer to Chapter 9 *"Documentation"* for additional guidelines.

Chapter 7 Channel Stability and Scour at Bridges

7.1 Introduction

7.1.1 Overview

Bridge design for stream crossings must consider and accommodate the impact of dynamic hydraulic and geomorphic processes on the bridge foundation. The consequences of failure—gradual or sudden—for any bridge crossing can be severe. Highway crossings of streams and rivers are particularly challenging to monitor because critical structural foundation elements are typically invisible—submerged in water or buried under sediment. If inspections and periodic maintenance of foundation components are neglected, gradual failure can go unnoticed. Sudden failure is particularly dangerous for structures that receive constant use.

Typically, engineers design bridge foundation elements to resist scour of the supporting bed material. Streams (and supporting bed material) are not static; they are dynamic and prone to evolution due to natural and manmade causes. Therefore, scour calculations must consider the potential range of conditions likely to develop at a given crossing, not just the conditions that are currently present.

This chapter addresses the subjects of scour and channel stability at bridges and provides references to relevant resources. It also includes guidance specific to design conventions for VTrans and the state of Vermont. The chapter is divided into four section that focus on the following subtopics:

1. *Assessing stream stability*—an understanding of stream stability is integral to selecting the appropriate initial conditions for the scour analysis.
2. *Performing scour calculations*—scour calculations are used to estimate potential scour depth based on selected initial conditions.
3. *Selecting countermeasures*—countermeasures reduce the impact of scour on the proposed structure.
4. *Designing bridge foundations*—VTrans recommends following a team-based procedure for designing bridge foundations.

Bridge foundation design is a multi-disciplinary task. The design team should include structural engineers, geotechnical engineers, geomorphologists, hydraulic engineers, and transportation engineers.

The hydraulic engineer should work with the geomorphologist to review and understand the following:

- Hydrologic conditions in the contributing watershed (see Chapter 4 *"Hydrology."*)
- Natural and anthropogenic influences on the contributing watershed
- Geomorphic characteristics of the stream channel
- Geologic composition of the streambed
- Hydraulic conveyance of the stream channel upstream and downstream of the crossing
- Historical flooding

A good understanding of the stream system is required to avoid provoking changes to the stream channel that could cause problems in the future, such as:

- Changing the course of the channel and undermining the bridge foundations;
- Changing the form of the channel, resulting in flow traveling around the abutments and over the approach embankments;
- Causing bed erosion (scour) that undermines the foundation components; or
- Causing bed aggradation that traps bed material and debris and blocks the opening.

A sufficient crossing structure design will successfully accommodate:

- Typical normal-flow conditions;
- Infrequent high-flow conditions;
- Potential changes in the horizontal alignment of the stream;
- Potential changes in the vertical profile of the stream; and
- Potential changes in the form (width, depth) of the stream channel.

The design should also balance the economic burdens of maintenance, repairs, and/or future retrofits.

7.1.2 Resources

The contents of this chapter are primarily based on the publications and guidance provided by three agencies: the Federal Highway Administration (FHWA), the Natural Resources Conservation Service (NRCS), and the Vermont Agency of Natural Resources (ANR). All three agencies have published methods for assessing stream stability. The FHWA and NRCS have published methods for estimating scour.

7.1.2.1 Federal Highway Administration

The FHWA provides a suite of excellent guidance documents for designing bridges for stability and resistance to impacts caused by scour. Refer to the following FHWA Hydraulic Engineering Circular (HEC) publications:

- HEC-18, *"Evaluating Scour at Bridges"*
- HEC-20, *"Stream Stability at Highway Structures"*
- HEC-23, *"Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance"* vol. 1 and vol. 2

Be sure to refer to the list of FHWA Current Hydraulic Engineering Publications for up-to-date methods and procedures.

7.1.2.2 Natural Resources Conservation Service

The NRCS's National Engineering Handbook (NEH) *"Part 654, Stream Restoration Design,"* includes technical descriptions of processes that affect rivers and streams and descriptions of techniques to stabilize systems undergoing change.

7.1.2.3 Vermont Agency of Natural Resources

The ANR's *"Vermont Stream Geomorphic Assessment Handbooks,"* offer separate but interrelated phases for examining and evaluating geomorphic (stream stability) and habitat conditions of a streams. The handbooks include:

- Phase 1, *"Watershed Assessment: Using Maps, Existing Data, and Windshield Surveys"*
- Phase 2, *"Rapid Stream Assessment: Field Protocols"*
- Phase 3, *"Survey Assessment: Field and Data Analysis Protocols"*

Each phase requires progressively more elaborate data collection and analysis. Analyses should progress through the phases until the appropriate level of detail is achieved for a specific crossing design.

7.1.3 Design Tools

7.1.3.1 Hydraulic Toolbox

The current version of the FHWA Hydraulic Toolbox includes twelve calculators for evaluating systems typical to hydraulic design for highway applications. A few of the calculators relevant to this chapter include channel capacity, channel lining, weir flows, sediment gradation, and bridge scour. VTrans recommends that users of the FHWA Hydraulic Toolbox verify their results with manual calculations and engineering judgment.

7.1.3.2 HEC-RAS

The U.S. Army Corps of Engineers (USACE) Hydraulic Engineering Center River Analysis System (HEC-RAS) is recommended as a computational tool for performing step-backwater analysis. The software was developed specifically to perform one-dimensional steady-state and unsteady-state flow hydraulics calculations for open channels.

Caution!

Do not use HEC-RAS to calculate scour. The scour equations in HEC-RAS do not reflect the latest methods published in the 5th edition of HEC-18, and the USACE has no intention of updating or maintaining the scour equations. Additionally, HEC-RAS may not always use the correct depths and velocities for computing scour.

VTrans recommends using HEC-RAS to model open channel hydraulics and calculate flow depths and velocities at the crossing structure under analysis. These calculated values should be pulled from the model and used in scour calculations performed outside of the framework of HEC-RAS in accordance with HEC-18. The hydraulic model may require slight adjustments for applications where bridge and scour calculations are required. Consider the following:

- Be sure to adjust the bank stations at internal and external bridge cross sections to represent channel flows through the opening during scour events.
- Be sure to adjust boundary conditions to test for supercritical flows. Velocities calculated for subcritical flows are not conservative in a scour analysis. If supercritical flows are likely, adjust the design for stability under supercritical conditions.
- If the subject reach is typically affected by a confluence downstream, be sure to assume a low tailwater condition to arrive at a conservative estimate of scour.

7.2 *Stream Stability*

Streams and rivers are dynamic features that respond to changes in hydrologic, hydraulic, and land cover conditions in the contributing watershed. A complete scour analysis must include an assessment of the stability of the stream to determine whether conditions at the crossing are likely to change in the future.

This chapter provides a brief introduction to the concept of stream stability, resources that may be used to evaluate stream stability, and field data that a geomorphologist would collect to assess stream stability. For an in-depth description of the history and research upon which these concepts are based, refer to the source documentation. References specific to the geomorphology of Vermont are available from the ANR. General technical references are available from the FHWA and the NRCS.

The ANR's *"Vermont Stream Geomorphic Assessment Handbooks,"* NEH "*Part 654, Stream Restoration Design*," and HEC-20, *"Stream Stability at Highway Structures,"* serve as primary references for Section 7.2.

This summary of stream stability and key concepts includes:

- Levels of Assessment
- Data Collection
- Geomorphic Classification
- Influences to Geomorphological Change

7.2.1 Stream Stability Resources

The ANR categorizes levels of assessment into three phases. The FHWA categorizes levels of assessment into three levels. In general, FHWA Level 1 corresponds with ANR Phase 1 studies. FHWA Level 2 corresponds with ANR Phase 2 and 3 studies. There is no ANR equivalent to a FHWA Level 3 study.

The ANR Phase 1 Handbook, *"Watershed Assessment: Using Maps, Existing Data, and Windshield Surveys"* summarizes methods for performing a watershed assessment using existing data consistent with a Phase 1/Level 1 analysis, described in Section 7.2.2.1.

HEC-20, *"Stream Stability at Highway Structures,"* Chapter 5, Section 4 and Appendix C summarize detailed methods for rapid assessment of stream stability for road crossing applications consistent with a Phase 1/Level 1 analysis, described in Section 7.2.2.1.

The ANR Phase 2 Handbook, *"Rapid Stream Assessment: Field Protocols"* summarizes methods for performing field data collection and rapid stream assessment consistent with a Phase 2 & 3/Level 2 analysis, described in Section 7.2.2.2.

The ANR Phase 3 Handbook. *"Survey Assessment: Field and Data Analysis Protocols"* summarizes methods for performing field data collection and field surveys consistent with a Phase 2 & 3/Level 2 analysis, described in Section 7.2.2.2.

HEC-20, *"Stream Stability at Highway Structures,"* Chapter 6 summarizes quantitative techniques for assessing stream stability consistent with an FHWA Level 3 analysis, described in Section 7.2.2.3.

NEH "*Part 654, Stream Restoration Design*," Chapter 3: Site Assessment and Investigations, includes detailed procedures for conducting geomorphological assessments.

Appendix E of the U.S. Department of Agriculture (USDA) Forest Service document, *"Stream Simulation: An Ecological Approach to Providing Passage for Aquatic Organisms at Road-Stream Crossings,"* contains detailed descriptions and procedures for assessing streambed stability for highway stream crossing applications.

7.2.2 Levels of Assessment

The preliminary assessment of a new bridge site or a bridge site undergoing a significant rehabilitation should include a site investigation performed by a geomorphologist in order to:

- Identify existing erosion or aggradation problems;
- Properly classify the study reach;
- Identify the likelihood of potential future evolution; and
- Observe the bed material for sediment transport capacity.

The level of detail of subsequent evaluations and supporting studies should be commensurate with the risk identified in this preliminary assessment.

7.2.2.1 Phase 1/Level 1—Qualitative Assessment

Use existing resources to perform a qualitative geomorphic assessment to identify potential stability problems. Examples are summarized in Section 7.2.3.1.

7.2.2.2 Phase 2 & 3/Level 2—Quantitative Assessment

Collect scientifically sound field data and use the quantitative data to supplement the Phase 1/Level 1 assessment. The ANR Phase 2 protocol uses field observations to help verify the

Phase I assessment and provide more information about the subject stream. A Phase 3 assessment may be necessary if conditions at the subject stream are not well understood and are undergoing significant adjustments. The FHWA Level 2 protocol includes using field data to build hydraulic models to calculate water surface profiles and scour depth potential. A Phase 2 & 3/Level 2 analysis will be adequate for evaluating conditions at most existing or proposed highway/stream crossings if conditions are well understood and the designed countermeasures will be sufficient to provide long term stability.

7.2.2.3 FHWA Level 3—Complex Quantitative Assessment

A Level 3 assessment is necessary for high-risk locations, extraordinarily complex problems, and forensic analyses where losses and liability costs are high. In such cases, use complex quantitative analyses to supplement the Level 2 assessment. Complex quantitative analyses may include detailed mathematical modeling and/or physical hydraulic modeling. Level 3 assessments typically require the services of professionals experienced with mathematical modeling techniques for sediment routing and/or physical modeling.

In order to assess stability, use field data including the following:

- Bed load and suspended load transport rates
- Properties of bed and bank materials, such as:
    - Size
    - Shape
    - Gradation
    - Fall velocity
    - Cohesion
    - Density
    - Angle of repose

7.2.3 Data Collection

The ANR Stream Geomorphic Assessment Data Management System (SGA-DMS) provides supporting background information on many existing highway stream crossing structures.

ANR also provides the Natural Resources Atlas, an online map viewer, of streams and rivers that have already been assessed. Use this map viewer to determine if previous studies have addressed the geomorphology of the subject reach.

Appendix A of the ANR *"Vermont Stream Geomorphic Assessment Handbooks,"* includes data collection checklists to support the use of the ANR geographic information system (GIS) Stream Geomorphic Assessment Tool (SGAT). If the geomorphic assessment is being conducted in collaboration with the ANR, this tool, which is described in detail in a separate document issued by ANR, must be used.

Appendix C of HEC-20, *"Stream Stability at Highway Structures,"* contains detailed field investigation checklists for stream stability and scour applications at bridge crossings. Data typically collected for stability and scour assessments is described below.

Data collection should focus on the following categories:

- Existing resources
- Reference reach
- Geomorphology
- Bed material
- Geometry
- Historical scour
- Hydrology

7.2.3.1 Existing Resources

Obtain readily available data including:

- Historic information
- Current site conditions
- Remote sensing data
    - Maps
    - Aerial photos
- Orthophotographs
- Topographic maps
- NRCS soil surveys
- Dam locations
- Hydraulic geometry curves
- Survey notes
- Bridge design files
- Maintenance records
- Interviews with longtime residents
- 'Windshield survey' data, which consists of general observations made while driving around the watershed

7.2.3.2 Reference Reach

A reference reach is a segment of the stream or river that is meant to act as a template for stream simulation design through the crossing structure. A reference reach should be stable, ideally nearby and upstream of the structure, outside the influence of the existing structure, of a similar gradient to the design gradient through the structure, and at least as long as the road-stream crossing.

The designer may not feel that identifying a reference reach is necessary for all projects involving crossing structures, however it is a useful step that can help the designer to evaluate the characteristics of the stream.

For more information, refer to the Vermont ANR Fish and Wildlife Department's (FWD) *"Guidelines for the Design of Stream/Road Crossings for Passage of Aquatic Organisms in Vermont"* and the 2009 NRCS Companion Document 580-8: *"Detailed Instructions for Reference Reaches."*

7.2.3.3 Geomorphology

Classify the geomorphology of the site. Determine the qualitative characteristics of the floodplain, such as whether it crosses a delta or an alluvial fan and whether it is young and evolving or mature and at equilibrium. Geomorphology is discussed in more detail in later sections of this chapter.

7.2.3.4 Bed Material

Assess the bed material at several locations along the stream: upstream and downstream of the proposed crossing and at the proposed crossing. Determine the grain size distribution (e.g. the D_{16}, D_{50}, D_{84}, and D_{90}) of the bed material using pebble count analyses or borings.

The pebble count method is described in the NRCS's Companion Document 580-8: *"Detailed Instructions for Reference Reaches."* The pebble count method is appropriate in reaches where pebbles exist and channel armoring has not occurred. Channel armoring is the phenomenon where fine-grained material is washed away, leaving a bed of disproportionally large material on the surface of the stream bed.

Borings may be preferred in reaches composed of fine-grained sediment, bedrock (if fracturing is suspected), and areas where channel armoring has occurred. Refer to the following guidance regarding soil borings, which has been modified from the Maryland State Highway Administration (MDSHA) *Manual for Hydrologic and Hydraulic Design*:

- At a minimum, collect borings at each foundation element and within the channel at locations upstream, downstream, and at the crossing.
- Tie boring depths into survey information so that elevations can be related to the water surface, the channel bottom, and foundation design elements.
- Conduct a sieve analysis on the collected material to determine streambed gradation. The particle size report should include the D_{16}, D_{50}, D_{84}, and D_{90}.
- If there is a significant change in the composition of the soil within a boring, note the elevation where the change occurs. Take additional samples of the soil below the change, gathering the same information as that discussed above.
- Note the elevation of the soil/rock interface.

Use the grain-size distribution along the reference reach segments to determine whether the existing bridge is sited in an area experiencing live-bed or clear-water scour. Use the grain-size distribution at the proposed crossing to determine whether the channel is protected by natural or man-made armoring and to determine the magnitude and flow rates of historic scour events.

7.2.3.5 Geometry

Survey the existing stream to obtain the following geometric data:

- Stream channel and floodplain cross sections
- Longitudinal streambed profile
- The stream's current (and where possible, historic) geomorphic plan form(s)

Identify and consider the potential impact of the following:

- Other bridges or constrictions in the area
- Nearby tributaries
- Bedrock controls
- Artificial controls (e.g. dams, old check structures, river training works)

Identify and quantify the distance from and height of any headcuts caused by natural or anthropogenic influences. A headcut is the sudden change in bed elevation at the leading edge of a gully. Identify and document features that may indicate possible plan form changes such as the rate of formation and/or migration of meanders.

7.2.3.6 Historical Scour

Review records and observe evidence of scour on other bridges or similar structures along the stream.

7.2.3.7 Hydrology

Identify the character of the stream hydrology. Classify the stream as ephemeral, intermittent, or perennial. Determine whether the stream responds slowly or rapidly to precipitation in the contributing area.

7.2.4 Geomorphologic Classification

Figure 7-1, excerpted from HEC-20, *"Stream Stability at Highway Structures,"* presents a summary of the geomorphic characteristics that affect stream stability. The relevant geomorphic characteristics are described below. The designer should complete the Geomorphic Factors Form (Figure 7-1) during the preliminary hydraulic study.

7.2.4.1 Size

In geomorphological classification, stream size is determined by the bank to bank channel width at the normal river stage. The normal river stage typically corresponds to the level of permanent vegetation along the banks. The potential for scour and lateral erosion increases as stream size increases.

7.2.4.2 Flow Habit

Flow habit may be classified as ephemeral, intermittent, perennial but flashy, or perennial.

7.2.4.3 Bed Material

Bed material may be classified as silt-clay, sand, gravel, or cobble/boulder based on field sampling and laboratory gradation analysis. The size of the bed material plays a role in whether clear-water or live-bed scour occurs and how long it takes for a scour hole to form. Scour holes form most rapidly in less stable sand and gravel beds and more slowly in cohesive or cemented beds. If constant flow conditions persist, a less stable sand or gravel bed could experience its maximum scour depth after a few hours, whereas a cohesive sediment would take a few days, and limestone or granite could take years or even centuries. In some cases, these maximum scour depths may be the same even though the bed materials differ.

7.2.4.4 Valley Setting

Valley setting is a measure of the slope, or relief, of the terrain within the contributing watershed. Relief is measured from the bottom of the valley to the top of the greatest adjacent divide. Classifications include: no valley, alluvial fan, low relief, moderate relief, and high relief.

7.2.4.5 Floodplains

The geomorphologic floodplain is the ground surface presently under construction by the stream, which in most cases is flooded by the bankfull discharge at an approximate frequency of 1.5 years. Floodplains are classified by width relative to the channel width: little or none, narrow, or wide.

7.2.4.6 Natural Levees

Natural levees are constructed during floods when stream stage exceeds bankfull conditions and sediment is deposited on the floodplain. Classifications are qualitative: little or none, mainly on concave (inside) bank, or well developed on both banks.

7.2.4.7 Apparent Incision

Incision is a measure of the relative height of the channel banks to the width at normal stage. High banks indicate probable incision. The risk for lateral erosion and horizontal migration is typically low for streams with incised banks.

7.2.4.8 Channel Boundaries and Vegetation

Channel boundaries may be classified as: alluvial, semi-alluvial, or non-alluvial. Non-alluvial channels are located in bedrock or very large material that does not move except during very high flows. Alluvial channels are located in loose unconsolidated soil or sediment (i.e. alluvium). Semi-alluvial channels have both bedrock and alluvium in their boundaries. Alluvial channels may undergo significant changes during periods of high flows.

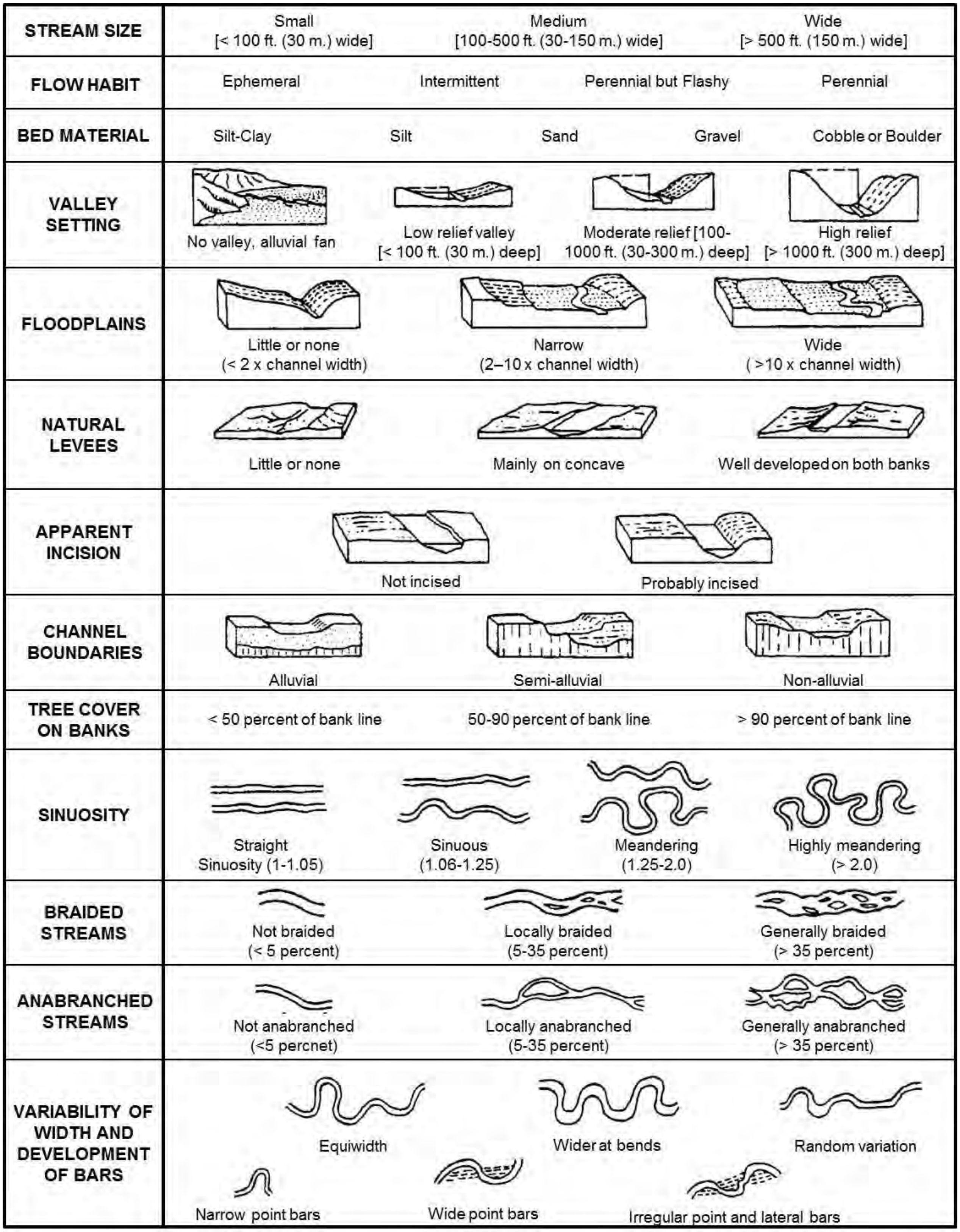
7.2.4.9 Bank Materials

Bank materials may be classified as non-cohesive, cohesive, or composite/stratified. Non-cohesive material is transported subject to shear stress from the velocity of flows adjacent to the bank, turbulence, seepage, piping, and/or waves. Cohesive material is more resistant to shear forces. A cohesive bank will typically fail by mass wasting. Composite/stratified materials are subject to surface erosion but may be protected by armoring.

7.2.4.10 Sinuosity

Sinuosity is a measure of the ratio of the length of a stream along its centerline to the length measured along the valley centerline. Classifications include: straight, sinuous, meandering, and highly meandering. Abrupt changes to channel sinuosity may provoke instability in a stream as the channel adjusts to a new equilibrium. Sinuosity does not necessarily correspond to the likelihood of lateral erosion.

Figure 7-1. Geomorphic Factors Form



(adapted from FHWA HEC-20)

7.2.4.11 Braided Streams

Braided streams consist of multiple and interlacing channels formed by large bed-material load deposited in high slope areas. Braided channels are unstable, dynamic, change alignment rapidly, carry large quantities of sediment, and are wide, shallow, and unpredictable. A channel may be classified as not braided, locally braided, or generally braided.

7.2.4.12 Anabranched Streams

Anabranched streams consist of channels divided by stable, often vegetated islands. Anabranches are relatively permanent, but, like braided streams, are unpredictable. Typically, bridge design engineers should avoid locating a stream crossing near anabranches.

7.2.4.13 Variability of Width and Development of Bars

The variability of unvegetated channel width is an indicator of the lateral stability of a channel. A channel is classified as uniform if the unvegetated width at a bend is not more than 1.5 times the average width of the narrowest straight section. The quality of vegetation on the inside of a bend will indicate if the bend is forming slowly (i.e. is well vegetated) or rapidly (i.e. is not vegetated).

7.2.5 Influences to Geomorphologic Change

The reaction of streams to natural and anthropogenic influences can be rapid or can gradually develop over time. Rapid changes can cause catastrophic failure of stream crossing structures. Gradually developing changes may be monitored and addressed as needed.

Natural disturbances include:

- Floods
- Droughts
- Landslides
- Forest fires

Human disturbances include:

- Alteration of vegetative cover
- Channelization
- Channel straightening
- Building levees and dikes
- Building bridges and culverts
- Building dams and reservoirs

Changes to channel slope and/or discharge can have significant impacts on the dimensions, shapes, and flow patterns of streams. For example, a simple increase in channel slope could influence a relatively tranquil, meandering stream to adjust into a rapidly varying, high velocity, braided stream with high sediment loads and sandbars. A decrease in channel slope could convert a braided stream into a meandering one. A simple change in hydrology can affect stream sinuosity, meander wave length, and channel width and/or depth.

HEC-20, *"Stream Stability at Highway Structures,"* presents Lane's calculations (1955) for determining the likelihood that a stream channel will be braided or meandering based on the channel slope and discharge. For channels with a sand bed, meanders will begin to form when:

$$SQ^{0.25} \leq 0.0017$$

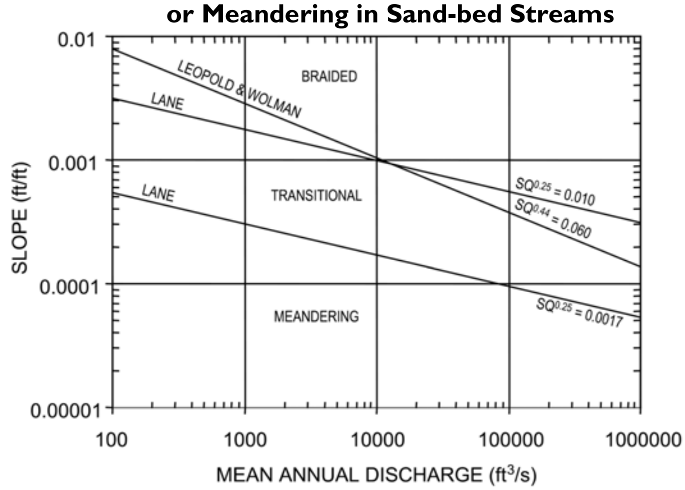
Similarly, braids will begin to form when:

$$SQ^{0.25} \geq 0.010$$

Where:
S = channel slope, ft/ft
Q = mean flow rate, cfs

Figure 7-2 illustrates the slope/discharge relationship to determine channel form for Lane's (1955) and for Leopold and Wolman's methods (1957). For more information about these geomorphic concepts, refer to HEC-20 or the original publications.

Figure 7-2. Slope/Discharge Relationship for Braiding or Meandering in Sand-bed Streams



(FHWA HEC-20, after Lane 1955)

7.2.6 Other Types of Scour

Refer to NEH Technical Supplement 14B, *"Scour Calculations"* for descriptions of methods and calculations for estimating scour at bends and within bedforms (such as meanders and braids).

Understanding the evolution—or potential evolution—of a stream channel at a bridge crossing is critical to bridge foundation design. Subsequent scour calculations must be performed using the appropriate initial conditions so that the bridge will accommodate potential future conditions within the design life of the structure.

