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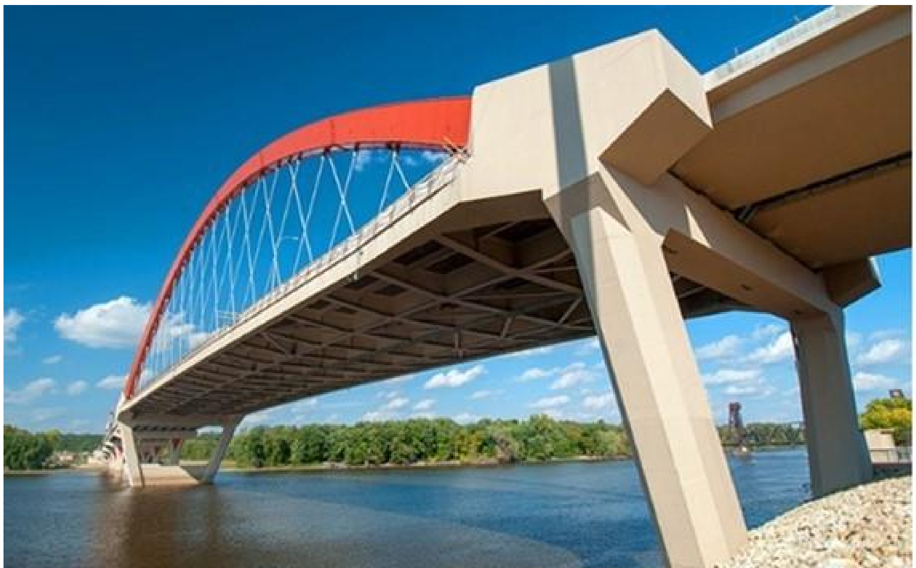

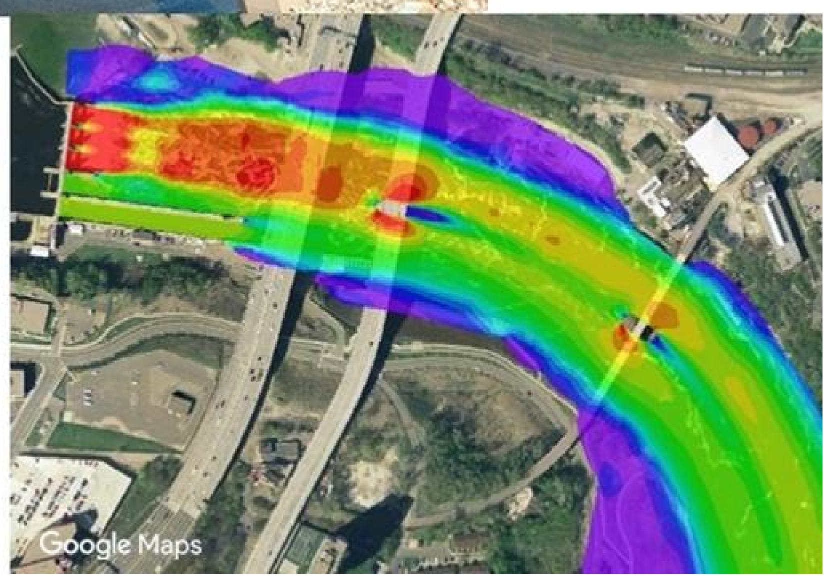

Hydraulic Design of Safe Bridges

Second Edition

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16. Abstract

This document provides technical information on the hydraulic analysis and design of bridges. The goal is to provide information so that bridges can be designed as safely as possible while optimizing costs and limiting impacts to property and the environment. Many significant aspects of bridge hydraulic design are discussed. These include regulatory topics, specific approaches for bridge hydraulic modeling, hydraulic model selection, bridge design impacts on scour and stream instability, and sediment transport.

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List of Acronyms

1D	One-dimensional
2D	Two-dimensional
3D	Three-dimensional
AASHTO	American Association of State Highway and Transportation Officials
AEP	Annual Exceedance Probability
AOP	Aquatic Organism Passage
ASCE	American Society of Civil Engineers
BFE	Base Flood Elevation
CEM	Coastal Engineering Manual
CFD	Computational Fluid Dynamics
CFR	Code of Federal Regulations
CLOMR	Conditional Letter of Map Revision
CSR	Capacity Supply Ratio
CWA	Clean Water Act
DOT	Department of Transportation
EGL	Energy Grade Line
ESA	Endangered Species Act
FEMA	Federal Emergency Management Agency
FESWMS	Finite Element Surface Water Modeling System
FHWA	Federal Highway Administration
FIS	Flood Insurance Study
FSA	Farm Service Agency
FST2DH	Flow and Sediment Transport - 2-Dimensional Horizontal plane
GIS	Geographical Information System
GMSLR	Global Mean Sea Level Rise
HDS	Hydraulic Design Series
HEC	Hydraulic Engineering Circular (FHWA)
HEC	Hydrologic Engineering Center (USACE)
HEC-RAS	Hydrologic Engineering Center River Analysis System
HGL	Hydraulic Grade Line
LIDAR	Light Detection and Ranging
LOMR	Letter of Map Revision
LRFD	Load Resistance Factor Design

MHHW	Mean Higher High Water
MHW	Mean High Water
MLLW	Mean Lower Low Water
MLW	Mean Low Water
MOU	Memorandum of Understanding
NAVD 88	North American Vertical Datum of 1988
NBIS	National Bridge Inspection Standards
NCHRP	National Cooperative Highway Research Program
NED	National Elevation Dataset
NEPA	National Environmental Policy Act
NFIP	National Flood Insurance Program
NGVD 29	National Geodetic Vertical Datum of 1929
NHPA	National Historic Preservation Act
NHS	National Highway System
NMFS	National Marine Fisheries Service
NOAA	National Oceanic and Atmospheric Administration
NOS	National Ocean Service
NRC	National Research Council
NRCS	Natural Resources Conservation Service
RAS	River Analysis System (HEC-RAS)
RMA2	Resource Management Associates 2-D hydraulic model
RSLR	Relative Sea Level Rise
SCS	Soil Conservation Service (now NRCS)
SFHA	Special Flood Hazard Area
SI	System International (metric system of units)
SHPO	State Historic Preservation Officer
SID	Standard Identifier
SMS	Surface-water Modeling System
SRH-2D	Sedimentation and River Hydraulics-Two-Dimensional model
SWE	Shallow Water Equations
SWL	Still Water Level
TAC	Transportation Agency of Canada
TRB	Transportation Research Board
TVA	Tennessee Valley Authority
US	United States

U.S.C.	United States Code
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
USCG	United States Coast Guard
USDA	United States Department of Agriculture
USDOT	United States Department of Transportation
USEPA	United States Environmental Protection Agency
USFWA	Unites States Fish and Wildlife Service
USGS	United States Geologic Survey
WSEL	Water Surface Elevation

Acknowledgments

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Notice

Non-Binding Contents

Quality Assurance Statement

CHAPTER 1
INTRODUCTION

1.1 Background & Purpose

1.1.1 Background

The Federal Highway Administration has oversight of the Nation's bridges through the National Bridge Inspection Standards (NBIS) and other regulatory policies and programs. Bridge failures resulting from natural and human factors led the U.S. Congress to express its concern about the safety, approaches, and oversight of the Nation's bridges.

Within the Conference Report for the Departments of Transportation and Housing and Urban Development, and Related Agencies Appropriations Act, 2010 (H.R. Report No. 111-366), the Congress recommended that the *"… (FHWA) use a more risk-based, data-driven approach to its bridge oversight"* to improve bridge safety. Congress stated its intention to monitor the progress that FHWA makes in identifying new approaches to bridge oversight, completing initiatives, and achieving results from its efforts. Congress directed FHWA to apply funds to focus and achieve these activities.

To address the conference report, FHWA undertook a combination of activities that contribute to four primary outcomes: more rigorous oversight of bridge safety; full NBIS compliance by all States; improved information for safety oversight and condition monitoring; and qualified and equipped bridge inspection personnel.

As hydraulic issues remain a leading factor in bridge failures, FHWA recognized that these activities need to include efforts to collect better, understand and deploy more recent and robust guidance and techniques to the accepted state of hydraulic and waterway-related practice. This document is one of the products of these efforts.

1.1.2 Purpose of this Reference Document

The purpose of Hydraulic Design Series 7 (HDS-7) *Hydraulic Design of Safe Bridges* is to provide technical information and describe widely accepted practices in bridge hydraulics. The first edition of HDS-7 replaced the last edition of HDS-1 *Hydraulics of Bridge Waterways* (FHWA 1978). The three decades after the last edition of HDS-1 saw dramatic changes in both the hydraulic design specifications of bridges and the methods and tools available for hydraulic analysis. HDS-7's first edition provided a comprehensive reference on the modern state of practice in bridge hydraulic design. The state of practice has continued to evolve since the first edition was published. This second edition provides updated content and discussion of bridge hydraulic engineering, reflecting these advances.

The hydraulic engineer aims to develop bridge designs that protect the safety of the traveling public and provide resilience against flood damage. A good design also optimizes project costs and limits impacts to adjacent properties and the environment. This document discusses the critical aspects of bridge hydraulic design. These include design considerations, regulatory topics, specific approaches for bridge hydraulic modeling, hydraulic model selection, bridge design impacts on scour and stream instability, and sediment transport.

1.2 History of Bridge Hydraulic Analysis & Design

1.2.1 The Early Years of Road Building in the United States

The hydraulic sizing of bridge and culvert waterways in the United States has continually evolved since the early days of road and railroad building. Chow (1962) and McEnroe (2007) provide helpful summaries of the methods used in the 1800s and early 1900s. The earliest methods relied on the individual engineer's practical experience and judgment, guided by observation of facilities' performance.

Manning's Equation (see Chapter 4) became popular in the 1890s. It established a relationship between the velocity and the channel's size, shape, slope, and resistance to flow for uniform steady-flow conditions. It represented an improvement in hydraulic analysis but did not account for non-uniform flow conditions such as backwater at a bridge.

By 1900, railroad engineers used formulas and tables to determine the waterway opening size. Among these were the Meyers formula, first presented in 1879; the Talbot formula, published in 1887; and the Dun Waterway Table of the Atchison, Topeka and Santa Fe Railway, published in 1909. These approaches involved determining the waterway area of the bridge or culvert based on the drainage area, with coefficients or corrections for watershed slopes and geographic factors. The Talbot formula included coefficients for rainfall intensity and the ground surface's relative imperviousness. Many highway engineers used the Dun table as late as the 1960s and the Talbot formula into the 1970s.

The methods described above, and many others like them, were used widely, but the engineers of the time recognized their shortcomings. In his paper, McEnroe (2007) includes a critique of the 1880s state of practice by A. M. Wellington, published in the *Railroad Gazette* in 1886:

> *For culverts, if we were called upon to suggest a formula, we could do no better than this: Estimate the necessary area as carefully as possible by the existing evidences of maximum flow, which let equal to A. Then will* $\sqrt[3]{8}$ *A equal the proper area for the culvert. In more popular language: 'Guess at the proper size and double it. ' We apprehend that this formula will give far more satisfactory and trustworthy results than that which our correspondent quotes [the Myers formula] or any other which purports to be of general application to a problem subject to such extremely diverse conditions. (Wellington, 1886).*

1.2.2 Advancements of the Mid-20th Century and the Rise of Computer Modeling

Flood-frequency-based sizing of culverts and bridges was a crucial advancement in transportation engineering. The Bureau of Public Roads (an FHWA predecessor) and others performed research in the 1940s that became the foundation of frequency-based design, and highway engineering practice began transitioning to this approach in the 1950s (McEnroe 2007).

The textbook *Open-Channel Hydraulics*, by V.T. Chow (1959), presented the standard-step method as a numerical technique for computing a steady-flow water surface profile in a natural channel or floodplain. The method drew from two fundamental principles: the conservation of mass and the conservation of energy. To account for non-uniform flow conditions, the Bureau of Public Roads first published HDS-1 in 1960 and updated it in 1970 and 1978 (FHWA 1978). HDS-1 presented equations and nomographs to estimate the backwater caused by a highway crossing based on a specific bridge geometry, the degree to which the crossing constricted the floodplain width, the skew angle of the flow, and other factors.

Based on Chow's work, engineers developed computer software for computing water surface profiles with the standard-step method. In the late 1960s, the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC) released the program Backwater Any Cross Section, commonly known as HEC-2 Water Surface Profiles (USACE 1991). HEC-2 became the most popular program in the USACE HEC series. This program continually improved and, in 1984, was made available in the personal computer (PC) environment.

HEC-2, a one-dimensional (1D) model, marked the transition of bridge hydraulic analysis in the United States from basic mathematical equations to numerical models. One-dimensional, in this context, means that the method only accounts for velocities, forces, and gradients in the streamwise direction (upstream or downstream). The method does not include velocities, forces, or gradients in the lateral or vertical directions. Numerical models have been the primary method of bridge hydraulic analysis since the introduction of HEC-2.

The FHWA, in cooperation with the United States Geological Survey (USGS), also developed a 1D computer program for water surface profile computations, but with particular emphasis on bridge waterways. The USGS/FHWA water surface profile computation model would become known as WSPRO (FHWA 1986). Like HEC-2, the WSPRO program computed water surface profiles for rivers and floodplains with natural cross sections, using the standard-step method.

In 1995, the USACE released the HEC's River Analysis System (HEC-RAS). This 1D program replaced HEC-2 with improved computations and, more significantly, with a graphical user interface for developing input and viewing results. HEC-RAS incorporated aspects of the WSPRO methodology as a bridge modeling option for users. HEC-RAS 1D modelling rapidly became the predominate approach for hydraulic engineering studies throughout the United States, including those carried out for transportation agencies.

1.2.3 The Emergence and Growth of Two-Dimensional Hydraulic Modeling

Bridge hydraulic engineers were commonly performing 1D analysis by the mid-1990s. Practitioners were increasingly aware of the limitations and inaccuracies of 1D models, particularly for complex situations. Two-dimensional (2D) hydraulic modeling was available and used by a limited number of agencies to simulate complex hydraulic problems as early as the 1970s. But until the 1980s and early 1990s, 2D models were run primarily on powerful mainframe computers because of their computational intensity. Processors for PCs became powerful enough in the 1980s to allow limited mainstream use of 2D modeling. Since that time, the number of projects using 2D modeling for transportation applications has steadily increased. Several computer programs are now available to perform 2D modeling. In this context, 2D means that the method accounts for velocities, forces, and gradients in all directions in the horizontal plane. The method neglects vertical components and gradients.

Early 2D models prevalent in the United States included the USACE's RMA2 and the USGS's FESWMS-2DH (Froehlich, 1989). The latter's application to transportation projects began in 1988. The University of Kentucky developed FESWMS-2DH with the sponsorship and oversight of FHWA. In 2003, the program name was changed to Flow and Sediment Transport Model (FST2DH), and FHWA published a User's Manual (FHWA 2003).

In 2012, FHWA published the first edition of HDS-7. This hydraulic design series document, HDS-1, identified several best practices in hydraulic modeling for bridge design. While HDS-7's first edition considered and explained both 1D and 2D modeling, it suggested 2D modeling for a wide range of situations. This second edition of HDS-7 also makes that suggestion and extends it to a broader range of situations, given the increasing availability of modeling software, 2D modeling training, and high-quality digital terrain data.

In 2013 FHWA began using the Sedimentation and River Hydraulics model (SRH-2D), developed and maintained by the United States Bureau of Reclamation (USBR), and suggested its use for transportation hydraulic analyses. SRH-2D marked a shift from the use of FST2DH. SRH-2D avoids the stability issues common with earlier 2D models like RMA2 and FST2DH. In 2013, the Surface Water Modeling System (SMS) software added SRH-2D as a supported modeling program. The FHWA partnered with the USBR to incorporate several advanced hydraulic structures modeling features into SRH-2D, including culvert hydraulics, bridge pressure flow, and piers. The FHWA's support of the development of a custom SRH-2D graphical user interface in SMS also led to a free community version of SMS. The FHWA's Hydraulic Toolbox is also part of SMS to support scour calculations through the SMS scour coverage in the map module. The scour coverage provides tools that enable users to accurately extract the hydraulic variables for scour calculations. The FHWA also produced the *Two-Dimensional Hydraulic Modeling for Highways in the River Environment: Reference Document* (FHWA 2019), which is intended as a comprehensive source of information on the use of 2D hydraulic modeling for highway structures in the river environment.

The USACE released version 5 of HEC-RAS in 2016. This version added 2D functionality to the widely used program. HEC-RAS version 6, released in 2021, incorporates several 2D enhancements, including bridge hydraulics, wind forces, a new shallow water equation (SWE) solver, and sediment transport modeling capabilities. Other 2D modeling software programs exist and may be suitable for bridge hydraulic analysis, especially those that accommodate unstructured grids, carry two-dimensional computations through the bridge waterway, and allow for pier drag modeling and pressure flow.

1.2.4 Looking Ahead

The FHWA and many State departments of transportation have made significant investments in advancing the state of practice in bridge hydraulics. Through the Every Day Counts-4 initiative Collaborative Hydraulics: Advancing to the Next Generation of Engineering (CHANGE) (2017-2018), the FHWA developed and disseminated resource materials and training to promote expanded use of 2D modeling in transportation hydraulics. The States have responded, with an increased use of 2D modeling for bridge design. It is reasonable to speculate that by 2025, 2D modeling will account for most new bridge design hydraulic studies in the United States.

Just as with 2D modeling in the early 1990s, the increase in personal computer capabilities now makes 3D/computational fluid dynamics (CFD) modeling accessible to a broader and growing base of hydraulic engineers in transportation agencies and consulting firms. The FHWA has conducted numerous scour research projects with CFD in recent years and envisions that future bridge design teams will employ CFD tools for scour analysis.

Bridge hydraulic analysis and design are still affected by several areas of uncertainty. However, researchers and practitioners' efforts and the explosion of computer power and availability through the decades have advanced bridge hydraulic engineering practice.

1.3 Document Organization

This document is organized into 12 chapters and two appendices as follows:

1.3.1 Chapter 1 – Introduction

This chapter discusses the purpose of HDS-7 and its legislative impetus. It explains key milestones in the history of bridge hydraulics in the United States.

1.3.2 Chapter 2 – Hydraulic Design Considerations for Bridges

Chapter 2 describes the most common considerations for the hydraulic design of bridges. Backwater considerations and constraints influence the design of the bridge waterway (bridge length, height of superstructure, and abutment type) and of the roadway profile. Freeboard provides clearance between the design water surface elevation and the bridge superstructure to accommodate floating debris and some of the uncertainty inherent in the discharge estimate and hydraulic analysis. The hydraulic engineer assesses scour potential and stream stability to inform the design of pier and abutment foundations. Chapter 2 also discusses bridge design practices that enhance the highway network's resilience against flood-related closures and damages.

1.3.3 Chapter 3 – Regulatory Requirements

Chapter 3 discusses design constraints and permit requirements that arise from Federal law. Several statutes and regulations govern the conduct of FHWA and affect the hydraulic design of bridges. Additional permit and procedural requirements arise from the statutes and regulations governing other agencies. These include environmental reviews under the National Environmental Policy Act, the Coast Guard Bridge Permit, Clean Water Act Section 404 Permits, historic preservation requirements, and the National Flood Insurance Program regulations.