7.3 Bridge Scour

Bridge foundations are composed of designed structural components and natural earthen components. The earthen components support the structural components. Sediment transport within the stream channel can dynamically change the condition of the natural earthen components by eroding (scouring) away bed material and undermining the designed structural components. Unless the foundation is designed to be free standing, erosion of the supporting bed material can cause the foundation and the bridge deck to collapse. A bridge that is undermined by scour is particularly dangerous because it often appears safe to the general public, but it could fail catastrophically at any time.

7.3.1 Bridge Scour Resources

HEC-18, *"Evaluating Scour at Bridges,"* describes the methods used to calculate contraction, pressure, and local scour at bridge foundation components.

NEH Technical Supplement 14B, "*Scour Calculations*," provides a general overview of scour and detailed methods for estimating the impacts of scour on the stream channel.

Appendix E of the USDA Forest Service document, "*Stream Simulation: An Ecological Approach to Providing Passage for Aquatic Organisms at Road-Stream Crossings*," contains detailed descriptions and procedures for assessing streambed mobility and performing analyses to design streambed substrate for stability.

NEH Technical Supplement 14C, "*Stone Sizing Criteria*" provides techniques to design earthen gradations to resist erosion due to shear stresses.

7.3.2 Bridge Foundation Design Considerations

Bridge foundations must be designed to accommodate both rapidly and gradually evolving changes in the streambed. The five primary types of streambed changes are listed below and described in more detail in subsequent subsections.

- Long-term profile changes due to aggradation and/or degradation
- Plan form changes due to lateral movement of the channel within the floodplain
- Local contraction scour/deposition
    - Local scour caused by turbulence at horizontal contractions in the stream channel
    - Local deposition caused by turbulence at horizontal expansions in the stream channel
- Local scour caused by turbulence at vertical contractions in stream depth as a result of pressure flow through bridge openings
- Local scour at fixed structural components (i.e. abutments or piers)

The following portion of this chapter describes methods to evaluate each of these items. The ANR's "*Vermont Stream Geomorphic Assessment Handbooks*," describe detailed methods for determining potential long-term and plan form changes responding to influences in the contributing watershed.

7.3.2.1 Long-Term Profile Changes

Evaluate the potential for long-term profile changes before determining the depths and elevations of structural footings. Design proposed structures for stability and sufficiency for current and future bed profiles. If the analysis of the long-term profile indicates that the stream will degrade, use the elevation after degradation as the base elevation for calculating contraction and local scour. Refer to Section 7.2 for information about estimating long-term profile changes in river and stream.

7.3.2.2 Plan Form Changes

Plan form changes include changes that influence the shape and alignment of the stream channel. Incised channels can evolve from meandering channels to braided channels. Meanders can migrate laterally and threaten to erode bridge approaches. Bank widening can change the bridge contraction ratio. Plan form changes are considered permanent future changes for the stream bed elevation at a bridge site and should be considered as part of the foundation design process. Refer to Section 7.2 for information about estimating plan form changes in river and stream.

7.3.2.3 Local Horizontal Contraction Scour/Deposition

Channel contraction scour is caused by a horizontal constriction of the channel, which may be partially caused by bridge foundation components in and adjacent to the flowing water. Deposition of bed material may be caused by an expansion of the channel, which may occur at sites where a bridge is located immediately downstream of a steeper reach or at the downstream end of a natural constriction.

7.3.2.4 Local Vertical Contraction (Pressure) Scour

Pressure scour is caused by a vertical constriction of the channel, which occurs when the bridge opening is submerged and excess water is forced through the opening under pressure. Pressure scour is calculated similarly to horizontal contraction scour with one main difference: calculations must account for a zone of ineffective flow referred to as the 'separation zone' that occurs between the top surface of the flowing water and the lower elevation of the bridge deck.

7.3.2.5 Local Scour at Piers and Abutments

Local scour is caused by the disruption of flow around fixed features such as pier and abutment foundation components. Local scour depends on the geometry of the fixed features and their orientation with respect to the flowing water. In areas experiencing subcritical flow, the maximum depth of potential local scour can be estimated well using existing formulas. The maximum depth of potential local scour for areas experiencing supercritical flow is not well researched.

7.3.2.6 Other Types of Scour

Scour can also occur at bends and within bedforms (such as meanders and braids). Typically, material scoured from the outside radius of a meander bend is redistributed to the inside radius. Braids consistently form and reform as a result of movement of bed material. Refer to Section 7.2.5 to determine if meanders and/or bends are likely to affect the crossing.

7.4 Scour Calculations

To perform scour calculations, first estimate long-term degradation, plan form changes, and meander/braiding conditions using the information provided in Section 7.2. Then use the resulting channel geometries as initial conditions for the subsequent calculations, described below.

7.4.1 Scour Design Events

Estimate scour conditions during these events:

- Scour design event (see Table 7-1)
- Scour check event (see Table 7-1)
- Largest event that does not overtop the structure—for cases in which the design or check events do overtop the structure (referred to herein as the incipient overtopping event)

Quick Tip

The frequency and magnitude of the incipient overtopping event may not be determined until the bridge and footing design is near finalization. However, this event must not be overlooked because it often puts the most stress on the bridge and can result in the greatest scour.

Determine which event results in the most severe potential scour condition. Design foundation components to be free standing (i.e. able to support the bridge) in the event that the most severe scour condition comes to fruition.

Keep in mind that hydraulic/scour analysis and crossing design are multi-disciplinary efforts, and the results of each component inform the entire effort. Communicate within the team and iterate the process as needed to achieve the project objectives.

Table 7-1. HEC-18 Scour Design and Check Event Selection

Hydraulic Design Event AEP* (%)	Scour Design Event AEP (%)	Scour Check Event AEP (%)
10%	4%	2%
4%	2%	1%
2%	1%	0.5%
1%	0.5%	0.2%

* *Refer to Chapter 4 "Hydrology" for the hydraulic design event for roads by classification. AEP stands for annual exceedance probability and refers to the percent likelihood that a storm event of a certain magnitude will occur in any given year.*

7.4.2 Clear-water and Live-bed Sediment Transport Conditions

The maximum potential depth of contraction scour and local pier and abutment scour are affected by the potential for sediment transport of bed material. In a live-bed sediment transport condition, bed material is transported during scour events; in a clear-water sediment transport condition, it is not.

Live-bed scour is cyclic in nature and occurs when bed material from upstream is transported to the crossing. A scour hole that develops in a live-bed condition will develop during the rising stage of a flood and fill during the falling stage.

Clear-water scour occurs when there is no movement of bed material in the flow upstream of the crossing. A scour hole that develops in a clear-water condition will form during the rising stage of a flood (or gradually deepen through many floods) and remain fixed during low-flow conditions. Clear-water scour depths are generally about 10% deeper than live-bed scour depths for a given set of initial conditions. Clear-water scour holes, unlike live-bed scour holes, do not get partially filled in by settling sediment after a flood event.

First, determine if a stream experiences live-bed or clear-water flow conditions using the median diameter (D_{50}) of the bed material upstream of the proposed crossing. Calculate the critical velocity that corresponds to the D_{50} using the equation below. Use the hydraulic model to calculate channel and overbank velocities during scour events.

$$V_c = 11.17\, y^{\frac{1}{6}} D^{\frac{1}{3}}$$

Where:

V_c = the critical velocity above which the bed material of particle size, D, and smaller will be transported, ft/s

y = the average depth of flow in the channel upstream of the bridge, ft

D = particle size, ft

If channel and/or overbank velocities exceed the critical velocity, assume live-bed flow conditions. If they do not, assume clear-water flow conditions.

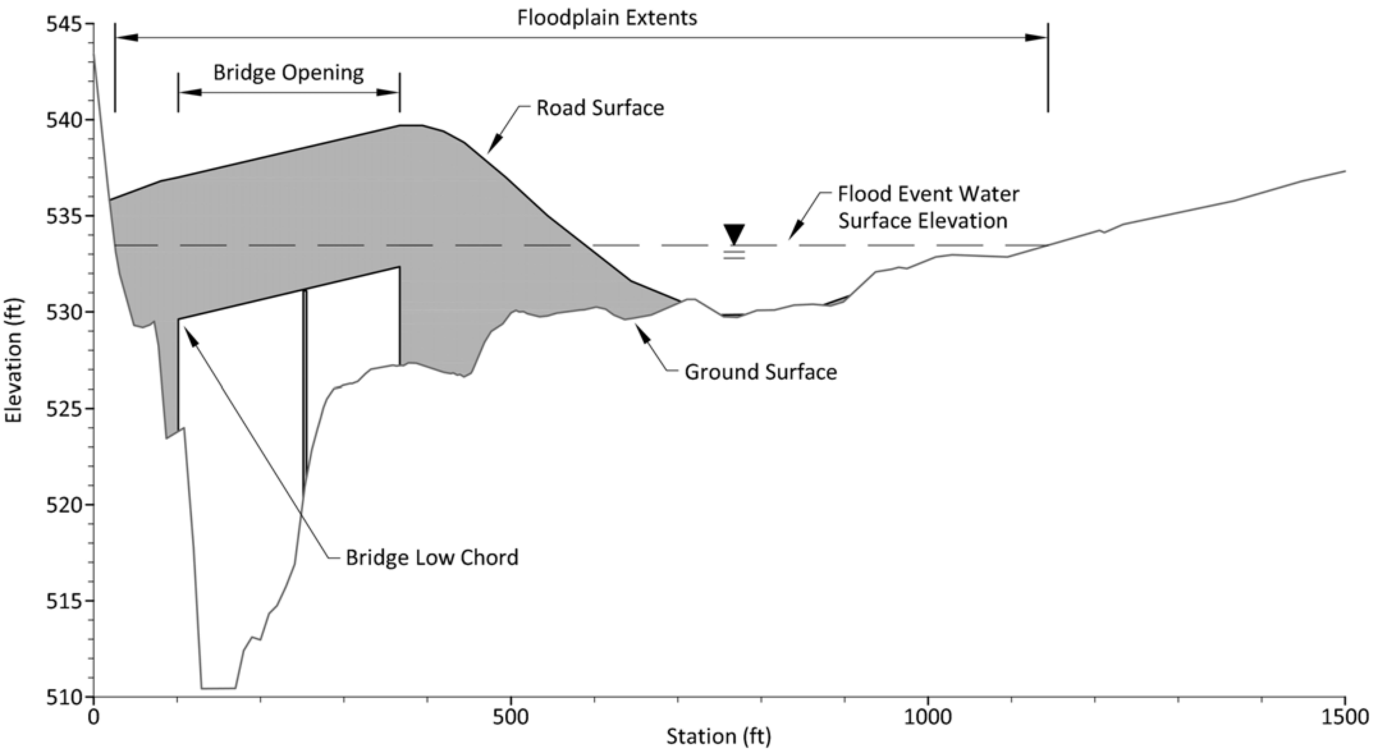
Always assess the sensitivity of the scour depth estimates to the value of D_{50} and use a value that provides appropriately conservative results.

7.4.3 Local Contraction Scour

Contraction scour occurs when the active flow area is reduced by a constriction. A horizontal constriction occurs when floodplain flows are forced back into the channel to pass through a narrow bridge opening. A vertical constriction occurs when the low chord of the bridge is submerged and flow through the bridge opening is pressurized.
Figure 7-3 illustrates a bridge crossing that experiences both horizontal and vertical constrictions during the design event.

The decrease in flow area results in an increase in velocity and shear stress on fixed and movable boundaries. The increase in shear stress mobilizes and erodes bed material until the flow area increases to reach a stable equilibrium with the movable material.

Figure 7-3. Bridge Crossing with Horizontal and Vertical Constrictions (Otter Creek and River Road, Rutland City)



If the bed material is comprised of cohesive sediments (i.e. bedrock), field testing of shear strength may be required. Refer to HEC-18, *"Evaluating Scour at Bridges,"* for methods to calculate ultimate scour depth and the time required to produce the ultimate scour depth in cohesive materials. Compare the time of formation of the scour hole to the design life of the structure and design the foundations accordingly.

7.4.3.1 Local Horizontal Contraction Scour

Chapter 6 of HEC-18 describes the concepts and techniques for calculating contraction scour. HEC-18 identifies four general cases for scour, which are briefly described here.

Case 1: Overbank flow is forced back to the main channel by the bridge approach.
Case 2: There is no overbank flow. Flow is confined to the main channel and the normal channel becomes narrower due to the bridge itself.
Case 3: Flow relief is provided in the overbank area. The relief area experiences clear-water scour.
Case 4: Flow relief is provided in the overbank area. The relief area experiences live-bed scour.

All cases are evaluated using the same principle scour equations. However, the case informs how the values of each input variable are determined. Refer to HEC-18 for more detail.

Calculate live-bed contraction scour depth using the following equation:

$$y_2 = \left[\left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}}\left(\frac{W_1}{W_2}\right)^{k_1}\right] y_1$$

Where:
y_2 = average depth of flow in the main channel in the constricted section, ft
y_1 = average depth of flow in the main channel upstream of the constriction, ft
Q_1 = flow rate in the upstream channel, cfs
Q_2 = flow rate in the constricted channel, cfs
W_1 = bottom width of the upstream channel, ft
W_2 = bottom width of the constricted section minus the width occupied by piers or other obstructions, ft
k_1 = scour coefficient, see Table 7-2

It is acceptable to use the top width of the channel for both W_1 and W_2 if the bottom widths are not known. Be consistent.

Table 7-2. Scour Equation Coefficient, k₁

V^*/T	k_1	Mode of Bed Material Transport
< 0.5	0.59	Mostly contact bed material discharge
0.5 to 2.0	0.64	Some suspended bed material discharge
> 2.0	0.69	Mostly suspended bed material discharge

Calculate the shear velocity in the upstream section using the following equation:

$$V^* = (g\, y_1 S_1)^{0.5}$$

Where:
V^* = shear velocity in the upstream section, ft/s
g = acceleration due to gravity, 32.2 ft/s^2
y_1 = average depth of flow in the main channel upstream of the constriction, ft
S_1 = slope of the energy grade line of the main channel, ft/ft
T = fall velocity of the bed material based on the D_{50}. See Figure 6.8 in HEC-18. Be sure to convert the result to consistent units, ft/s

Calculate the clear-water contraction scour depth using the following equation:

$$y_2 = \left(\frac{0.0077\, Q^2}{D_m^{\frac{2}{3}} W^2}\right)^{\frac{3}{7}}$$

Where:
y_2 = average depth of flow in the main channel in the constricted section, ft
Q = flow in the constricted channel, cfs
W = bottom width of the constricted section minus the width occupied by piers or other obstructions, ft
D_m = diameter of the smallest non-transportable particle in the bed material ($1.25 * D_{50}$) in the constricted section, ft
D_{50} = median diameter of the bed material (ft)

Scour depth is the difference between the existing average depth of the constricted section and the average depth of the constricted section after the scour hole forms. Calculate the scour depth using the following equation:

$$y_s = y_2 - y_o$$

Where:

y_s = scour depth, ft

y_2 = average depth of flow in the main channel in the constricted section, ft

y_0 = the existing depth of flow in the contracted section before the scour occurs, ft

7.4.3.2 Local Vertical Contraction (Pressure) Scour

HEC-18 Section 6.10 describes the concepts and techniques for calculating vertical contraction scour (pressure flow scour).

Pressure flow occurs when the flow of water through a constricted opening no longer meets the criteria of open channel flow. This may occur at bridge crossings when headwater conditions result in a submerged opening.

Vertical contraction scour begins to occur when the bridge opening becomes submerged. It increases until the maximum discharge through the opening is reached. It decreases as overflow relief is provided either by weir flow over the bridge or overland floodplain flow over the bridge approach sections.

Calculate the vertical contraction scour using the same methods and equations used to calculate horizontal contraction scour with minor modifications, described below:

$$y_2 = y_s - t + h_b$$

Where:

y_2 = average depth of flow in the main channel in the constricted section, ft

y_s = scour depth, ft

t = the flow separation thickness, ft

h_b = distance from the low chord of the bridge to the average elevation of the stream bed before scour, ft

If no relief flows bypass or overtop the bridge, calculate y_2 using the appropriate horizontal contraction scour equations for live-bed or clear-water scour with the variable definitions described previously.

If flows bypass the constricted opening, be sure to adjust flow rates to include only the flow going through the opening.

In live-bed conditions where flows overtop the bridge, adjust Q_1 and y_1 before applying the horizontal contraction equations according to the following:

$$h_{ue} = h_b + T$$

$$Q_{ue} = Q_1 \left(\frac{h_{ue}}{h_u}\right)^{\frac{8}{7}}$$

Where:

h_{ue} = effective upstream channel flow depth to be used in the live-bed horizontal contraction equation in place of y_1, ft

Q_{ue} = the effective channel discharge to be used in the live-bed horizontal contraction equation in place of Q_1, cfs

h_b = distance from the low chord of the bridge to the average elevation of the stream bed before scour, ft

T = the height of the obstruction including girders, deck, and parapet, ft

Q_1 = channel discharge upstream of the constriction, cfs

h_u = upstream channel flow depth, ft

The separation zone thickness, t, is calculated with the following:

$$\frac{t}{h_b} = 0.5\left(\frac{h_b - h_t}{{h_u}^2}\right)^{0.2}\left(1 - \frac{h_w}{h_t}\right)^{-0.1}$$

Where:

t = the flow separation thickness, ft

h_b = distance from the low chord of the bridge to the average elevation of the stream bed before scour, ft

h_u = upstream channel flow depth, ft

h_t = distance from the water surface to the low chord of the bridge ($h_u - h_b$), ft

h_w = weir flow depth, ft

7.4.4 Local Scour at Piers and Abutments

Local scour at piers and abutments is primarily caused by turbulence and two well-described vortices, the horseshoe vortex and wake vortex. The horseshoe vortex forms at the base of the structure. The wake vortex forms downstream from the leading edge of the structure. The wake vortex loses influence along long structures oriented parallel to flow.

The magnitude of the local scour depth is influenced by the velocity and depth of the approach flow, the shape, width, and length of the structure, the orientation of the structure with respect to flow, the size and gradation of bed material, and the presence of ephemeral or permanent debris. Considerations for reducing local scour depths are listed in Section 7.5.

7.4.4.1 Pier Scour

Pier scour depths can be calculated using the methods presented in HEC-18, which include the HEC-18 pier scour equation and the Florida Department of Transportation (FDOT) pier scour methodology.

The HEC-18 pier scour equation is based on the Colorado State University (CSU) method. Depths calculated using this approach are conservative and rarely under predict measured scour depths.

FDOT provides an alternative methodology for calculating pier scour. Consider using the FDOT method in applications with wide piers, where the ratio of flow depth to pier width is less than 0.2, or in shallow flow areas with fine bed material. Refer to HEC-18 for details regarding this methodology.

Most research on pier scour depths has been conducted on areas experiencing subcritical flow ($Fr < 1.0$). If the subject bridge crossing is located in an area that may experience supercritical flow, be sure to test for mobility of bed material and design channel substrate appropriately.

This manual does not include instructions for calculating pier scour in cohesive materials. If the bed material is comprised of cohesive sediments, field testing of shear strength may be required. Refer to HEC-18 for methods to calculate ultimate pier scour depth and the time required to produce the ultimate pier scour depth in cohesive materials. As a rule of thumb, pier scour depths range up to 2.4 or 3.0 times the width of the pier.

HEC-18 suggests:

If $Fr < 0.8$, then $y_s < 2.4\,a$

If $Fr > 0.8$, then $y_s < 3.0\,a$

Where:

a = pier width, ft

y_s = scour depth, ft

Fr = Froude number directly upstream of the pier

HEC-18, in accordance with the CSU method, uses the following equation to calculate pier scour depth:

$$y_s = 2.0\, y_1 K_1 K_2 K_3 K_w \left(\frac{a}{y_1}\right)^{0.65} Fr^{0.43}$$

Where:

y_s = scour depth, ft

y_1 = is the flow depth directly upstream of the pier, ft

K_1 = correction factor for nose shape

K_2 = correction factor for angle of attack of flow

K_3 = correction factor for bed condition

K_w = correction factor for pier width

a = pier width, ft

Fr = Froude number directly upstream of the pier

And where:

$$Fr = \frac{V_1}{\sqrt{g y_1}}$$

V_1 = velocity of flow directly upstream of the pier, ft/s

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

y_1 = is the flow depth directly upstream of the pier, ft

Correction Factor for Nose Shape

Table 7-3. Nose Shape Correction Factor, K1

Shape of Nose	K_1
Square	1.1
Round	1.0
Circular/Cylinder	1.0
Group of Cylinders	1.0
Sharp	0.9

Note: *If the angle of attack (θ) is greater than 5 degrees (0.087 radians), assume K_1 = 1. 0*

7.4.4.2 Correction Factor for Angle of Attack of Flow

When flow is parallel to the pier, the angle of attack correction factor, K_2, is equal to 1.0. When flow is perpendicular to the pier, the maximum value of K_2 should never exceed 5.0.

$$K_2 = \left[\cos\theta + \frac{L}{a}\sin\theta\right]^{0.65}$$

Where:

K_2 = correction factor for angle of attack of flow
θ = the angle of attack of the flow on the pier, radians
L = pier length, ft
a = pier width, ft

Caution!

Use the correction factor for angle of attack of flow wisely. HEC-18 cautions that, "The values of the correction factor K_2 should be applied only when the field conditions are such that the entire length of the pier is subjected to the angle of attack of the flow. Use of this factor will result in a significant over-prediction of scour if (1) a portion of the pier is shielded from the direct impingement of the flow by an abutment or another pier; or (2) an abutment or another pier redirects the flow in a direction parallel to the pier. For such cases, judgment must be exercised to reduce the value of the K_2 factor by selecting the effective length of the pier actually subjected to the angle of attack of the flow."

Correction Factor for Bed Conditions

Most rivers and streams experience the plane-bed condition during the flows used in scour design. If medium or large dunes are present, refer to HEC-18 for further guidance.

Table 7-4. Bed Condition Correction Factor, K_3

Bed Condition	Dune Height, H (ft)	K_3
Clear-water scour	n/a	1.1
Plane bed and anti-dune flow	n/a	1.1
Small dunes	10 > H ≥ 2	1.1
Medium dunes	30 > H ≥ 10	1.1 to 1.2
Large dunes	H ≥ 30	1.3

Correction Factor for Pier Width

Under most conditions, the correction factor for pier width should be set equal to 1.0. However, the CSU pier scour equation can overestimate scour for cases with wide piers and shallow flows. If the ratio of the depth of flow to pier width is less than 0.8, consider adjusting the value of K_w using the following equations. This adjustment is not conservative; i.e. it will reduce the calculated scour depth. Use it if engineering judgment indicates that the CSU overestimates pier scour for the particular design under review.

If $y/a \leq 0.8$, then

If $V_1/V_c \leq 1$, then $K_w = 2.58(y/a)^{0.34}Fr^{0.65}$

If $V_1/V_c > 1$, then $K_w = 1.0(y/a)^{0.13}Fr^{0.25}$

Where:

K_w = correction factor for pier width
V_1 = velocity of flow directly upstream of the pier, ft/s
V_c = the critical velocity above which the bed material of particle size, D, and smaller will be transported, ft/s
y = the average depth of flow in the channel upstream of the bridge, ft
a = pier width, ft
Fr = Froude number directly upstream of the pier

A discussion of complex pier scour is omitted from this handbook. See Section 7.5.2 for more information on VTrans conventions for foundation design. If the multi-disciplinary bridge foundation design team recommends a footing design that allows for exposed piles, refer to the HEC-18 chapter on Complex Pier Foundations.

7.4.4.3 Pier Scour within Coarse Bed Material

If the scour analysis determines that the study area experiences clear-water scour conditions where the following conditions are met:

- The approach flow velocity is less than the critical velocity for bed material motion
- The median diameter (D_{50}) of the bed materials is greater than 2 inches, and
- The sediment gradation coefficient, σ, is greater than or equal to 1.5,

consider using the following equation to estimate pier scour depth:

$$y_s = 1.1K_1K_2a^{0.62}{y_1}^{0.38}\,tanh\left[\frac{H^2}{1.97\sigma^{1.5}}\right]$$

Where:

y_s = scour depth, ft
K_1 = correction factor for nose shape
K_2 = correction factor for angle of attack of flow
a = pier width, ft
y_1 = is the flow depth directly upstream of the pier, ft
σ = sediment gradation coefficient, D_{84}/D_{50}
H = the densimetric particle Froude Number

And where:

$$H = \frac{V_1}{\sqrt{g(S_g - 1)D_{50}}}$$

H = the densimetric particle Froude Number
V_1 = velocity of flow directly upstream of the pier, ft/s
S_g = sediment specific gravity
g = acceleration due to gravity, 32.2 ft/s^2
D_{50} = median diameter of bed material, ft or in

7.4.4.4 Abutment Scour

Abutment scour depths can be calculated using the Froehlich, Highways in the River Environment (HIRE), and National Cooperative Highway Research Program (NCHRP) 24-20 equations presented in HEC-18.

In general, VTrans experience with abutment scour indicates that the existing methods for estimating scour depths are too conservative. VTrans recommends countermeasures to protect against abutment scour (i.e. placement of armoring stone fill). In general, VTrans recommends that abutments be designed with a footing that is free standing to the greater of the following two depths:

- The sum of contraction scour and long-term degradation.
- Six feet below the depth of the thalweg.

If abutment scour is a concern at the site of interest, refer to HEC-18 for guidance.

7.4.5 Debris and Ice

The accumulation of debris, (e.g. brush, logjams, ice, and trash) along stream banks and at stream crossings can have a significant impact on local scour conditions. If the debris becomes lodged permanently, it can also alter the overall stream geometry (e.g. stream width, profile, and plan form geometry). Debris accumulation at bridge crossings should be cleared as soon as possible to prevent the formation of unanticipated scour holes. If seasonal ice jams are likely, design bridge foundations to accommodate them. USACE provides technical references and support for ice jam analyses via the Cold Regions Research and Engineering Laboratory (CRREL).

7.4.6 Scour Prism Plots

The documentation associated with a scour analysis should include the calculations associated with the five primary types of streambed changes (refer to Section 7.3.2 for a list of the types). It should also include a plot of the scour prism for each scour design event.

Figure 7-4 illustrates a typical plot of the scour prism. The plot should include the following features:

- The existing cross section.
- The elevation of the cross section adjusted for long-term degradation.
- The depth of the horizontal contraction scour below the lowest point (the thalweg) of the cross section that represents the long-term degradation.
- The depth of the vertical scour (in the special case of pressure flow) below the horizontal contraction scour line.
- The pier scour from and below the contraction scour line.
- The abutment scour from and below the long-term degradation line. Unless geomorphic assessments have determined that the stream channel is likely to migrate to a location that will have a significant impact on the abutment, assume the abutment scour extends 6 feet below the thalweg.
- The pier scour hole widths.

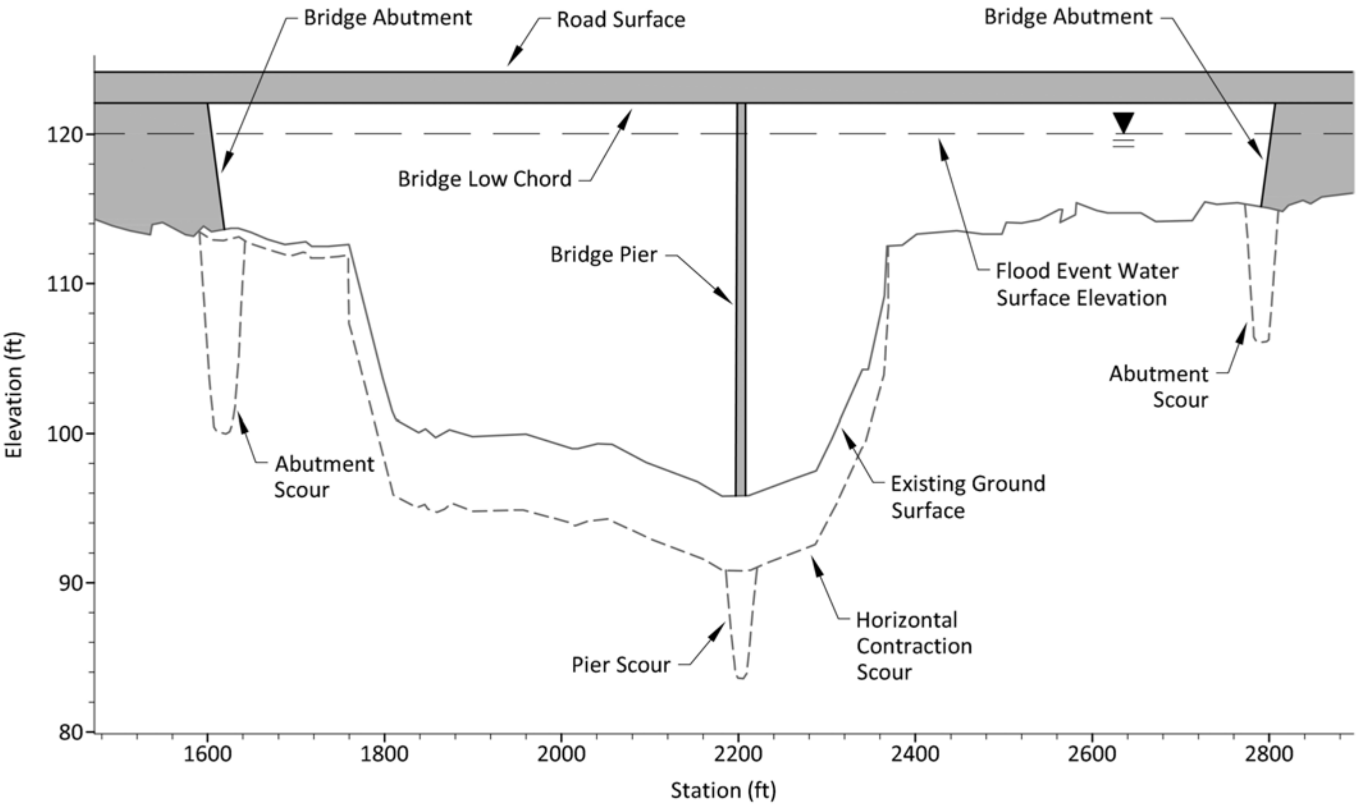
7.5 Countermeasures

HEC-23, *"Bridge Scour and Stream Instability Countermeasure: Experience, Selection, and Design Guidance,"* vol. 1 and vol. 2 describe and provide design guidance for stream stability and scour reduction countermeasures that have been implemented by various U.S. and international transportation agencies.

Countermeasures include:

- Protection for bridge piers
- Protection for bridge abutments
- Riprap or other armoring
- Protection for environmentally sensitive channel and banks

Figure 7-4. Scour Prism at Bridge Crossing



Countermeasures control, inhibit, change, delay, or minimize the impact of stream stability and erosion (including scour) on highway stream crossings. They may be designed and installed with original construction or may be retrofit to resolve developing problems. Retrofitting may be appropriate, in terms of economics and engineering practice, because the magnitude, location, and nature of potential stability problems are not always discernible at the design stage, and may take a period of several years to develop.

> **Caution!**
> Do not incorporate countermeasures in order to raise the required elevations of new bridge foundations. Bridge foundations must be designed to remain freestanding in the wake of a scour check event without taking into account protection provided by scour countermeasures. Refer to Section 7.4.4.4 for special considerations regarding this rule-of-thumb.

Countermeasures may be classified into three principal groups:

1. Hydraulic
    a. River training structures
        i. Transverse structures
        ii. Longitudinal structures
        iii. Areal structures
    b. Armoring countermeasures
        i. Revetments and bed armor
        ii. Local scour armoring
2. Structural
    a. Foundation strengthening
    b. Pier geometry modification
3. Biotechnical

Hydraulic countermeasures are designed to modify the flow (e.g. river training structures) or resist erosive forces (e.g. armoring countermeasures) caused by the flow. Structural countermeasures involve modification of the bridge foundation to prevent failure from scour. Biotechnical countermeasures involve using vegetation to control stream bank erosion by stabilizing the banks. Countermeasure selection requires thorough understanding of technical, economic, and social aspects of design including:

- The mechanism of erosion
- Stream hydrology and hydraulics
- Construction and maintenance requirements
- Potential for vandalism
- Cost

Countermeasures can be designed to minimize the impacts of changes such as:

- Meander migration
- Channel braiding
- Degradation
- Aggradation
- Scour

To minimize the risk of impacts due to meander migration:

- Construct the crossing on a relatively straight reach of stream between bends.
- Protect the existing bank line.
- Establish a new preferred flow or alignment.
- Control the constriction of channel flow.

To minimize the risk of impacts due to channel braiding and anabranching, confine multiple channels to one channel to increase velocities and sediment transport capacity.

To minimize the risk of impacts due to long-term degradation, consider constructing grade control and/or drop structures and lining the channel to limit headcutting and downward migration of the streambed.

To minimize the risk of impacts due to channel aggradation, consider maintenance activities such as dredging or clearing debris to increase the velocity and conveyance. If low-maintenance structural solutions are desired, consider channelizing the stream and constructing off-line basins to trap debris.

To minimize the risk of impacts due to scour, consider designing a stone fill to armor the channel. More detailed guidance on designing stone fill is provided in Chapter 5 *"Open Channels."*

Countermeasures must be designed with consideration for the environmental impact of such measures on the stream channel and banks. Refer to Chapter 6 *"Crossing Structures"* for more information about aquatic organism passage (AOP) provisions.

7.5.1 Evaluate Alternative Configurations

Consider reducing the magnitude of contraction scour by:

- Relocating the crossing to a more amenable location
- Increasing the width of the bridge opening
- Increasing the height of the bridge opening

Consider reducing the magnitude of the pier scour by:

- Aligning the pier parallel to the direction of flow
- If parallel alignment is not possible, replacing a long rectangular pier with a cylindrical pier
- Minimizing the width of the pier
- Modifying the spacing of piers to avoid overlapping scour holes with other piers and abutments
- Specifying a streamlined nose. In order of smallest to largest scour depth: (1) sharp-nosed, (2) round nosed, and (3) square –nosed

Consider reducing the magnitude of abutment scour by:

- Widening the open area of the bridge and moving the abutments back
- Angling embankments downstream
- Replacing vertical wall abutments with sloped stone fill

7.5.2 Foundation Design Guidance

The following criteria, adapted from HEC-18, *"Evaluating Scour at Bridges,"* provide guidance for designing scour-resistant bridges. In general, HEC-18 encourages the designer to seriously consider designing all footings/foundations located in the floodplain to the same depth as those located in the main channel. That way, if a migrating stream channel threatens an abutment, the abutment is already constructed to resist scour. As previously stated, VTrans recommends taking a team-based approach to designing bridge foundations.

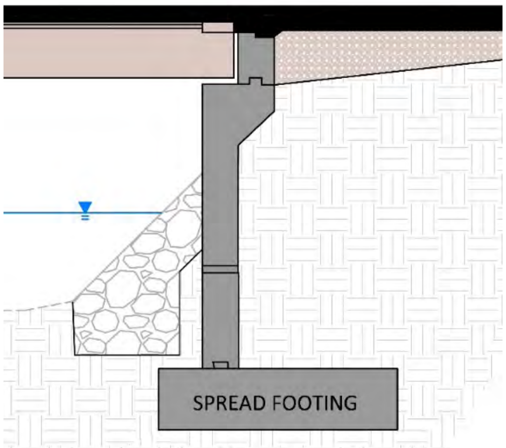
If ANR Phase 1 and Phase 2 assessments indicate that horizontal and vertical channel migration is likely, the crossing design must either include measures to control the horizontal location of the stream channel or include footings designed to perform safely if such horizontal migrations are allowed to occur.

In accordance with the *VTrans Structures Design Manual*, spread footings must be a minimum of 2 feet thick, and footings on piles must be a minimum of 3 feet thick.

7.5.2.1 Footings and Foundations on Floodplains

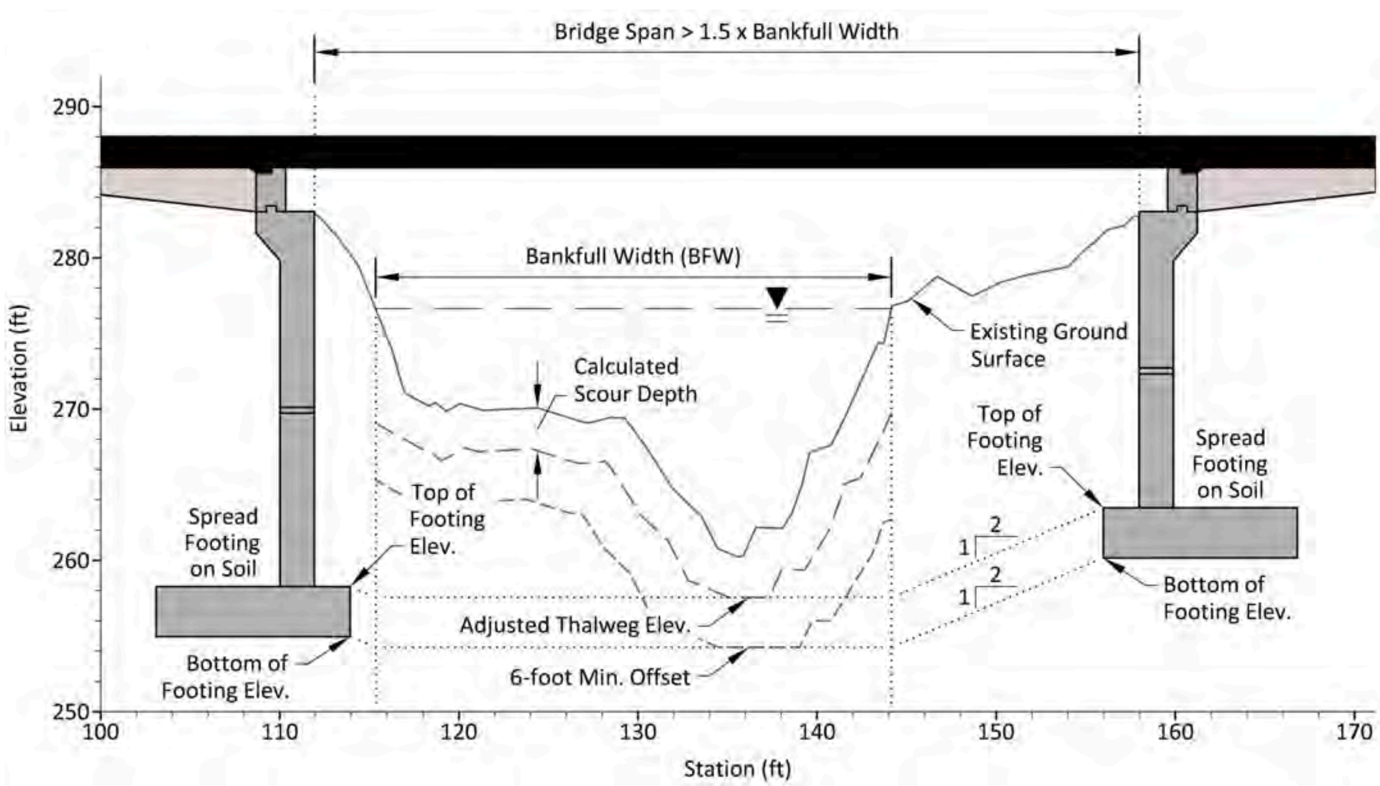
- Conduct a geomorphologic analysis to determine the stream's lateral stability and the likelihood that the thalweg will shift laterally.
- Consider placing footings/foundations on floodplains at similar elevations to what would be required if they were in the main channel.

7.5.2.2 Spread Footings on Soil

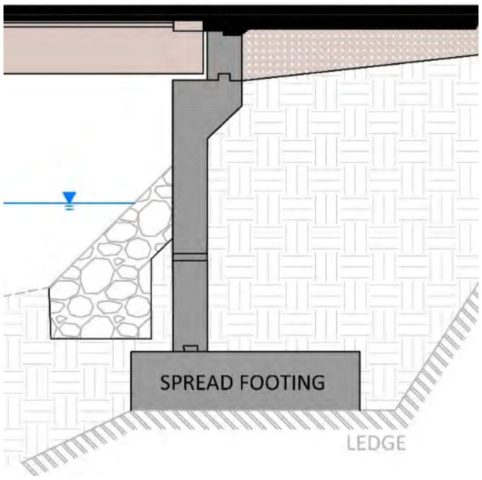


- Place the top of the footing below the adjusted thalweg elevation (i.e. the elevation of the thalweg lowered to account for long-term degradation, contraction scour, and pier scour, where applicable).
- The bottom of the footing should be 6 feet below the thalweg, as a minimum.
- If the bridge span is greater than 1.5 times bankfull width (BFW), the footing may be raised above the previously identified elevation, provided that an ANR Phase 2 geomorphic assessment has been completed. Proceed only if the stream is regarded as laterally and vertically stable. Refer to Figure 7-5 for an illustration depicting how to establish the new elevation.
    - o To find the elevation of the footing top, draw a line at a 2:1 (H:V) slope beginning at the edge of the channel at the adjusted thalweg elevation and terminating at the station of the footing.
    - o To find the elevation of the footing bottom, draw a line at a 2:1 (H:V) slope beginning at the edge of the channel 6 feet below the thalweg elevation and terminating at the station of the footing.
    - o The deepest of the newly calculated top and bottom elevations will control. The footing thickness must be in accordance with the *VTrans Structures Design Manual.*

Figure 7-5. Adjusted Elevation of Spread Footing on Soil for Bridges with Spans >1.5 x Bankfull Width

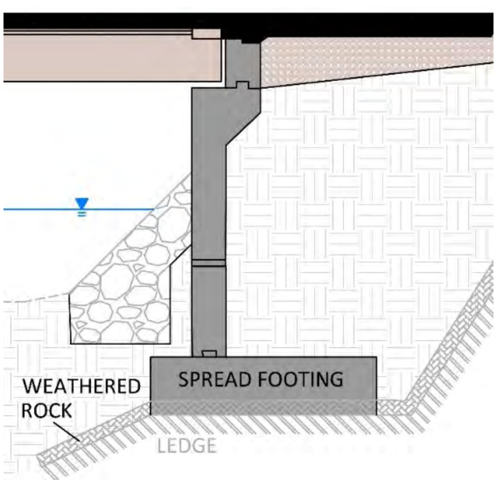


7.5.2.3 Spread Footings on Rock Highly Resistant to Scour



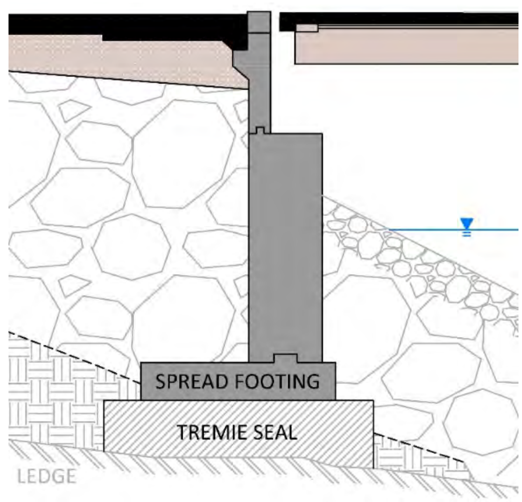
- For massive rock formations that are highly resistant to scour, place the bottom of the footing directly on the cleaned rock surface.
- Avoid embedments (keying) because blasting to achieve keying frequently damages the sub-footing rock structure and makes it more susceptible to scour.
- If footings on smooth massive rock surfaces require lateral constraint, call for steel dowels to be drilled and grouted into the rock below the footing.

7.5.2.4 Spread Footings on Erodible Rock



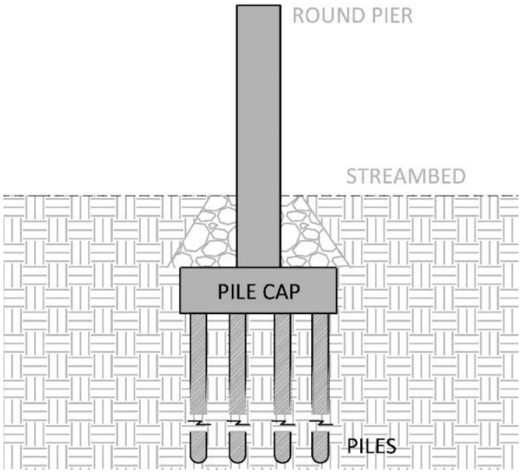
- Carefully assess weathered or other potentially erodible rock formations for scour.
- Consult an engineering geologist familiar with the area's geology to determine if rock, soil, or other criteria should be used to calculate the support for the spread footing foundation. Base the decision on an analysis of intact rock cores, including rock quality designations and local geology, as well as hydraulic data and anticipated service life of the structure.
- If high quality rock is present below a thin weathered zone, place the footing on the high quality rock.
- For deep deposits of weathered rock, place the top of the footing below the adjusted thalweg elevation (i.e. the elevation of the thalweg lowered to account for long-term degradation, contraction scour, and pier scour, where applicable).
- The bottom of the footing should be 6 feet below the thalweg, as a minimum.
- Proceed with care when excavating into weathered rock. If blasting is required, use light, closely spaced charges to minimize overbreak beneath the footing level. Remove loose rock pieces and fill the zone with clean concrete.
- Pour the final footing in contact with the sides of the excavation for the full designed footing thickness to minimize water intrusion below footing level.
- Refer to the 1991 FHWA memorandum "*Scourability of Rock Formations*" for guidance on scourability of rock formations.

7.5.2.5 Spread Footings on Tremie Seals



- Place the top of the footing below the adjusted thalweg elevation (i.e. the elevation of the thalweg lowered to account for long-term degradation, contraction scour, and pier scour, where applicable).
- The bottom of the tremie seal should be 6 feet below the thalweg, as a minimum.

7.5.2.6 Deep Foundations with Footings or Caps (Drilled Shafts and Driven Piles)



The guidance for deep foundations is divided into three categories according to the level of scour risk at the crossing:

High Risk—For bridges that do not span the BFW and/or where the footing or cap will significantly obstruct flow during a scour event (more than two rows of piles, etc.):

- Place the bottom of the footing/cap a minimum of 6 feet below the thalweg.
- Drive shafts/piles below the adjusted thalweg elevation (i.e. the elevation of the thalweg lowered to account for long-term degradation, contraction scour, and pier scour, where applicable). This depth is likely to exceed the established minimum 6-foot adjustment.
- If supporting piles could be damaged by erosion and corrosion from exposure to river or tidal currents, it may advisable to place the top of the footing/cap below the adjusted thalweg elevation. For more discussion on pile and drilled shaft foundations, see the FHWA manuals on design and construction of driven pile foundations and drilled shafts.

Caution!
Deep foundations for piers will almost always fall under the High Risk category because they are typically within BFW.

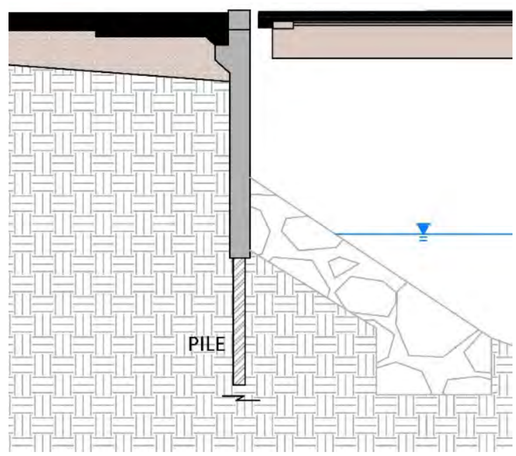
Moderate Risk—For bridges that span the natural channel and have footing/cap dimensions that have been minimized and will not significantly obstruct flow during a scour event:

- Place the top of the footing/cap below the thalweg.
- Drive shafts/piles below the adjusted thalweg elevation (i.e. the elevation of the thalweg lowered to account for long-term degradation and contraction scour). If the calculated adjustment is less than 6 feet, use 6 feet as a minimum.
- The shafts/piles must be free standing and maintain structural integrity in the event the thalweg shifts laterally and the bed material around the piles scours to the adjusted thalweg elevation.

Low Risk—For bridges with spans that are significantly longer than BFW over streams with stable channels that are unlikely to experience scour at the foundation:

- Place the top of the footing/cap wherever reasonable.
- Drive shafts/piles below the adjusted thalweg elevation (i.e. the elevation of the thalweg lowered to account for long-term degradation and contraction scour). If the calculated adjustment is less than 6 feet, use 6 feet as a minimum.
- The shafts/piles must be free standing and maintain structural integrity in the event the thalweg shifts laterally and the bed material around the piles scours to the adjusted thalweg elevation.

7.5.2.7 Stub Abutments on Piles



- For stub abutments positioned in embankments, drive piles below the adjusted thalweg elevation (i.e. the elevation of the thalweg lowered to account for long-term degradation and contraction scour). If the calculated adjustment is less than 6 feet, use 6 feet as a minimum.
- The piles must be free standing and maintain structural integrity in the event the thalweg shifts laterally and the bed material around the piles scours to the adjusted thalweg elevation.

7.6 Procedure to Evaluate Bridge Scour to Support Foundation Design

1. Determine the design events to be used to evaluate scour (refer to Table 7-1). The frequency of the overtopping event may not be determined until the final iteration of this design procedure.

2. Estimate the peak flow rate of the river or stream during the design events using the methods described in Chapter 4 *"Hydrology."* The peak flow rate during the overtopping event may not be determined until the final iteration of this design procedure.

3. Build a hydraulic model of the study area (preferably in HEC-RAS) using the methods described in Chapter 6 *"Crossing Structures"* and supplemental guidance specific to scour applications described in this chapter. Calculate the water surface profiles.

4. Determine the size of a bed material which will resist movement and cause armoring to occur.

5. Assess the bridge crossing for profile bed scour changes to be expected from degradation or aggradation. The designer can obtain a quick estimate of degradation by identifying downstream headcuts and projecting the profile downstream of the headcut up to the crossing. A more thoughtful estimate would require sediment transport modeling during simulated flood events.

6. Assess the bridge crossing for historic and/or developing plan form changes including meanders, braiding, etc. Attempt to forecast whether encroaching meanders could reach the crossing within the service life of the structure.

7. Adjust the fixed-bed hydraulic variables in the HEC-RAS model based on the likelihood of impacts due to aggradation, degradation, meanders, and/or braiding. Recalculate the water surface profiles.

8. Compute the magnitude of the scour components:
   a. Long term streambed elevation
   b. Contraction (live-bed vs. clear-water)
   c. Pressure
   d. Local pier scour (live-bed vs. clear-water)
   e. Local abutment scour
   f. Lateral stream migration (thalweg)
   g. Bends

9. Plot the scour depths.

10. Evaluate the results obtained in Steps 7, 8, and 9. Are they reasonable, considering the limitation in current scour estimating procedures? The scour depth(s) adopted may differ from the equation value(s) based on engineering judgment.

11. Review results with multi-disciplinary team. Identify modifications and/or countermeasures that might minimize scour impacts.

12. Revise the hydraulic model to test alternative countermeasures. Recalculate water surface profiles.

13. Once an acceptable scour threshold is reached, the geotechnical and structural engineers can establish a preliminary foundation design.

Chapter 8 Storm Drainage Systems

8.1 *Introduction*

8.1.1 Overview

Highway storm drainage systems collect stormwater runoff and convey it within 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 systems can convey flow using a combination of curbs, gutters, pipes, channels, and culverts. The placement and hydraulic capacities of storm drainage structures and conveyances should be designed to take into consideration damage to adjacent property and to ensure that the risk of flooding and the corresponding traffic interruption is consistent with the importance of the road.

Quick Tip

This chapter is meant to describe transportation-specific drainage standards. It is not meant to be a fully inclusive stormwater manual. Refer to the Vermont Agency of Natural Resources (ANR) *"Vermont Stormwater Management Manual"* (VSMM) and other more inclusive documentation when designing stormwater infrastructure.

8.1.2 Resources

The Federal Highway Administration (FHWA) provides a suite of excellent guidance documents pertaining to storm drainage systems. The FHWA Hydraulic Engineering Circular (HEC) publications include supplementary information regarding design principles and engineering techniques related to channel design. These publications include:

- HEC-21, *"Design of Bridge Deck Drainage"*
- HEC-22, *"Urban Drainage Design Manual"*

8.1.3 Design Tools

The FHWA offers additional technical resources that may be used to support evaluations of storm drainage systems.

The current version of the FHWA Hydraulic Toolbox includes twelve calculators for evaluating systems typical to hydraulic design for highway applications including curb and gutter analysis, Rational Method analysis, detention basin analysis, median/ditch drop-inlet analysis, and horizontal grade inlet analysis. VTrans recommends that users verify their results with manual calculations and engineering judgment to validate the performance of the calculator.

Proprietary software is available to aid with the design of storm drainage systems. Such software can help with capacity and hydraulic grade line (HGL) calculations.

8.2 *General Guidelines*

8.2.1 Surface Channels

Surface channels are used to intercept runoff and conduct it to an adequate outfall. They should have adequate capacity for the design runoff and should be located and shaped in a manner that does not present a traffic hazard.

Where permitted by the design velocities, channels should have a vegetative lining. Channel linings shall be used where vegetation will not control erosion. See Chapter 5 *"Open Channels"* for more guidance.

8.2.2 Curbs, Inlets, Chutes

Curbs, inlets, and chutes or flumes are used to prohibit pavement runoff from eroding fill slopes and/or to reduce the right-of-way needed for shoulders, channels, etc. Pavement sections are usually curbed at locations where a storm drainage system is necessary.

8.2.3 Drainage Inlets

Drainage inlets in urban settings are sized and located to limit the spread on traffic lanes to tolerable widths for the design storm event. Because grates may become blocked by debris, curb openings—or combination inlets with both grate and curb openings—are advantageous for some locations. Grate inlets and depressions of curb-opening inlets should be located outside the through-traffic lanes to minimize the shifting of vehicles attempting to avoid them. Inlet grates should safely accommodate bicycle and pedestrian traffic where appropriate.

Inlets should be selected, sized, and located to prevent silt and debris carried in suspension from being deposited on the traveled way where the longitudinal gradient is decreased.

Inlets at vertical curve sags in the roadway grade should also be capable of limiting the spread to tolerable limits. The width of water spread in sags should not be substantially greater than the width of spread encountered on continuous grades. At high discharges, this can only be accomplished if inlets are included just upstream of the sag inlet on either or both sides of the sag. These additional inlets, often referred to as flanking inlets, pick up the runoff before it reaches the sag and limit the spread in the event that the sag inlet is clogged by debris. For locations where adjacent property could be damaged by runoff overtopping the curb in a sag, flanking inlets should be used and the location should be checked to insure that the curb is not overtopped due to insufficient inlet capacity.

Inlets should be located so that concentrated flow and heavy sheet flow will not cross traffic lanes. Where pavement surfaces are warped, such as at cross streets or ramps, surface water should be intercepted just before the change in cross slope. Also, inlets should be located just upgrade of pedestrian crossings.

Inlets should be placed upstream of locations where the pavement cross slope reverses, such as on the high side of superelevated horizontal curves, to prevent concentrated flows from crossing the roadway.

Special care should be given to inlet placement to ensure adequate capacity at bridge approaches and at sag vertical curves where ponding deeper than the curb height could occur.