1.3.4 Chapter 4 – Principles of River & Floodplain Hydraulics

Chapter 4 provides background on the fundamentals of fixed-bed open channel flow. It provides the engineer with a practical conceptual understanding of flow classification and the governing equations of open channel flow. With this understanding, the engineer can review and interpret hydraulic model results, check questionable results, explain the results to others, predict how changes in input can affect the results, and identify why to select one approach over another.

1.3.5 Chapter 5 – Bridge Hydraulic Analysis Considerations

Chapter 5 builds on the background from Chapter 4 to discuss hydraulic modeling. This chapter compares and contrasts 1D, 2D, and 3D numerical modeling along with physical model studies. Chapter 5 also discusses model data, boundary conditions, and hydrologic analysis and explains steady- versus unsteady-flow modeling. A wide range of considerations can influence the appropriate selection of the modeling approach. Skewed crossings and multiple-opening crossings are two examples of conditions calling for 2D modeling instead of 1D. Increasingly, at some sites, it takes less time and effort to develop a 2D model than a 1D model.

1.3.6 Chapter 6 – One-Dimensional Bridge Hydraulic Analysis

Chapter 6 provides information on using 1D models for bridge hydraulic analysis. The chapter often refers to HEC-RAS, but the information applies to other 1D models. The chapter covers typical applications as well as special cases. The 1D modeling approach solves for a water surface profile of a natural river and floodplain using the standard-step solution of the energy equation. The computation points are cross sections, or transects, across the floodplain. The engineer can select from multiple available approaches to bridge hydraulic calculations, both for free-surface flow through the bridge waterway and for higher flood conditions that create pressure flow or overtopping conditions.

1.3.7 Chapter 7 – Two-Dimensional Bridge Hydraulic Analysis

Chapter 7 provides information on using 2D models for bridge hydraulic analysis. This chapter occasionally refers to the SRH-2D and SMS programs, but the information can apply to other 2D hydraulic models. The chapter explains the assumptions and input data and parameters of 2D models. It covers general model applications and special techniques for modeling highway structures. Two-dimensional modeling using unstructured mesh models is rapidly becoming widespread in the United States. Adopting superior practices in terrain representation, mesh development, roughness assignment, and boundary condition development enables the engineer to create representative simulation models. In turn, the models help the engineer prepare effective bridge designs. This chapter extensively refers to another FHWA document: *Two-Dimensional Hydraulic Modeling of Highways in the River Environment* (FHWA 2019).

1.3.8 Chapter 8 – Unsteady Flow Hydraulic Analysis & Bridges over Tidal Waterways

Chapter 8 discusses modeling unsteady flow conditions with 1D and 2D models. Topics discussed in this chapter include the fundamental equations that define unsteady flow, upstream and downstream model extents, floodplain storage and connections, and boundary conditions. The chapter discusses the types of tidal waterways commonly crossed by highways. The chapter explains tides, currents, and waves. Highways crossing tidal waterways are subject to fundamentally different hydraulic conditions than riverine bridges. Tidal fluctuations lead to continually changing discharge rates and regular flow direction reversals through the bridge waterway. Currents associated with storm surge conditions can attain high, albeit transient, velocity magnitudes and extreme wave action can pose a significant threat to bridge resilience. This chapter extensively refers to another FHWA document: HEC-25 *Highways in the Coastal Environment* (FHWA 2020).

1.3.9 Chapter 9 – Bridge Scour Considerations & Scour Countermeasures

Chapter 9 discusses a crucial aspect of bridge safety. Scour during floods is a significant aspect of bridge design and scour prediction is a primary contribution of the hydraulic engineer to the bridge structural design. This chapter discusses the types of scour and explains methods of obtaining appropriate hydraulic variables for scour calculations from 1D and 2D models. The chapter explains how to use the FHWA Hydraulic Toolbox software to conduct scour calculations. It describes how to use hydraulic modeling results to design scour countermeasures and analyze certain countermeasure designs. This chapter refers to several FHWA documents, including HEC-18 *Evaluating Scour at Bridges* (FHWA 2012b), HEC-20 *Stream Stability at Highway Structures* (FHWA 2012c), HEC-23 *Design of Bridge Scour and Stream Instability Countermeasures* (FHWA 2009a), HEC-25 *Highways in the Coastal Environment* (FHWA 2020) and *Hydraulic Considerations for Shallow Abutment Foundations* (FHWA 2018).

1.3.10 Chapter 10 – Sediment Transport & Alluvial Channel Concepts

Chapter 10 provides an overview of the application of sediment transport concepts to bridge projects and aids in determining whether sediment transport analysis is needed. It discusses sediment transport concepts and equations and describes the sediment continuity approach to analysis. This chapter draws from and extensively refers to another FHWA document HEC-16 *Highways in the River Environment: Roads, Rivers, and Floodplains* (FHWA 2023a).

1.3.11 Chapter 11 – Other Considerations & Bridge Hydraulics Topics

Chapter 11 is a resource for hydraulic engineers to identify additional factors that may impact bridge design and structure safety. These topics include bridge deck drainage, hydraulic forces on bridge decks, piers and pile groups, coincident flows at confluences of rivers, physical modeling, and computational fluid dynamics.

1.3.12 Chapter 12 – Literature Cited

Chapter 12 lists all the references cited in the document. For references produced by government agencies (FHWA, USGS, NCHRP etc.), the agency is indicated as the author. This format was selected to group all agency documents together in the reference list. Each citation also lists the author(s).

1.3.13 Appendix A – Glossary

Provides useful terms and definitions used in the document.

1.3.14 Appendix B – Units

An appendix provides information on units and unit conversions.

1.4 Units of this Document

This document uses customary (English) units consistent with FHWA policy. However, in limited situations, it uses both customary (English) units and SI (metric), or only SI units, because these are the predominant measure used nationwide and globally for such topics. In these situations, the document provides the rationale for the use of units.

CHAPTER 2
HYDRAULIC DESIGN CONSIDERATIONS FOR BRIDGES

2.1 Introduction

A bridge design team faces a wide variety of decisions across multiple disciplines. When the bridge is over water, many of those decisions are in the hydraulics discipline. Hydraulic issues are ideally considered early in the design process, as they will influence many decisions throughout design and construction. The hydraulic design will also affect the crossing's performance through its service life. This chapter provides a general discussion of the design considerations that drive the hydraulic design of bridges, emphasizing those related to bridge hydraulic capacity and bridge foundation stability. This document's later chapters discuss each of these issues in detail. Beyond being essential engineering design issues, these considerations also have regulatory and permitting implications. Chapter 3 describes the regulatory framework of bridge hydraulics.

2.2 Hydraulic Capacity Considerations

2.2.1 Backwater

A highway crossing over a stream and floodplain has the potential to cause an increase in the flood profile compared to natural conditions. This increase, if it occurs, is called backwater, and it is a crucial consideration in bridge hydraulic design. Backwater is essentially a buildup of potential energy needed to accelerate flow into the relatively smaller bridge area. Designing the crossing to cause little or no backwater is usually desirable, and in many cases, it is limited by policy or regulations. Figure 2.1 illustrates backwater. Excessive backwater increases flood risk to neighboring properties and exposes the highway owner to liability. It also increases scour potential at a bridge. Excessive backwater may also violate local, State, or Federal floodplain management regulations. Early determination of the backwater constraints is advantageous.

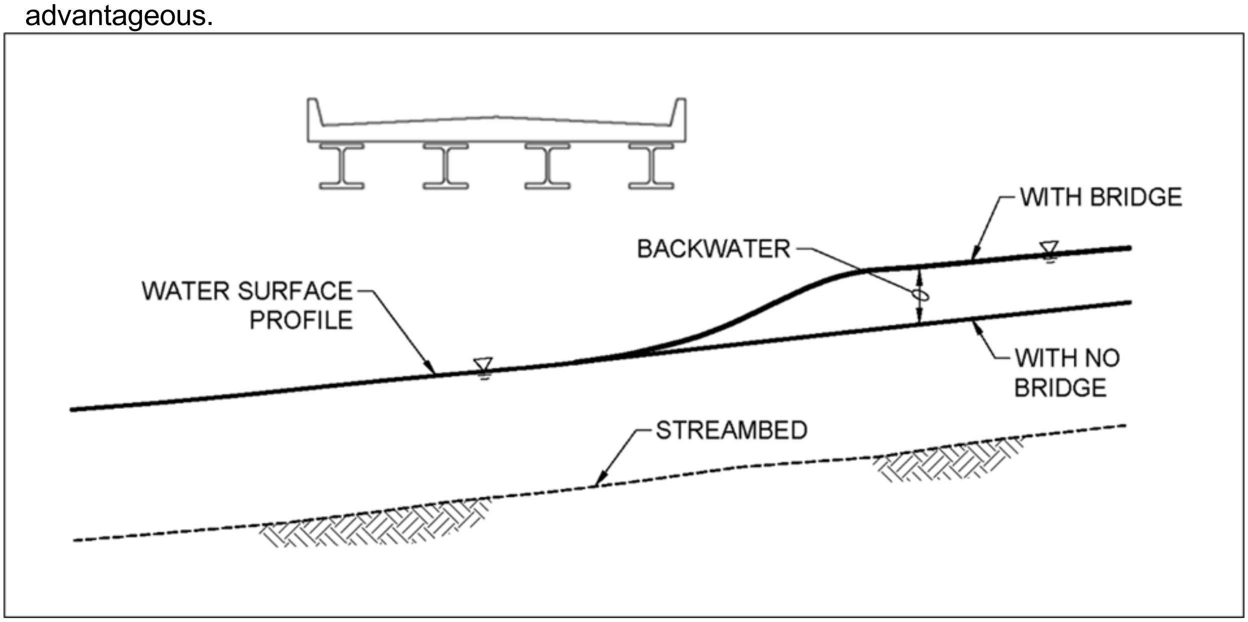

Figure 2.1. Profile-view illustration of backwater at a bridge.

2.2.2 Bridge Waterway Width

The bridge length determines the bridge waterway width and is a dominant factor influencing the crossing capacity and the potential backwater. Intuitively, given a particular design discharge at a given crossing location, a shorter bridge provides less hydraulic capacity and causes more backwater than a longer bridge. The engineer ideally performs at least a rough hydraulic analysis early in the project design to establish the minimum bridge length needed to avoid causing an unacceptable backwater elevation.

2.2.3 Road Profile Design

Highways crossing floodplains usually include encroachments into the floodplains by approach road embankments in addition to the bridge structure. The extent of encroachment by the road embankment is the difference between the floodplain width and the bridge waterway width. Flow that does not overtop the road embankment contracts to pass through the bridge opening and then expand downstream of the bridge to reoccupy the full floodplain width. This process of contraction and expansion is usually the primary cause of backwater. When the water surface profile of a flood is above the road profile, the resulting overtopping flow capacity can limit the backwater for that flood. “By definition, the highway will not be inundated from the stage of the design flood” [23 CFR § 650.105(d)]. Designing the crossing to avoid overtopping is advantageous for the safety of the traveling public, maintaining the highway's level of service, and preventing costly roadway damage. However, allowing overtopping reduces the backwater potential and may allow a shorter bridge. The design team weighs this trade-off of competing objectives in developing the overall crossing design.

2.2.4 Bridge Superstructure & Freeboard

As noted in Chapter 3, FHWA regulations (23 CFR § 650.115) require the provision of freeboard, where practicable. Most State departments of transportation also include freeboard requirements in their design standards and criteria. In the context of transportation agency bridge hydraulic design criteria, freeboard is the vertical clearance from the water surface elevation of the hydraulic design flood to the superstructure's low chord. Freeboard allows for the passage of floating debris and ice and provides a measure of protection against wave attack. Another role of freeboard is to account for a degree of uncertainty in the hydrologic and hydraulic analysis (FHWA (2019). In that 2019 document, Section 8.1.1 provides a detailed description of freeboard estimation from 2D hydraulic model results. The area of rapid drawdown in the water surface at the upstream bridge face does not provide an adequate basis for freeboard determination in hydraulic design. One practice is to measure freeboard from the lowest bridge member to the water surface approximately one bridge deck width upstream of the bridge face. Using Figure 2.1 as an example, one bridge deck width upstream is an acceptable location for measuring freeboard, but individual situations may differ.

A technical justification for freeboard is that a pressure-flow condition (flood flow submerging the bridge low chord) significantly increases the backwater. Figure 2.2 illustrates a pressure-flow condition that would aggravate backwater at a bridge.

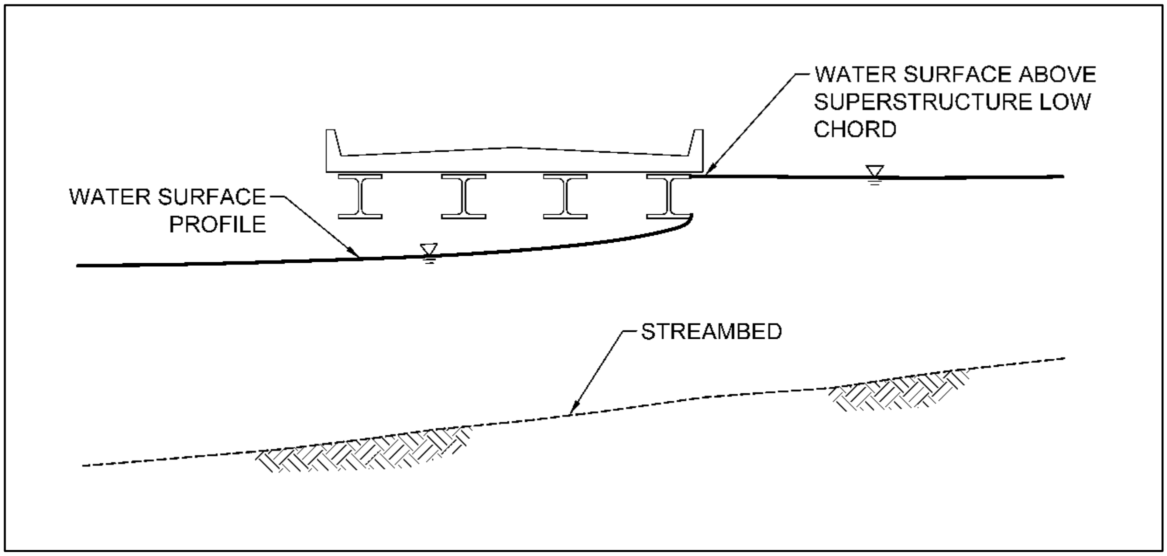

Figure 2.2. Profile-view illustration of pressure flow.

2.2.5 Bridge Piers & Abutments

Bridge piers typically have a smaller influence on crossing capacity and backwater than the bridge length, the bridge superstructure, and the road profile. However, when the flow direction does not align with the pier axis, those piers can form significant obstructions to flow that increases drag and causes undesirable backwater. Whenever possible, engineers align the pier axis to the anticipated flow direction. For piers skewed with the flow direction, the increased obstruction area of the pier increases the drag force (and thus the backwater potential) exerted by the pier. Chapter 4 provides drag coefficient values for analysis of or piers of different shapes. Piers with squared-off ends have the highest drag coefficient. Oval-shaped piers with semi-circular ends have a lower drag coefficient. Long elliptical-shaped piers have a much lower drag coefficient than either. Pier shape, dimensions, and orientation to the flow also affect pier scour. Using single, large cylindrical piers avoids the drag and scour issues related to pier alignment and is considered a best practice by many States.

Beyond the obvious relationship between abutment location and bridge length, the abutments' configuration and alignments influence the backwater potential. Spill-through abutments generally lead to less backwater than vertical abutments. Vertical abutments present an abrupt flow transition, but the engineer can mitigate this to a degree by adding angled wingwalls at the abutment corners. Abutments aligned with the flow, whether vertical or spill-through, are likely to cause less backwater than misaligned abutments. Figure 2.3 illustrates a pier and two abutments that are misaligned to the flow direction.

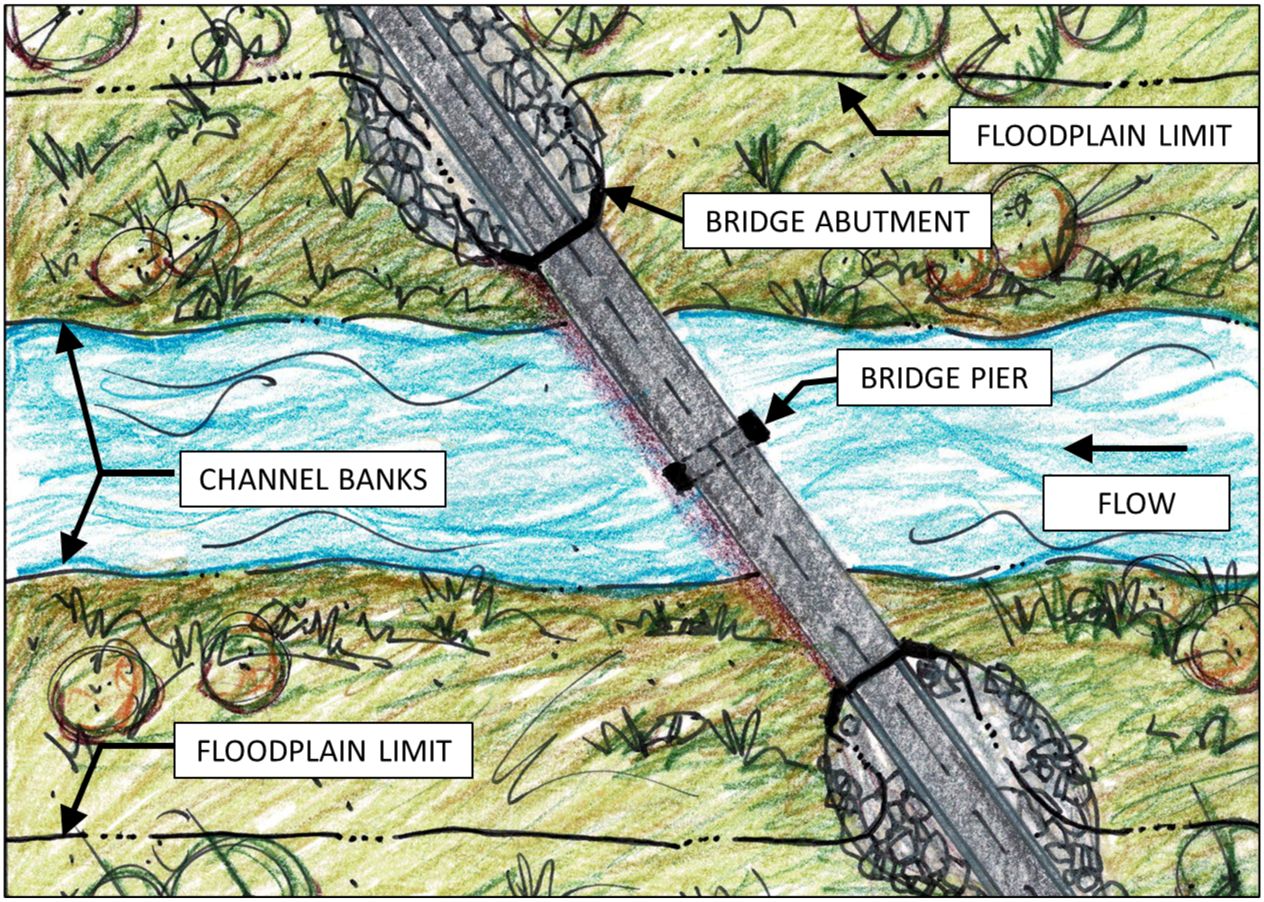

Figure 2.3. Plan-view illustration of a pier and abutments that are misaligned to the flow direction.

2.3 Scour & Stream Stability Considerations

Floods eroding bed material from the foundation of piers and abutments are the most common cause of bridge failures. The full range of flows have the potential to erode channel banks, cause the channel to shift, and cause the bed to raise or lower (aggrade or degrade). When not considered in the design process, these stream stability processes can cause bridge failures. Chapter 9 discusses scour and stream stability processes in detail, and they are the subjects of two FHWA documents: HEC-18 *Evaluating Scour at Bridges* and HEC-20 *Stream Stability at Highway Structures* (FHWA 2012b and 2012c). The main types of scour considered in the design process are scour due to channel instability (channel shift and long-term degradation), contraction scour, abutment scour, and local scour at piers.

2.3.1 Channel Instability

River channels are natural systems and have inherent variability with respect to both space and time. One expects the riverbed elevation at a given location to fluctuate over time in response to variable flow conditions. The thalweg (the lowest point in the channel bottom) may shift from side to side between the channel banks. These variations are expected in stable channel reaches. In the bridge design context, channel instability refers to potential channel changes that can threaten a bridge foundation. The engineer is concerned with potential vertical instability that could cause a long-term lowering (degradation) of the channel bed through the bridge reach and with lateral instability that could cause a threatening horizontal shift in the channel bank locations. Channel instability leads to channel geometry changes that could expose bridge foundations and increase scour potential during floods. The channel changes associated with instability may be gradual or episodic. They are treated as long-term changes because they alter the channel over the bridge's service life.

Given the freedom to choose a crossing location, a stable reach is preferable to an unstable river reach. In some cases, the alignment is shifted as an adaptation approach to improve the crossing resilience. However, there is often limited latitude in the selection of the crossing location. In this case, the bridge hydraulic engineer strives to design the crossing to accommodate apparent instabilities in the river reach. Such accommodations may include:

- Setting the bridge abutments well back from the channel banks to provide separation from the main channel.
- Designing overbank pier foundations that are near to the channel as if they were in the main channel in case of future channel migration.
- Including angle of attack in the pier design to account for channel shifting or selecting pier configurations that are insensitive to flow direction.
- Enhancing the channel bank stability by installing bank protection measures, which often need to extend beyond the highway right-of-way, both upstream and downstream.
- Accounting for channel degradation that could occur over the life of the bridge and incorporating the estimates into the bridge scour evaluation and foundation design.

2.3.2 Contraction Scour

Contraction scour is caused by constricting the floodplain flow into the bridge waterway's constrained width. The constriction leads to higher velocities and shear stresses. Contraction scour may reach its peak depth at or near the peak flow of a flood then refill as the flood recedes, leaving little or no evidence after the flood that it had occurred. Contraction scour can occur in the main channel portion of the bridge waterway and in the overbank portions between the abutments and the main channel.

Contraction scour is minimal if there is no encroachment into the floodplain by the road embankments or bridge abutments. The engineer can limit contraction scour potential by reducing the floodplain encroachment but weighs the cost trade-off of a longer bridge versus longer embankments. Increasing bridge length to reduce contraction scour has an added benefit of increasing resilience to changing basin and climate conditions.

Providing freeboard to avoid pressure flow is another design measure that reduces contraction scour potential. Pressure flow creates a vertical constriction that increases velocity, shear stress, and contraction scour at the bridge. Pressure flow also creates a flow separation area under the bridge superstructure that increases the vertical constriction. In most cases, pressure flow scour adds to the contraction scour from blocking floodplain flow.

2.3.3 Abutment Scour

Scour occurs at an abutment when the approaching road embankment obstructs the flood flow. The flow passing through the bridge from directly upstream of the abutment abruptly converges with the redirected flood flow near the upstream corner of the abutment. This condition leads to a locally high velocity and strong vortices adjacent to the abutment, causing localized scour. It also produces a wake eddy and consequent scour near the abutment's downstream corner. Even though it is a local scour phenomenon, abutment scour is integrally related to contraction scour in the bridge opening.

The most dominant influence on abutment scour potential is the amount of encroachment by the road embankment into flood flows. Abutment scour is heavily influenced by the proximity of the abutments to the main channel banks. The engineer can limit abutment scour potential by locating the abutments well back from the channel banks. In addition to the careful location of the abutments to minimize abutment scour potential, bridge designers work in interdisciplinary teams to evaluate abutment scour and account for it in either the foundation design or in designing abutment scour countermeasures that protect the foundation from this scour component. Figure 2.4 is an aerial view showing a very large remnant abutment scour hole at an interstate highway bridge. The abutment scour in Figure 2.4 is an extreme case. The embankment on the right overbank (i.e., lower right of photo) obstructs over 6,000 feet of floodplain flows.

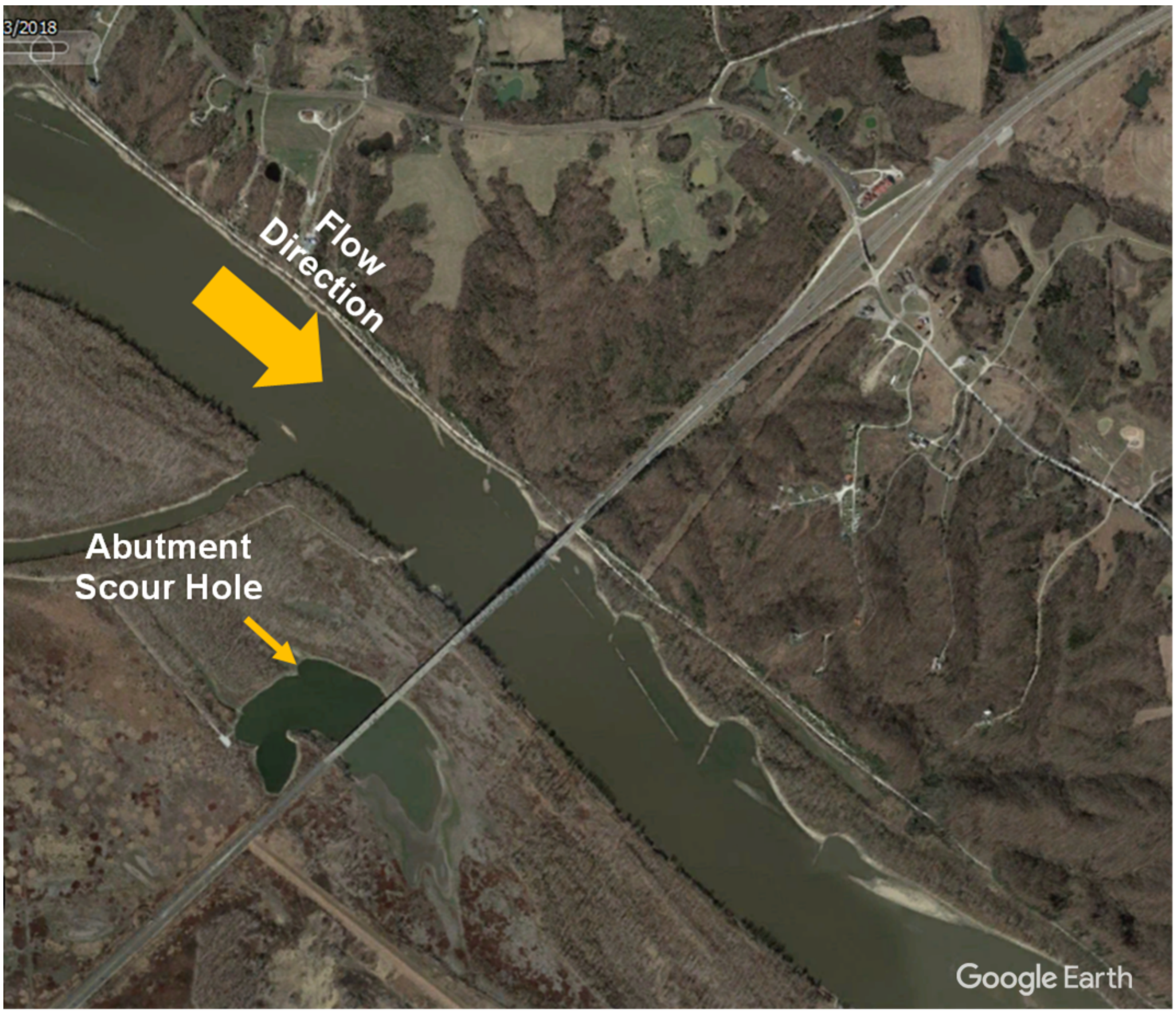

Figure 2.4. Aerial view of a remnant abutment scour hole at an interstate highway bridge.

2.3.4 Pier Scour

A bridge pier obstructs the flow, leading to local flow accelerations and vortices that produce scour at the pier's base. Pier scour potential is driven by the velocity and flow depth upstream of the pier as well as the width and shape of the pier. A pier aligned to the flow direction has less scour potential than a skewed pier, and debris accumulation on a pier also increases the scour potential. The engineer can reduce pier scour potential by aligning the pier axes with the flow direction. It is important that bridge designers consider whether the flow direction may be different for flood flows than for typical low flows contained in the main channel. Bridge engineers design new bridge piers to withstand the total estimated scour without the aid of scour countermeasures. This paradigm usually calls for deep foundations, such as driven piles or drilled shafts.

2.4 Considerations when crossing floodplains mapped under the National Flood Insurance Program

Chapter 3 includes an explanation of the regulatory basis of the National Flood Insurance Program (NFIP) administered by the Federal Emergency Management Agency (FEMA). Bridge design projects that cross or encroach on a floodplain regulated under the NFIP or local ordinances may be subject to a floodplain development permit issued by the relevant Floodplain Administrator(s). Communities that participate in the NFIP establish land-use regulations that enforce the NFIP regulations. FEMA publishes guidelines that present FEMA's standard methods for developing, maintaining, and updating floodplain and floodway mapping. The Floodplain Administrator’s actions and decisions are subject to FEMA audits for compliance with the NFIP. The Floodplain Administrator documents and justifies any divergence from FEMA's standards. An unsatisfactory audit could cause FEMA to sanction the participating community.

Floodplain Administrators typically expect designers of projects that encroach onto a mapped FEMA or local regulatory floodway to evaluate the potential impacts of the project on the Base Flood Elevations (BFE). The Floodplain Administrator may not permit encroachments onto the regulatory floodway unless a project impact evaluation using standard engineering methods demonstrates that the proposed project would cause no rise compared to existing conditions at any evaluation line (2D model) or cross-section (1D model). The bridge hydraulic engineer of record documents such a finding with a no-rise certification. *FEMA Guidance Document 79* presents typical no-rise certification practices.

If the project would cause an increase in flood risk to an insurable structure, the design is modified to mitigate the increase. The Floodplain Administrator typically has no authority to approve a project that causes an un-mitigated impact to an existing insurable structure. If the project’s impacts would cause an increase in BFEs and would not cause an impact to any insurable structure, then the Floodplain Administrator may require that any impacts be quantified prior to construction and the floodplain/floodway remapped as a condition of the floodplain development permit. This effort is a two-step process:

1. Before construction, the design team submits a request for a Conditional Letter of Map Revision (CLOMR) to quantify the project's potential impacts if constructed as designed. Construction is not authorized to begin until the CLOMR is approved by FEMA.
2. Once construction is complete, a Letter of Map Revision (LOMR) documents the actual changes to the regulatory floodplain and floodway. When approved, the LOMR is an official update to the mapping.

The CLOMR/LOMR process typically involves more modeling, stakeholder engagement, public notification, and design coordination effort than a no-rise certification. *FEMA Guidance Document 106* describes the typical CLOMR/LOMR process.

The Floodplain Administrator may also expect a LOMR submittal as a stipulation to the floodplain development permit, even if a CLOMR was not required. Whenever a LOMR is submitted, FEMA requires an as-built survey to verify the project installation, inform the updated hydraulic modeling, and accurately map the project’s actual floodplain impacts. *FEMA Guidance Document 47* describes minimum mapping standards.

If a bridge design project requires a CLOMR/LOMR, the pre-construction review cycle will likely be longer, and closing out the floodplain development permit after construction will involve more effort than the no-rise certification process. Many States primarily design bridges using the no-rise certification process to simplify floodplain permitting. However, a bridge that would require CLOMR and LOMR submittals may be significantly less expensive to construct or may have other highly desirable design features compared to a bridge that does not require a map revision. The design team ideally weighs the trade-off of a more costly project with simpler floodplain permitting and less schedule impact versus a potentially more cost-effective or beneficial project with a longer and more complex floodplain permit process.

2.5 Risk & Resilience

The FHWA document HEC-17 *Highways in the River Environment-Floodplains, Extreme Events, Risk and Resilience* (FHWA 2016) defines **risk** as the product of the probability of an undesirable event and the consequences of the event. The HEC-17 document also discusses **resilience**; defined under statute as the *“… the ability to anticipate, prepare for, or adapt to conditions or withstand, respond to, or recover rapidly from disruptions …”* [23 U.S.C. § 101(a)(24)]. The bridge design team can enhance the highway network's resilience by designing the bridge crossing to limit the damage from flooding or changing basin conditions and prevent flood-related closures of the crossing or minimize the duration and frequency of closures.

2.5.1 Design Flood Events

Highway engineering agencies in the United States have long used flood-frequency-based design standards and criteria. In so doing, they have incorporated the concept of risk into bridge hydraulic design. The flood frequency defines the probability of the undesirable event (exceedance of criteria). Design standards reference the design flood frequency as either the design recurrence interval (for instance, the 50-year flood) or the 2 percent annual exceedance probability (AEP). Typically, the design standards vary based on the facility's importance, which is categorized by route classification or traffic volume and can be influenced by the presence of other critical infrastructure. The importance of the facility represents the magnitude of consequences of a flood event exceeding the criteria. Higher traffic volumes mean more motorists exposed to safety hazards, greater mobility impacts from a road closure, and likely larger and more costly facilities to repair or replace.

In discussing bridge design flood frequencies, it is important to mention the nuance that the scour design flood is typically more severe than the hydraulic design flood, as explained in HEC-18 (FHWA 2012b). This difference is justified by the severe potential consequence of bridge foundation instability if the scour design flood is exceeded. A bridge is designed to a higher level for scour than for the hydraulic design because if the hydraulic design flood is exceeded then a greater amount of scour will occur which could lead to bridge failure rather than temporary loss of service due to road closure from overtopping. Also, designing for a higher level of scour than the hydraulic design flood ensures a level of redundancy after the hydraulic design event occurs.

2.5.2 Operational Performance of the Facility

To properly account for risk and to promote a resilient design, the design team can give thought to the performance of the crossing over its expected service life. A typical modern bridge project in the United States has a design life of many decades. The 9th edition of AASHTO's *LRFD Bridge Design Specifications* (AASHTO 2020) calls for a minimum 75-year design life. One can reasonably expect at least one and possibly multiple exceedances of the design flood in the crossing's service life. Table 2.1 from HEC-17, gives the probabilities of extreme event occurrences for various lengths of service.

Table 2.1. Probability of extreme event occurrence for various lengths of service, from HEC-17.

Service Life (years)	10-yr Flood (AEP 10%)	25-yr Flood (AEP 4%)	50-yr Flood (AEP 2%)	100-yr Flood (AEP 1%)	500-yr Flood (AEP 0.2%)
1	0.10	0.04	0.02	0.01	0.002
10	0.65	0.34	0.18	0.10	0.02
25	0.93	0.64	0.40	0.22	0.05
50	0.99	0.87	0.64	0.39	0.10
75	1.00	0.95	0.78	0.53	0.14
100	1.00	0.98	0.87	0.63	0.18

Consider a bridge with a 75-year design life and a hydraulic design based on the 50-year design flood (AEP = 2 percent). Table 2.1 shows that the bridge has a 78 percent probability of experiencing an exceedance of its design flood at least once. This example illustrates that a bridge is likely to experience one or more exceedances of its hydraulic design flood during its service life. Exceeding the hydraulic design flood does not mean that the bridge has failed or necessarily even loss of service. Even exceeding the scour design flood does not mean bridge failure will occur. The hydraulic engineer contributes to a resilient design by considering how the crossing will perform in events exceeding the design flood.

2.5.3 Nonstationarity: Evolution of Conditions over the Service Life

A highway crossing's flood risk may change during its service life. Nonstationarity in this context refers to changes in the watershed's land use or in climate. Urbanization that replaces permeable surfaces in the watershed with pavements and rooftops is an example of nonstationarity. Climate changes that lead to increased annual rainfall or increased frequency of intense rainfall are another example. Yet another example is the abrupt change in flood risk that could come with a dam removal upstream of the site. HEC-17 provides information and techniques for detecting nonstationarity and adjusting for it in hydrologic analysis.

The risk conditions at a bridge crossing could change over time for other reasons besides hydrologic nonstationarity. Long-term channel changes could raise or lower the streambed, the former reducing the hydraulic capacity and the latter increasing scour risk. Debris could accumulate at a pier or in a span between piers if the bridge owner's maintenance resources are insufficient. Bridge designs that accommodate channel shifting address this type of nonstationarity. Future unrelated projects could result in new hydraulic controls downstream that would raise the flood profile for a given flowrate. Other projects could remove existing downstream channel bed controls and initiate channel degradation at the bridge.

The bridge owner could make a future decision to upgrade the functional classification of the route carried by the bridge. By increasing the consequence of design flood exceedance, that decision would increase the risk even with no change in the probability of exceedance.

It would be impracticable to anticipate and design for all possible non-stationarities and changes in conditions that could increase the risk to the crossing. However, investing some time and study toward identifying the more likely possible future changes or evolutions is a good practice and well justified.

2.5.4 Strategies for Resilience

Resilient designs can recover from, adjust to, or withstand exposure to flood events (FHWA 2016). Examples of resilient service life performance of a crossing include but are not limited to:

- Sustaining exposure to the hydraulic design discharge with no occurrence of pressure flow or road overtopping.
- Limiting backwater that could harm upstream properties in floods equal to the 1 percent AEP flood. In some cases, this may involve reducing existing backwater potential versus existing conditions.
- Avoiding redistribution of flow compared to natural conditions that could cause harm to downstream properties.
- Surviving a flood discharge greater than the scour design flood without catastrophic failure of the bridge.
- Withstanding storm surge and severe wave conditions in a hurricane.
- Promoting complete reopening shortly after the passage of a flood that overtopped the roadway.
- Accommodating channel changes due to natural processes within the bridge waterway without risk of foundation problems.

A current practice in bridge scour design provides an example of a resilience strategy. Bridge designers typically consider both a scour design flood and a scour check flood. Properly designed bridges withstand the scour design flood at the service limit state (able to remain in service) and withstand the scour check flood at the extreme limit state (not open to traffic but still intact). This practice provides a good measure of resilience in the bridge's foundation stability.

Analyzing the hydraulic performance of a crossing for a flood exceeding the hydraulic design flood is another practice that promotes resilience. For example, if a bridge has a 2 percent AEP hydraulic design flood, one also performs a hydraulic analysis for the 1 percent AEP flood. The engineer is responsible for assessing the bridge hydraulic performance to ensure the project would cause no harm to neighboring properties and that the crossing would not sustain significant damage in a flood of that magnitude.

Designing the bridge with freeboard is an example of a resilience strategy as it accommodates the passage of a flood larger than the design flood and can also accommodate floating debris. As stated in 23 CFR § 650.115(a)(3) “Freeboard shall be provided, where practicable, to protect bridge structures from debris- and scour-related failure.” Without adequate freeboard, debris may collect along the deck and reduce flow conveyance. Increasing the freeboard height beyond the standard requirements is justified when hydrologic nonstationarity is detected or reasonably anticipated. One can promote the crossing's resilience in the event of overtopping by protecting the embankment and pavement from scour damage along the lowest segment of the road embankment, where the greatest overtopping would occur.