8.2.4 Storm Drainage Piping

Storm drain pipes are used to convey water from inlets to an acceptable outlet. Cross drain pipes "hydraulically designed" to function as a culvert are exceptions to that statement. Storm drain pipes should have adequate capacity to accommodate runoff that enters the system.

The storm drainage system for sag vertical curves should have a higher level of flood protection to decrease the depth of ponding on the roadway and bridges.

Storm drainage systems should be designed to protect the roadway from flooding at the appropriate return period. Reserve capacity should be available at critical locations such as vertical curve sags and at bridge approaches. Where feasible, storm drainage systems should be designed to avoid existing utilities. Erosion control measures at storm drain outfalls should be designed similar to those required at culvert outlets. See Chapter 6 *"Crossing Structures."*

8.2.5 Costs

The cost of drainage is neither incidental nor minor on most roads. Careful attention to requirements for adequate drainage and protection of the highway from stormwater in all phases of planning and design will prove to be effective in reducing costs in both construction and maintenance. Unless drainage is properly accommodated, maintenance costs will be unduly high. However, the designer should also keep in mind that unnecessary infrastructure adds maintenance costs. The life-cycle cost of sheet flow is much cheaper than closed infrastructure if such a design can be stably implemented.

The designer should limit direct discharges to surface waters and implement Green Stormwater Infrastructure (GSI) wherever possible. Otherwise, future retrofits may be required to employ practices that promote infiltration and slow peak discharges.

8.2.6 Bridge Drainage

Short, continuous span bridges, particularly overpasses, may be built without inlets. The water from the bridge deck should be carried downslope by open or closed chutes near the end of the bridge structure. Some type of bridge end drainage should be provided at all bridges.

Longitudinal drainage on long bridges should be provided by scuppers or inlets that should be of sufficient size and number to drain the gutters adequately. Downspouts, where required, should be made of rigid corrosion-resistant material not less than 6 inches in diameter.

Deck drainage systems should not discharge stormwater onto any portion of the structure or onto moving traffic below, and they should have measures in place to prevent erosion at the outlet of the downspout. Deck drainage may be connected to conduits leading to stormwater outfalls at ground level. Overhanging portions of concrete decks should be provided with a drip bead or notch. Water in a roadway gutter section should be intercepted prior to the bridge. For more information, refer to HEC-21, *"Design of Bridge Deck Drainage."*

8.3 Design Concepts

8.3.1 Introduction

The primary aim of storm drainage design is to limit the amount of water flowing along the gutters and ponding at sags to quantities that will not interfere with the passage of traffic during the design event. This aim is accomplished by placing inlets to intercept flows and control spread or by promoting sheet flow over vegetated surfaces to avoid the use of subsurface collection systems.

The most serious effects of an inadequate roadway drainage system are:

- Risk and delay to traffic caused by excessive ponding in sags or excessive spread along the roadway
- Damage to surrounding or adjacent property, resulting from water overflowing the roadway curbs and entering such property
- Weakening of base and subgrade due to saturation from frequent ponding of long duration

8.3.2 System Planning

8.3.2.1 Introduction

The proper design of any storm drainage system involves the accumulation of certain basic data, familiarity with the project site, and a basic understanding of the hydrologic and hydraulic principles and drainage policy associated with that design. The general order in which the design should be conducted is outlined below:

1. Collect the required data.
2. Coordinate with other agencies and account for nearby projects.
3. Evaluate whether a closed drainage system is likely to be needed and whether GSI can be incorporated.
4. Develop a preliminary layout of the project with respect to the surrounding area.
    - Locate the primary outfall(s).
    - Determine the flow direction.
    - Delineate contributing drainage areas.
    - Select and evaluate the proposed inlet type, spacing, and capacity.
    - Survey the locations of existing storm drainage features and other existing utilities.
5. Size the drain pipes.
6. Perform an HGL analysis.
7. Prepare the design plans.
8. Document the design.

8.3.2.2 Required Data

The designer should be familiar with the land use patterns, the nature of the physical development of the area(s) to be served by the storm drainage system, and the ultimate pattern of drainage (both overland and by pipe) to some existing outfall location. Furthermore, there should be an understanding of the nature of the outfall because it usually has a significant influence on the storm drainage system.

Actual surveys of these and other features are the most reliable means of gathering the required data. Photogrammetric mapping has become one of the most important methods of obtaining the large amounts of data required for drainage design, particularly for busy urban roadways with all the attendant urban development. Existing topographic maps (available from the U. S. Geological Survey, the Natural Resources Conservation Service (NRCS), some local governments, and even private developers) are also valuable sources of the kind of data needed for a proper storm drainage design.

Developers and governmental planning agencies should be consulted regarding plans for the area in question. Often, in rapidly growing urban areas, the physical characteristics of an area to be served by a storm drainage system may change drastically in a very short time. In such cases, the designer should anticipate these changes and consider them in the storm drainage design.

8.3.2.3 Preliminary Layout

Preliminary schematics that feature the basic components of the intended design are an important early step in the design process. Such sketches should include the following, where applicable:

- Watershed areas and land use
- A street and driveway layout with respect to the project roadway
- Underground utility locations and elevations
- Logical inlet and manhole locations
- Preliminary lateral and trunk line layouts
- A clear identification of the outfall location and characteristics.

With this sketch or schematic, the designer can 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. Whenever possible, develop the preliminary layout using software that facilitates changes and updates as the design progresses.

8.3.2.4 Special Considerations

Avoid utilities and deep cuts wherever possible and give careful consideration to actual trunk line layout. Some situations may favor a trunk line on both sides of the roadway with very few laterals, while others may call for a single trunk line. Such features are usually a function of economy but may be controlled by other physical features.

Except in special circumstances, storm drainage systems should discharge to a single outfall. A system which branches, thereby distributing the discharge, should be avoided. However, the use of flow splitters are often needed to route low flow to treatment areas.

Storm drain pipes should not decrease in size in a downstream direction regardless of the available pipe gradient.

8.3.3 Hydrology

8.3.3.1 Introduction

The first factor to consider when determining the necessary capacity of a storm drainage system is the anticipated runoff. Designers often favor proprietary software to quickly calculate runoff for storm drainage system design, but the hydrology methods that the programs draw upon can also be performed manually or with the help of spreadsheet software.

Most watersheds contributing to storm drainage systems comprise areas on the order of fractions of acres up to tens of acres. Additionally, they are often in developed areas. The Rational Method and the Runoff Curve Number and Unit Hydrograph (RCN/UH) Method are both applicable to these situations. Refer to Chapter 4 *"Hydrology"* for more information about performing these methods.

8.3.3.2 Rational Method

The simplicity of the Rational Method makes it a popular choice for manual hydrology calculations.

In the design of storm drainage systems, the time of concentration should be regarded as consisting of two components:

- The time required for water to flow from the hydraulically most distant point of the drainage area to the inlet, which is known as the inlet time.
- The time required for the water to flow through the storm drain under consideration

In other words, 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. Where there is more than one source of runoff to a given point in a storm drainage system, the longest time of concentration is used to estimate the intensity. For each storm drain pipe of the drainage system, the travel time may be estimated by the relation:

$$t_c = \frac{L}{V * 60}$$

Where:

t_c = time of concentration, min

L = length of water course over which runoff must travel, ft

V = flow velocity, ft/s

Note: This velocity is based upon normal depth of flow for the design discharge in a run of pipe.

In municipal areas, a minimum time of concentration of ten minutes or greater is recommended for design.

8.3.3.3 Other Hydrologic Methods

Some situations may lend themselves to the use of some alternate hydrologic estimating methods. Refer to Chapter 4 *"Hydrology"* for a discussion of these methods.

8.3.4 Pavement Drainage

8.3.4.1 Introduction

Roadway features considered during gutter, inlet, and pavement drainage calculations include:

- Longitudinal slope
- Cross slope
- Curb and gutter sections
- Roadside and median channels
- Bridge decks

8.3.4.2 Design Frequencies

All roadway runoff should be designed to meet the following criteria:

- Roadside, median, and storm drainage systems should pass the 10% annual exceedance probability (AEP) storm event without flooding
- The HGL at inlets and access holes along the storm drain system should not interfere with the intended function of an inlet or reach an access hole cover during a 10% AEP storm event.
- Spread on the roadway should not exceed the limits shown in Table 8-1.

The resultant inlet spacing shall not exceed 250 feet and the maximum depth of flow shall be limited to 0.35 feet regardless of computed encroachment.

Table 8-1. Allowable Water Spread

Roadway Classification	Storm Event AEP (%)	Design Spread
Freeways	10%	Shoulder Only
Sag Points	2%	Shoulder Only
Principal & Minor Arterials	10%	Shoulder + 3 ft
Sag Points	10%	Shoulder Only
Collector Roads & Streets	10%	½ Driving Lane
Sag Points	10%	½ Driving Lane
Local Roads & Streets	10%	½ Driving Lane
Sag Points	10%	½ Driving Lane

8.3.4.3 Longitudinal Slope

A minimum longitudinal gradient is more important for a curbed pavement because it is susceptible to stormwater spread. Flat gradients on uncurbed pavements can also lead to a spread problem if vegetation is allowed to build up along the pavement edge.

Desirable gutter grades should not be less than 0.3% for curbed pavements and 0.2% in very flat terrain. Minimum grades can be maintained in very flat terrain by use of a sawtooth profile.

To provide adequate drainage in sag vertical curves, a minimum slope of 0.3% should be maintained within 50 feet of the level point in the curve. Special gutter profiles should be developed to maintain a minimum slope of 0.2% up to the inlet. Although ponding is not usually a problem at crest vertical curves, a similar minimum gradient should be provided to facilitate drainage on extremely flat curves.

8.3.4.4 Cross Slope

The *Vermont State Standards for the Design of Transportation Construction, Reconstruction and Rehabilitation on Freeways, Roads and Streets* outlines the standard practice.

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. The FHWA reports that cross slopes of 2% have little effect on driver effort in steering or on friction demand for vehicle stability (FHWA, 2013). Use of a cross slope steeper than 2% on pavements with a central crown line is not desirable. In areas of intense rainfall, a somewhat steeper cross slope may be necessary to facilitate drainage. In such areas, the cross slope may be increased to 2.5%.

If three or more lanes are inclined in the same direction on multi-lane pavements, wherever possible, design cross-slopes so that each successive pair of lanes outward from the crown line have an increased slope. The two lanes adjacent to the crown line should be pitched at the normal slope, and successive lane pairs, or portions thereof outward, should be increased by about 0.5–1%. Where three or more lanes are provided in each direction, the maximum pavement cross slope should be limited to 4%.

Wherever possible, provide a break in cross slope at two lanes, with three lanes being the upper limit. Although it is not widely encouraged, inside lanes can be sloped toward the median. This should not be used unless four continuous lanes or some physical constraint on the roadway elevations occurs, because inside lanes are used for high-speed traffic and the allowable water depth is lower. Median areas should not be drained across traveled lanes. A careful check should be made of designs to minimize the number and length of flat pavement sections in cross slope transition areas, and consideration should be given to increasing cross slopes in sag vertical curves, crest vertical curves, and 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.

Shoulders should generally be sloped to drain away from the pavement, except with raised, narrow medians.

8.3.4.5 Curb and Gutter

Typical practice is to place curbs at the outside edges of shoulders or parking lanes. The gutter width may be included as a part of the parking lane.

8.3.4.6 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. Due to right-of-way limitations, roadside channels cannot be used on most urban highways. They can be used in cut sections, depressed sections, and other locations where sufficient right-of-way is available and driveways or intersections are infrequent.

Curbed highway sections are relatively inefficient at conveying water, and the area tributary to the gutter section should be kept to a minimum to reduce the hazard from water on the pavement. Where practicable, the flow from major areas draining toward curbed highway pavements should be intercepted by channels.

Slope median areas and inside shoulders to a center swale to prevent drainage from the median area from running across the pavement. This is particularly important for high-speed roadways and for roadways with more than two lanes of traffic in each direction. Temporary storage in shallow medians must be carefully engineered to handle high-intensity rainfall.

8.3.4.7 Bridge Decks

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. Bridge deck construction usually requires a constant cross slope. 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 several spans to the bridge end for disposal.

Zero gradients and sag vertical curves should be avoided on bridges. The minimum desirable longitudinal slope for bridge deck drainage should be 0.2%. For locations where bridges are placed at a vertical curve and the longitudinal slope is less than 0.2%, the gutter spread should be checked to ensure a safe, reasonable design. Scuppers are the recommended method of deck drainage. They have a low initial cost and are relatively easy to maintain. However, the use of scuppers should be evaluated for site-specific concerns. Scuppers should not be located over embankments, slope pavement, slope protection, navigation channels, driving lanes, or railroad tracks. Runoff collected and transported to the end of the bridge should generally be collected by inlets and down drains. Runoff should also be handled in compliance with applicable stormwater quality regulations.

For situations where traffic under the bridge or environmental concerns prevent the use of scuppers, grated bridge drains should be used.

8.3.4.8 Median/Median Barriers

Medians are commonly used to separate opposing lanes of traffic on divided highways. Wherever possible, slope median areas and inside shoulders to a center depression to prevent drainage from the median area from running across travel lanes. If median barriers are used (particularly on horizontal curves with associated superelevations), provide relief for the water that accumulates against the barrier. This can be done with weep holes in the barrier. In order to minimize flow across traveled lanes, a more preferred method of relief is to collect the water into a subsurface system that ultimately connects with the main storm drainage system.

8.3.4.9 Hydroplaning

The potential for hydroplaning can be evaluated using an empirical equation based on studies conducted for the FHWA. Estimates for the vehicle speed at which hydroplaning occurs can be derived from HEC-21, *"Design of Bridge Deck Drainage."* For additional information, the University of South Florida has conducted an in-depth analysis of previous hydroplaning studies and the relationship between contributing factors in the 2012 report, *"Hydroplaning on Multi Lane Facilities."*

The following considerations help reduce the potential for hydroplaning problems:

- A permeable surface course or a high macrotexture surface course appears to have the highest potential for reducing hydroplaning accidents. This has been accomplished using friction courses.
- Pavement cross slope is the dominant factor in removing water from the pavement surface. A minimum cross slope of 2% is recommended.
- As a guideline, a wheel path depression of 0.2 inches is the threshold to indicate the potential for pavement drainage problems where dense asphaltic concrete or Portland cement concrete pavements are used. The potential for hydroplaning is greater from wheel path depressions than from sheet flow depth. This is also true for most multilane roadways. Surface drains located parallel to the lane lines will probably not solve potential drainage problems caused by the creation of wheel path depressions.
- Do not use transverse surface drains located on the pavement surface.
- Grooving may be a corrective measure for severe localized hydroplaning problems.

8.3.4.10 Pavement Texture

The pavement texture is an important consideration for roadway surface drainage. Hydroplaning may be forestalled to some extent if the pavement is of a rough texture. Grooving the pavement can encourage the removal of small amounts of water, such as in a light drizzle. A very rough pavement texture is beneficial to inlet interception but has the negative effect of causing a wider spread of water in the gutter.

8.3.5 Gutter Flow

8.3.5.1 Introduction

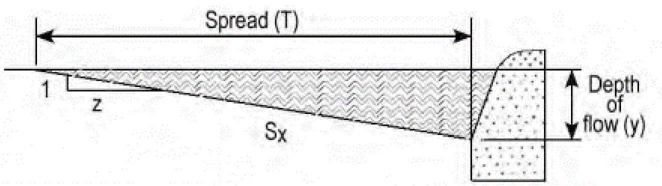
Gutters are used to convey stormwater from the edge of the pavement toward drainage system inlets. They operate according to principles of open channel hydraulics and should be appropriately sized to handle the design flows.

The FHWA Hydraulic Toolbox, discussed in Section 8.1.3, provides a calculator for curb and gutter analysis. The calculator is capable of evaluating gutters with uniform cross slopes or with composite cross slopes.

8.3.5.2 Uniform Cross Slope

Gutters with uniform cross slopes typically maintain the same cross slopes as the adjacent travel lane(s), as shown in Figure 8-1.

Figure 8-1. Uniform Gutter Cross Section



Cross Slope, $S_x = \frac{1}{z}$

(Virginia Department of Transportation (VDOT), 2002)

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$Q = \left(\frac{0.56}{n}\right) S_x^{\frac{5}{3}} S^{\frac{1}{2}} T^{\frac{8}{3}}$$

Where:

Q = total gutter flow rate, cfs

n = Manning's roughness coefficient (Table 8-2)

S_x = pavement cross slope, ft/ft

S = longitudinal gutter slope, ft/ft

T = total width of gutter flow or spread, ft

The equation can be rearranged and used to calculate spread such that:

$$T = 1.243(Qn)^{\frac{3}{8}} S_x^{-\frac{5}{8}} S^{-\frac{3}{16}}$$

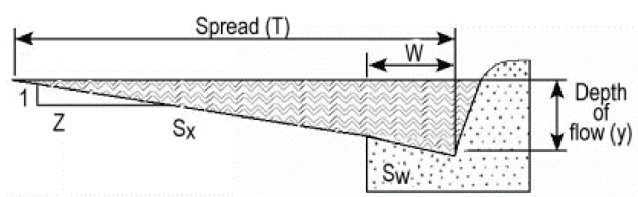
Table 8-2. Manning's n Values for Street and Pavement Gutters

Type of Gutter or Pavement	Range of Manning's n
Concrete gutter, troweled finish	0.012
Asphalt pavement	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with 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 the above values of n by:	0.002

8.3.5.3 Composite Gutter Sections

Gutters with composite cross slopes are often configured with a gutter section that is more depressed and thus has a steeper cross slope than the adjacent travel lane(s), as shown in Figure 8-2. The process of manually evaluating the hydraulics of a composite gutter section is iterative based on the known input parameters. Detailed steps are available in Section 8.4.2.

Figure 8-2. Composite Gutter Cross Section



(VDOT, 2002)

8.3.6 Drainage Inlets

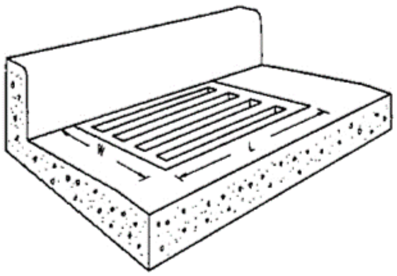
8.3.6.1 Introduction

Inlets are drainage structures that collect surface water through grate or curb openings and convey it to storm drainage pipes or directly outlet to culverts. Grate inlets subject to traffic should be bicycle and pedestrian safe and be adequately load bearing. Appropriate frames should be provided. The FHWA Hydraulic Toolbox, discussed in Section 8.1.3, provides a few calculators that aid with inlet capacity calculations.

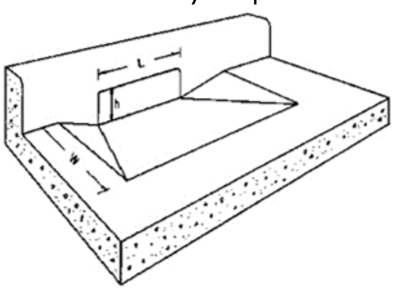
8.3.6.2 Types

Inlets used for the drainage of highway surfaces can be divided into four major classes. These classes are:

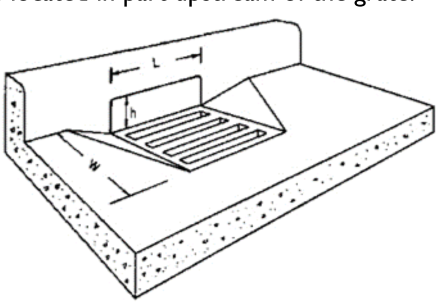
1. *Grate Inlets.* These inlets include grate inlets consisting of an opening in the gutter covered by one or more grates.



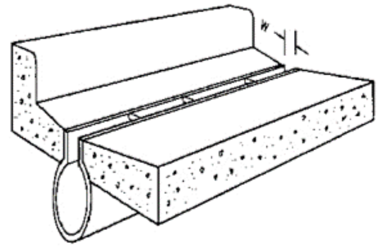
2. *Curb-Opening Inlets.* These inlets are vertical openings in the curb covered by a top slab



3. *Combination Inlets.* These inlets usually consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part upstream of the grate.



4. *Slotted Drain.* These inlets consist of a slotted opening with bars perpendicular to the opening. Slotted inlets function in essentially the same manner as curb opening inlets—that is, as weirs with flow entering from the side.

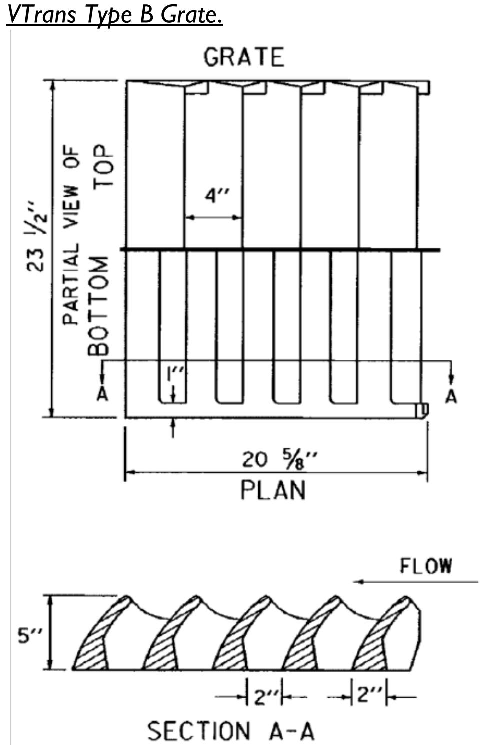


For locations where significant ponding can occur, such as underpasses and in sag vertical curves, it is good engineering practice to place flanking inlets on each side of the inlet at the low point in the sag. The flanking inlets should be placed so that they will limit spread on low gradient approaches to the level point and act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded.

8.3.6.3 VTrans Standard Grates

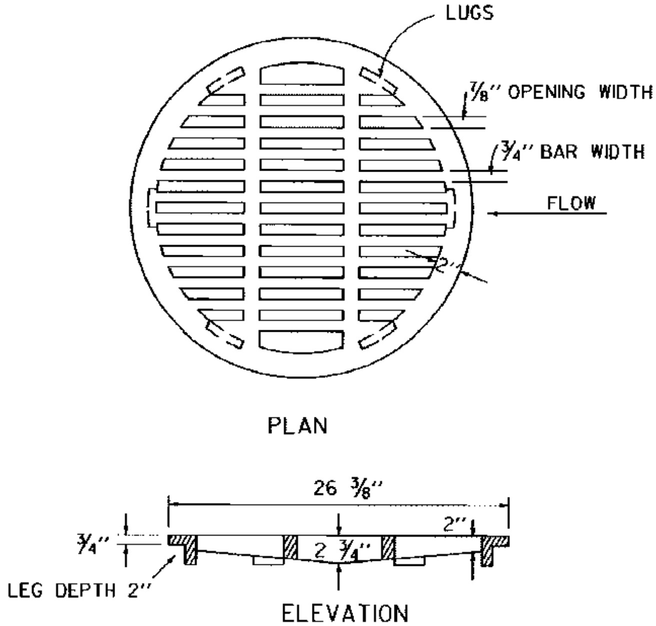
VTrans has four standard cast iron grates that are typically specified for projects. The details for VTrans Standard Grate Types B, C, D, and E are available for download and presented in the list below:

1. *VTrans Type B Grate.*



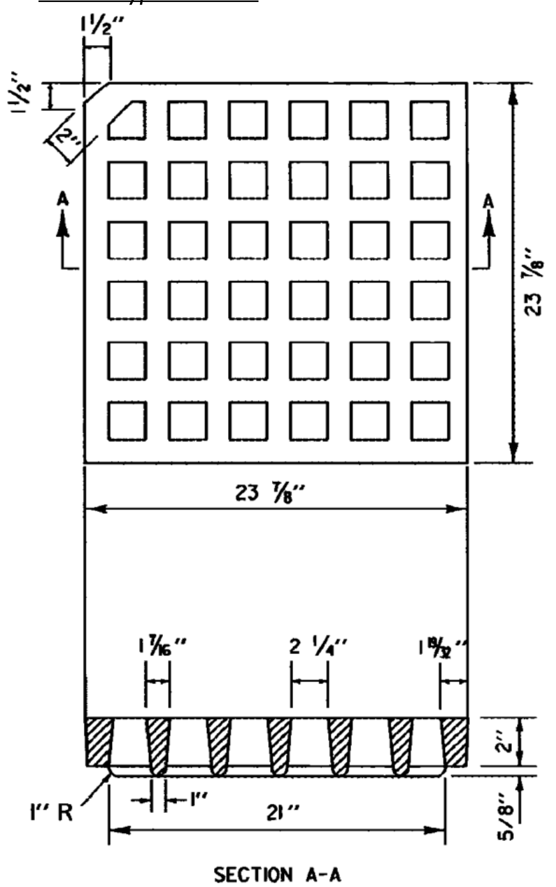
CAST IRON GRATE, TYPE B

2. *VTrans Type C Grate.*



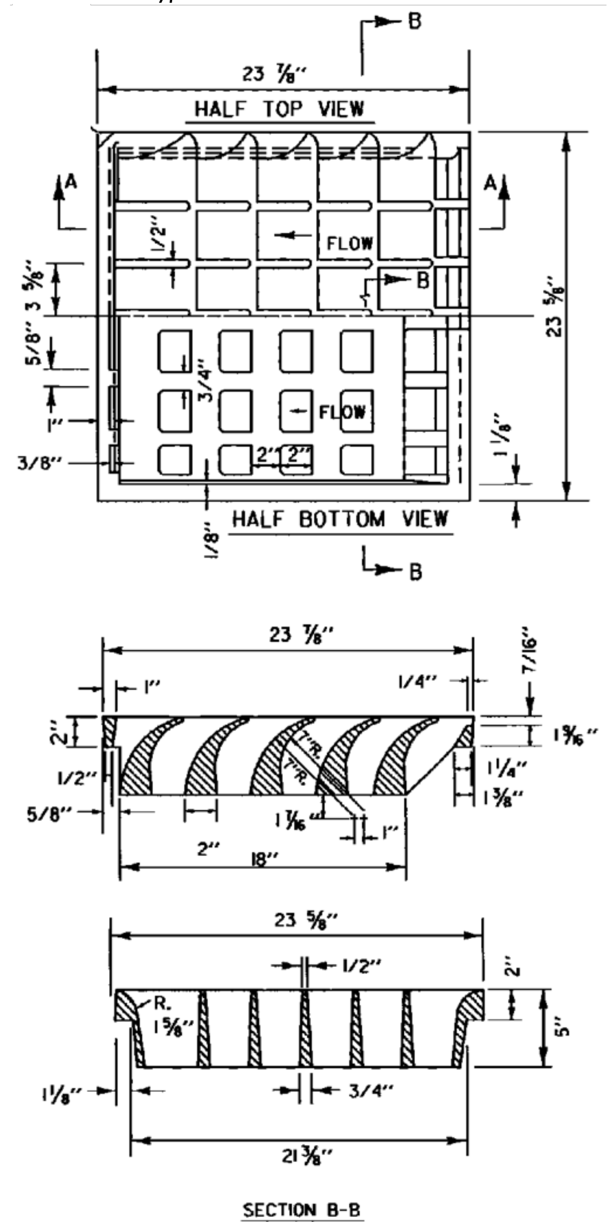
CAST IRON GRATE, TYPE C
TO BE USED IN CONJUNCTION WITH 24" CORRUGATED STEEL PIPE

3. *VTrans Type D Grate.*



CAST IRON GRATE, TYPE D

4. *VTrans Type E Grate.*



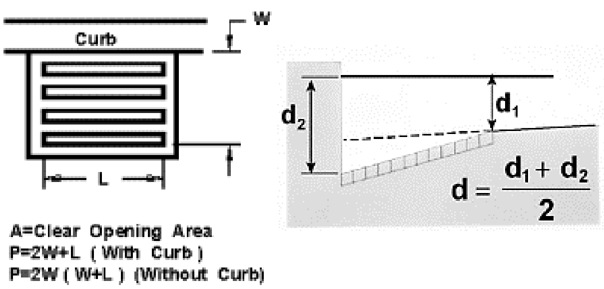
CAST IRON GRATE, TYPE E

8.3.6.4 Grate Inlets

Grate Inlets on Grade:
The capacity of an inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow, and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb-opening inlets. Figure 8-3 depicts the variables used for referring to grate inlet dimensions.