CHAPTER 3
REGULATORY REQUIREMENTS FOR BRIDGE HYDRAULIC DESIGN

Federal policy for highways is at the nexus of two broad Federal policy arenas:

- Highway engineering.
- River management.

Each has its own evolving history that influences the highway system throughout the country. This chapter provides background on these policy arenas, provides applicable highway engineering statutes and regulations, and provides an overview of other Federal statutes and regulations that may affect bridge hydraulic design projects.

3.1 Federal Highways & Rivers: National Overview

The FHWA has the primary responsibility for Federal policy on highways. Legislation for the Federal road system dates back over a century. The Federal-Aid Road Act of 1916 created the Federal-Aid Highway Program, which funded State highway agencies so they could make road improvements “to get the farmers out of the mud.” This 1916 Act charged the Bureau of Public Roads with implementing the program. The growth of the Federal highway system, including the addition of the Interstate Highway System and concerns about how all of these highways affected the environment, city development, and the ability to provide public mass transit, led to the 1966 establishment of the United States Department of Transportation (USDOT). The same enabling legislation renamed the Bureau of Public Roads to the FHWA. Currently, the FHWA continues to administer United States Federal policy on highways and coordinates extensively with other Federal agencies on environmental policies and permits, floodplains, and other compliance issues related to highway program and project delivery.

In contrast, United States Federal policy on river management is not concentrated in any one agency but dispersed over several according to their historical missions. The Federal Emergency Management Agency (FEMA) oversees the National Floodplain Insurance Program (NFIP). The United States Fish and Wildlife Service (USFWS) and the National Oceanic and Atmospheric Administration’s (NOAA) National Marine Fisheries Service (NMFS) administer and enforce the Endangered Species Act (ESA). Almost every project involving work or activities in rivers is subject to the Clean Water Act (CWA) of 1972, which is administered by the United States Environmental Protection Agency (USEPA) in coordination with State governments.

3.2 Highway Statutes & Regulations

The FHWA provides financial and technical assistance to State and local governments to ensure that United States roads and highways continue to be among the safest and most technologically sound in the world. The FHWA authority for this document's subject matter includes the following statutes and regulations. The section below provides a synopsis of these various authorities as well as pertinent Congressional findings and statements, policy, and guidance.

3.2.1 FHWA Statute

The FHWA operates under the statutory authority of Title 23 (Highways) of the United States Code (U.S.C.). Sections relevant to this document include:

- **Standards [23 U.S.C. § 109].** It is the intent of Congress that Federally funded projects to resurface, restore, and rehabilitate highways shall *"be constructed in accordance with standards to preserve and extend the service life of highways and enhance highway safety"* [23 U.S.C. § 109(n)]. Designs for new, reconstructed, resurfaced, restored, or rehabilitated highways on the National Highway System must consider, among other criteria, the *"constructed and natural environment of the area."* [Id. at (c)(1)(a)].
- **Maintenance [23 U.S.C. § 116].** Preventive maintenance is eligible for Federal assistance under Title 23 if a State department of transportation can demonstrate that it is a "cost-effective means of extending the useful life of a Federal-aid highway." [23 U.S.C. § 116(e)]
- **National Highway Performance Program [NHPP] [23 USC. § 119].** The NHPP allows FHWA to provide Federal-aid funds for *"construction, replacement …, rehabilitation, preservation, and protection (including … protection against extreme events) of bridges on the National Highway System"* [23 U.S.C. § 119(d)(2)(B)]. The NHPP also allows Federal-aid funds for *"[construction, replacement …, rehabilitation, preservation, and protection (including … protection against extreme events) of tunnels on the National Highway System"* [Id. at (d)(2)(C)].
- **Surface Transportation Block Grant [STBG] Program [23 U.S.C. § 133].** The STBG program allows FHWA to provide Federal-aid funds for protection of *"bridges (including approaches to bridges and other elevated structures) and tunnels on public roads"* including *"painting, scour countermeasures, seismic retrofits, impact protection measures, security countermeasures, and protection against extreme events."* [23 U.S.C. § 133(b)(9)]. The STBG program also allows Federal-aid funds for "inspection and evaluation of bridges and tunnels and other highway assets." [Id.]
- **Metropolitan transportation planning [23 U.S.C. § 134].** In the context of metropolitan transportation planning, Congress has found that it *"is in the national interest … to encourage and promote the safe and efficient management, operation, and development of surface transportation systems … within and between States and urbanized areas"* including taking "resiliency needs" into consideration. [23 U.S.C. § 134(a)(1)].
- **National bridge and tunnel inventory and inspection standards [23 U.S.C. § 144]**. Congress has found that *"continued improvement to bridge conditions is essential to protect the safety of the traveling public."* [23 U.S.C. § 144(a)(1)(A)]. Congress has further found that "the systematic preventative maintenance of bridges, and replacement and rehabilitation of deficient bridges, should be undertaken." [Id. at (a)(1)(B)]. Congress has also declared that *"it is in the vital national interest"* to use a "data-driven, risk-based approach" toward meeting these ends." [Id. at (a)(2)(B)]. Considering these findings and declarations, Section 144 requires FHWA to maintain an inventory of bridges and tunnels on public roads both "on and off Federal-aid highways." [Id. at (b)]. The FHWA is also required to *"establish and maintain inspection standards for the proper inspection and evaluation of all highway bridges and tunnels for safety and serviceability."* [Id. at (h)(1)(A)] Section 144 also provides an exception to the requirement to obtain a bridge permit from the United States Coast Guard for certain bridges over a limited subset of navigable waters. [Id. at (c)(2)].
- **National goals and performance management measures [23 U.S.C. § 150].** Congress has declared that it is *"in the interest"* of the United States to focus the Federal-aid highway program on certain national transportation goals, including Infrastructure Condition, or the objective to *"maintain highway infrastructure in a state of*

good repair;" and System Reliability, or the objective to *"improve the efficiency of the surface transportation system."* [23 U.S.C. § 150(b)].

- **Research and technology development and deployment [23 U.S.C. § 503].** In carrying out certain highway and bridge infrastructure and research and development activities, FHWA must *"study vulnerabilities of the transportation system to … extreme events and methods to reduce those vulnerabilities."* [23 U.S.C. § 503(b)(3)(B)(viii)].

3.2.2 FHWA Regulations

The FHWA's regulations are found within the Code of Federal Regulations (CFR), Title 23, Highways (23 CFR). The FHWA requires compliance with Federal law and the regulations in Chapter I, Subchapter A, Part 1 of 23 CFR for a project to be eligible for Federal-aid or other FHWA participation or assistance. [23 CFR § 1.36]. The following FHWA regulations apply to highway projects and actions interacting with and within rivers and floodplains (paraphrased for brevity):

Scope of the statewide and nonmetropolitan transportation planning process [23 CFR § 450.206]. State Departments of Transportation (DOTs) must "carry out a continuing, cooperative, and comprehensive statewide transportation planning process that provides for consideration and implementation of projects, strategies, and services that will … improve the resiliency and reliability of the transportation system. [23 CFR § 450.206(a)].

Transportation Asset Management Plans (TAMPs) [23 CFR Part 515]. Part 515 establishes processes that a DOT must use to develop an asset management plan. Two notable sections include:

- **Section 515.7(b).** "A State DOT shall establish a process for conducting life-cycle planning for an asset class or asset sub-group at the network level (network to be defined by the State DOT). As a State DOT develops its life-cycle planning process, the State DOT should include future changes in demand; information on current and future environmental conditions including extreme weather events, climate change, and seismic activity; and other factors that could impact whole of life costs of assets."
- **Section 515.7(c).** "A State DOT shall establish a process for developing a risk management plan. This process shall, at a minimum, produce the information: Identification of risks that can affect condition of NHS pavements and bridges and the performance of the NHS, including risks associated with current and future environmental conditions, such as extreme weather events, climate change, seismic activity, and risks related to recurring damage and costs as identified through the evaluation of facilities repeated damaged by emergency events carried out under part 667 of title 23 of the CFR. Additional information that must be produced is specified in the regulation at 23 CFR 515.7(c)."

Design Standards [23 CFR Part 625]. Part 625 describes structural and geometric design standards.

- **Sections 625.3(a)(1), 625.3(b) and § 625.4(b)(3).** The FHWA, in cooperation with DOTs, has approved the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications. Based on FHWA's approval, certain National Highway System (NHS) projects must follow those Specifications, including sections related to hydrology, hydraulics, and bridge scour.
- **Section 625.3(a)(2).** Non-NHS projects must follow DOT standard(s) and specifications on drainage, bridges, and other topics.

Location and Hydraulic Design of Encroachments on Flood Plains [23 CFR Part 650, Subpart A]. One of the FHWA's most important river-related regulations, 23 CFR Part 650, Subpart A, sets forth policies and procedures for location and hydraulic design of highway encroachments in base (1-percent chance) floodplains. Section 650.111 sets forth requirements for location hydraulic studies to identify the potential impact of the highway alternatives on the base floodplain; these studies are commonly used during NEPA. The regulations prohibit significant encroachment unless FHWA determines that such encroachment is the only practicable alternative. [23 CFR § 650.113(a)]. This finding must be included in the NEPA documents for a project and supported information, including the reasons for the finding and considered alternatives. [Id]. The procedures also provide minimum standards for Interstate Highways, set freeboard requirements to account for debris and scour, and require highway encroachments to be consistent with certain established design flow standards for hydraulic structures, including standards from FEMA and State and local governments related to administration of the National Flood Insurance Program (NFIP). [23 CFR § 650.115(a)]. Notably, this Subpart's policies and procedures apply to encroachments in all base floodplains, not just the floodplains regulated by the Federal Emergency Management Agency (FEMA) in the NFIP. [23 CFR § 650.107]. Additionally, the Subpart incorporates a requirement for project-by-project risk assessments or analyses. [23 CFR § 650.115(a)(1)]. Two notable sections include:

- **Section 650.115 [Hydraulic Design Standards].** This regulation applies to all Federal-aid projects, whether on the NHS or Non-NHS. Federal, State, local, and AASHTO standards may not change or override the design standards set forth under § 650.115 — although certain State and local standards must also be satisfied under that section. That section requires development of a "Design Study" for each highway project involving an encroachment on a floodplain. [23 CFR § 650.115(a)].
- **Section 650.117 [Content of Design Studies].** This regulation requires studies to contain the "hydrologic and hydraulic data and design computations." [23 CFR § 650.117(b)]. As both hydrologic and hydraulic factors and characteristics lead to scour formation, data and computations applicable to scour should be provided as well. Project plans must show the water surface elevations of the overtopping flood and base flood (i.e., 100-year flood) if larger than the overtopping flood. [23 CFR §650.117(c)].

National Bridge Inspection Standards (NBIS) [23 CFR § 650 Subpart C]. This regulation implements requirements found in 23 U.S.C. § 144. In addition to the inspection and inventory requirements, the regulation specifically focuses on scour at bridges. The regulation applies to both new and existing bridges. It establishes definitions of scour appraisal, scour assessment and scour evaluation, which have been included in the glossary of this second edition of HDS-7.

Mitigation of Impacts to Wetlands and Natural Habitat [23 CFR § 777]. This regulation provides policy and procedures for evaluating and mitigating adverse environmental impacts to wetlands and natural habitat resulting from Federal-aid funded projects.

3.3 Other Federal Agency Statutes & Regulations

Civil engineering projects in the river environment are subject to numerous Federal laws, policies, and regulations. This section describes some of the common Federal statutes, regulations, and other authoritative guidance that may govern highway projects.

3.3.1 Rivers and Harbors Act of 1899 [33 U.S.C. § 401 and § 403]

River and coastal highway engineering projects are subject to Section 9 [33 U.S.C. § 401] and Section 10 [33 U.S.C. § 403] of the Rivers and Harbors Act of 1899. Section 9 of this act restricts the construction of any bridge, dam, dike, or causeway over or in United States navigable waterways. With the exception of bridges and causeways under Section 9 [33 U.S.C. § 401], the United States Army Corps of Engineers (USACE) is responsible for maintaining the standards set by and for issuing permits under the Rivers and Harbors Act. Authority to administer Section 9, applying to bridges and causeways, was redelegated to the United States Coast Guard under the provisions of the Department of Transportation Act of 1966 (as discussed below).

3.3.2 General Bridge Act of 1946 [33 U.S.C. § 525 through 533]

The General Bridge Act of 1946 requires the location and plans of bridges and causeways across the navigable waters of the United States to be submitted to and approved by the United States Coast Guard prior to construction. [33 U.S.C. § 525]. The USACE may also impose conditions relating to the structure's maintenance and operation. [Id]. The General Bridge Act of 1946 is cited as the legislative authority for bridge construction in most cases. Although the General Bridge Act of 1946 originally provided authority for issuing bridge permits to the USACE, subsequent legislation transferred these responsibilities from the USACE to the United States Coast Guard (USCG).

3.3.3 Transportation Act of 1966 [Public Law 89-670]

The Transportation Act of 1966 transferred the United States Coast Guard (USCG) to USDOT (subsequent legislation has since transferred the USCG to the Department of Homeland Security). One of USCG’s assigned duties in the Transportation Act of 1966 was to issue bridge permits. This legislation, along with the Rivers and Harbors Act and General Bridge Act, made the USCG responsible for ensuring that bridges and other waterway obstructions do not interfere with the navigability of waters of the United States without the express permission of the United States Government. Subsequent legislation amended 23 U.S.C. § 144 to provide certain exceptions to USGC’s authority under 33 U.S.C. § 401 and 33 U.S.C. § 525 for bridges constructed, reconstructed, rehabilitated, or replaced using Federal-aid funds. [23 U.S.C. § 144(c)(2)].

3.3.4 National Environmental Policy Act (NEPA) [42 U.S.C. § 4321, et seq.]

The National Environmental Policy Act of 1969 (NEPA) establishes the continuing policy of the Federal government to use all practicable means and measures “to foster and promote the general welfare, … create and maintain conditions under which man and nature can exist in productive harmony, and fulfill the social, economic, and other requirements of present and future generations of Americans [42 U.S.C. § 4331]. To achieve this goal, NEPA creates a requirement for Federal agencies to consider the environmental impacts of their actions before undertaking them [42 U.S.C. § 4332(C)].

Section 102(2)(C) of NEPA requires Federal agencies develop a detailed statement on proposals for major Federal actions significantly affecting the quality of the human environment [42 U.S.C. § 4332(C)]. Environmental impact statements address items including *“the environmental impact of” and “alternatives to” the proposed action.”* [Id.] FHWA implements NEPA according to the Council on Environmental Quality (CEQ) NEPA regulations at 40 CFR Part 1500 et seq. and the FHWA-FRA-FTA joint regulations at 23 CFR Part 771.

3.3.5 Clean Water Act [33 U.S.C. § 1251-1387]

Almost every project involving work or activities in rivers is subject to the Clean Water Act (CWA) of 1972, which is administered by the United States Environmental Protection Agency

(USEPA) in coordination with State governments. The CWA is the primary Federal statute governing the protection of the Nation's surface waters. Engineering of highways in the river environment is often subject to Section 404 of the CWA, which regulates the discharge of dredged or fill material in waters of the United States, including wetlands. [33 U.S.C. § 1344]. This includes the use of dredged or fill material for development, water resource projects, and infrastructure development (e.g., roads, bridges, etc.). The USACE handles the day-to-day administration and enforcement of the Section 404 program, including issuing permits. In circumstances where Section 404 is triggered, permit applicants also obtain a Section 401 certification from the State in which the discharge of dredged or fill material originates. [13 U.S.C. § 1341]. The Section 401 certification assures that discharge of materials to waters of the United States complies with the CWA's relevant provisions, including water quality standards. In addition, Section 402 of the CWA establishes the National Pollutant Discharge Elimination System (NPDES) Program. [33 U.S.C. § 1342]. The NPDES Program requires a permit for discharges of pollutants into waters of the United States, including storm water discharges.

3.3.6 Endangered Species Act [16 U.S.C. § 1531-1544]

Highway engineering projects have the potential to impact fish, wildlife, and marine mammals. The purposes of the Endangered Species Act of 1973 (ESA) include conserving "ecosystems upon which endangered species and threatened species depend may be conserved" and providing "a program for the conservation of such endangered species and threatened species." [16 U.S.C. § 1531]. The United States Fish and Wildlife Service (USFWS) and the NOAA National Marine Fisheries Service (NMFS) administer and enforce the ESA. The USFWS and NMFS review public notices and environmental documents (e.g., Environmental Assessments, Environmental Impact Statements) provided by the lead Federal agency for compliance with the ESA. They also conduct consultations with the lead Federal agency when a proposed project may affect Federally endangered or threatened species. USFWS or NMFS involvement in a project depends on the affected species and the nature and extent of anticipated impacts (direct and indirect) to that species and its designated critical habitat. If anticipating a "take" of a Federally listed species, USFWS or NMFS issues a biological opinion, the terms and conditions of which are generally binding on the lead Federal agency. [16 U.S.C. § 1536.]

3.3.7 Wild and Scenic Rivers Act [16 U.S.C. § 1271 et seq.]

This Act establishes a policy to preserve designated rivers *"in free-flowing condition"* and to protect *"their immediate environments … for the benefit and enjoyment of present and future generations."* [16 U.S.C. § 1271] Section 7(a) provides that *"no department or agency of the United States shall assist by loan, grant, license, or otherwise in the construction of any water resources project that would have a direct and adverse effect on the values for which such river was established."* [16 U.S.C. § 1278(a)] A water resources project is *"any dam, water conduit, reservoir, powerhouse, transmission line, or other project works under the Federal Power Act . . . or other construction of developments which would affect the free-flowing characteristics of a Wild and Scenic River or Study River."* [36 CFR 297.3] *"Federal assistance means any assistance by an authorizing agency including, but not limited to, . . . [a] license, permit, or other authorization granted by the Corps of Engineers, Department of the Army, pursuant to the Rivers and Harbors Act of 1899 and section 404 of the Clean Water Act (33 U.S.C. 1344)."* [Id.]

3.3.8 Fish and Wildlife Coordination Act [16 U.S.C. §§ 661-666c]

The Fish and Wildlife Coordination Act (FWCA) requires adequate consideration for the "conservation, maintenance, and management of wildlife resources" whenever the *"waters of* *any stream or other body of water are impounded, diverted, the channel deepened, or the stream or other body of water otherwise controlled or modified for any purpose … including navigation and drainage, by any department or agency of the United States.* [16 U.S.C. § 663(a)] This generally includes consultation with the USFWS, the NMFS, and State wildlife agencies for activities that affect, control, or modify waters of any stream or bodies of water in order to minimize the adverse impacts of such actions on fish and wildlife resources and habitat. This consultation is generally incorporated into the process of complying with Section 404 of the Clean Water Act, NEPA, or other Federal permit, license, or review requirements.

3.3.9 National Historic Preservation Act [54 U.S.C. 300101 et seq.]

Highway engineering projects are often subject to the National Historic Preservation Act of 1966 (NHPA). Section 106 of the National Historic Preservation Act (NHPA) (commonly called "Section 106") requires Federal agencies to consider the impacts of projects that they carry out, approve, or fund on historic properties. [54 U.S.C. § 306108]. The implementing regulations for the Section 106 process are found in 36 CFR part 800. Those regulations provide that Federal agencies, in consultation with State Historic Preservation Officers (SHPO) and certain other interested parties, identify and assess adverse effects to historic properties and seek ways to avoid, minimize, or mitigate those effects. [36 CFR § 800.4-800.6]. Under Section 106, "historic property" is defined as any prehistoric or historic district, site, building, structure, or object included in, or eligible to be included in, the National Register of Historic Places [36 CFR 800.16(l)(1); see also 54 U.S.C. 300311 and 302102]. The particular responsibilities of SHPOs are set forth at 54 U.S.C. § 302303.

In addition to Section 106, Section 4(f) of the United States Department of Transportation Act of 1966 [23 U.S.C. 138 and 49 U.S.C. 303] requires that FHWA not approve the use of historic sites for a project unless there is no prudent and feasible alternative and the project incorporates all possible planning to minimize harm, or any impacts to historic sites are determined to be de minimis. The FHWA's regulations for implementing Section 4(f) are found at 23 CFR part 774.

3.3.10 National Flood Insurance Act of 1968 [42 U.S.C. § 4001 et seq.]

The National Flood Insurance Act of 1968 instituted the National Flood Insurance Program (NFIP) to help protect the United States against flood losses. The NFIP adopted the 100-year flood as the standard, or base flood, for mapping United States floodplains. See, e.g., 44 CFR § 9.4 (defining *"base floodplain"*). The area inundated by the 100-year flood determines the Special Flood Hazard Area (SFHA) on Flood Insurance Rate Maps (FIRMs) developed by FEMA and used to determine flood insurance rates for structures. FEMA implements the NFIP using its regulations found in 44 CFR. See, for example, 44 CFR § 59.1 (defines "area of special flood hazard" and "regulatory floodway") and 44 CFR § 60.3 (defines *"regulatory floodway,"* defines floodway *"encroachments including fill, new construction, substantial improvements and other development,"* establishes the *"Conditional/Final Letter of Map Revision"* (CLOMR/LOMR) processes detailed in 44 CFR § 65.12 and explained in FEMA Guidance Documents 79 and 106; see below, and establishes the *"no-rise"* exception to CLOMR/LOMR requirements).

The FHWA's policies require bridge replacement projects to be consistent with the standards and criteria in the NFIP, where appropriate. See 23 CFR § 650.115. To assist DOTs in complying with this policy, FHWA developed coordination procedures for Federal-aid highway projects with encroachments in NFIP regulated floodplains. FEMA agreed to these procedures by signing a 1982 Memorandum of Understanding (MOU) with FHWA. Chapter 2 includes a discussion of considerations for projects crossing NFIP floodplains.

CHAPTER 4
PRINCIPLES OF RIVER & FLOODPLAIN HYDRAULICS

4.1 Introduction

This chapter provides background on the fundamentals of fixed-bed open channel flow. In open channel flow, the water is in contact with the atmosphere, the channel boundary, vegetation, and various obstructions. Because there is a free surface, flow depends primarily on gravity and has an additional degree of freedom compared to closed conduits flowing full. This chapter provides sufficient information to act as a reference on flow classification and equations commonly used in open channel and bridge hydraulics. There are many textbooks available for more in-depth investigations of this topic, including Chow (1959), Henderson (1966), and Chaudhry (2008).

Engineers predominantly perform open channel flow and bridge hydraulic calculations using computer programs, but they can also use online calculators or spreadsheet tools. Computer software performs the computations based on the engineer entering channel, floodplain, and structure geometries, surface roughness, boundary conditions (boundary discharges and water surfaces), and other parameters. The engineer's work is not finished when the program executes and provides results. It is necessary to review the results for reasonableness and correctness. No one can check every calculation for most hydraulic simulations but accepting results without review treats the model as a black box. With a suitable understanding of flow classification and the fundamental equations of open channel flow, the modeler can:

- review and interpret results,
- check questionable results,
- predict how changes in variables can affect results,
- provide explanations of the results to others,
- identify the most important variables and factors for specific situations,
- identify why one model or approach is preferred over another, and
- identify how model assumptions affect results.

The following sections emphasize how open channel flow is (1) uniform or nonuniform, (2) steady or unsteady, and (3) subcritical or supercritical. A fourth flow classification, laminar versus turbulent, is not discussed because laminar flow has few practical applications in open channels as it occurs from a combination of extremely slow and shallow flows. Most open channel flow is turbulent.

The fundamental equations of open channel flow include (1) continuity (conservation of mass), (2) energy (conservation of energy), and (3) momentum (conservation of linear momentum in two or three dimensions). Solving these equations is a primary reason for the range of software programs available to hydraulic engineers and other modelers.

This chapter also discusses other important equations for open channel flow and bridge hydraulic calculations. These include (1) flow resistance, (2) drag forces, (3) weir flow, and (4) orifice flows. Flow resistance plays a crucial role in addressing flow classification and in the solutions of both the energy and momentum equations. At bridge crossings, piers produce drag forces, roads can act like weirs, and submerged bridge decks can act like orifices.

4.1.1 Definitions

Velocity: For the purposes of this discussion, the velocity of a fluid particle is the rate of displacement of the particle from one point to another and, as shown in Figure 4.1, is a vector quantity having both magnitude and direction. The mathematical formulation of velocity magnitude is:

$$V = \frac{ds}{dt}. \tag{4.1}$$

where:

V	=	flow velocity, ft/s
S	=	distance along the particle flow path, ft
t	=	time, s

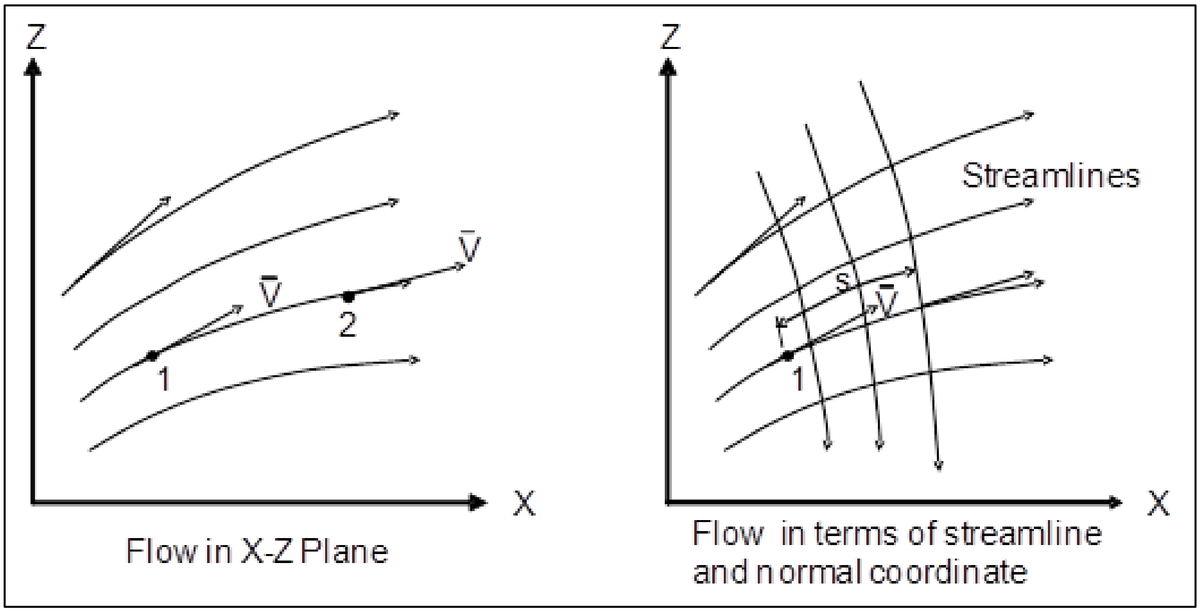

Figure 4.1. Flow in the X-Z plane and flow in terms of streamline and normal coordinates.

Streamline: A streamline is an imaginary line within the flow that is tangent everywhere to the velocity vector, see Figure 4.1. Since the flow is tangent to the streamline, there cannot be any net movement of fluid across the streamline in any direction.

Velocity is a vector quantity with a magnitude and direction. As the particle moves it follows a particular path which is governed by its velocity vector. The location of the particle along the path is a function of where the particle started and velocity along the path. If the flow is steady (i.e., nothing changes with time at a given location in the flow field), each successive particle that passes through a given point such as point (1) in Figure 4.1, follows the same path. For such cases the path is a fixed line in the X-Z plane. Neighboring particles that pass either side of point (1) follow their own paths, which may be of different shape but do not cross the path passing through (1). The entire X-Z plane is filled with such paths.

Streamtube: A streamtube is an element of fluid bounded by a pair of streamlines that enclose or confine the flow. Since there can be no net movement of fluid across a streamline, it follows that there can be no net movement of fluid in or out of the streamtube, except at the ends. The development of the continuity equation uses this fact.

For steady flow each particle progresses along its path and its velocity vector is everywhere tangent to the path. The lines that are tangent to the velocity vectors throughout the flow field are called streamlines. For many situations it is easiest to describe the flow in terms of the streamline coordinates based on the streamlines as shown in Figure 4.1. The particle motion is described in terms of its distance along the streamline. The distance along the streamline is related to the particle speed and the radius of curvature is related to the shape of the streamline.

Acceleration: Acceleration is the time rate of change in magnitude or direction of the velocity vector. Acceleration is expressed by the total derivative of the velocity vector as follows:

$$a = \frac{dv}{dt} \tag{4.2}$$

where:

a = flow acceleration, ft/s^2

The vector acceleration, a, has components both tangential and normal to the streamline, the tangential component representing the change in magnitude of the velocity, and the normal component reflecting the change in direction:

$$a_s = \frac{dv_s}{dt} = \frac{\partial v_s}{\partial t} + 1/2\frac{\partial(v_s^2)}{\partial s} \tag{4.3}$$

$$a_n = \frac{dv_n}{dt} = \frac{\partial v_n}{dt} + \frac{(v^2)}{r} \tag{4.4}$$

where:

variables with subscripts s are along, or tangential, to the streamline,

variables with subscripts n are across, or normal, to the streamline, and

r = local radius of curvature of the flow path, ft

The first partial derivative terms in Equations 4.3 and 4.4 represent the change in velocity, both magnitude and direction, with time at a given point. This is called local acceleration. The second term in each equation, called the convective acceleration, is the change in velocity, both magnitude and direction, with distance.

4.2 Open-Channel Flow Classifications

Open channel flow lends itself to several types of classifications. One flow classification is steady versus unsteady. Flow can also be classified as uniform or non-uniform (varied). Non-uniform is either gradually varied or rapidly varied. The flow can also be subcritical or supercritical, and depending upon the turbulence of the flow field, the flow is either laminar or turbulent. Since nearly all open channel flow situations are turbulent, the following sections do not discuss laminar flow. For further clarification of laminar and turbulent flow in open channels the reader is directed to Henderson (1966), Chow (1959), and Fundamentals of Fluid Mechanics by Munson et al. (2010).

4.2.1 Steady versus Unsteady Flow

Steady flow occurs when discharge is not changing with time at a channel location. Conversely, flow is unsteady when discharge is changing with time. Floods, with distinct rising limbs, peak flow conditions, and falling limbs, rarely produce steady conditions. However, engineers typically perform riverine bridge hydraulic analyses as steady-flow analyses of the peak discharge. This is because the peak discharge dominates bridge hydraulic results rather than storage and other unsteady effects. Unsteady flow analyses also involve substantially more data and computations. Therefore, as discussed in Chapter 8, unsteady flow analyses are less common for riverine bridge hydraulics though very common for coastal bridges where tides and storm surges drive hydraulic conditions.

4.2.2 Uniform, Gradually Varied, and Rapidly Varied Flow

Uniform flow occurs in open channels when there is no change in hydraulic conditions with distance along the channel. For uniform flow conditions to exist, not only does the channel have to be prismatic (unvarying channel geometry and slope), but the roughness, discharge, depth, and velocity cannot vary along the channel. Artificial channels are often prismatic though natural channels are rarely so. Figure 4.2 illustrates uniform flow in a channel with constant velocity, depth, and channel bed slope. The figure includes two other profile lines: the water surface and the energy grade line. The slopes of the water surface, or hydraulic grade line (HGL), and energy grade line (EGL) are equal to the bed slope in uniform flow conditions. The slope of the energy grade line is the rate of energy dissipation along the channel and is referred to as the energy or friction slope.

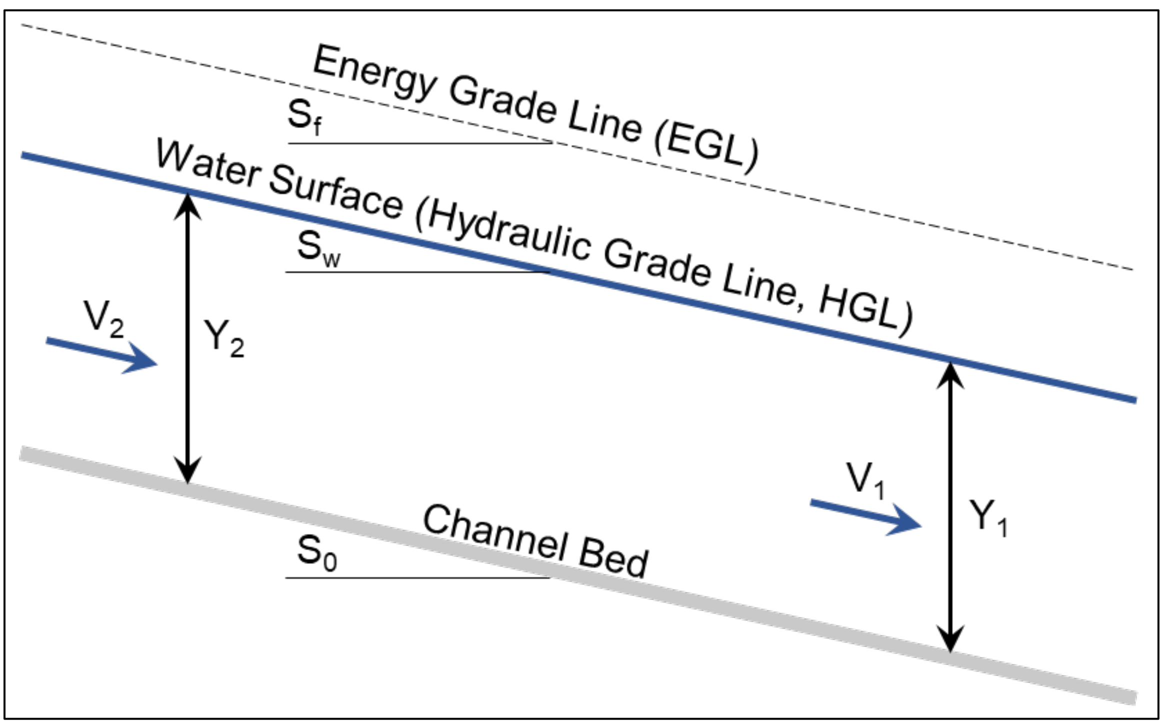

Figure 4.2. Example of uniform flow.

For a fluid at rest, the pressure at a point below the surface (assuming the surface is exposed to the atmosphere) is equal to the distance below the surface (depth of submergence) multiplied by the unit weight of the fluid. This is defined as the hydrostatic pressure for an incompressible flow. Hydrostatic pressure is the water pressure due only to gravity. No shear stresses act on the fluid or the boundary since the fluid is at rest. The equation for hydrostatic pressure is:

$$P = \rho g y = \gamma y \tag{4.5}$$

where:

P	=	water pressure, lb/ft^2
ρ	=	water density, slugs/ft^3
g	=	acceleration due to gravity, ft/s^2
y	=	water depth from the water surface, ft
γ	=	unit weight of water, lb/ft^3

For uniform flow, the pressure would also be hydrostatic since the streamlines in uniform flow are all parallel to one another and there are no vertical accelerations. Uniform flow conditions are rare in natural channels. Flows through bridge reaches cannot be uniform when floodwaters extend across the floodplain, converge into the bridge opening, and expand back into the floodplain downstream of the bridge. Although uniform flow concepts are useful for roadway drainage designs, their primary use in bridge hydraulic analyses is for setting water surface boundary conditions in bridge hydraulic models. When no better information exists, one can estimate the boundary water surface elevation by setting the energy slope equal to the prevailing downstream channel or floodplain slope.

Nonuniform flow conditions are expected in natural channels and are used to analyze hydraulic conditions at bridges. Figure 4.3 shows nonuniform flow with varying depths and velocities along a channel, producing unequal channel bed, water surface, and energy grade line slopes. These hydraulic variables change throughout a river reach during flood conditions, especially at bridges.

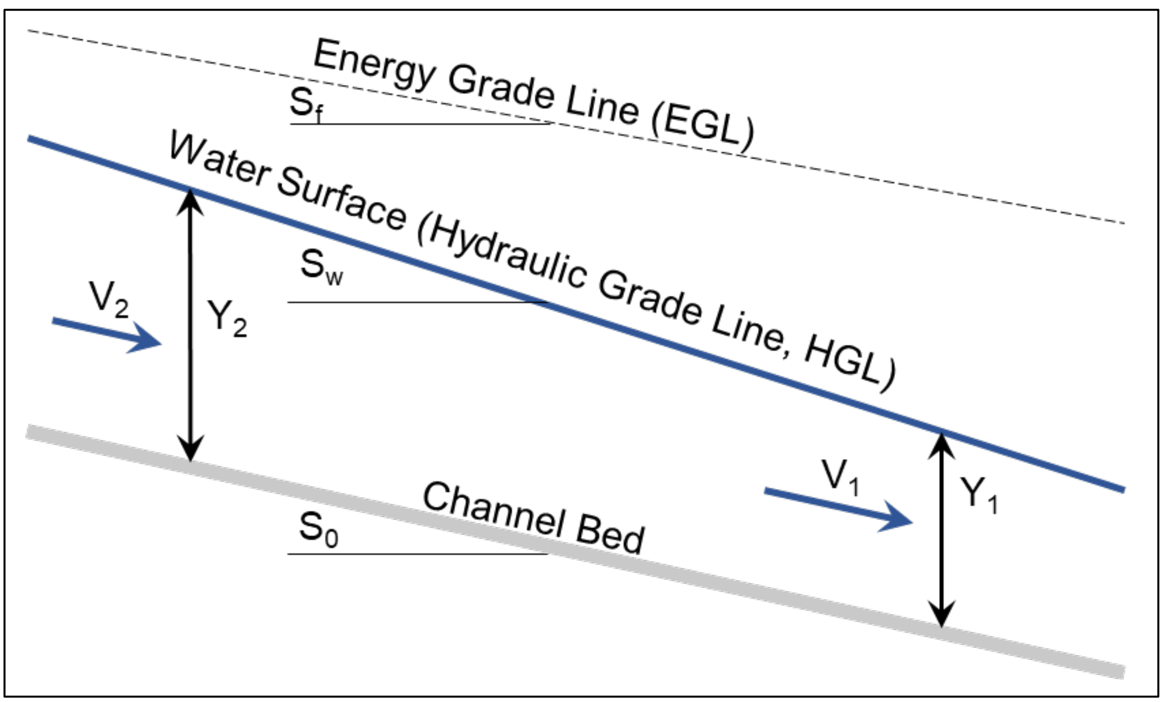

Figure 4.3. Example of nonuniform flow.

As the name implies, gradually varied flow is nonuniform, with gradual changes in hydraulic variables. Hydrostatic pressure exists throughout the water column, which is another distinguishing feature of gradually varied flow.