At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs (vane grates in many situations are efficient). For grates less than 2 feet long, intercepted flow is small.

Figure 8-3. Grate Inlet Dimension Variables



Inlet interception capacity has been investigated by agencies and manufacturers of grates. For inlet efficiency data for various sizes and shapes of grates, refer to FHWA's HEC-22, *"Urban Drainage Design Manual,"* and inlet grate capacity charts prepared by the grate manufacturers. For the designer's convenience, VTrans has included some of the necessary charts and nomographs within this chapter of the manual.

A parallel bar grate is the most efficient type of gutter inlet; however adding crossbars for bicycle safety greatly reduces the efficiency. If bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features.

For locations where debris is a problem, refer to laboratory test results that rank the efficiencies of grate inlets. Table 8-3, taken from HEC-22, presents the laboratory-tested debris handling efficiencies of several grates.

Table 8-3. Grate Debris Handling Efficiencies

Rank	Type of Grate	Longitudinal Slope	
		(0.005)	(0.04)
1	Curved Vane	46	61
2	30° - 85 Tilt Bar	44	55
3	45° - 85 Tilt Bar	43	48
4	P – 50	32	32
5	P – 50x100	18	28
6	45° - 60 Tilt Bar	16	23
7	Reticuline	12	16
8	P – 30	9	20

(Originally from FHWA HEC-22, 2013)

The ratio of frontal flow to total gutter flow for a uniform cross slope is expressed by the following equation:

$$E_o = \frac{Q_w}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67}$$

Where:

E_o = ratio of frontal flow to total gutter flow
Q = total gutter flow rate, cfs
Q_w = gutter flow rate above depressed gutter, W, cfs
W = width of depressed gutter or grate, ft
T = total width of gutter flow or spread, ft

Figure 8-4 provides a graphical solution of E_o for either uniform cross slopes or composite cross slopes.

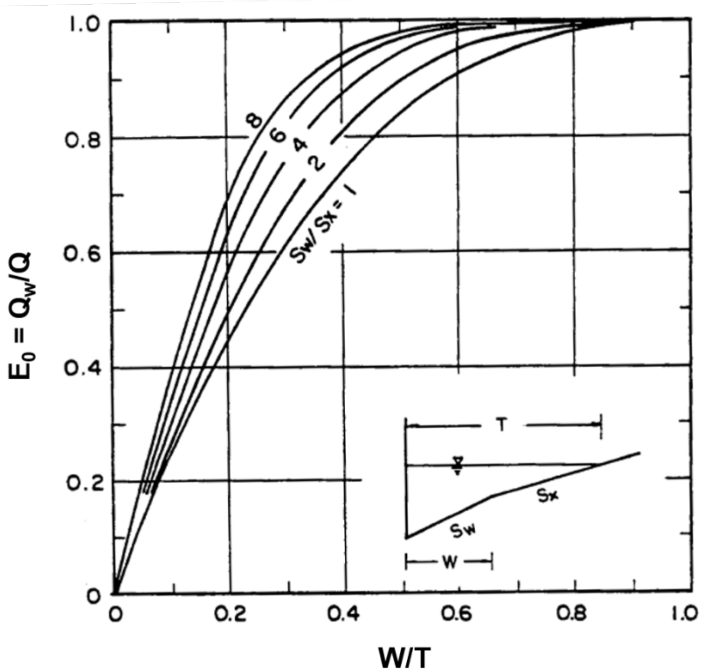
The ratio of side flow to total gutter flow is given by:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o$$

Where:

Q_s = gutter side flow rate, cfs
Q = total gutter flow rate, cfs
Q_w = gutter flow rate above depressed gutter, W, cfs
E_o = ratio of frontal flow to total gutter flow

Figure 8-4. Ratio of Frontal Flow to Total Gutter Flow



The ratio of intercepted frontal flow to total frontal flow is expressed by the following equation:

$$R_f = 1 - 0.09(V - V_o)$$

Where:

R_f = ratio of intercepted frontal flow to total frontal flow (Note: R_f cannot exceed 1.0)

V_o = gutter velocity where splash-over first occurs, ft/s

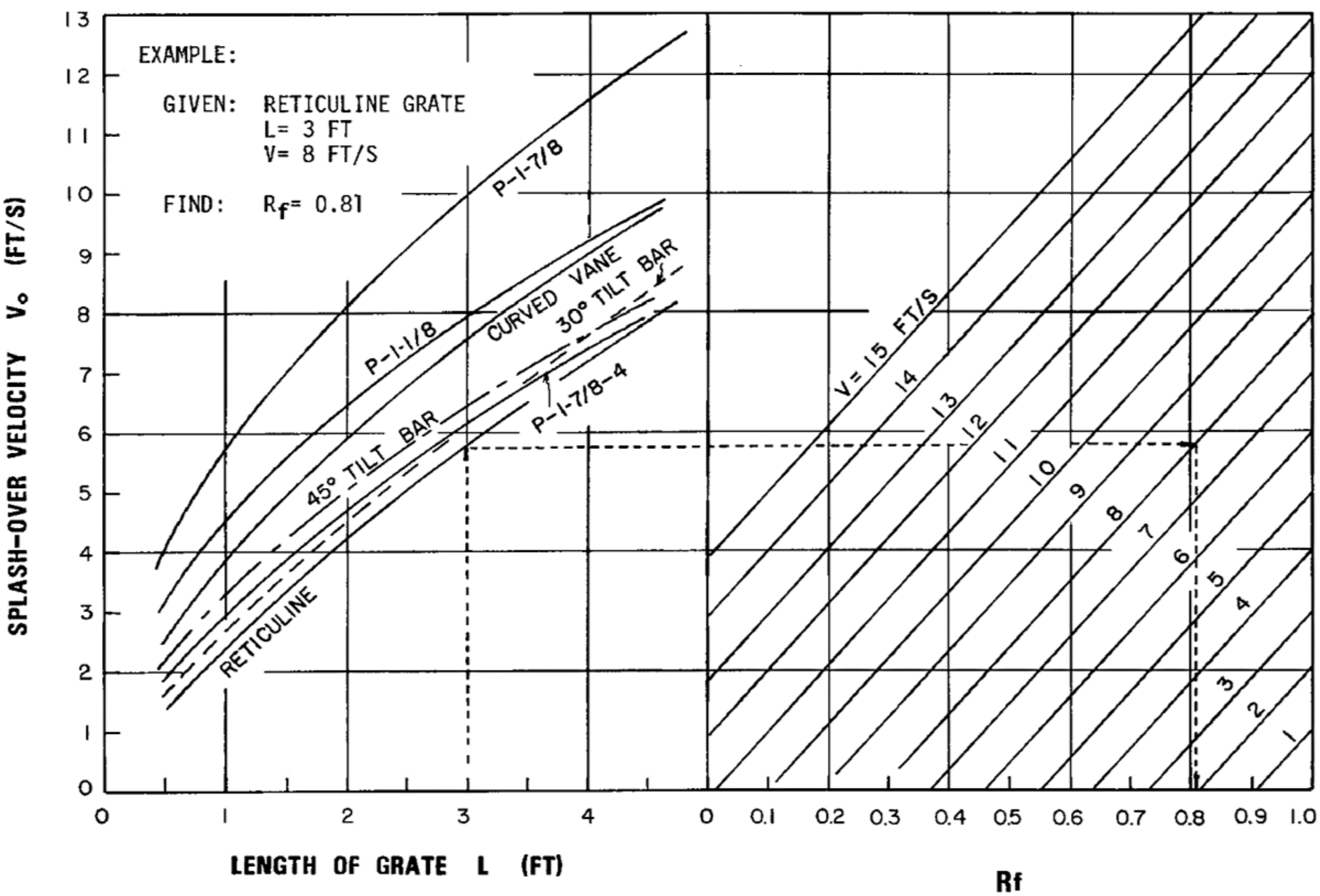
V = velocity of flow in the gutter, ft/s (= Q/A)

Q = total gutter flow rate, cfs

A = cross sectional area of gutter, ft^2

This ratio is equivalent to frontal flow interception efficiency. Figure 8-5 provides a solution for the equation that takes into account grate length, bar configuration, and gutter velocity at which splash-over occurs.

Figure 8-5. Grate Inlet Frontal Flow Interception Efficiency



The ratio of intercepted side flow to total side flow is expressed by:

$$R_s = \left(\frac{1}{\left[1 + \left(\frac{0.15V^{1.8}}{S_x L^{2.3}}\right)\right]} \right)$$

Where:
R_s = ratio of intercepted side flow to total side flow (Note: R_s cannot exceed 1.0)
L = length of the grate, ft
S_x = pavement cross slope, ft/ft
V = velocity of flow in the gutter, ft/s (= Q/A)
Q = total gutter flow rate, cfs
A = cross sectional area of gutter, ft^2

The efficiency of a grate is expressed as:

$$E = R_f E_o + R_s(1 - E_o)$$

Where:
E = grate inlet efficiency
R_f = ratio of intercepted frontal flow to total frontal flow (Note: R_f cannot exceed 1.0)
E_o = ratio of frontal flow to total gutter flow
R_s = ratio of intercepted side flow to total side flow (Note: R_s cannot exceed 1.0)

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q\left[R_f E_o + R_s(1 - E_o)\right]$$

Where:
Q_i = flow interception capacity, cfs
E = grate inlet efficiency
Q = total gutter flow rate, cfs

Grate Inlets in Sag

A grate inlet in a sag can operate as a weir or as an orifice depending on the depth of flow over the opening. For standard gutter inlet grates, weir operation continues to a depth of about 0.4 feet above the top of grate. When the depth of water exceeds about 1.4 feet, the grate begins to operate as an orifice. Between depths of about 0.4 feet and about 1.4 feet, a transition from weir to orifice flow occurs.

The capacity of a grate inlet operating as a weir is:

$$Q_i = 3Pd^{1.5}$$

Where:
Q_i = flow interception capacity, cfs
C = 3.0 (weir coefficient)
P = perimeter of grate excluding bar widths and the side against the curb, ft
d = average depth of the water above grate, ft

The capacity of a grate inlet operating as an orifice is:

$$Q_i = CA(2gd)^{0.5}$$

Where:
Q_i = flow interception capacity, cfs
C = 0.67 (orifice coefficient)
A = clear opening area of the grate, ft^2
g = acceleration due to gravity, 32.2 ft/s^2
d = average depth of the water above grate, ft

As depicted in Figure 8-6, grate size influences the depth at which a grate transitions from weir flow to orifice flow. Grate capacity during the transitional period can be approximated by drawing in a curve between the lines representing the perimeter and the net area of the grate.

8.3.6.5 Curb Inlets

Curb Inlets on Grade

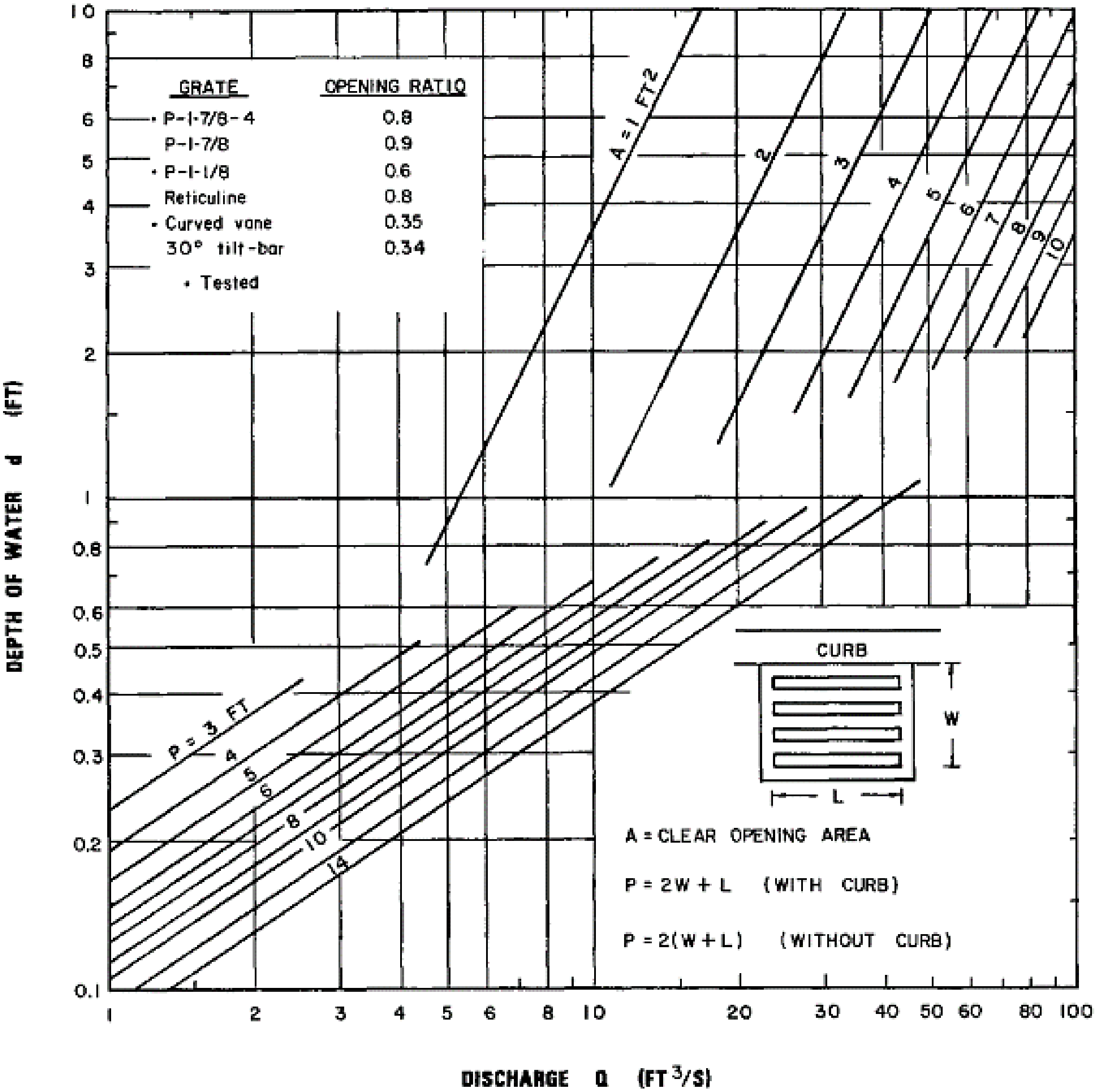
Curb-opening inlets are effective at draining highway pavements when flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length that a curb-opening inlet must be in order to completely intercept gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = C\, Q^{0.42} S^{0.3} \left(\frac{1}{nS_x}\right)^{0.6}$$

Where:
L_T = curb opening length required to intercept 100% of the gutter flow, ft
C = 0.6 (coefficient)

Figure 8-6. Grate Inlet Capacity in Sump Conditions



Q = total gutter flow rate, cfs

S = longitudinal gutter slope, ft/ft

n = Manning's roughness coefficient (Table 8-2)

S_x = pavement cross slope, ft/ft

The efficiency of curb-opening inlets that are shorter than the length required for total interception is expressed by:

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8}$$

Where:

E = curb inlet efficiency

L = curb opening length, ft

L_T = curb opening length required to intercept 100% of the gutter flow, ft

The equivalent cross slope should be used in place of S to calculate the required curb-opening length for complete flow interception if the curb-opening inlet lies in a depressed gutter section. It is given by the formula:

$$S_e = S_x + S'_w E_o$$

Where:

S_e = equivalent cross slope, ft/ft

S_x = pavement cross slope, ft/ft

S'_w = cross slope of the gutter measured from the cross slope of the pavement, ft/ft ($= a/12W$)
a = gutter depression, in
W = width of depressed gutter or grate, ft
E_o = ratio of flow in the depressed section to total gutter flow

Curb Inlets in Sag

The capacity of a curb-opening inlet in a sag depends on the water depth at the curb, the curb opening length, and the height of the curb opening. The inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transitional stage.

The equation for the interception capacity of a curb-opening inlet on a pavement section with a straight cross slope is expressed by:

$$Q_i = C\,L\,d^{1.5}$$

Where:
C = 2.3 (coefficient)
Q_i = flow interception capacity, cfs
L = curb opening length, ft
d = depth of water at curb measured from the normal cross slope gutter flow line, ft

The equation for the interception capacity of a depressed curb opening inlet operating as a weir is given by:

$$Q_i = 2.3\,(L + 1.8W)d^{1.5}$$

Where:
Q_i = flow interception capacity, cfs
L = curb opening length, ft
W = width of depression, ft
d = depth of water at curb measured from the normal cross slope gutter flow line, ft

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 times the height of the curb-opening orifice. The interception capacity can be computed by:

$$Q_i = C\,A\left[2g\left(d_l - \frac{h}{2}\right)\right]^{0.5}$$

Where:
Q_i = flow interception capacity, cfs
C = 0.67 (orifice coefficient)
A = clear area of curb opening, ft^2
g = acceleration due to gravity, 32.2 ft/s^2
d_l = depth of water at lip of curb opening, ft
h = height of curb opening orifice, ft

Figure 8-7 and Figure 8-8 represent graphical solutions for calculating curb inlet capacity. Capacity during the transitional period can be approximated by drawing in a curve between the lines representing weir flow and orifice flow.

8.3.6.6 Combination Inlet

The interception capacity of a combination inlet on a continuous grade is not appreciably greater than that of a grate alone. In computing the inlet capacity, the curb opening is neglected and only grate opening is considered. The use of a combination inlet in a sag is desirable. The curb opening provides a relief opening if the grate should become clogged. The capacity of a combination inlet in a sag is essentially the same as the grate alone in weir flow conditions unless the grate opening becomes clogged. In orifice flow, the capacity is equal to the total capacity of grate and curb opening.

8.3.6.7 Slotted Drains

Wide experience with the debris-handling capabilities of slotted drains is not available. Deposition in the pipe is the most commonly encountered problem, and the inlet is accessible for cleaning only with a high-pressure water jet. Slotted drains are effective pavement drainage inlets that have a variety of applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations.

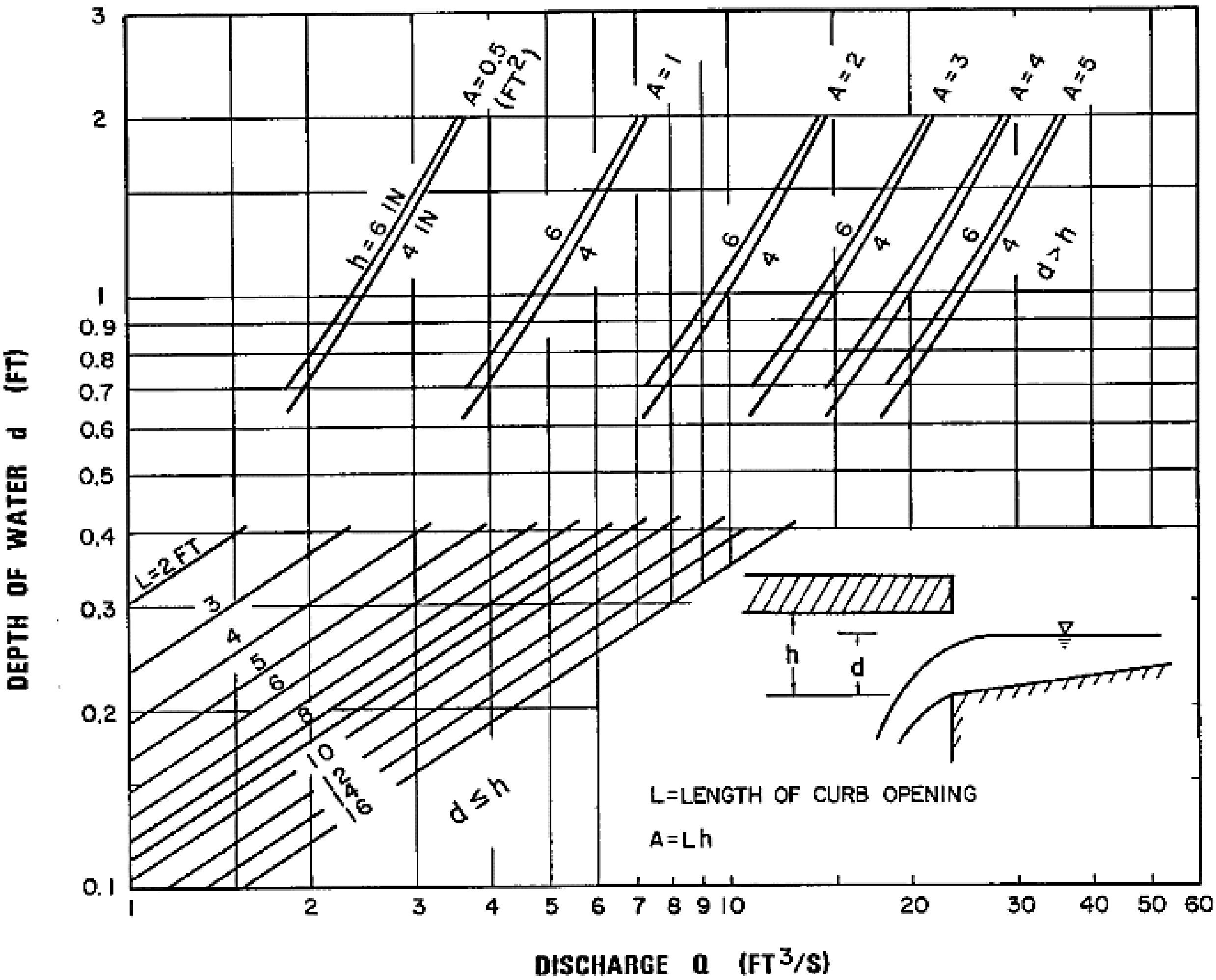
Slotted Drains on Grade

Flow interception by slotted drains and curb-opening inlets is similar in that each is a side weir and the flow accelerates laterally due to the cross slope of the pavement. Thus the equations used for curb-opening inlets on grade can be used for the design and analysis of slotted drains on grade.

Slotted Drains in Sag

Slotted drains in sag locations perform as weirs to depths of about 0.2 feet, depending on slot width and length. At depths greater than about 0.4 feet, they perform as orifices.

Figure 8-7. Undepressed Curb-opening Inlet Capacity in Sump Locations



Between these depths, flow is in a transition stage. The interception capacity of a slotted drain operating as an orifice can be computed by the following equation:

$$Q_i = 0.8\, L\, W\, (2gd)^{0.5}$$

Where:

- Q_i = flow interception capacity, cfs
- L = length of slot, ft
- W = width of slot, ft
- g = acceleration due to gravity, 32.2 ft/s^2
- d = depth of water at slot, ft

For slot widths of 1.75 inches, the above equation becomes:

$$Q_i = 0.94\, L\, d^{0.5}$$

The interception capacity of slotted drains at depths between 0.2 feet and 0.4 feet can be computed using the orifice equation. The orifice coefficient varies with depth, slot width, and the length of the slotted drain.

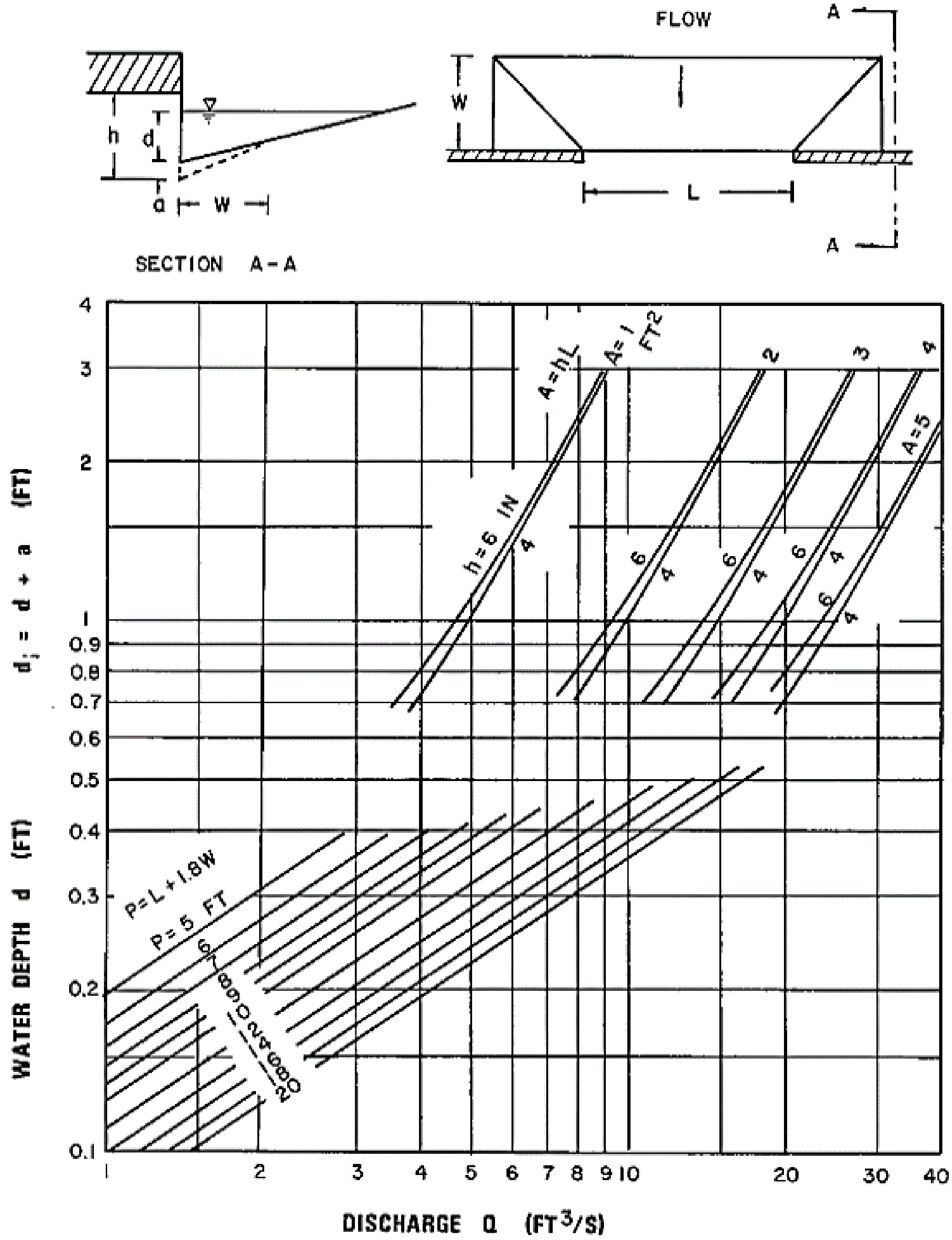
8.3.7 Storm Drainage Piping

8.3.7.1 Introduction

A closed drainage system consists of piping and/or structures that link two or more inlets and outlets in series, and the system eventually conveys runoff to a discharge point where it enters a channel or stream. A storm drainage system may include one or more closed drainage systems in conjunction with other conveyance methods such as open ditch systems.

After the preliminary layout has been developed and inlets have been selected and sized, the next step is to compute the flow that each storm drain pipe must convey and determine an appropriate pipe size and gradient to accomplish that flow conveyance.

Figure 8-8. Depressed Curb-opening Inlet Capacity in Sump Locations



Storm drain pipes should be sized by proceeding in steps from the upstream end of a pipe to the downstream end. The required flow capacity for a drain pipe is calculated, which in turn dictates the required pipe size, and the process is repeated for the next drain pipe downstream. Note that the required capacity of any particular section of drain pipe is not necessarily the sum of the inlet design discharge rates of all inlets upstream of that drain pipe. As a general rule, it is somewhat less than this total. The time of concentration is highly influential, and as the time of concentration grows larger, the proper rainfall intensity to be used in the design grows smaller.

Quick Tip
For VTrans projects, use a minimum storm drain pipe size of 12 inches.

For ordinary conditions, drain pipes should be sized based on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. Manning's Equation is recommended for capacity calculations.

8.3.7.2 Capacity

The most widely used formula for determining the hydraulic capacity of storm drainage pipes is Manning's Equation and it is expressed by the following equation:

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

Where:
V = flow velocity, ft/s
R = hydraulic radius, defined as the area of the flow divided by the wetted flow surface or wetted perimeter, ft
S = slope of the HGL, ft/ft
n = Manning's roughness coefficient

In terms of discharge, the above formula becomes:

$$Q = \frac{1.486\ A\ R^{\frac{2}{3}}\ S^{\frac{1}{2}}}{n}$$

Where:
Q = flow rate, cfs
A = cross sectional area of flow, ft^2

For storm drainage pipes flowing full, the above equations become:

$$V = \frac{0.590\left(D^{\frac{2}{3}}\right)\left(S^{\frac{1}{2}}\right)}{n} \qquad Q = \frac{0.464\left(D^{\frac{8}{3}}\right)\left(S^{\frac{1}{2}}\right)}{n}$$

Where:
D = diameter of pipe, ft

8.3.7.3 Minimum Grade

Design storm drain piping so that flow velocities will not be less than 3 feet per second at the design flow. For very flat flow lines, attempt to design the components so that flow velocities will increase progressively throughout the length of the pipe system. Check the storm drainage system to make sure there is sufficient velocity in all of the pipes to keep solids moving toward the outlet and deter particle settling.

The minimum slope required to achieve a velocity of 3 feet per second can be calculated using Table 8-4 or Manning's Equation, where the terms are as previously defined:

$$S = \left[\frac{V\ n}{1.486\ R^{\frac{2}{3}}}\right]^2$$

Table 8-4. Minimum Pipe Slopes to Achieve 3 ft/s Velocities in Drainage Pipes Flowing Full

Pipe Size	Full Pipe Flow (cfs)	Minimum Slopes (ft/ft)			
		n=0.010	n=0.012	n=0.013	n=0.024
8	1.05	0.0044	0.0064	0.0075	0.0256
10	1.64	0.0033	0.0048	0.0056	0.0190
12	2.36	0.0026	0.0037	0.0044	0.0149
15	3.68	0.0019	0.0028	0.0032	0.0111
18	5.30	0.0015	0.0022	0.0026	0.0087
21	7.22	0.0012	0.0018	0.0021	0.0071
24	9.43	0.0010	0.0015	0.0017	0.0059
27	11.93	0.00087	0.0013	0.0015	0.0051
30	14.73	0.00076	0.0011	0.0013	0.0044
33	17.82	0.00067	0.00097	0.0011	0.0039
36	21.21	0.00059	0.00086	0.0010	0.0034
42	28.86	0.00048	0.00070	0.00082	0.0028
48	37.70	0.00041	0.00059	0.00069	0.0023
54	47.71	0.00035	0.00050	0.00059	0.0020
60	58.90	0.00030	0.00044	0.00051	0.0017
66	71.27	0.00027	0.00038	0.00045	0.0015
72	84.82	0.00024	0.00034	0.00040	0.0014

8.3.7.4 Curved Alignment

Curved storm drain pipes are permitted in special cases. Example requirements are listed below.

For curved pipes 24 inches in diameter and smaller, consider the following:

- *Location*. Curved alignments should follow the general alignment of streets.
- *Curve Type.* Only simple curve design is acceptable.
- *Radius of Curvature.* The minimum allowable radius of curvature is 300 feet.
- *Manholes.* Manholes or inlets are required at the beginning and end of all curves.
- *Velocity.* The minimum velocity shall be 3 feet per second for full flow condition and the maximum velocity should not exceed 10 feet per second.

- *Joints.* Compression joints are required. The American Society for Testing and Materials (ASTM) maximum allowable deflection of the pipe joints shall not be exceeded (25% maximum).

Curved pipes larger than 24 inches in diameter shall meet the requirements given above for smaller pipes except that the joints may be manufactured so that they fit together securely without deflection at the design curvature and the radius of curvature may be less than 300 feet.

Consult the local pipe manufacturer regarding manufacturing and installation feasibility, and check the availability of proper cleaning equipment for storm drain maintenance. Many manufacturers have standardized joint configurations and deflections for specific radii.

8.3.8 Underdrains

Underdrains can refer to networks of perforated (or otherwise permeable) pipe, French drains, or collector fields. They are typically used to protect foundations, substructures, subgrades, and other highway components from groundwater. In most soils where groundwater is a problem, a system of underdrains installed for the removal of excess moisture can be a very useful feature in the overall roadway design. Where such appurtenances are needed, the cost of their installation is usually recouped in terms of future savings in roadway and structure maintenance costs.

Percolation rates for groundwater can be obtained using field measurements, NRCS data, or estimation. Collector pipe sizes and networks can then be established for the removal of that water. French drains can be very useful in applications 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 geo-textile filter material.

8.3.9 Hydraulic Grade Line

8.3.9.1 Introduction

The HGL through a drainage system must be analyzed as part of the design process in order to determine if design flows can be accommodated without causing flooding. The HGL is the sum of the pressure and elevation heads versus position along the pipe or channel. Head losses through the system must also be considered, and they occur due to both friction and form. Friction losses occur as forces act on the flow at the boundary material, and form losses (often called minor losses) occur at hydraulic structures, expansions, contractions, bends, and junctions.

These calculations can be performed manually with charts and calculators or automated using proprietary software. The following sections provide the equations for calculating head losses through a drainage system and determining the corresponding water surface elevation (WSE) needed to overcome the head losses.

8.3.9.2 Friction Head Losses

Energy losses from pipe friction may be determined by rewriting Manning's Equation. If uniform flow conditions are present, the slope of the HGL is equivalent to the friction slope, and the friction slope can be calculated as follows:

$$S_f = \left[\frac{V\,n}{1.486\,R^{\frac{2}{3}}}\right]^2 = \left[\frac{Q\,n}{1.486\,A\,R^{\frac{2}{3}}}\right]^2$$

Where:

S_f = friction slope, ft/ft
V = flow velocity, ft/s
n = Manning's roughness coefficient
R = hydraulic radius, defined as the area of the flow divided by the wetted flow surface or wetted perimeter, ft
Q = flow rate, cfs
A = cross sectional area of flow, ft^2

Then the head losses due to friction may be determined by the formula:

$$H_f = S_f L$$

Where:

H_f = friction head loss, ft
S_f = friction slope, ft/ft
L = length of outflow pipe, ft

8.3.9.3 Form (Minor) Head Losses

From the time stormwater first enters the storm drainage system at the inlet until it discharges at the outlet, it will encounter a variety of hydraulic structures such as inlets, manholes, junctions, bends, contractions, enlargements, and transitions, which will cause minor head losses. Minor head losses may be expressed in a general form derived from the Bernoulli and Darcy-Weisbach Equations:

$$H_m = \frac{K(V^2)}{2g}$$

Where:

H_m = minor head loss, ft
K = loss coefficient
V = flow velocity, ft/s
g = acceleration due to gravity, 32.2 ft/s^2

8.3.9.4 Entrance and Exit Losses

The following are the equations used for entrance and exit losses for flows. The terminal loss occurs at the pipe outlet for the entire system due to a sudden expansion. The K term is omitted from that equation because it is equal to 1.0. Entrance losses occur when flow through a structure is contracted as it enters into a pipe.

$$H_{m_t} = \frac{V^2}{2g} \qquad H_e = \frac{K(V^2)}{2g}$$

Where:

H_{m_t} = terminal (exit) head loss, ft
H_{m_e} = entrance head loss for structure, ft
K = 0.5 (assuming square-edge inlet)

8.3.9.5 Junction Losses

Incoming Opposing Flows

For a junction where two almost equal and opposing flows meet head-on with the outlet direction perpendicular to both incoming directions, the head loss is considered to be the total velocity head of the outgoing flow. The equation is given as:

$$H_{m_{j1}} = \frac{{V_{out}}^2}{2g}$$

Where:

$H_{m_{j1}}$ = junction head loss, ft
V_{out} = velocity of outgoing flow, ft/s
g = acceleration due to gravity, 32.2 ft/s^2

Changes in Direction of Flow

When main storm drain pipes or lateral lines meet at a junction, velocity is reduced within the chamber and specific head increases to develop the velocity needed in the outlet pipe. The sharper the bend (approaching 90°), the more severe this energy loss becomes. When the outlet conduit is sized, determine the velocity and compute head loss in the chamber using the formula:

$$H_{m_b} = \frac{K({V_{out}}^2)}{2g}$$

Where:

H_{m_b} = bend head loss, ft
V_{out} = velocity of outgoing flow, ft/s
K = bend/change in direction loss coefficient

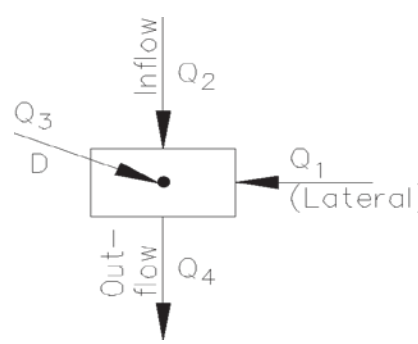
Table 8-5 lists the values of K for various bend/junction angles.

Table 8-5. K Values for Change in Flow Direction

Degree of Turn (in Junction)	K
15	0.19
30	0.35
45	0.47
60	0.56
75	0.64
90 and greater	0.70

Several Entering Flows

Computing losses in a junction with several entering flows requires the principle of conservation of energy, which involves both position energy (elevation of water surface) and momentum energy (mass times velocity head). Thus, for a junction with several entering flows, the energy content of the inflows is equal to the energy content of outflows plus additional energy required by the collision and turbulence of flows passing through the junction. Also, when two nearly equal flows enter the junction from opposing directions, head loss is considered to be the total velocity head of the outgoing flow.



The total junction losses at the sketched intersection above is as follows:

$$H_{m_{j2}} = \frac{[(Q_4V_4^2) - (Q_1V_1^2) - (Q_2V_2^2) - (KQ_1V_1^2)]}{[(2gQ_4)]}$$

Where:

$H_{m_{j2}}$ = junction head loss, ft

Q = flow rates, cfs

V = flow velocities, ft/s

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

K = bend/change in direction loss coefficient

Subscript nomenclature for the equation is as follows:

Q_1 = 90° lateral, cfs

Q_2 = straight through inflow, cfs

Q_4 = main outfall/total computed discharge, cfs

V_1, V_2, V_4 are the horizontal velocities of foregoing flows, respectively in ft/s

Also assume:

- $H_{m_b} = {K(V_1{}^2)}/{2g}$ for change in direction
- No velocity head of an incoming line is greater than the velocity head of the outgoing line.
- Water surface of inflow and outflow pipes in junction to be level.

When losses are computed for any junction condition with the same or a lesser number of inflows, the above equation will be used with zero quantities for those conditions not present. If more directions or quantities are at the junction, additional terms will be inserted with consideration given to the relative magnitudes of flow and the coefficient of velocity head for directions other than straight through.

8.3.9.6 Summary

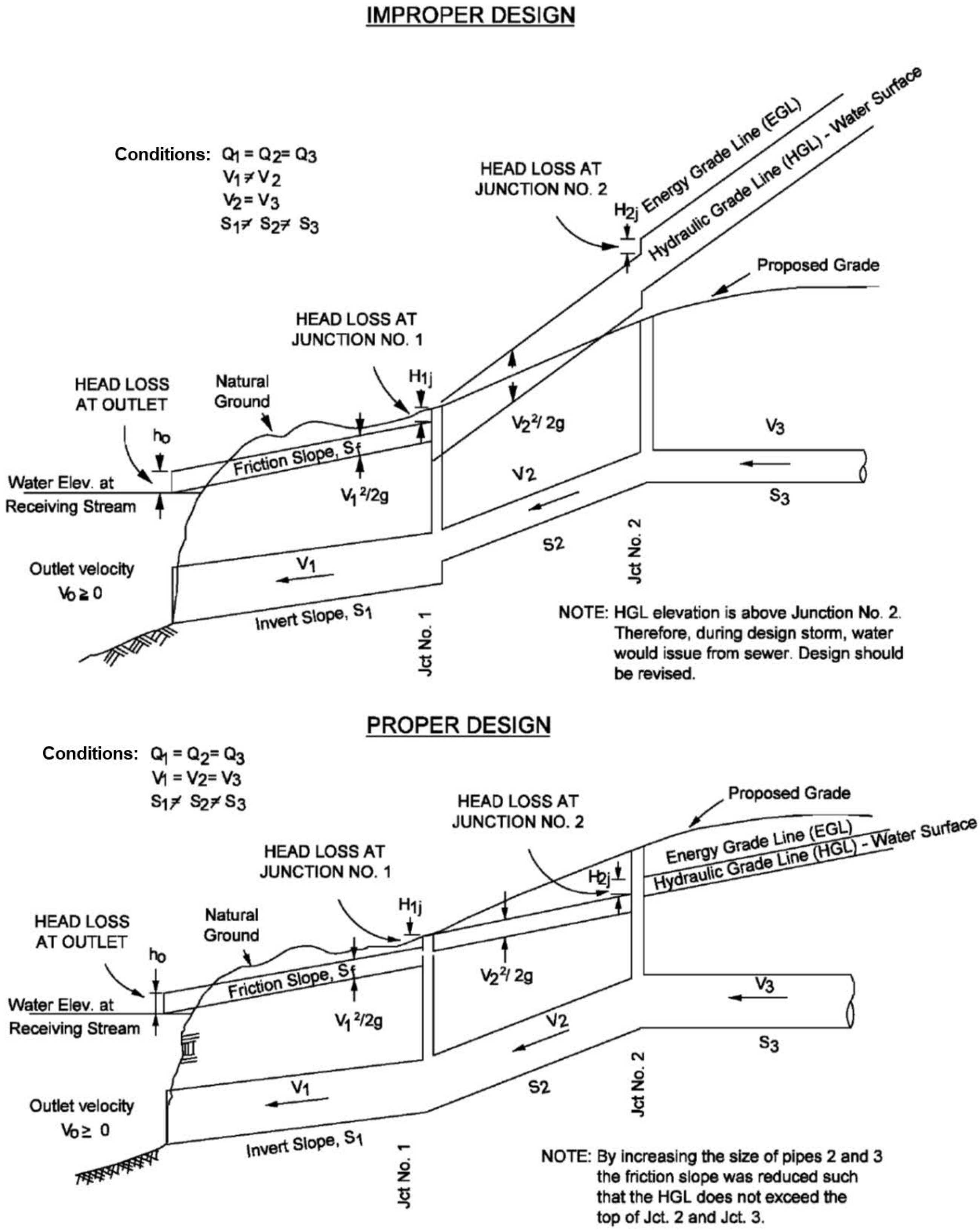
The final step in designing a storm drain system is to check the HGL as described in Section 8.4. Computing the HGL will determine the elevation to which water will rise in various inlets, manholes, junctions, etc. under design conditions.

Figure 8-9 is a sketch showing the proper and improper use of energy losses in developing a storm drain system.

8.3.10 Tailwater and Outfall Considerations

For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth. In determining the HGL, begin with the actual tailwater elevation or an elevation equal to 0.8 times the diameter of the outlet pipe (0.8D), whichever is higher.

Figure 8-9. Use of Energy Losses in Developing a Storm Drainage System



8.4 Design Procedures

8.4.1 Introduction

The design of storm drainage systems is generally divided into the following operations.

- Determine inlet location(s) and spacing.
- Prepare a plan layout of the storm drainage system establishing the following design data:
  - Location of drainage structures and pipes
  - Direction of flow
  - Location of manholes
  - Location of existing utilities such as water, gas, or underground cables
- Delineate drainage areas to inlets.
- Calculate runoff using the Rational Method.
- Compute the hydraulic capacity through the system using Manning's Equation.

8.4.2 Composite Gutter Calculation Procedures

Refer to Section 8.3.5 for background information to aid with carrying out the procedure.

The variables for this configuration are defined as follows, where:

Q = total gutter flow rate, cfs
Q_s = gutter side flow rate, cfs
Q_w = gutter flow rate above depressed gutter, W, cfs
n = Manning's roughness coefficient (see Table 8-2)
S = longitudinal gutter slope, ft/ft
S_x = pavement cross slope, ft/ft
S_w = depressed gutter cross slope, ft/ft
T = total width of gutter flow or spread, ft
T_s = spread above pavement, not including W, ft
W = width of depressed gutter or grate, ft
E_0 = ratio of frontal flow to total gutter flow

8.4.2.1 <u>Spread Calculations</u>

1. Determine the input parameters for S, S_x, S_w, W, n, Q, and a trial value of Q_s.
2. Use the following to calculate Q_w:
   $Q_w = Q - Q_s$
3. Calculate E_0 and the ratio S_w/S_x. Use Figure 8-4 to find an appropriate value for W/T. If W/T is greater than 1.0, use W/T = 1.0.
4. Calculate T using the known value for W and W/T from Step 3.
5. Use the following to calculate T_s:
   $T_s = T - W$
6. Use the following equation to calculate Q_s:
   $$Q_s = \left(\frac{0.56}{n}\right) S_x^{\frac{5}{3}} S^{\frac{1}{2}} T_s^{\frac{8}{3}}$$
7. Compare the value of Q_s from Step 6 to the trial value from Step 1. If the values are not comparable, select a new value of Q_s and return to Step 1. If the values are comparable, the calculated parameters for spread are assumed correct.

8.4.2.2 <u>Flow Calculations</u>

1. Determine the input parameters for S, S_x, S_w, n, T, W, and T_s.
2. Calculate Q_s using the following equation:
   $$Q_s = \left(\frac{0.56}{n}\right) S_x^{\frac{5}{3}} S^{\frac{1}{2}} T_s^{\frac{8}{3}}$$
3. Calculate the ratios W/T and S_w/S_x. If W/T is greater than 1.0, use W/T = 1.0. Use Figure 8-4 to find an appropriate value for E_0.
4. Calculate Q using the following equation:
   $$Q = \frac{Q_s}{1 - E_0}$$
5. Calculate Q_w if needed.

8.4.3 Preliminary Storm Drain Computation Procedure

Storm drainage systems are normally designed for full gravity flow conditions using the design event peak flow rates. The exceptions are depressed roadways and underpasses where ponded water can be removed only through the storm drainage system. In these situations, a 2% AEP storm event should be used to locate the inlets at the sag locations and to size the storm drain pipes. A minimum of three inlets at each curb 15 feet to 20 feet apart is recommended. If a storm drainage system discharges into a pumping station, the drainage system must be designed for a 2% AEP storm event under gravity flow conditions.

Storm drain pipes with free outfalls draining depressed roadways are designed by computing HGLs for pressure flow conditions to keep the WSEs below the grates and/or established critical elevations for 2% AEP flows in the system.

The design procedure should include the following:

- Storm drain design computation can be made on forms as illustrated in Figure 8-10.
- All computations and design sheets should be clearly identified. The engineer's initials and date of computations should be shown on every sheet. Voided or superseded sheets should be so marked. The origin of data used on one sheet but computed on another should be given.

8.4.4 Hydraulic Grade Line Procedure

Calculating the HGL is a critical step in the process of designing a storm drainage system. The storm drainage system should be configured and sized such that the HGL does not exceed any critical elevation during the design storm. A critical elevation is a level above which there would be unacceptable inundation of travel lanes or adjacent property. Often times, critical elevations correspond to the elevations of manhole rims or tops of inlet grates. The HGL should be computed for storm drain systems at underpasses and depressed roadways for the 2% AEP storm event.

The hydraulic control is a set WSE from which the hydraulic calculations are begun. Typically, storm drainage systems are assumed to be operating under outlet control, which means that the system is not capable of conveying as much flow as the inlet opening will accept. Under an outlet-control scenario, HGL calculations should begin at downstream control points and head losses should be calculated from downstream to upstream.

Under an inlet-control scenario, the reverse is true. HGL calculations should begin at upstream control points, and head losses should be calculated from upstream to downstream.

The head losses are calculated beginning from the control point to the first junction and the procedure is repeated for the next junction. The computation for an outlet control may be tabulated on Figure 8-11 using the following procedure:

1. In Col. 1, enter the station for the junction immediately upstream of the outflow pipe. HGL computations will begin at the outfall and continue upstream, taking each junction into consideration.
2. In Col. 2, enter whichever of the following two values is greater:
   a. The WSE at the outlet (if the outlet will be submerged during the design storm)
   b. The elevation of the outlet pipe invert out plus 0.8 times the outlet pipe diameter (D_o).
3. In Col. 3, enter the diameter of the outlet pipe (D_o).
4. In Col. 4, enter the design discharge (Q_o) for the outlet pipe.
5. In Col. 5, enter the length (L_o) of the outlet pipe.
6. In Col. 6, enter the friction slope (S_{f_o}) of the outlet pipe in ft/ft.
7. In Col. 7, calculate the friction loss (H_f) by multiplying the friction slope (S_{f_o}) in Col. 6 by the length (L_o) in Col. 5. For curved alignments, calculate the curve losses using the following formula, and add it to the friction loss.

   $$H_{m_c} = 0.002\,\Delta\,\frac{V_o^2}{2g}$$

   Where:

   H_{m_c} = curvature head loss, ft

   Δ = angle of curvature, degrees

   V_o = outlet pipe velocity, ft/s

   g = acceleration due to gravity, 32.2 ft/s^2
8. In Col. 8, enter the outlet pipe velocity of the flow (V_o).
9. In Col. 9, enter the entrance loss (H_o) that occurs as flow leaves the upstream junction and enters the outlet pipe using the formula:

   $$H_o = 0.25\,\frac{V_o^2}{2g}$$
10. In Col. 10, enter the design discharge (Q_i) for each pipe flowing into the junction. Exclude lateral pipes with inflows that are less than 10% of the mainline outflow. Inflow must be adjusted to th
11. \e mainline outflow duration time before a comparison is made.
12. In Col. 11, enter the velocity of flow (V_i) for each pipe flowing into the junction. Apply the same exception for lateral pipes described in Step 10.
13. In Col. 12, enter the product of $Q_i * V_i$ for each pipe flowing into the junction. When several pipes flow into the junction, the pipe producing the greatest $Q_i * V_i$ is the pipe that will produce the greatest expansion loss (H_i). Apply the same exception for lateral pipes described in Step 10.

14. In Col. 13, enter the controlling expansion loss (H_i), using the following formula:

    $$H_i = 0.35\ \frac{V_i^2}{2g}$$

15. In Col. 14, enter the skew angle of each pipe contributing flow to the outlet pipe. Apply the same exception for lateral pipes described in Step 10.

16. In Col. 15, enter the greatest bend loss (H_Δ) calculated using the following formula:

    $$H_\Delta = K\ \frac{V_i^2}{2g}$$

    Where:

    K = the bend loss coefficient corresponding to the various skew angles of the inflowing pipes.

17. In Col. 16, enter the total head loss (H_t) by summing the values in Col. 9, Col. 13, and Col. 15. The resulting formula is given by:

    $$H_t = H_o + H_i + H_\Delta$$

18. If the junction incorporates surface inflow (such as through a drop inlet), and this flow accounts for 10% or more of the mainline outflow, increase H_t by 30% and enter the adjusted H_t into Col. 17.

19. If the junction incorporates inlet shaping using partial pipe diameters to provide a smooth flow transition between pipes, reduce the value of H_t by 50% and enter the adjusted value into Col. 18.

20. In Col. 19, enter the final H, which is calculated as the sum of H_f and H_t, where H_t is the final adjusted value of H_t.

21. In Col. 20, enter the sum of the elevation in Col. 2 and the final H in Col. 19. This elevation is the potential WSE for the junction under the design conditions.

22. In Col. 21, enter the rim elevation or the gutter flow line, whichever is lowest, of the junction under consideration in Col. 20. If the potential WSE exceeds the rim elevation or the gutter flow line, adjustments must be made to the system to reduce the elevation of the HGL.

23. Repeat the procedure starting with Step 1 for the next junction upstream

Figure 8-10. Storm Drainage System Computation Form

COMPUTED BY	______	DATE	______	ROUTE	______
CHECKED BY	______	DATE	______	SECTION	______
PAGE	______	OF	______	COUNTY	______

STR. ID		LENGTH	DRAINAGE AREA		RUNOFF COEFF. "C"	"AREA" X "C"		TIME OF CONCENTRATION		RAIN "I"	RUNOFF "Q"	PIPE DIA.	Q FULL	VELOCITY		SEC TIME	INVERT ELEV.		CROWN DROP	SLOPE
FROM	TO	( )	INC ( )	TOTAL ( )		INC. ( )	TOTAL ( )	INLET (min)	SYSTEM (min)	(/hr)	(³/s)	( )	(³/s)	FULL (/s)	DESIGN (/s)	(min)	U/S ( )	D/S ( )	( )	(/)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)

Figure 8-11. Hydraulic Grade Line Computation Form

PROJECT:______________________________

HYDRAULIC GRADE LINE

SHEET:__________ of ________

							JUNCTION LOSS														
INLET NUMBER (1)	Outlet Water Surface Elev. (2)	D_o (3)	Q_o (4)	L_o (5)	S_{fo}' (6)	H_f (7)	V_o (8)	H_o (9)	Q_i (10)	V_i (11)	Q_iV_i (12)	$\frac{V_i^2}{2g}$	H_i (13)	Angle (14)	H_Δ (15)	H_t (16)	1.3 H_t (17)	0.5 H_t (18)	Final H (19)	Inlet Water Surface Elev. (20)	Rim Elev. (21)

Total head loss formulas

$H_i = 0.35 \frac{V_i^2}{2g}$ $H_o = 0.25 \frac{V_o^2}{2g}$ $H_\Delta = K \frac{V_i^2}{2g}$

$H_t = H_o + H_i + H_\Delta$

FINAL $H = H_f + H_t$

Loss coefficients for different bend angles

90° K = 0.70	50° K = 0.50	20° K = 0.25
80° K = 0.66	40° K = 0.43	15° K = 0.19
70° K = 0.61	30° K = 0.35	10° K = 0.13
60° K = 0.56	25° K = 0.30	5° K = 0.06

Chapter 9 Documentation

9.1 Introduction

9.1.1 Overview

Documentation is an important part of the design or analysis of any hydraulic facility. Appropriate documentation is essential because it can provide:

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

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

A projects documentation file should be created for each project to document assumptions, data collection, calculations, and design decisions for others' review and future use. In order to make each documentation file more easily navigable to others, be sure to include a narrative at the beginning of the file to act as an outline to assist users in finding detailed information. If the documentation file is stored electronically, include this narrative in the main project directory with a file name that is easily identifiable.

9.1.2 Terminology

The term "hydrologic and hydraulic documentation," as used in this chapter, refers to the compilation and preservation of a design, its related details, and all pertinent information on which the design and decisions were based. This includes drainage area(s) and other maps, field survey information, source references, photographs, computer model runs, engineering calculations and analyses, measured and other data, and flood history—including narratives from local publications and individuals such as highway maintenance personnel and local residents who witnessed or had knowledge of an unusual event.