Rapidly varied flow exists when the streamlines (either vertically or horizontally) have pronounced curvature and the hydrostatic pressure assumption is not valid. The 1D and 2D hydraulic models discussed in this document assume hydrostatic pressure and gradually varied flow conditions in their computations. The models may apply hydrostatic pressure for some rapidly varied conditions, though with some loss of accuracy in the results. In many situations, such as weir flow, gates, and other hydraulic structures, the models use specialized equations to account for rapidly varied flow conditions.

4.2.3 Subcritical versus Supercritical Flow

Open-channel flows are free to adjust the water surface in response to varying channel and discharge conditions. The range of practical open-channel and bridge hydraulic analyses can be classified as subcritical or supercritical, although supercritical flow is not common in natural channels. The Froude number, which is the ratio of inertial forces to gravitational forces, reveals whether the flow is super- or subcritical. The equation for the Froude number is:

$$F_R = \frac{V}{\sqrt{gy}} \tag{4.6}$$

where:

F_R	=	Froude number
V	=	flow velocity, ft/s
g	=	acceleration due to gravity, ft/s^2
y	=	water depth, ft

When the Froude number is equal to one, "critical" flow conditions exist. A Froude number greater and less than one indicates supercritical and subcritical flow in the same way that Mach numbers indicate supersonic and subsonic conditions in aeronautics.

Various types of waves and surges may occur in open channels and cause a locally unsteady flow. The simplest is the small surface wave which progresses radially outward from a point as a rock would cause if thrown into a lake. The rate that this wave progresses outward is called its celerity. Subcritical flow is when the flow velocity is less than the celerity of a gravity wave and supercritical flow is when the flow velocity is greater than the wave velocity.

Dropping a rock in a pond causes a wave to propagate in all directions at the same velocity (Figure 4.4a). By dropping the rock in a stream with subcritical flow and superimposing the average velocity, V, the wave propagates upstream at a velocity of the wave speed minus the flow velocity and downstream at the wave speed plus the flow velocity (Figure 4.4b). If a higher, supercritical velocity is imposed, the wave is washed downstream with no affect upstream (Figure 4.4c). The conclusion is that for subcritical flow any disturbance in the flow field will translate upstream, which is the reason 1D water surface computations progress from downstream to upstream. On the other hand, for supercritical flow the computations progress from upstream to downstream since any disturbance in the flow field will not translate upstream. In 1861 William Froude presented a paper in which he defined the ratio of the characteristic velocity (average) V to a gravitational wave velocity, which was later called the Froude Number.

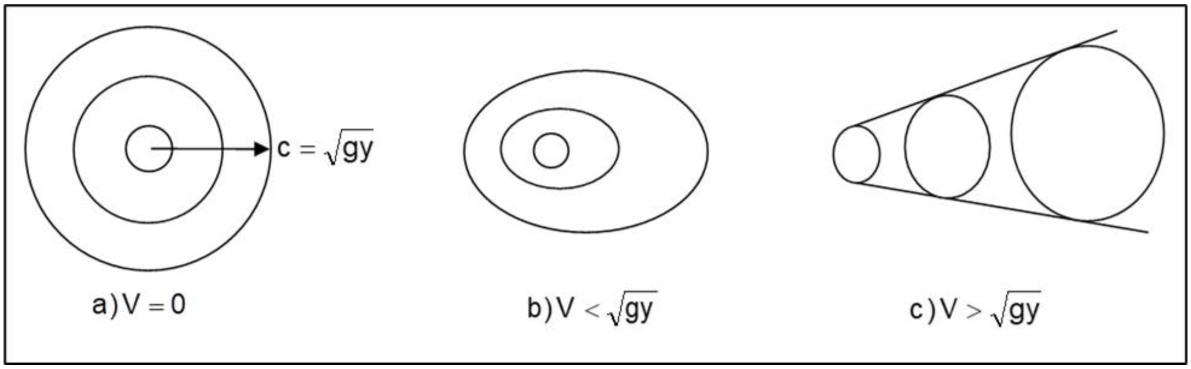

Figure 4.4. Propagation of waves in shallow water illustrating subcritical and supercritical flow.

4.3 Modeling Fundamentals & Equations

The basic equations of flow in open channels derive from three conservation laws: (1) the conservation of mass, (2) the conservation of energy, and (3) the conservation of linear momentum. The conservation of mass is another way of stating that matter can neither be created nor destroyed. The conservation of energy is an empirical law of physics that the energy remains constant over time in a closed system. Like matter, energy is neither nor destroyed, although it is transformable from one state to another (e.g., kinetic energy to potential or thermal energy). The principle of conservation of linear momentum is based on Newton's second law of motion, which states that a mass (of fluid) accelerates in the direction of and in proportion to the applied forces on the mass. In other words, force equals mass times acceleration. Conservation of mass and Newton's second law also provide the basis for fluid motion in two and three dimensions.

The following sections describe the application of the three conservations laws as the equations that are solved in 1D and 2D models. Other useful equations are also described. The fundamental computational component of 1D models is the cross section. A cross section is defined by a series of elevation and distance pairs crossing the channel and floodplains. A series of cross sections represents the river channel and floodplains in a 1-D model. Elements are the computational components of a 2D model. Each element represents a small area of the channel and floodplains, and the amalgamation of elements represents the entire river and floodplain area. A line, or transect, cut through a 2D model domain is used to visualize and evaluate model results across the model but is not used in the model computations.

The equations representing the conservations laws are solved between cross sections in 1D models and across elements in 2D models. The spatial step size (cross section spacing and element dimensions) affects the accuracy of the numerical solutions. Whether using a 1D or 2D model, the engineer selects the step size to accurately represent channel geometry, floodplain topography, variations in surface roughness, and hydraulic controls. Additional refinement (additional cross sections or smaller elements) may be needed to ensure reliable results in the solutions to the equations. Similarly, the engineer selects the temporal step size, or time step, to obtain reliable results in unsteady flow models.

4.3.1 Continuity (1D and 2D Modeling)

For steady flow, the continuity equation expresses the conservation of mass in a simplified, though very useful, form that allows a model to track the volume of flow or discharge passing a given location. In hydraulic modeling, matter is conserved, meaning that it is neither created nor destroyed within a control volume. The basic continuity equation is:

$$Q = V\,A = V_1\,A_1 = V_2\,A_2 \tag{4.7}$$

where subscripts 1 and 2 designate Cross Sections 1 and 2, and the variables are:

Q	=	discharge across a cross section or transect, ft^3/s (cfs)
V	=	average flow velocity across the cross section or transect, ft/s
A	=	flow area, ft^2

This equation applies when the fluid density is constant, the flow is steady, and there are no lateral inflows or seepage. The concept applies equally to 1D model cross sections and transects across 2D model domains. Failure to maintain flow continuity at the cross sections or transects indicates a fundamental error in the model calculations or that the model has not fully converged on a stable solution. Checking that flow continuity has been maintained is an easy and necessary step in the hydraulic modeling process.

Velocities vary in both direction and magnitude. In 1D models, cross sections are oriented perpendicular to the anticipated flow direction. The engineer is responsible for setting up the cross sections with a reasonably correct orientation. Two-dimensional models account for the velocity direction throughout the model, and element orientations are set based on describing channel bathymetry and floodplain topography.

For flow in two or three dimensions, consider flow through a small control volume as shown in Figure 4.5. Assuming a general flow, V (x, y, z, t), flows through surfaces 1 and 2 are perpendicular to the Y-Z plane. Note that the efflux rate, or flow, through area 1 is the fluid density times V_x times area and the flow through area 2 is the flow through area 1 plus the change in density times V_x times area 2 over the distance dx. Performing similar computations for the other sides and adding the results, the total net efflux rate is:

$$\text{Net Efflux Rate} = \left\{\frac{\partial(\rho V_x)}{\partial x} + \frac{\partial(\rho V_y)}{\partial y} + \frac{\partial(\rho V_z)}{\partial z}\right\} dx\, dy\, dz \tag{4.8}$$

The rate of decrease of mass inside the control volume equals the change in fluid density with time multiplied by (dx dy dz). Dividing both sides of the equation by dx dy dz yields the following relationship:

$$\frac{\partial(\rho V_x)}{\partial x} + \frac{\partial(\rho V_y)}{\partial y} + \frac{\partial(\rho V_z)}{\partial z} = \frac{-\partial \rho}{\partial t} \tag{4.9}$$

And for steady incompressible flow, fluid density is constant and the time derivative is zero, which gives the simplified, three-dimensional differential form of the continuity equation as:

$$\frac{\partial(V_x)}{\partial x} + \frac{\partial(V_y)}{\partial y} + \frac{\partial(V_z)}{\partial z} = 0 \tag{4.10}$$

The equation states that for steady flow the rate of flow into the control volume is equal to the rate of flow out.

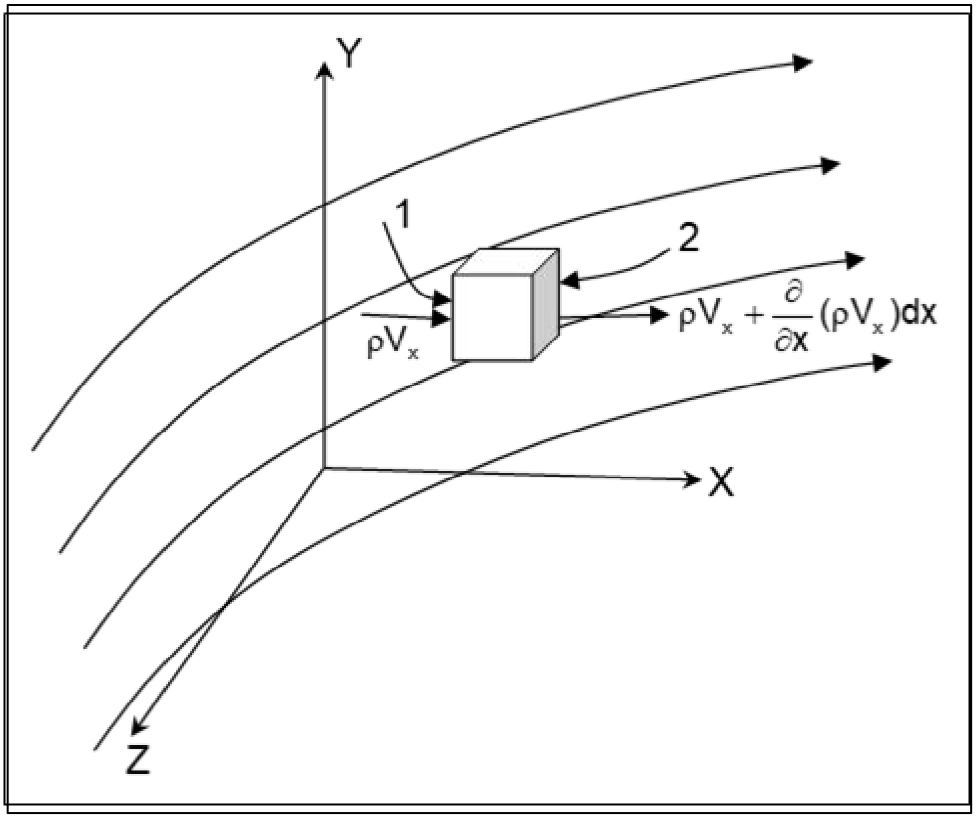

Figure 4.5. Net flow through a control volume.

In free surface unsteady modeling, or if there are lateral flow sources or losses, the continuity equation includes additional terms. The 2D flow continuity equation below illustrates source term and the effect of a free water surface by including the change in water depth with respect to time and distance.

$$\frac{\partial h}{\partial t} + \frac{\partial hU}{\partial x} + \frac{\partial hV}{\partial y} = q_m \tag{4.11}$$

where:

h	=	water depth, ft
U	=	depth-averaged velocity in the x direction, ft/s
V	=	depth-averaged velocity in the y direction, ft/s
q_m	=	inflow per unit area, ft/s

In Equation 4.11, the differential of depth with time reflects the flow unsteadiness. Depth and velocity vary with distance, and other inflow, in this case, vertical inflow from seepage or rainfall, is also included. Although 2D models solve Equation 4.11, it is important to remember that the simplified equation (Equation 4.7) still applies to transects across 2D steady-flow models.

4.3.2 Energy Equation (1D Modeling)

The energy equation asserts the conservation of energy along a streamline, which for 1D models is the channel between cross sections. Figure 4.6 illustrates that the total energy at a cross section is the sum of two types of potential energy, elevation and water depth, and the flow kinetic energy represented by the velocity-squared term. Each of the terms has units of length (feet in this document), and implicit in the water depth term is the assumption of hydrostatic pressure. In Figure 4.6, the total energy is greater at the upstream cross section than downstream, so to conserve the system's energy, some energy losses occur between the cross sections. The energy is not lost in a global sense because it becomes thermal energy.

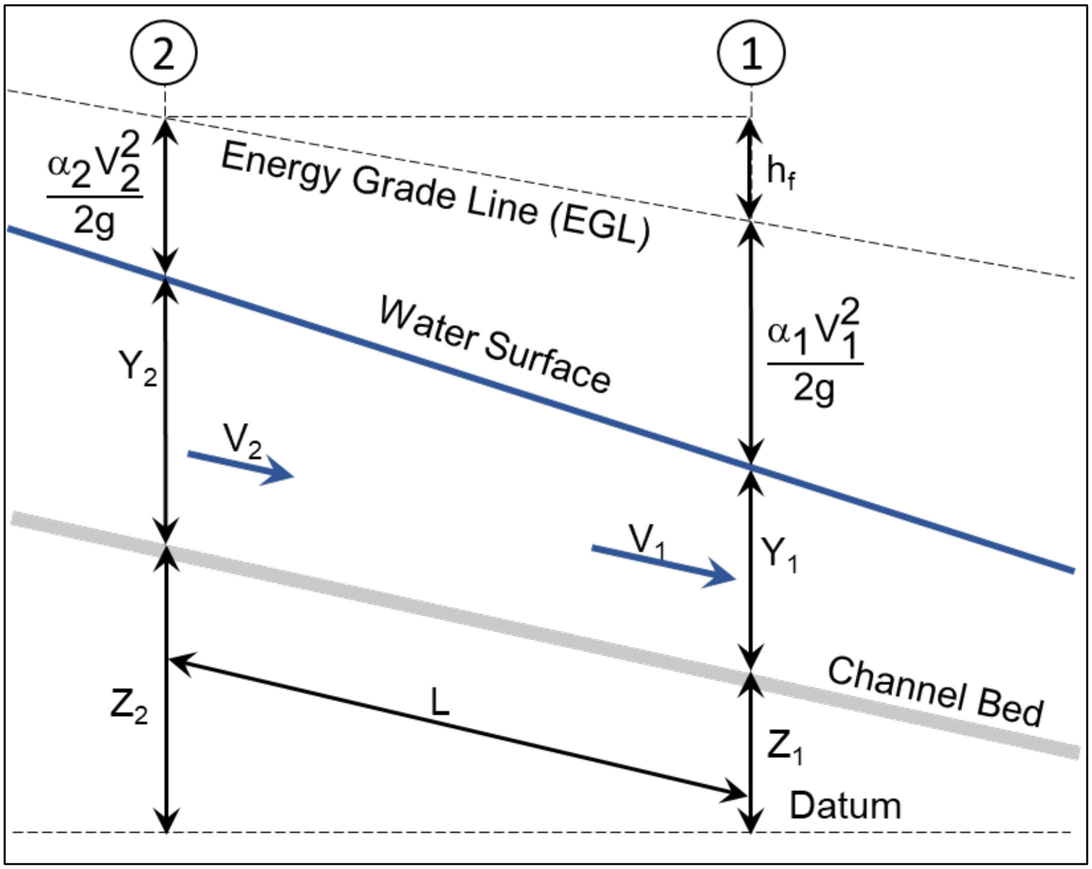

Figure 4.6. Components of the energy equation between two cross sections.

The energy equation applied to 1D models is:

$$Y_1 + \frac{\alpha_1 V_1^2}{2g} + Z_1 + h_f + h_e = Y_2 + \frac{\alpha_2 V_2^2}{2g} + Z_2 \tag{4.12}$$

where subscript 1 designates the downstream cross section, subscript 2 designates the upstream cross section, and the variables are:

Y	=	water depth, ft
α	=	energy correction coefficient
V	=	cross-section average velocity, ft/s
g	=	acceleration due to gravity, ft/s^2
Z	=	bed elevation, ft
h_f	=	friction loss between the cross sections, ft
h_e	=	expansion, contraction, and other losses between the cross sections, ft

In the kinetic energy term, the energy correction coefficient a is a consequence of using the average velocity for the cross section in the energy equation. Since the complete expression of kinetic energy is mass flux (ρ times V times A) times velocity squared, kinetic energy is actually proportional to velocity cubed. Therefore, high-velocity flow areas contribute far more kinetic energy than the average cross-section velocity, and low-velocity flow areas contributes very little kinetic energy. Including (integrating) kinetic energy over a cross section with varying velocity often results in far more total kinetic energy than would be calculated from the average velocity alone. The energy correction coefficient is typically less than two for in-channel flows and is often ignored. During floods, the highly varying velocities in channels versus floodplains produce energy correction coefficient values that easily exceed five. Therefore, 1D models necessarily include the correction term for flood flows.

The solution of the energy equation is the central aspect of 1D water surface profile models. Assuming subcritical flow, computations progress from known hydraulic conditions (velocity, water surface elevation, and α) at Cross Section 1 to determine the hydraulic conditions at Cross Section 2. The standard-step method is a calculation method applicable to a wide range of channel conditions and is especially useful for natural and other non-prismatic channels. The solution is iterative because the velocity at Cross Section 2 is a function of the water surface elevation at Cross Section 2. The velocities at both cross sections contribute to the energy losses h_f and h_e between the cross sections. The standard-step method uses a trial water surface at Cross Section 2 to estimate velocity and energy losses. The water surface is adjusted until the two sides of Equation 4.12 balance. With the water surface and hydraulic variables now known for Cross Section 2, the model proceeds to the next cross section. The standard-step method assumes the flow is gradually varied between cross sections.

Another step method is the direct-step method. It applies to prismatic channels only (constant bed slope and unchanging channel geometry), so it can be used for determining profiles through culverts and other designed channels. In this method, the step is the change in water surface elevation, and the method computes the distance between cross sections. The calculations progress from a location of known hydraulic conditions to the next cross section, which also has known hydraulics based on the change in the water surface. The two known energy slopes enable calculating the distance between the cross sections. This process is not iterative as it makes a direct calculation.

Any numerical method depends on using suitable step sizes. In 1D models, the spatial step size is the cross-section spacing. If the cross sections are spaced too far apart in the standard-step method or the water surface steps are too large in the direct-step method, then the water surface profile and other hydraulic results can be inaccurate. This is primarily because the friction loss term, h_f, derives from an average of the two cross sections' energy slopes.

4.3.3 Momentum Equation (1D Modeling)

The momentum equation is an expression of Newton's second law of motion, which states that the change in momentum is equal to the applied force. The change in momentum is the mass times the change in velocity, or acceleration. The application of the equation, as expressed below, is that the unbalanced force causes the fluid mass to accelerate.

$$\sum F_x = ma \tag{4.13}$$

where:

F_x	=	forces in the x direction, lb
m	=	mass, slugs

a = acceleration, ft/s^2

There are several possible applied forces in fluids, some acting in the direction of flow and others resisting flow. This implies a coordinate system with positive values in the direction of flow. In Figure 4.7, the two acting forces are the pressure force at the upstream cross section and the component of weight of the volume of water caused by the channel's slope. If the channel slope were negative (adverse slope), the weight component would be a resisting rather than acting force. The resisting forces are the water pressure force at the downstream cross section, the friction force between the water and channel boundary (friction times the wetted surface area), and the drag forces of the pier and the obstruction on the channel bed. This chapter discusses friction and drag forces in more detail in later sections.

The water accelerates in the downstream direction if the acting forces exceed the resisting forces and decelerates when the resisting forces exceed the acting forces. Note that the pressure used in the pressure force term (pressure times area) is the pressure at the centroid of the cross-section area. Other forces that could act on this system can include some types of vegetation or wind-induced stress on the water surface.

Equation 4.14 is the momentum equation applied to the channel segment shown in Figure 4.7. The equation includes other forces not shown in Figure 4.7, which could include pier drag force at bridge or forces from other obstructions. Note that the signs of the equation terms arise from the reference system, which is positive x in the direction of flow from Cross Section 2 to Cross Section 1.

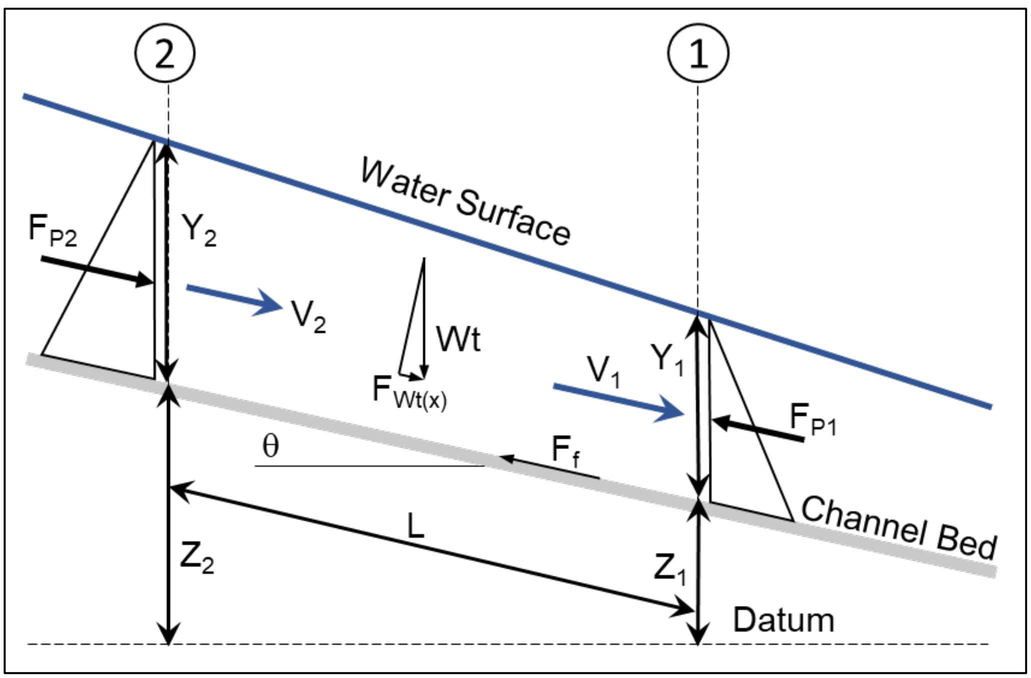

Figure 4.7. Forces acting on a control volume for the conservation of linear momentum.

The 1D momentum equation is:

$$F_{P2} - F_{P1} \pm F_{Wt(x)} - F_f - F_{other} = \rho Q (\beta_1 V_1 - \beta_2 V_2) \tag{4.14}$$

where subscript 1 designates the downstream cross section, subscript 2 designates the upstream cross section, and the variables are:

F_P	=	hydrostatic pressure force, lb
$F_{Wt(x)}$	=	component of the water weight force in the x direction, lb
F_f	=	friction force, lb
F_{other}	=	other forces, lb
ρ	=	water density, slugs/ft^3
Q	=	discharge, cfs
β	=	momentum correction coefficient
V	=	cross-section average velocity, ft/s

Like the energy correction coefficient, α, in the energy equation, the momentum correction coefficient, β, is applied when the average velocity is used to compute momentum because the total momentum of a velocity distribution is not equal to the momentum computed from the average velocity. The complete expression of momentum is mass flux (ρ times V times A) times velocity, so the momentum is proportional to velocity squared. A cross section with varying velocity has more total momentum than would be calculated from the average velocity alone.

One-dimensional modeling only infrequently uses the momentum equation, but in its expanded two- and three-dimensional forms, it is the basis for 2D and CFD (Computation Fluid Dynamics) models. The momentum equation applies to some bridge hydraulic applications in the HEC-RAS 1D model (USACE, 2021a). As noted in the previous section, excessive distances can reduce the accuracy of the computed results, though this is unlikely to affect calculations at a bridge.

4.3.4 Two-Dimensional Equations of Motion (2D Modeling)

FHWA (2019) provides a detailed description of the 2D equations of motion, known as the shallow water equations (SWE), commonly used in 2D models. Individual models vary in their solution of these equations and the terms they include, but they all aim to solve the Navier-Stokes differential equations representing fluid motion. Like the momentum equation described above, these equations derive from Newton's second law, that force equals mass times acceleration. Equations 4.15 and 4.16 are the differential equations representing Newton's second law in two dimensions.

The terms in their order in the equation on the left side are forces, including the hydrostatic pressure (partial derivative of h with respect to distance x or y), weight component (partial derivative of Z_b with respect to distance), shear stress, and turbulence stresses. The right side of the equation is mass times acceleration, which, as shown in Equations 4.2 through 4.4, have temporal and convective terms. Although the differential form of the equation can be challenging to interpret, it is important to recognize that Equations 4.15 and 4.16 represent Newton's second law (force equals mass times acceleration) just as the 1D momentum equation does (Equation 4.14). The 1D momentum equation excludes the turbulence terms and is shown for steady flow so only the convective acceleration term is included.

rho times g times h open bracket the partial of h with respect to x plus the partial of Z subscript b with respect to x closed bracket minus tau subscript b x minus the partial of h times tau subscript xx with respect to x minus the partial of h times tau subscript x y with respect to y equals rho open bracket the partial of h times U with respect to t plus the partial of h times U times U with respect to x plus the partial of h times U times V with respect to y closed bracket

$$\rho g h\left[\frac{\partial h}{\partial x}+\frac{\partial Z_b}{\partial x}\right]-\tau_{bx}-\frac{\partial h\tau_{xx}}{\partial x}-\frac{\partial h\tau_{xy}}{\partial y}=\rho\left[\frac{\partial hU}{\partial t}+\frac{\partial hUU}{\partial x}+\frac{\partial hUV}{\partial y}\right] \tag{4.15}$$

$$\rho g h\left[\frac{\partial h}{\partial y}+\frac{\partial Z_b}{\partial y}\right]-\tau_{by}-\frac{\partial h\tau_{yy}}{\partial y}-\frac{\partial h\tau_{yx}}{\partial x}=\rho\left[\frac{\partial hV}{\partial t}+\frac{\partial hVV}{\partial y}+\frac{\partial hVU}{\partial x}\right] \tag{4.16}$$

where:

ρ = water density, slugs/ft^3
g = acceleration due to gravity, ft/s^2
h = water depth, ft
Z_b = bed elevation, ft
τ_{bx} = bed shear stress in the x direction, lb/ft^2
τ_{by} = bed shear stress in the y direction, lb/ft^2
τ_{xx} and τ_{xy} = shear stresses due to turbulence in the x direction, lb/ft^2
τ_{yy} and τ_{yx} = shear stresses due to turbulence in the y direction, lb/ft^2
U = depth-averaged velocity in the x direction, ft/s
V = depth-averaged velocity in the y direction, ft/s

The 2D equations of motion (Equations 4.15 and 4.16) apply the forces in Newton's second law to a control volume in conjunction with the continuity equation. Figure 4.8 illustrates a control volume of flowing water in three-dimensional space and includes the primary forces acting on the control volume in two-dimensions. The figure also shows the calculated variables of velocity and depth. Two-dimensional models neglect vertical velocity components and assume hydrostatic pressure. Velocity is a vector quantity that can be expressed as a magnitude and direction or as the x and y velocity components U (x direction) and V (y direction). The elevation of the bed (Z_b) and water depth (h) vary over the area. The force variables shown are pressure (P) at the control volume horizontal surfaces, water weight (Wt), which has a component in the direction of the bed slope, and bed shear stress (τ_b), which acts in the opposite direction as velocity. Forces not shown in Figure 4.8 include water surface shear stress from wind, turbulence stresses acting on the element boundaries, and forces from objects, such as bridge piers, located within the element. For a set of unbalanced forces, the mass associated with the control volume accelerates or decelerates. Another term included in some 2D models is the Coriolis force term, which is an acceleration term that is treated as a force term because the model domain is in a fixed (inertial) coordinate system in an accelerating global system (the rotating earth).

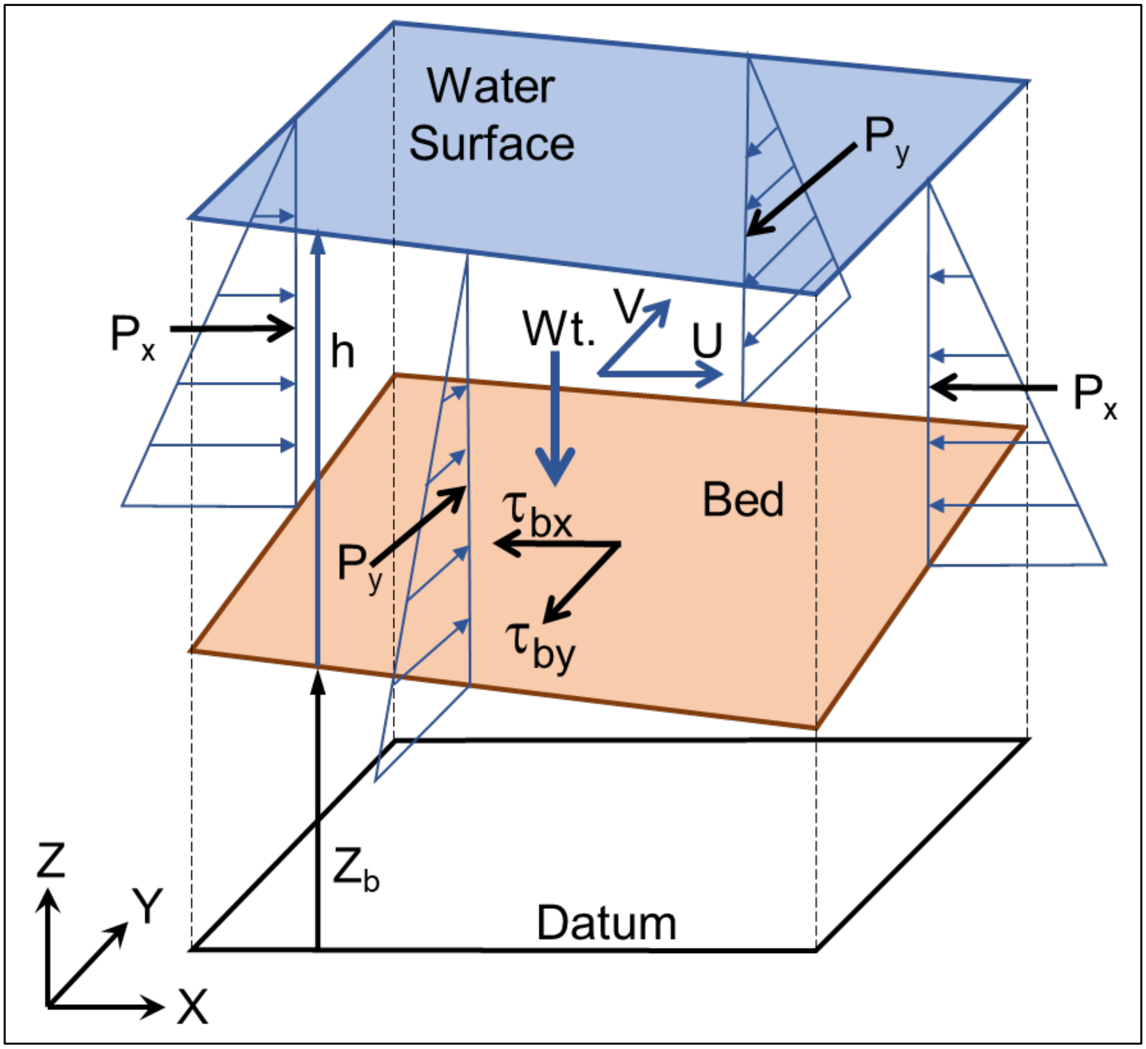

Figure 4.8. Two-dimensional hydraulic variables in a control volume.

4.3.5 Computational Fluid Dynamics

Computational Fluid Dynamics (CFD) models add the vertical dimension to the flow field's computational framework. The vertical dimension is not included as a single element is with 2D models but is discretized into multiple layers. CFD models calculate vertical velocities and velocity gradients, and the assumption of hydrostatic pressure does not apply. The substantial computational demands of CFD models limit their application to research and complex engineering problems. Like the current transition from 1D to 2D models in bridge hydraulics, CFD applications will likely become more available to the engineering community in the future.

4.4 Flow Resistance

Accurately representing the flow resistance is essential to most types of hydraulic analysis and specifically to open channel flow and bridge hydraulic analyses. Hydraulic model accuracy depends primarily on the accurate depiction of channel and floodplain geometries. Second to geometry is selecting appropriate values of flow resistance parameters. The friction loss term in the energy equation (Equation 4.12), the friction force term in the momentum equation (Equation 4.14), and the shear stress term in the SWE (Equations 4.15 and 4.16) are expressions of flow resistance. Any hydraulics textbook provides detailed information on the range of flow resistance equations and parameters for closed conduit and open channel flow applications. The commonly applied Manning flow resistance formula is the focus of this document, and the following section describes its use.

4.4.1 Manning's n

Robert Manning developed his flow resistance formula in the late nineteenth century and documented his work in a series of papers, including Manning (1889). He focused on uniform flow conditions where the channel bed, water surface, and energy slopes are equal. For this condition, the energy equation reduces to friction loss equals the difference in bed elevation, and the momentum equation reduces to friction force equals the weight force. Manning's Equation also applies to gradually varied flow with the substitution of energy slope shown below. In U.S. customary units, Manning's Equation is:

$$V = \frac{1.486}{n} R^{2/3} S_e^{1/2} \tag{4.17}$$

where:

V	=	flow velocity, ft/s
n	=	Manning's resistance coefficient
R	=	hydraulic radius, ft
S_e	=	energy slope

The factor of 1.486 is unity (1.0) in SI units to maintain the same value of Manning's n in both systems. The hydraulic radius, R, is determined as the flow area divided by the wetted perimeter (length of the wetted boundary), and the value of Manning's n is the composite or representative value over that boundary.

The popularity of Manning's n partly owes to it being less affected by depth changes than other open-channel flow resistance formulations. Another reason for its popularity, as discussed later in this section, is that it is a lumped parameter that can account for a variety of energy loss sources. The fact that it accounts for multiple sources of energy loss is often the main challenge in selecting an appropriate n value, but this is true for any flow resistance formulation.

By applying the continuity equation, Manning's Equation can calculate the discharge as:

$$Q = \frac{1.486}{n} A R^{2/3} S_e^{1/2} \tag{4.18}$$

where:

Q	=	discharge, cfs
A	=	area, ft^2

This form can apply to an entire cross-sectional area when the n value represents the entire boundary but can also determine the discharge in subareas, such as channel and floodplain areas, when the area, n value, and hydraulic radius are established for each subarea. This leads to a convenient simplification of Equation 4.18 that groups all the terms except energy slope into the conveyance parameter as:

$$K = \frac{1.486}{n} A R^{2/3} \tag{4.19}$$

where:

K	=	conveyance, cfs

The conveyance equation can also apply to the entire cross section or subareas when the equation variables represent the selected area. A 1D model can subdivide the discharge at a cross section in a 1D model by assuming that the energy slope is the same for each subarea. It can distribute the discharge within the cross section in proportion to the conveyance and determine each subdivision's velocity through continuity. Note that the underlying assumption of equal energy slope within a cross section is not applied in 2D models. The absence of that assumption is one reason for the substantially better results from more advanced models.

Another manipulation of Manning's Equation for gradually varied flow is in calculating the energy slope. A 1D model calculates the energy slope at a cross section based on total discharge and conveyance. It can also be calculated based on the velocity, n value, and hydraulic radius of a subarea. The relationships for energy slope are:

$$S_e = \left(\frac{Q}{K}\right)^2 = \left(\frac{Vn}{1.486}\right)^2 \frac{1}{R^{4/3}} \tag{4.20}$$

The first part of this equation represents the energy slope at a cross section for 1D models. The equation's expanded form can evaluate the energy slope throughout a 2D model domain using local values of velocity, Manning's n, and depth (recognizing that the local value of hydraulic radius is depth).

In the solution of the energy equation (Equation 4.12) using the standard- or direct-step methods described above, the friction slope between cross sections is the average of the two cross-section energy slopes. There are several methods for calculating the average. Determining the friction slope enables the calculation of friction loss by Equation 4.21.

$$h_f = LS_f \tag{4.21}$$

where:

h_f	=	friction loss between two cross sections, ft
L	=	flow length between cross sections, ft
S_f	=	friction slope (average energy slope between two cross sections)

Manning's n plays a vital role in the solution of the energy equation in 1D models as described above. Manning's n also influences the shear stress and friction forces in the momentum equation and 2D model elements.

The equation for shear stress is:

$$\tau = \gamma R S_e = \gamma \left(\frac{Vn}{1.486}\right)^2 \frac{1}{R^{1/3}} \tag{4.22}$$

where:

τ	=	shear stress, lb/ft^2
γ	=	unit weight of water, lb/ft^3

The shear stress applied over the wetted area between cross sections provides the friction force in the momentum equation as:

$$F_f = \tau\, WP\, L \tag{4.23}$$

where:

F_f	=	friction force, lb
WP	=	wetted perimeter, ft
L	=	flow length between cross sections, ft

Similarly, the friction force over a 2D model element is the shear stress integrated over the element bed surface area. In the 1D momentum equation, the friction force acts upstream, and in 2D models, the shear stress is a vector quantity acting in the opposite direction as the velocity vector.

Energy slope and shear stress are both directly proportional to velocity and Manning's n squared. Thus, the model results can be very sensitive to the selected value of Manning's n, especially in high-velocity areas. Although model calibration, where n is adjusted to match observed hydraulic conditions, may be the most definitive approach to establishing applicable n values, several other methods for estimating Manning's n are available. One method is Cowan's (USGS 1956) equation, which emphasizes the lumped parameter aspect of Manning's Mn. Cowan's Equation for estimating Manning's n is:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)\, m_5 \tag{4.24}$$

where:

n	=	Manning's n
n_0	=	base Manning's n for the bed material in a straight, uniform channel
n_1	=	Manning's component for channel surface irregularities
n_2	=	Manning's component for variation in channel cross section
n_3	=	Manning's component for obstruction effects
n_4	=	Manning's component for vegetation
m_5	=	coefficient for degree of meandering

The base value In Cowan's method represents the channel bed material's grain roughness: gravel and other large particles create more grain roughness than sand-sized particles. The engineer typically considers the other components of the total roughness on a river reach basis. These include channel bedforms (see FHWA 2012c) and other surface irregularities, channel cross-section variability, obstructions, and vegetation. Since meandering channels redirect the flow at each bend, one can adjust the Manning's n value based on channel sinuosity much in the way that pipe flow hydraulic analyses include so-called minor losses for pipe bends. Table 4.1 gives base and component values for calculating Manning's n with Cowan's method. Cowan's method applies to channels, not floodplains. Chow (1959) presents a table listing typical n values for a range of conditions, including channels and floodplains. The table includes minimum, normal, and maximum values for various materials and channel types.