9.1.3 Purpose

This chapter presents the documentation that should be included in the design files and on the construction plans. VTrans documentation requirements for existing and proposed drainage facilities are similar, but some differences do exist in the data that is retained. This chapter focuses on the documentation of the findings obtained by using the other chapters of this manual, so designers should already be familiar with all the hydrologic and hydraulic design procedures associated with this chapter. This chapter identifies the VTrans system for organizing the documentation of hydraulic designs and reviews so as to provide as complete a history of the design process as is practical.

Quick Tip

There is a common 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 documentation should be viewed as the record of reasonable and prudent design analysis based on the best available technology.

The major purpose of providing good documentation is to define the design procedure that was used and to show how the final design and decisions were made. Good documentation can:

- Protect VTrans by proving that reasonable and prudent actions were, in fact, taken (such proof should certainly not increase the potential court award and may decrease it by disproving any claims of negligence by the plaintiff);
- Identify the situation at the time of design, which might be very important if legal action occurs in the future;
- Document the fact that rationally accepted procedures and analysis were used at the time of the design, which were commensurate with the

perceived site importance and flood hazard (this should further disprove any negligence claims);
- Provide a continuous site history to facilitate future reconstruction;
- Provide the file data necessary to quickly evaluate any future site problems that might occur during the facility's service life; and
- Expedite plan development by clearly providing the reasons and rationale for specific design decisions.

9.1.4 Types

There are several types of documentation, including:

- Planning
- Design
- Construction
- Operation and maintenance (O&M)

9.1.4.1 Planning Documentation

Include the following as applicable:

- Aerial photographs
- Topographic data
- Watershed map or plan, including:
  - Flow directions
  - Watershed boundaries and areas
  - Natural storage areas
- Survey data, including:
  - Existing hydraulic facilities
  - Existing controls
  - Profiles—roadway, channels, driveways
  - Cross sections—roadway, channels, faces of structures
- Federal Emergency Management Agency (FEMA) Flood Insurance Studies (FISs) and Flood Insurance Rate Maps (FIRMs)
- Natural Resources Conservation Service (NRCS) soil maps
- Site visit report(s), which may include:
  - Photographs
  - Written analysis of findings with sketches
  - Data collected digitally, possibly with the use of a Global Positioning System (GPS) unit
- Reports from other agencies (local, state or federal), VTrans personnel, and abutting property owners

9.1.4.2 Design Documentation

Document all information used to justify the design, including:

- Calculations and modeling files, where applicable
- Reports from other agencies
- Hydrologic Data Form (see Chapter 4 *"Hydrology"*)
- Hydraulic report
- Approvals

Preserving and carefully documenting modeling files used for permitting and final design can be very useful after a project has been completed. Designers can more quickly develop new models of a site when a past model is available as a starting point. Deciphering past models is much easier if the creator used descriptive and consistent file naming conventions.

For example, part of what makes the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Centers River Analysis System (HEC-RAS) a powerful modeling tool is its ability to pull together different modeling files (i.e. geometry, flow) to create a single "plan" representing a set of hydraulic conditions. This feature allows the designer to run different combinations of scenarios easily and without much additional work. However, having many model files associated with a HEC-RAS project can make it difficult to keep track of what conditions each file represents. Establish a clear file naming convention for geometry, flow, and plan files so that both the designer and others can easily distinguish what is represented.

9.1.4.3 Construction Documentation

Include the following construction documentation:

- Plans (PDF submittal and electronic drafting files)
- Revisions
- As-built plans and subsurface borings
- Photographs
- Record of operation during flooding events
- Complaints and resolutions

9.1.4.4 Operation and Maintenance Documentation

Include the following O&M documentation:

- Photographs
- Record of operation during flood events
- Complaints and resolutions
- Any problems

It is very important to prepare and maintain as-built plans for every drainage structure in a permanent file. These should document subsurface foundation elements such as footing types and elevations, pile types, and (driven) tip elevations. The designer should use discretion and incorporate additional information into the documentation file if the situation warrants.

9.1.5 Chronology

Documentation should not just 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. Instead, documentation should be an ongoing process and part of each step in the hydrologic and hydraulic analysis and design process. Taking this approach will increase the accuracy of the documentation, provide data for future steps in the plan development process, and provide consistency in the design even when different designers are involved at different times of the plan development process.

Provide accurate documentation during the following steps or phases of the plan development process:

- Environmental Assessment (EA) and Environmental Impact Statement (EIS) development
- Reconnaissance phase
- Route location phase
- Survey phase (drainage surveys)
- Design phase
- Revised design phase
- Construction phase to include as-built plans
- Operational phase—documentation should be continuous over the structure's life cycle.

9.1.6 Responsibility

The designer should be responsible for determining what hydrologic analyses, hydraulic design, and related information to document during the plan development process. The designer should also decide when documentation is complete during the plan development process—up to and including the final design.

9.2 Procedure

9.2.1 Introduction

Maintain a complete hydrologic and hydraulic design and analysis (documentation file) for each waterway encroachment or crossing. Where practical, this file shall include such items as:

- Identification and location of the facility
- Photographs (ground and aerial)
- Vicinity maps and topographic maps
- Contour maps
- Hydrologic investigations
- Drainage area maps
- Interviews (local residents, adjacent property owners, and maintenance forces)
- Design notes and correspondence relating to design decisions
- History of performance of existing structure(s)
- Assumptions

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

9.2.2 Practices

The following are VTrans practices related to documentation of hydrologic and hydraulic designs and analyses:

- Compile 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—in a documentation file.
- Provide a narrative at the beginning of the documentation to act as an outline of the documentation file to assist users in finding detailed information.
- Document all design assumptions and selected criteria, including the decisions related thereto.
- Ensure that the amount of documentation detail for each design or analysis is commensurate with the risk and the importance of the facility.
- Organize documentation to be as concise and complete as practicable so that knowledgeable designers can understand years hence what was done by predecessors.
- Circumvent incriminating statements whenever possible by stating uncertainties in less than specific terms—for example, "the culvert may back water" rather than "the culvert will back water."
- Provide all related references in the documentation file to include such things as published data and reports, memos and letters, and interviews. Include dates and signatures where appropriate.
- Include data and information from the conceptual stage of project development through service life so as to provide successors with all information.
- Organize documentation to logically lead the reader from past history through the problem background, into the findings, and through the performance.
- Include the Geomorphic Factors Form (see Chapter 7 *"Channel Stability and Scour at Bridges"*) and the Final Hydraulics Report Form (see Appendix C *"Hydraulics Form"*) for each documentation file, as applicable.

9.2.3 Documentation Assembly and Storage

9.2.3.1 Hard-Copy Files

When storing hard-copies of hydraulic data, maintain uniformity and accessibility by organizing the hydraulic design folders in the following order:

1. Correspondence—from latest date backward
2. Drainage area map
3. Bridge inventory—list existing, upstream and downstream bridges, and include distance from project site
4. Hydrology data and Hydrologic Data Form, see Chapter 4 *"Hydrology"*
5. Hydraulics—preliminary sizing; summary for final structure selected, and stage-discharge plot
6. USACE and U.S. Coast Guard (USCG) computations and permit applications
7. Appendix—old bridge reports, calculations, and all appropriate computer outputs
8. Geomorphic Factors Forms (see Chapter 7 *"Channel Stability and Scour at Bridges"*)
9. Final Hydraulics Report Form (see Appendix C *"Hydraulics Form"*)

All sections should begin on separate sheets and all information, regardless of date, should be put in the appropriate section. Each sheet should be dated and initialed. Maximize the use of tables and diagrams to summarize material.

The designer should maintain the documentation files, including microfilm, microfiche, magnetic media, etc., in a secure location where they will be readily available to VTrans for use during construction, for defense of litigation, and for future replacement or extension. Only retain documentation that is not retained elsewhere. Original plans, project correspondence files, construction modifications, and inspection reports are the types of documentation that usually do not need to be duplicated.

9.2.3.2 Electronic Files

Store electronic copies of project files on a secure server. Use a logical file directory structure that mirrors the organization of the hard copy documentation folder as closely as possible. Distinguish key submittal files from intermediate design files, and keep record of file naming conventions to facilitate ease of navigation and understanding for later access.

9.3 Categorical Considerations

9.3.1 Introduction

The following lists are intended to act as guidelines for assembling categorical documentation, but they are not conclusive lists of all documents that may be included in a documentation file. Include additional items that are useful in understanding the hydrologic and hydraulic analyses, design, findings, and final recommendations.

If the designer has sized a hydraulic structure using procedures that differ from normal or if the size of the structure is governed by factors other than hydrologic or hydraulic factors, include a narrative summary detailing the design basis in the documentation file.

9.3.2 Hydrology

Include the following items, as applicable:

- Contributing watershed area size and identification of source (map name, etc.)
- Design frequency and decision for selection
- Hydrologic discharge and hydrograph estimating method and findings
- Flood frequency curves including design and 1% annual exceedance probability (AEP) flood events, discharge hydrographs, and any historical floods
- Clearly labeled input data and output results from the models/computational tools used for the selected methodologies (e.g. Peak FQ, HEC-HMS, and StreamStats—see Chapter 4 *"Hydrology"* for more information)

9.3.3 Open Channels

Include the following items, as applicable:

- Consideration of potential flood hazards to adjacent properties
- Observed high water, dates, and discharges
- Design event high water elevation for existing and proposed conditions
- Information on the method used for design high-water determination
- Cross section(s) used in the design event water surface determinations and their locations
- Roughness coefficient (Manning's n) assignments
- Stage-discharge curve for existing and proposed conditions
- Velocity calculations and locations for design and 1% AEP events

- Water surface profiles through the reach for the design and 1% AEP events
- Design or analysis of materials proposed for the channel bed and banks
- Energy dissipation calculations and designs
- Clearly labeled input data and output results from the models/computational tools used for evaluating the selected alternatives during the selected storm events (e.g. HEC-RAS and FHWA Hydraulic Toolbox—see Chapter 5 *"Open Channels"*)

9.3.4 Bridges

Include the following items, as applicable:

- Roadway geometry (plan and profile)
- Consideration of potential flood hazards to adjacent properties
- Observed high water, dates, and flow rates
- Design and 1% AEP flood event high water elevation for existing and proposed conditions
- Information on the method used for design high-water determination
- Cross section(s) used in the design high-water determination
- Roughness coefficient (Manning's n) assignments
- Stage-discharge curve for existing and proposed conditions
- Velocity calculations and locations (include both the through-bridge and channel velocities) for design and 1% AEP events
- Water surface profiles through the reach for the design event, 1% AEP event, and any historical flood events
- Magnitude and frequency of overtopping flood
- Scour evaluation showing calculated backwater elevations, velocities, and scour depths for the incipient overtopping event, the scour design event, and the scour check event (see Chapter 7 *"Channel Stability and Scour at Bridges"* for help selecting the AEPs for each event and performing the required calculations)
- Clearly labeled input data and output results from the models/computational tools used for evaluating the selected alternatives during the selected storm events (e.g. HEC-RAS and FHWA Hydraulic Toolbox—see Chapter 6 *"Crossing Structures"* and Chapter 7 *"Channel Stability and Scour at Bridges"* for more information)
- Geomorphic Factors Forms (see Chapter 7 *"Channel Stability and Scour at Bridges"*)
- Final Hydraulics Report Form (see Appendix C *"Hydraulics Form"*)

9.3.5 Culverts

Include the following items, as applicable:

- Roadway geometry (plan and profile)
- Consideration of potential flood hazard to adjacent properties
- Observed high water, dates, and flow rates
- Allowable headwater elevation and basis for its selection
- Design and 1% AEP flood event high water elevation for existing and proposed conditions
- Information on the method used for design high-water determination
- Cross section(s) used in the design high-water determinations
- Roughness coefficient (Manning's n) assignments
- Stage-discharge curve for existing and proposed conditions
- Velocity calculations and locations (include both the through-bridge and channel velocities) for design and 1% AEP events
- Water surface profiles through the reach for the design event, 1% AEP event, and any historical flood events
- If a scour evaluation was performed, include calculated backwater elevations, velocities, and scour depths for the incipient overtopping event, the scour design event, and the scour check event (see Chapter 7 *"Channel Stability and Scour at Bridges"* for help selecting the AEPs for each event and performing the required calculations)
- Type of culvert entrance condition
- Culvert outlet appurtenances and energy dissipation calculations and designs
- Clearly labeled input data and output results from the models/computational tools used for evaluating the selected alternatives during the selected storm events (e.g. HEC-RAS and HY-8—see Chapter 6 *"Crossing Structures"*)
- Geomorphic Factors Forms (see Chapter 7 *"Channel Stability and Scour at Bridges"*)
- Final Hydraulics Report Form (see Appendix C *"Hydraulics Form"*)

9.3.6 Storm Drainage Systems

Include the following items, as applicable:

- Complete drainage area map

- Design frequency
- Information concerning outfalls, existing storm drains, and other design considerations
- A schematic indicating storm drainage system layout
- Computations for inlets and pipes, including HGLs.
- Copies of the standard computation sheets given in Chapter 8 *"Storm Drainage Systems."*
- Clearly labeled input data and output results from the models/computational tools used for evaluating the system during the selected storm events (e.g. FHWA Hydraulic Toolbox —see Chapter 8 *"Storm Drainage Systems"*)

9.3.7 Associated VTrans Forms

Include the following hydraulics forms in the documentation folder when appropriate:

- *The Geomorphic Factors Form.* Fill out this form during the preliminary hydraulic study. The form is included in Chapter 7 *"Channel Stability and Scour at Bridges"*)
- *The Final Hydraulics Report Form.* Fill out this form for once the final hydraulics study is completed. The Final Hydraulics Report Form should be used to summarize the hydraulic performance of existing and proposed crossing structures with clear spans of 6 feet or greater. It can also be used for structures with clear spans that are less than 6 feet if site conditions warrant that amount of detail. All of the information from this form should be included on project plans. The Final Hydraulics Report Form is available electronically for use with spreadsheet software. Include the information from the Final Hydraulics Report Form on the project plan. The form is included in Appendix C *"Hydraulics Form."*

Appendix A Manning's n Values

Extracted from:

Federal Highway Administration (FHWA). 1961. "Design Charts for Open-Channel Flow." *Hydraulic Design Series No. 3 (HDS-3).* Publication No. FHWA-EPD-86-102. http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hds3.pdf

Table A-1. Closed Conduits

Closed Conduits	Manning's *n* Range
A. Concrete pipe	0.011 – 0.013
B. Corrugated metal pipe or pipe arch	
1. 2 ⅔ x ½-inch corrugation (riveted pipe)	
a. Plain or fully coated	0.024
b. Paved invert (range values are for 50% and 25% of circumference paved)	
i. Flow full depth	0.018 – 0.021
ii. Flow 0.8 depth	0.016 – 0.021
iii. Flow 0.6 depth	0.013 – 0.019
2. 6 x 2-inch corrugation	0.030
C. Vitrified clay pipe	0.012 – 0.014
D. Cast-iron pipe, uncoated	0.013
E. Steel pipe	0.09 – 0.011
F. Brick	0.014 – 0.017
G. Monolithic concrete	
1. Wood forms, rough	0.015 – 0.017
2. Wood forms, smooth	0.012 – 0.014
3. Steel forms	0.012 – 0.013
H. Cemented rubble masonry walls	
1. Concrete floor and top	0.017 – 0.022
2. Natural floor	0.019 – 0.025
I. Laminated treated wood	0.015 – 0.017
J. Vitrified clay liner plates	0.015

Table A-2. Open Channels, Lined (straight alignment)

Open Channels, Lined	Manning's *n* Range
A. Concrete, with surfaces as indicated:	
1. Formed, no finish	0.013 – 0.017
2. Trowel finish	0.012 – 0.014
3. Float finish	0.013 – 0.015
4. Float finish, some gravel on bottom	0.015 – 0.017
5. Gunite, good section	0.016 – 0.019
6. Gunite, wavy section	0.018 – 0.022
B. Concrete bottom, float-finished, sides as needed	
1. Dressed stone in mortar	0.015 – 0.017
2. Random stone in mortar	0.017 – 0.020
3. Cement rubble masonry	0.020 – 0.025
4. Cement rubble masonry, plastered	0.016 – 0.020
5. Dry rubble (rip-rap)	0.020 – 0.030

Open Channels, Lined	Manning's *n* Range
C. Gravel bottom, sides as indicated	
1. Formed concrete	0.017 – 0.020
2. Random stone in mortar	0.020 – 0.023
3. Dry rubble (rip-rap)	0.023 – 0.033
D. Brick	0.014 – 0.017
E. Asphalt	
1. Smooth	0.013
2. Rough	0.016
F. Wood, planed, clean	0.011 – 0.013
G. Concrete-lined excavated rock	
1. Good section	0.017 – 0.020
2. Irregular section	0.022 – 0.027

Table A-3. Open Channels, Excavated (straight alignment, natural lining)

Open Channels, Excavated	Manning's *n* Range
A. Earth, uniform section	
1. Clean, recently completed	0.016 – 0.018
2. Clean, after weathering	0.018 – 0.020
3. With short grass, few weeds	0.022 – 0.027
4. In gravelly soil, uniform section, clean	0.022 – 0.025
B. Earth, fairly uniform section	
1. No vegetation	0.022 – 0.025
2. Grass, some weeds	0.025 – 0.030
3. Dense weeds or aquatic plants in deep channels	0.030 – 0.035
4. Sides clean, gravel bottom	0.025 – 0.030
5. Sides clean, cobble bottom	0.030 – 0.040
C. Dragline excavated or dredged	
1. No vegetation	0.028 – 0.033
2. Light brush on banks	0.035 – 0.050
D. Rock	
1. Based on design section	0.035
2. Based on actual mean section	
a. Smooth and uniform	0.035 – 0.050
b. Jagged and irregular	0.040 – 0.045
E. Channels not maintained, weeds and brush uncut	
1. Dense weeds, high as flow depth	0.080 – 0.120
2. Clean bottom, brush on sides	0.050 – 0.080
3. Clean bottom, brush on sides, highest stage of flow	0.070 – 0.110
4. Dense brush, high stage	0.100 – 0.140

Table A-4. Highway Channels with Maintained Vegetation (range values are for velocities of 6 ft/s and 2 ft/s)

Highway Channels with Maintained Vegetation	Manning's *n* Range
A. Depth of flow up to 0.7 feet	
B. Bermudagrass, Kentucky bluegrass, Buffalograss	
1. Mowed to 2 inches	0.045 – 0.070
2. Length 4 to 6 inches	0.050 – 0.090
C. Good stand, any grass	
1. Length about 12 inches	0.090 – 0.180
2. Length about 24 inches	0.150 – 0.300
D. Fair stand, any grass	
1. Length about 12 inches	0.080 – 0.140
2. Length about 24 inches	0.130 – 0.250
E. Depth of flow 0.7 to 1.5 feet	
1. Bermudagrass, Kentucky bluegrass, Buffalograss	
a. Mowed to 2 inches	0.035 – 0.050
b. Length 4 to 6 inches	0.040 – 0.060
2. Good stand, any grass	
a. Length about 12 inches	0.070 – 0.120
b. Length about 24 inches	0.100 – 0.200
3. Fair stand, any grass	
a. Length about 12 inches	0.060 – 0.100
b. Length about 24 inches	0.090 – 0.170

Table A-5. Values of Manning's Roughness Coefficient *n*—Street and Expressway Gutters

Street and Expressway Gutters	Manning's *n* Range
A. Concrete gutter, troweled finish	0.012
B. Asphalt pavement	
1. Smooth	0.013
2. Rough	0.016
C. Concrete gutter with asphalt pavement	
1. Smooth	0.013
2. Rough	0.015
D. Concrete Pavement	
1. Float finish	0.014
2. Broom Finish	0.016
E. For gutters with small slope, where sediment may accumulate, increase above values of *n* by	0.002

Table A-6. Values of Manning's Roughness Coefficient *n*—Natural Stream Channels

Natural Stream Channels	Manning's *n* Range
A. Minor streams (surface width at flood stage less than 100 feet)	
1. Fairly regular section	
a. Some grass and weeds, little or no brush	0.030 – 0.035
b. Dense growth of weeds, depth of flow	0.035 – 0.050
c. Some weeds, light brush on banks	0.035 – 0.050
d. Some weeds, heavy brush on banks	0.050 – 0.070
e. Some weeds, dense willows on banks	0.060 – 0.080
f. For trees in channel with branches submerged at high stage, increase all above values by	0.010 – 0.020
g. Irregular sections with pools, slight channel meander, increase values in 1.a. – 1.e. by	0.010 – 0.020
2. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks sub-merged at high stage	
a. Bottom of gravels, cobbles and few boulders	0.040 – 0.050
b. Bottom of cobbles with large boulders	0.050 – 0.070
B. Floodplains (adjacent to natural streams)	
1. Pasture, no brush	
a. Short grass	0.030 – 0.035
b. High grass	0.035 – 0.050
2. Cultivated areas	
a. No crop	0.030 – 0.040
b. Mature row crops	0.035 – 0.045
c. Mature field crops	0.040 – 0.050
3. Heavy weeds, scattered brush	0.050 – 0.070
4. Light brush and trees	
a. Winter	0.050 – 0.060
b. Summer	0.060 – 0.080
5. Medium to dense brush	
a. Winter	0.070 – 0.110
b. Summer	0.100– 0.160
6. Dense willows, summer, not bent over by current	0.150 – 0.200
7. Cleared land with tree stumps, 100 to 150 per acre	
a. No sprouts	0.040 – 0.050
b. With heavy growth of sprouts	0.060 – 0.080
8. Heavy stand of timber, a few down trees, little undergrowth	
a. Flood depth below branches	0.100 – 0.120
b. Flood depth reaches branches	0.120 – 0.160
C. Major streams (surface width at flood stage more than 100 feet): Roughness coefficient usually less than for minor streams of similar description because less effective resistance is offered by irregular banks or vegetation on banks. Values of *n* may be somewhat reduced. Follow recommendation in publication cited, if possible. The value of *n* for larger streams of most regular section, with no boulders or brush, may be in the range of	0.028 – 0.033

Appendix B Field Investigation Forms

Hydraulics Unit Vermont Agency Of Transportation National Life Building, Fourth Floor Drawer 33 Montpelier, VT 05633-5001					
		Town:		**Field Investigation Form**	
		Highway:		Prepared by:	
		Structure:		Date:	
		Stream:		Road Classification:	
		Tributary to:		Project No.:	
Site Information:		**Stream Information:**			
Type of Structure:		OHW:		Bed Material:	
Clear Span:		OLW:		D_{50} (estimated):	
Clear Height:		Scour:		Manning's n :	
No. of Spans:		Channel Erosion:		Flow Class:	Subcritical / Supercritical
Abutment Type:		Ponding:		Stream Grade:	Flat / Moderate / Steep
Depth of Beams:		Debris:		Sinuosity:	Straight / Sinous / Meandering
Inlet Headwall:		Chan Width Up:		Tailwater:	
Outlet Headwall:		Chan Width Down:		Flood Plains:	
Stone Fill:		Terrain & Land Use:			
Upstream:					
Town:		Roadway No.:		Bridge No.:	
Type:		Clear Span:		Clear Height:	
Waterway Area:		Year Built:		Distance:	
Downstream:					
Town:		Roadway No:		Bridge No.:	
Type:		Clear Span:		Clear Height:	
Waterway Area:		Year Built:		Distance:	

NOTES:

Inlet/Upstream Cross Section: Cover:______	**Outlet/Downstream Cross Section:** Cover:______
Fill Height (Streambed to Road):__________ Relief Height:_______	Fill Height (Streambed to Road):__________

*NOTE: plan view and additional notes on the reverse side

Hydraulics Unit
Vermont Agency Of Transportation
National Life Building, Fourth Floor
Drawer 33
Montpelier, VT 05633-5001

Plan View and Additional Notes

Hydraulic Survey Field Inspection Checklist

Vermont Agency of Transportation
Montpelier, VT 05633-5001

General Project Data

1) Project Number: 2) Site Name:
3) Road Name: 4) Station:
5) Town: 6) M.P.:
7) Survey Conducted By: 8) Date Survey Received:
9) Site Inspected By: 10) Date Inspected:
11) Survey Source: ☐ Field ☐ Aerial ☐ Other:
12) Site Description: ☐ Cross drain ☐ Storm drain ☐ Long encroach. ☐ Channel Change ☐ Other:

Office Preparation for Inspection

1) Reviewed:
- Aerial photos: ☐ None available ☐ Yes, photo numbers:
- Mapping/maps: ☐ None available ☐ Yes, map numbers:
- Reports: ☐ Yes ☐ No ☐ None available at this time
- VTrans Permanent File: ☐ Yes ☐ No ☐ No file data found

2) Special requirements and problems identified for field checking:
- ☐ Hydrologic boundary — Obtain hydrologic channel geometry
- ☐ Adverse flood history — Obtain HW marks, dates, and eye witnesses
- ☐ Adverse channel stability and alignment history — Check for headcutting, bank caving, brainding, increased meander activity
- ☐ Structure scour — Check flow alignment, scour at culvert outlet, or evidence of bridge scour
- ☐ Aquatic Organism Passage Requirements — Check ecological connectivity and sediment continuity
- ☐ National Flood Insurance Program — Check published study transects and model values near site
- ☐ Check bed and bank samples at:
- ☐ Other:

Field Inspection *(The following details that were obtained at the site are annotated on the drainage survey.)*

1) Does the survey appear correct? ☐ Yes ☐ No, apparent errors are:
which were resolved by:

2) Is flooding apparent? ☐ No ☐ Yes, and HW marks obtained ☐ Yes, but HW marks not obtained because:

3) Do all floods reach site? ☐ Yes ☐ No, and details obtained ☐ Yes, but details not obtained because:

4) Has channel geometry changed? ☐ Yes ☐ No, because:

5) Is the channel unstable? ☐ No ☐ Yes, because (check all that apply):
☐ Headcutting observed and ☐ Amount/location obtained ☐ Bank caving
☐ Braiding ☐ Increased meander activity ☐ Other:

6) Evidence of structure scour? ☐ No ☐ Yes, and:
☐ Obtained bed/bank samples ☐ Bed/bank samples not obtained because:
☐ Noted flow alignment problems ☐ Flow alignment not noted because:

7) Was Manning's "n" obtained? ☐ Yes ☐ No, because:

8) Property damage due to backwater? ☐ No ☐ Yes, and elevation/property type checked
☐ Yes, but elevation/property type not obtained because:

9) Are environmental hazards present? ☐ No ☐ Yes, and details obtained ☐ Yes, but details not obtained because:

10) Were ground photos taken? ☐ No ☐ Yes (check all that apply):
☐ Channel material with scale ☐ Evidence of channel instability ☐ Upstream
☐ Existing structure inlet/outlet ☐ Evidence of scour ☐ Downstream
☐ Other:

11) Was the effective drainage area visually verified? ☐ Yes ☐ No, because:

Designer's signature Date

Appendix C Hydraulics Form

VAOT FINAL HYDRAULICS REPORT (Guidance Sheet)

Date: __________

TOWN: __________ COUNTY: __________
PROJECT # : __________ STREAM: __________
HIGHWAY # : __________ STRUCTURE # : __________

HYDROLOGIC DATA

(Refer to Chapter 4 of Hydraulics Manual)

DRAINAGE AREA : (Show sq. mi. or sq. km. to nearest tenth)
CHARACTER OF TERRAIN : (Based on field investigation observation and topoquads)
STREAM CHARACTERISTICS : (List characteristic geomorphic factors)
NATURE OF STREAMBED : (Based on field investigation observation or geology lab sample results)

PEAK FLOW DATA (BY ANNUAL EXCEEDANCE PROBABILITY)
(Rounding of units depends on size of numbers and the units used. Use judgement and show units.)

Q 43% = __________ Q 2% = __________
Q 10% = __________ Q 1% = __________
Q 4% = __________ Q 0.2% = __________

DATE OF FLOOD OF RECORD : (From local info sources, USACE studies, nearby gage station, or FIS reports)
ESTIMATED DISCHARGE: (")
WATER SURFACE ELEV.: (")
NATURAL STREAM VELOCITY : (Report at design Q. Use the highest predicted average channel velocity near the existing structure, generally taken from HEC-RAS. Pull from the six bridge cross sections or "Six XS Bridge," which include the two cross sections upstream of the crossing, the two cross sections downstream of the crossing, and the two internal cross sections. If no existing structure is present, use most appropriate cross sections in the vicinity of the proposed structure.)
ICE CONDITIONS : (i.e. Light, moderate, or heavy. Based on local information or other sources.)
DEBRIS: (")
DOES THE STREAM REACH MAXIMUM HIGHWATER ELEV. RAPIDLY? (Is it flashy?)
IS ORDINARY RISE RAPID? (Flashy? Judgment call, but sometimes discussed in the FIS or USACE reports)
IS STAGE AFFECTED BY UPSTREAM OR DOWNSTREAM CONDITIONS? (Yes or No)
IF YES, DESCRIBE: __________

WATERSHED STORAGE: (%)
HEADWATERS: (X approximately)
UNIFORM: (")
IMMEDIATELY ABOVE SITE: (")

EXISTING STRUCTURE INFORMATION

(Refer to Chapter 6 of Hydraulics Manual. VTrans bridge inspection files have useful information.)

STRUCTURE TYPE: __________
YEAR BUILT: __________
CLEAR SPAN (NORMAL TO STREAM): (Measured at narrowest constriction caused by structure)
VERTICAL CLEARANCE ABOVE STREAMBED: (Generally measured from thalweg to bridge low chord, taken as the lowest point on the bottom of the bridge deck. Also list the low chord elevation here. Round to nearest tenth.)
WATERWAY OF FULL OPENING: (Calculated in HEC-RAS as the variable "BR Open Area")
DISPOSITION OF STRUCTURE: (i.e. Removal, rehabilitation, etc.)
TYPE OF MATERIAL UNDER SUBSTRUCTURE: (Refer to borings if available)

WATER SURFACE ELEVATIONS AT:
(Report max WSE two model cross sections upstream of the existing structure. Standard practice is often to find the predicted upstream WSE at a distance equal to the bridge span, but the key is to get outside of the area contracted by the bridge, which could be more or less than one bridge span upstream.)

VELOCITY AT:
(Report the highest predicted average channel velocity near the existing structure, generally taken from HEC-RAS. Pull from the six bridge cross sections or "Six XS Bridge," which include the two cross sections upstream of the crossing, the two cross sections downstream of the crossing, and the two internal cross sections. If no existing structure is present, use most appropriate cross sections in the vicinity of the proposed structure.)

Q 43% =	(Round elev. to tenths)	"	(Round vel. to tenths)
Q 10% =		"	
Q 4% =		"	
Q 2% =		"	
Q 1% =		"	

LONG TERM STREAMBED CHANGES: (Compare surveyed channel profile to older survey or other available channel profile [FIS report, USACE study, aerial photos, etc.] and note changes/trends.)

IS THE ROADWAY OVERTOPPED BELOW Q 1%: (Yes or No)
FREQUENCY: (Use stage discharge curve to estimate overtopping return interval based on relief elev.)
RELIEF ELEVATION: ______
DISCHARGE OVER ROAD @Q 1%: ______

UPSTREAM STRUCTURE

(Town Highway maps, USGS topoquads, VTrans bridge inspection files, and field visit useful)

TOWN: ______ DISTANCE: ______
HIGHWAY # : ______ STRUCTURE #: ______
CLEAR SPAN: ______ CLEAR HEIGHT: ______
YEAR BUILT: ______ FULL WATERWAY: ______
STRUCTURE TYPE: ______

DOWNSTREAM STRUCTURE

(Town Highway maps, USGS topoquads, VTrans bridge inspection files, and field visit useful)

TOWN: ______ DISTANCE: ______
HIGHWAY # : ______ STRUCTURE #: ______
CLEAR SPAN: ______ CLEAR HEIGHT: ______
YEAR BUILT: ______ FULL WATERWAY: ______
STRUCTURE TYPE: ______

PROPOSED STRUCTURE

(Refer to Chapter 6 of Hydraulics Manual. VTrans bridge inspection files have useful information.)

STRUCTURE TYPE: (From PI sheet of plans)

CLEAR SPAN (NORMAL TO STREAM): (Measured at narrowest constriction caused by structure)
VERTICAL CLEARANCE ABOVE STREAMBED: (Generally measured from thalweg to bridge low chord, taken as the lowest point on the bottom of the bridge deck. Round to nearest tenth.)
WATERWAY OF FULL OPENING: (Calculated in HEC-RAS as the variable "BR Open Area")

WATER SURFACE ELEVATIONS AT:
(Report max WSE two model cross sections upstream of the proposed structure.)

VELOCITY AT:
(Report the highest predicted average channel velocity near the proposed structure, generally taken from HEC-RAS.)

Q 43% =	(Round elev. to tenths)	"	(Round vel. to tenths)
Q 10% =		"	
Q 4% =		"	
Q 2% =		"	
Q 1% =		"	

IS THE ROADWAY OVERTOPPED BELOW Q 1%: (Yes or No)

FREQUENCY: (Use stage discharge curve to estimate overtopping return interval based on relief elev.)

RELIEF ELEVATION:

DISCHARGE OVER ROAD @Q 1%:

BRIDGE LOW CHORD ELEVATION:

FREEBOARD: (Measured from max WSE during design storm to bridge low chord, taken as the lowest point on the bottom of the bridge deck. Round to nearest tenth.)

SCOUR: (Refer to Chapter 7 of Hydraulics Manual. Calculate in accordance with HEC-18 guidance. Report only contraction and pier scour for worst case of scour design event, scour check event, or incipient overtopping event.)

REQUIRED CHANNEL PROTECTION: (Stone Fill Type I, II, III, or IV based on velocity and engineering judgement. Consider ice, debris, and bank failure history)

PERMIT INFORMATION

(Refer to Chapter 4 of Hydraulics Manual)

		DEPTH OR ELEVATION:
AVERAGE DAILY FLOW:		
ORDINARY LOW WATER:		
ORDINARY HIGH WATER:		

TEMPORARY BRIDGE REQUIREMENTS

(Refer to Chapter 6 of Hydraulics Manual)

STRUCTURE TYPE: (i.e. Bridge, culvert, etc. Note if bridge must be removed before winter.)

CLEAR SPAN (NORMAL TO STREAM): (Minimum clear span required to adequately span the channel)

VERTICAL CLEARANCE ABOVE STREAMBED: (Generally measured from thalweg to bridge low chord, taken as the lowest point on the bottom of the bridge deck. Also list the low chord elevation here. Round to nearest tenth.)

WATERWAY AREA OF FULL OPENING:

ADDITIONAL INFORMATION

(Add any additional information here that is relevant to hydraulics of the site.)

VAOT FINAL HYDRAULICS REPORT

Date: ____________

TOWN: ______________________ COUNTY: ______________
PROJECT # : ______________________ STREAM: ______________
HIGHWAY # : ______________________ STRUCTURE # : ______________

HYDROLOGIC DATA

DRAINAGE AREA : ______________________
CHARACTER OF TERRAIN : ______________________
STREAM CHARACTERISTICS : ______________________
NATURE OF STREAMBED : ______________________

PEAK FLOW DATA

Q 43% = ______________ Q 2% = ______________
Q 10% = ______________ Q 1% = ______________
Q 4% = ______________ Q 0.2% = ______________

DATE OF FLOOD OF RECORD : ______________________
ESTIMATED DISCHARGE: ______________________
WATER SURFACE ELEV.: ______________________
NATURAL STREAM VELOCITY : @ Q?? = ______________________
ICE CONDITIONS : ______________________
DEBRIS: ______________________
DOES THE STREAM REACH MAXIMUM HIGHWATER ELEV. RAPIDLY? ______________
IS ORDINARY RISE RAPID? ______________________
IS STAGE AFFECTED BY UPSTREAM OR DOWNSTREAM CONDITIONS? ______________
IF YES, DESCRIBE: ______________________

____

WATERSHED STORAGE: ______________ HEADWATERS: ______________
UNIFORM: ______________
IMMEDIATELY ABOVE SITE: ______________

EXISTING STRUCTURE INFORMATION

STRUCTURE TYPE: ______________________
YEAR BUILT: ______________________
CLEAR SPAN (NORMAL TO STREAM): ______________________
VERTICAL CLEARANCE ABOVE STREAMBED: ______________________
WATERWAY OF FULL OPENING: ______________________
DISPOSITION OF STRUCTURE: ______________________
TYPE OF MATERIAL UNDER SUBSTRUCTURE: ______________________

WATER SURFACE ELEVATIONS AT:

Q 43% = ______ VELOCITY = ______
Q 10% = ______ " ______
Q 4% = ______ " ______
Q 2% = ______ " ______
Q 1% = ______ " ______

LONG TERM STREAMBED CHANGES: ______

IS THE ROADWAY OVERTOPPED BELOW Q 1%: ______
FREQUENCY: ______
RELIEF ELEVATION: ______
DISCHARGE OVER ROAD @Q 1%: ______

UPSTREAM STRUCTURE

TOWN: ______ DISTANCE: ______
HIGHWAY # : ______ STRUCTURE #: ______
CLEAR SPAN: ______ CLEAR HEIGHT: ______
YEAR BUILT: ______ FULL WATERWAY: ______
STRUCTURE TYPE: ______

DOWNSTREAM STRUCTURE

TOWN: ______ DISTANCE: ______
HIGHWAY # : ______ STRUCTURE #: ______
CLEAR SPAN: ______ CLEAR HEIGHT: ______
YEAR BUILT: ______ FULL WATERWAY: ______
STRUCTURE TYPE: ______

PROPOSED STRUCTURE

STRUCTURE TYPE: ______

CLEAR SPAN (NORMAL TO STREAM): ______
VERTICAL CLEARANCE ABOVE STREAMBED: ______
WATERWAY OF FULL OPENING: ______

WATER SURFACE ELEVATIONS AT:

Q 43% = ______ VELOCITY = ______
Q 10% = ______ " ______
Q 4% = ______ " ______
Q 2% = ______ " ______
Q 1% = ______ " ______

IS THE ROADWAY OVERTOPPED BELOW Q 1%: ______
FREQUENCY: ______
RELIEF ELEVATION: ______
DISCHARGE OVER ROAD @Q 1%: ______
______
BRIDGE LOW CHORD ELEVATION: ______
FREEBOARD: @ Q?? = ______

SCOUR: ______
______
REQUIRED CHANNEL PROTECTION: ______

PERMIT INFORMATION

		DEPTH OR ELEVATION:
AVERAGE DAILY FLOW:	______	
ORDINARY LOW WATER:	______	______
ORDINARY HIGH WATER:	______	______

TEMPORARY BRIDGE REQUIREMENTS

STRUCTURE TYPE: ______
CLEAR SPAN (NORMAL TO STREAM): ______
VERTICAL CLEARANCE ABOVE STREAMBED: ______
WATERWAY AREA OF FULL OPENING: ______

ADDITIONAL INFORMATION

______
______
______
______

Acronyms

Acronym	*Complete Title*
AASHTO	American Association of State Highway and Transportation Officials
AEP	Annual Exceedance Probability
ANR	Vermont Agency of Natural Resources
ANR GP	Vermont Agency of Natural Resources, *"Stream Alteration General Permit"*
AOP	Aquatic Organism Passage
AOS	Apparent Opening Size
ARC	Antecedent Runoff Condition
ASTM	American Society for Testing and Materials
BFW	Bankfull Width
CAD	Computer-Aided Design
CE	Categorical Exclusion
CRREL	Cold Regions Research and Engineering Laboratory
CSU	Colorado State University
DEC	Vermont ANR Department of Environmental Conservation
DTM	Digital Terrain Model
EA	Environmental Assessment
EIS	Environmental Impact Assessment
FDOT	Florida Department of Transportation
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FWD	Vermont ANR Fish and Wildlife Department
GIS	Geographic Information System
GPS	Global Positioning System
GSI	Green Stormwater Infrastructure
HDS	Hydraulic Design Series
HEC	Hydraulic Engineering Circular
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Centers River Analysis System
HEC-SSP	Hydrologic Engineering Center Statistical Software Package
HGL	Hydraulic Grade Line
HIRE	Highways in the River Environment
HSG	Hydrologic Soil Group
LID	Low Impact Development
LiDAR	Light Detection and Ranging
MDSHA	Maryland State Highway Administration
NACSE	Northwest Alliance for Computational Science and Engineering
NED	National Elevation Dataset
NEH	National Engineering Handbook
NEPA	National Environmental Policy Act
NETC	New England Transportation Consortium
NFIP	National Flood Insurance Program
NRB	Vermont Natural Resources Board
NRCC	Northeast Regional Climate Center
NRCS	Natural Resources Conservation Service
NSS	National Streamflow Statistics
NWIS	National Water Information System
O&M	Operation and Maintenance
OHW	Ordinary High Water
OSU	Oregon State University
PRISM	Parameter-elevation Regressions on Independent Slopes Model
RCN/UH	Runoff Curve Number and Unit Hydrograph
RECP	Rolled Erosion Control Product
RI	Recurrence Interval
SGA	Stream Geomorphic Assessments
SGA-DMS	Stream Geomorphic Assessment Data Management System
SGAT	Stream Geomorphic Assessment Tool
SRH-2D	Sediment and River Hydraulics – Two Dimensional
TR-55	Technical Release 55
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USCG	U.S. Coast Guard
USDA	U.S. Department of Agriculture
USFWS	U.S. Fish and Wildlife Service
USGS	U.S. Geological Survey
UVM	University of Vermont
VCGI	Vermont Center for Geographic Information
VDOT	Virginia Department of Transportation
VSMM	*"Vermont Stormwater Management Manual"*
VT SRMPP	Vermont Standard River Management Principles and Practices
VTrans	Vermont Agency of Transportation
WSE	Water Surface Elevation

Definitions

A

Aggradation – Increase in land elevation due to deposition.

Alluvial fan – A stream geomorphology describing a fan or cone-shaped deposit of sediment crossed and built up by streams.

Annual Exceedance Probability (AEP) – The likelihood that a natural event (i.e. storm/flood) will occur in any given year, reported as a percent. Calculated as the reciprocal of the *Recurrence Interval (RI)*.

B

Backwater – The increase in the upstream water surface level resulting from an obstruction to flow, such as a roadway fill with a bridge or culvert opening placed on the floodplain.

Bankfull Width (BFW) – The distance between channel bankfull elevations, which is the elevation at which flow first floods over the bank into the floodplain.

C

Channel armoring – The phenomenon where fine-grained material is washed away, leaving a bed of disproportionally large material on the surface of the stream bed.

Clear-water hydraulics – The hydraulic condition where there is little to no movement of sediment and aquatic organisms.

Clear-water scour – Occurs when there is no movement of bed material in the flow upstream of a crossing structure. A scour hole that develops in a clear-water condition will form during the rising stage of a flood (or gradually deepen through many floods) and remain fixed low-flow conditions. Clear-water scour depths are generally about 10% deeper than live-bed scour depths for a given set of initial conditions.

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

Concentrated flow – Flowing water that has been accumulated into a single, fairly narrow, stream.

Conveyance – A measure of the flow capacity of a channel.

Critical depth – The depth of critical flow, representing the minimum specific energy for a given flow rate.

Critical elevation – The highest level above a stormwater feature that water can rise before causing unacceptable inundation of travel lanes or adjacent property.

Critical flow – The type of flow that occurs when the *Froude number* has a value of 1.0, indicating that the inertial forces and gravitational forces are equal.

Critical velocity – The flow velocity above which the bed material of particle size, D, and smaller will be transported.

Crown – The inside top of the culvert.

Curb-opening inlet – A drainage inlet consisting of an opening in the roadway curb.

D

Debris control countermeasure – The implementation of targeted engineering strategies to prevent structural damage and failure specifically due to debris accumulation.

Degradation – Decrease in land elevation due to erosion.

Dendritic (see *Drainage pattern*)

Design event – The physical counterpart to *Design frequency*. The representation of natural processes that have the potential to affect the performance and use of an engineered structure. The simulated storm or flood used to predict the behavior of a proposed hydraulic system.

Design frequency – An event with a designated *Annual Exceedance Probability (AEP)* that a proposed hydraulic system must be hydraulically capable of conveying without flooding and becoming impassable.

Drainage – (1) The process of removing surplus ground or surface water by artificial means. (2) The system by which the waters of an area are removed. (3) The area from which waters are drained; a drainage basin.

Drainage area – (also drainage basin, basin, catchment, and watershed). The specific portion of the earth's surface upon which falling precipitation flows to a given location. With respect to a highway, this location may either be a culvert, the farthest point of a channel, or an inlet to a roadway drainage system.

Drainage pattern – A descriptor of the means by which surface water flows through its watershed. A radial drainage pattern occurs where streams or channels radiate outward from a single high point. A dendritic drainage pattern is the most common and occurs in V-shaped watersheds where smaller tributaries feed into larger streams and rivers, forming branch-like systems. A parallel drainage pattern occurs where the watershed has an elongated, pronounced slope. A trellis drainage pattern occurs where mountain tributaries feed into larger streams and rivers in valleys at 90-degree angles.

Drop inlet – A drainage inlet with a horizontal or nearly horizontal opening.

E

Equivalent cross slope – An imaginary straight cross slope having conveyance capacity equal to that of the given compound cross slope.

F

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

Flood routing – A technique used to predict the changes in the shape of water as it moves through a river channel or reservoir.

Free outlet – An outlet that has a tailwater equal to or lower than critical depth. For culverts with free outlets, lowering the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

Frontal flow interception – The portion of flow that passes over the upstream side of a grate and is intercepted.

Froude number – A dimensionless parameter representing the ratio of inertia forces to gravity forces.

G

Gaging Station – A location along a stream where measurements of stage or discharge are customarily made. The location includes a reach of channel through which the flow is uniform, a control downstream from this reach, and usually a small building to house the recording instruments.

Geomorphic – Relating to the physical features and characteristics of the earth's surface.

Geomorphology – The study of the Earth's surfaces and the processes that shape them.

Gradually varied flow – Flow in which depth and velocity change gradually enough in the flow direction that vertical accelerations can be neglected.

Grain size distribution – The relative amount, typically by mass, of the particles present in a sample falling under set size categories.

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, with consideration to the fact that any side adjacent to a curb is not considered a part of the perimeter in weir flow computations.

Gumbel distribution – A skewed statistical distribution for extreme value analysis.

Gutter – The portion of the roadway section adjacent to the curb that is used to convey stormwater runoff. It may include a portion, or all, of a traveled lane, shoulder or parking lane. A limited width adjacent to the curb may be of different materials and have a different cross slope.

H

Headcut – The sudden change in bed elevation at the leading edge of a gully.

Headwater – The water depth measured from the flow line (invert) of the culvert inlet to the water surface elevation.

Hydraulic Grade Line (HGL) – The level to which water in a pipe would rise when exposed to atmospheric pressure. If the HGL is below the crown of a pipe, open channel flow is occurring. If the HGL is above the crown of a pipe, pressure flow is occurring.

Hydraulic jump – An abrupt and observable transition from supercritical to subcritical flow in the flow direction. Depth and velocity change significantly in the jump, and energy is dissipated.

Hydrograph – A graph showing stage, flow, velocity, or other property of water with respect to time.

Hydrologic – Pertaining to the cyclic phenomena of waters of the earth; successively as precipitation, runoff, storage, and evaporation, and quantitatively as to distribution and concentration.

Hydrology – The science of dealing with the occurrence and movement of water upon and beneath the surface of the earth. Overlaps and includes portions of other sciences such as meteorology and geology. The particular branch of hydrology that a design engineer is generally interested in is surface runoff which is the result of excess precipitation.

I

Improved inlet – An inlet with an entrance geometry that decreases the flow constriction 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).

Incipient overtopping event – The largest design frequency flood event that does not overtop a crossing structure. The incipient overtopping event is used for scour analysis because it often puts the most stress on the bridge and can result in the greatest scour.

Inlet efficiency – The ratio of flow intercepted by an inlet to total flow in the gutter. (see *Interception capacity)*

Interception capacity – The amount of stormwater runoff that an inlet captures. Flow that is not intercepted bypasses the inlet and is carried in the street or channel to the next inlet down grade. (see *Inlet efficiency)*

Invert – The inside bottom of the structure; for an open-bottom culvert or bridge, the invert is the elevation of the channel's low point at the location of analysis.

L

Live-bed scour – Occurs when bed material from upstream is transported to the crossing. A scour hole that develops in a live-bed condition will develop during the rising stage of a flood and fill during the falling stage.

N

Natural levee – A buildup of sediment, sand, or debris on the sides of a river or stream's flood plain that occurs during flooding.

Non-uniform flow – Flow in which the velocity and depth vary in the direction of motion. Non-uniform flow can occur either in a prismatic channel or in a natural channel with variable properties.

Normal flow – Occurs in a channel reach when the discharge, velocity, and depth of flow do not change throughout the reach. The water surface profile and channel bottom slope will be parallel. Normal flow will typically exist in a culvert operating on a steep slope provided the culvert is long enough.

O

Ordinary High Water (OHW) – Relevant to the USACE 404 Permit. With respect to non-tidal waters, the line on the shore established by the fluctuations of water indicated by physical characteristics such as a clear, natural line impressed on the bank, shelving, changes in the character of the soil, destruction of terrestrial vegetation, the presence of litter and debris, or other appropriate means to consider the characteristics of the surrounding areas.

Overtopping flood – The flood described by the probability of exceedance and water surface elevation at which flow occurs over the highway, over the watershed divide, or through structure(s) provided for emergency relief.

P

Parallel (see *Drainage pattern*)

Peak flow – Maximum momentary stage or discharge of a stream in flood.

Pebble count – A method for systematically and proportionately sampling all bed features within a bankfull channel through a designated reach.

Perennial stream – A watercourse or a portion, segment, or reach of a watercourse, generally exceeding 0.5 square miles in watershed size, in which surface flows are not frequently or consistently interrupted during normal seasonal low flow periods. Perennial streams that begin flowing subsurface during low flow periods, due to natural geologic conditions, remain defined as perennial. All other streams, or stream segments of significant length, shall be termed intermittent. A perennial stream shall not include the standing waters in wetlands, lakes, and ponds.

Piping – The loss of finer soil particles through the coarser material.

Plan form changes – Changes that influence the shape and alignment of the stream channel. Incised channels can evolve from meandering channels to braided channels. Meanders can migrate laterally and threaten to erode bridge approaches. Bank widening can change the bridge contraction ratio. Plan form changes are considered permanent future changes for the stream bed elevation at a bridge site and should be considered as part of the foundation design process.

Pressure flow – Occurs when the flow of water through a constricted opening no longer meets the criteria of open channel flow.

Pressure head – The height of a column of water that would exert a unit pressure equal to the pressure of the water. The pressure head is also known as static head.

R

Radial (see *Drainage pattern*)

Rainfall – (1) Point precipitation: that which registers at a single gage. (2) Area precipitation: Adjusted point rainfall for area size.

Rapidly varied flow – Flow in which there is a pronounced curvature of the streamlines and the assumption of hydrostatic pressure is no longer valid.

Recurrence Interval (RI) – The average interval of time expected to pass before a natural event (i.e. storm/flood) of the same magnitude occurs again.

Reference reach – A segment of the stream or river that is meant to act as a template for stream simulation design through the crossing structure. A reference reach should be stable, ideally nearby and upstream of the structure, outside the influence of the existing structure, of a similar gradient to the design gradient through the structure, and at least as long as the road-stream crossing.

Return period (see *Recurrence Interval (RI)*)

Roadway Classification – The functional type of roadway to which a road belongs and the design and construction standards that are associated with it.

Runoff – (1) Water that runs off at the surface during a precipitation or snowmelt event when infiltration and/or storage is exceeded or unavailable. (2) Drainage or flood discharge after a rainfall or snowmelt event which leaves an area as surface flow or as piped flow and is not infiltrated.

S

Sag point – A roadway depression designed to contain and direct stormwater to an inlet.

Scour depth – The depth to which the stream bed is eroded beyond the normal depth of the stream bed due to factors such as increased flow velocity of the stream.

Scupper – A vertical hole through a bridge deck for the purpose of deck drainage. A horizontal opening in the curb or barrier is sometimes also referred to as a scupper.

Shear stress – The force applied to the cross-sectional area of a material, which is parallel to the applied force vector. Shear stress is calculated by dividing force per unit area.

Sheet flow – Any flow spread out and not confined; i.e. flow across a flat open field.

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

Sieve analysis – A procedure used to assess particle size distribution in a stream bed, also known as gradation test.

Slip-lining – A trenchless method of rehabilitating a segment of existing pipeline by fitting a smaller pipe inside the original pipe, filling the space between the pipes with grout, and sealing both the upstream and downstream ends.

Slope conditions – Steep slope occurs where the critical depth is greater than the normal depth. Mild slope occurs where critical depth is less than normal depth.

Slotted drain – A drainage inlet composed of a continuous slot built into the top of a pipe that serves to intercept, collect, and transport the flow.

Specific energy – The energy head relative to the channel bottom.

Splash-over – The portion of the frontal flow at a grate which skips or splashes over the grate and is not intercepted.

Spread – The width of flow in the gutter measured laterally from the roadway curb.

Spur dikes – A structure used to protect stream banks from erosion and to encourage stable pools along a stream. It is a linear structure with one end projecting into the stream and the other end on the bank of the stream.

Stage – The elevation of a water surface above its minimum; also above or below an established “low water” plane; hence above or below any datum of reference; gage height.

Steady-state flow – Flow that occurs when the discharge passing a given cross section is constant with respect to time. The maintenance of steady flow in any reach requires that the rates of inflow and outflow be constant and equal.

Step-backwater analysis – A method for calculating the water surface profile along a reach at incremental stages using the energy equation. The calculations typically proceeds from downstream to upstream.

Storm – A disturbance of the ordinary, average conditions of the atmosphere which, unless specifically qualified, may include any or all meteorological disturbances, such as wind, rain, snow, hail, or thunder.

Stream – Water flowing in a channel or conduit, ranging in size from small creeks to large rivers.

Subcritical flow – Flow that occurs when the *Froude number* has a value that is less than 1.0, indicating that the gravitational forces are greater than the inertial forces.

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

Surface runoff – The movement of water on earth's surface, whether flow is over surface of ground or in channels.

Supercritical flow – Flow that occurs when the *Froude number* has a value that is greater than 1.0, indicating that the inertial forces are greater than the gravitational forces.

T

Tailwater – The waters located directly downstream from a hydraulic structure.

Thalweg – The line of lowest elevation along a stream bottom.

Time of concentration – The time required for storm runoff to flow from the most remote point of a drainage area to the point under consideration (design point).

Total energy head – The specific energy head plus the elevation of the channel bottom with respect to a datum.

Trellis (see *Drainage pattern*)

Tremie seal – A concrete placement method using a pipe to place concrete below water.

Trunk line – The mainline of a stormwater drainage system.

U

Underdrain – A perforated pipe at the bottom of a detention basin, channel or swale that allows the feature to drain.

Uniform flow – Flow that occurs in a channel with a constant cross section, roughness, and slope in the flow direction.

Unsteady-state flow – Flow that occurs when the discharge passing a given cross section varies over time.

V

Velocity head – The kinetic energy of flowing water expressed as a height of water. The velocity head is also known as the dynamic head.

Watershed (see *Drainage area)*

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