Table 4.1. Values for calculating Manning's n using Cowan's method (after Chow 1959).

Material involved	n_0
Earth	0.020
Rock cut	0.025
Fine gravel	0.024
Coarse gravel	0.028
Degree of irregularity	n_1
Smooth	0.000
Minor	0.005
Moderate	0.010
Severe	0.020
Variation in channel cross section	n_2
Gradual	0.000
Alternating occasionally	0.005
Alternating frequently	0.010 to 0.015
Relative effect of obstructions	n_3
Negligible	0.000
Minor	0.010 to 0.015
Appreciable	0.020 to 0.030
Severe	0.040 to 0.060
Vegetation	n_4
Low	0.005 to 0.010
Medium	0.010 to 0.025
High	0.025 to 0.050
Very high	0.050 to 0.100
Degree of meandering	m_5
Minor	1.00
Appreciable	1.15
Severe	1.30

Table 4.2 includes the Natural Streams portion of Chow's table. The channel Manning's n values in Cowan's method range from 0.025 to greater than 0.20, and in Chow's table, n values range from 0.025 to 0.15. Floodplain n values in Chow's table range from 0.020 to 0.16.

Other sources for estimating Manning's n include pictures of selected streams and floodplains to use as a guide for selecting n. The publication *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains* (FHWA 1984b) is a resource for consideration of potential Manning n values for both channel and floodplain flows. The U.S. Geological Water Supply Paper 1849, *Roughness Characteristics of Natural Channels*, (USGS 1967) is another example of photographs of natural channels with the associated n values. These publications can be found on the internet by searching the document titles. Other publications with photos of calibrated streams are: Arcement and Schneider (USGS 1989), Chow (1959), Hicks and Mason (1991), and NRCS (1963).

Figure 4.9 shows an example floodplain with a computed Manning n of 0.20. The floodplain vegetation is a mixture of small and large trees, including oak, gum, and ironwood. The base is firm soil and has minor surface irregularities. Obstructions are minor; ground cover is medium, with a large amount of undergrowth that includes vines and palmettos (FHWA 1984b). Similarly, Figure 4.10 is taken from the USGS (1967) Water Supply Paper and shows a channel with a computed Manning n of 0.026.

Figure 4.9. Floodplain roughness example with Manning’s n of 0.20. (USGS 1989).

Table 4.2. Values of Manning's n for natural channels (after Chow 1959).

Type of channel and description	Minimum	Normal	Maximum
D. Natural Streams			
D-1 Minor stream (top width at flood stage < 100 ft)			
a. Stream on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, and few boulders	0.03	0.04	0.05
2. Bottom: cobbles with large boulders	0.04	0.05	0.07
D-2 Flood Plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.05	0.07
2. Light brush and trees, in winter	0.035	0.05	0.06
3. Light brush and trees, in summer	0.040	0.06	0.08
4. Medium to dense brush, in winter	0.045	0.07	0.11
5. Medium to dense brush, in summer	0.070	0.10	0.16
d. Trees			
1. Dense willows, summer, straight	0.11	0.15	0.20
2. Cleared land with tree stumps, no sprouts	0.03	0.04	0.05
3. Same as above, but with heavy growth of sprouts	0.05	0.06	0.08
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.08	0.10	0.12
5. Same as above, but with flood stage reaching branches	0.10	0.12	0.16
D-3 Major streams (top width at flood stage > 100 ft). n values are less than similar minor streams because banks contribute less resistance.			
a. Regular section with no boulders or brush	0.025	up to	0.06
b. Irregular and rough section	0.035	up to	0.10

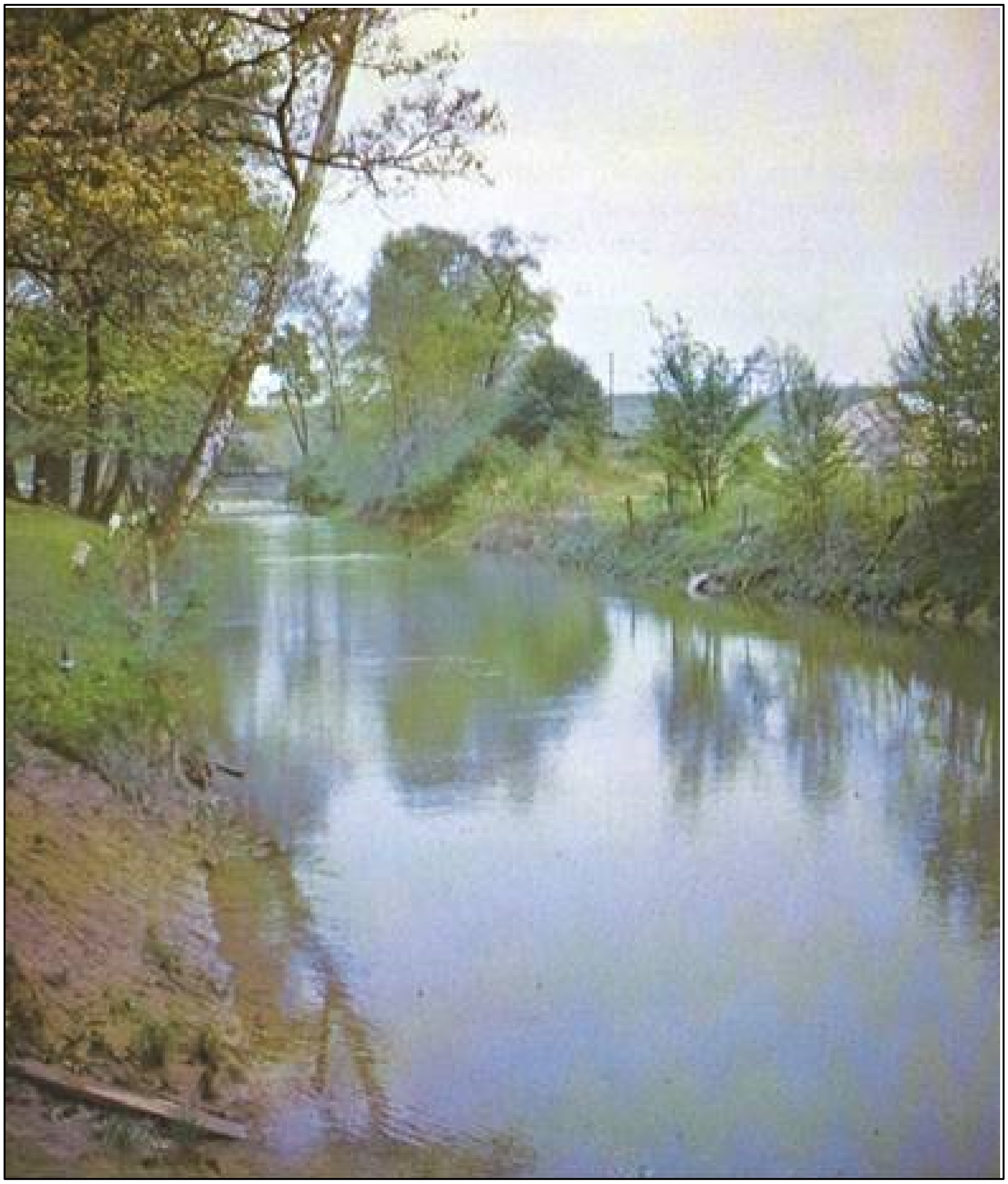

Figure 4.10. Channel roughness example with Manning's n of 0.026. (Indian Fork near New Cumberland, Ohio, USGS 1967).

Figure 4.9 and Figure 4.10 further illustrate the concept of Manning's n as a lumped parameter. In each case, there is a surface shear attributed to the roughness of the channel bed and banks, and floodplain surface. Some vegetation is predominantly on the surface and contributes to the surface shear. Other vegetation, especially trees, presents obstructions to flow. Together these factors contribute to an overall energy loss effect by increasing Manning's n even though they act individually. Some obstructions merit direct inclusion in hydraulic computations as flow resistance from drag forces, which is the subject of the next section.

As described above, the energy equation uses Manning's n to determine friction loss. The various forms of the momentum equation use it to determine friction force. It is also true that Manning's n is a lumped parameter that can include a variety of processes contributing to energy loss. Since 2D models explicitly include changes in flow direction in meandering channels, it appears unlikely that the coefficient for meandering in Cowan's method would be fully included in 2D models. The other components in Cowan's method may also be excluded from 2D models depending on the degree that the 2D model surface geometry depicts channel irregularities and obstructions.

One can supplement the information provided above with information provided in model reference documents to select Manning's n values. Model calibration is also a useful exercise for refining n values when observed data are available.

4.4.2 Drag Forces

As described earlier, resistance to flow includes surface shear stresses and forces from obstructions in the flow. As flow moves around and over an obstruction, the resulting pressure difference across the object creates a drag force or form drag. The form drag force depends on the size and shape of the object in the flow field. The empirically determined drag coefficient depends on the obstruction's shape and is considered constant for turbulent flow conditions. The equation for drag force is:

$$F_D = C_D \rho A_p \frac{V^2}{2} \tag{4.25}$$

where:

F_D	=	drag force, lb
C_D	=	drag coefficient
ρ	=	fluid density, slugs/ft^3
A_p	=	projected area of the obstruction, ft^2
V	=	flow velocity approaching the obstruction, ft/s

Application of Drag to Piers: Bridge piers in a channel or the floodplain cause an additional backwater due to their form drag. One can calculate the drag force created by the pier using Equation 4.25 and assigning the drag coefficient. Table 4.3 includes typical drag coefficients of piers from the *HEC-RAS Reference Manual* (USACE 2021a).

Table 4.3. Typical drag coefficients for different pier shapes.

Pier Shape	Drag Coefficient C_D
Circular pier	1.20
Elongated piers with semi-circular ends	1.33
Elliptical piers with 2:1 length to width	0.60
Elliptical piers with 4:1 length to width	0.32
Elliptical piers with 8:1 length to width	0.29
Square nose piers	2.00
Triangular nose with 30-degree angle	1.00
Triangular nose with 60-degree angle	1.39
Triangular nose with 90-degree angle	1.60
Triangular nose with 120-degree angle	1.72

Drag Force due to the Addition of Debris: The accumulation of debris on bridge piers, as illustrated in Figure 4.11, and on the superstructure can create significant forces on the structure. One-dimensional models can analyze the hydraulic effects of debris. However, the complexity of the hydraulics and the risk of failure of the structure may justify 2D models, CFD models, or physical model studies. The reader is referred to NCHRP Report 445, *Debris Forces on Highway Bridges* (NCHRP 2000), and Hydraulic Engineering Circular No. 9 (FHWA 2005), *Debris Control Structures Evaluation and Countermeasures*, Chapter 4 *Analyzing and Modeling Debris Impacts to Structures*.

Figure 4.11. Example of debris accumulation on piers.

4.5 Flow Controls at Roads & Bridges

The energy and momentum methods described above are well suited for free surface flow conditions at bridges prior to road overtopping. The model includes additional factors for flood flows when the road overtops or when the water surface comes in contact with the bridge superstructure (pressure flow). Pressure flow and road overtopping can involve flow conditions that vary rapidly, making the energy and momentum equations less applicable. These situations may call for specialized equations. The following sections describe the weir equation for roadway and bridge overtopping conditions and orifice equations for pressure flow conditions.

4.5.1 Weir Flow

Weirs have a wide range of applications for flow control and discharge measurement in laboratory and field applications. Of the many types and shapes of weirs used in practice, the broad-crested weir can closely represent roadway overtopping conditions. Figure 4.12 illustrates flow over a broad-crested weir. Critical depth, with a Froude number equal to 1.0, occurs at some point on the weir. Flows at several locations have substantial vertical velocity components, so the flow is rapidly varied.

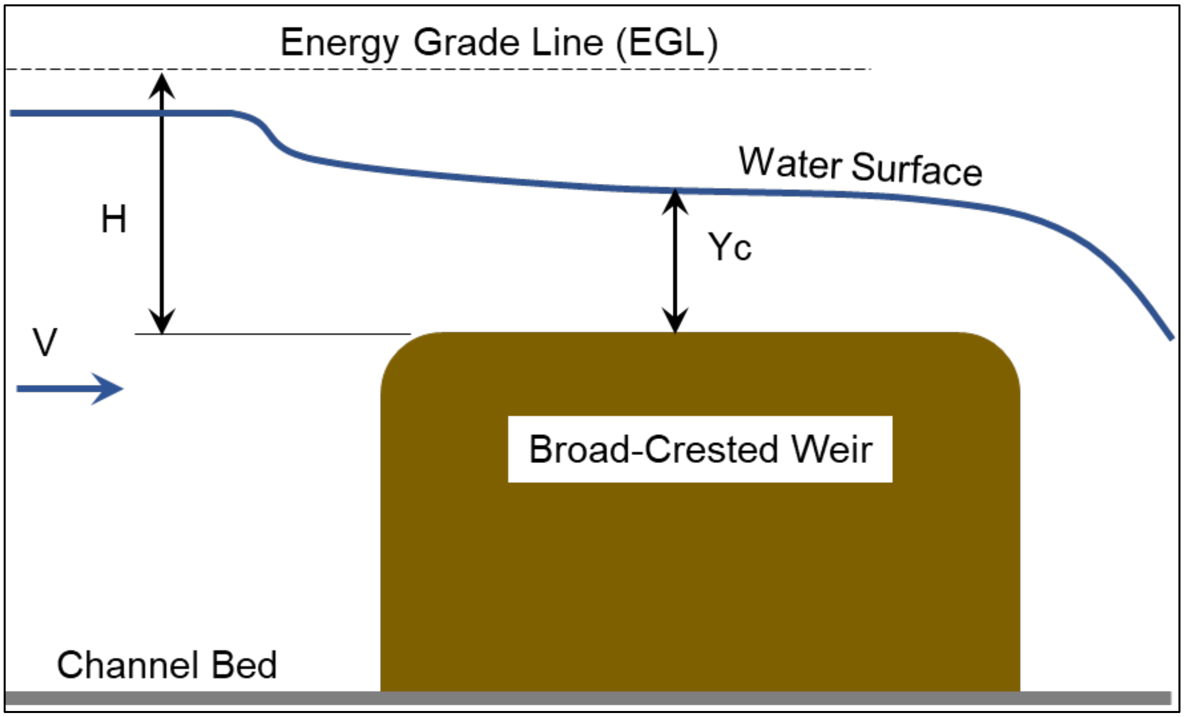

Figure 4.12. Broad-crested weir.

The following equation calculates the discharge over a broad-crested weir:

$$Q = C\,L\,H^{3/2} \tag{4.26}$$

where:

Q	=	discharge over the weir, cfs
C	=	weir coefficient, $ft^{0.5}/s$
L	=	weir length perpendicular to flow, ft
H	=	head above weir crest, ft

In roadway overtopping applications, the weir coefficient ranges from 2.6 to 3.1 in U.S. customary units. Many analysis programs leave the velocity head out of the calculation because the upstream velocity is small. The equation's derivation ignores friction loss, which can be substantial for long weir crests in the direction of flow and for shallow overtopping conditions. Submergence of the weir from high tailwater also reduces flow over the weir. As described in Chapter 6 of this document, practitioners can appropriately reduce the weir coefficient's value in these situations.

Engineers can simulate road overtopping in some 2D models using the weir equation by creating internal boundaries adjacent to each side of the roadway, specifying roadway elevations, and selecting the appropriate weir coefficient. It is also common practice in 2D models to simulate the roadway geometry as part of the network of model elements. The model then simulates flow over the roadway using the SWEs. Although this approach applies the SWEs to a rapidly varied flow condition, the advantage is that it includes friction forces and tailwater submergence in the solution. This approach appears to work well in many applications.

4.5.2 Pressure Flow

The occurrence of pressure flow can reduce the hydraulic capacity at a bridge because the flow area does not increase with increasing water surface elevations. In addition to the constrained area, the wetted perimeter includes the bottom chord of the bridge girders, reducing the hydraulic radius. One-dimensional models can use the energy equation to simulate pressure flow. This approach is often acceptable for low amounts of submergence, but it excludes rapidly varied flow conditions. The orifice equation, which accounts for rapidly varied flow, is:

$$Q = C_d A \sqrt{2 g \Delta H} \tag{4.27}$$

Q	=	discharge through the bridge opening, cfs
C_d	=	discharge coefficient
A	=	bridge opening area, ft^2
ΔH	=	head differential, ft

Bridge analysis considers orifice conditions as submerged orifice when both the upstream and downstream bridge faces are submerged, and free-flowing orifice, when the upstream bridge face is submerged, and the rest of the bridge opening has a free surface. The flow condition – submerged or free-flowing – determines the head differential's definition and the discharge coefficient values. For the submerged orifice condition, the head differential is the difference in energy grade line between the cross sections upstream and downstream of the bridge. For a free-flowing orifice, the head differential is the energy grade upstream of the bridge minus the mid-height elevation between the bridge low chord and the channel bed. The orifice equation relies on selecting discharge coefficients that account for rapidly varied flow conditions, including flow separation under the bridge deck. For submerged orifice flow at bridges, the discharge coefficient ranges from 0.7 to 0.9. Discharge coefficients for free-flowing orifices at bridges typically range from 0.35 to 0.5.

The *HEC-RAS Hydraulic Reference Manual* (USACE, 2021a) provides information on orifice flow equations used for 1D bridge hydraulic modeling. Two kinds of orifice flow are included: orifice and submerged-orifice flows. For orifice flow conditions only the upstream bridge face is submerged, and the downstream conditions do not influence the solution. For submerged-orifice flow the bridge deck is submerged at the upstream and downstream bridge faces. For this condition the discharge is determined based on the head differential across the bridge. Each of these methods rely on selection of discharge coefficients that account for rapidly varied flow conditions including flow separation under the bridge deck.

Although some 2D models include orifice equations at bridges, essentially providing a 1D solution at the bridge, they can also incorporate the bridge superstructure geometry directly into the network. This approach is similar to using the energy equation for pressure flow in 1D models because the bridge low chord is added as a surface in the shear force calculations of pressure flow elements. The advantage of this approach is that two-dimensional hydraulic calculations are performed within the bridge opening. The disadvantage is that the rapidly varied flow conditions, including flow separation and vertical velocity components, are not simulated.

CHAPTER 5
BRIDGE HYDRAULIC ANALYSIS CONSIDERATIONS

5.1 Introduction

Chapter 4 provides principles of the fundamental concepts for most hydraulic modeling calculations encountered in open channel flow and bridge hydraulic analysis. The calculations are often complex and may entail iterative solution techniques due to the interaction between variables. Therefore, computer programs are the primary tool for hydraulic engineers. As computer technology has advanced, so has numerical hydraulic modeling.

For decades, the primary analysis approach for bridge hydraulics has been 1D modeling. However, 2D hydraulic modeling is becoming the preferred approach across a broad spectrum of hydraulic study types, and increasingly, 3D modeling is employed to analyze complex flow fields. Chapters 6 and 7 provide information on using 1D and 2D models for bridge hydraulic analysis. *Two-Dimensional Hydraulic Modeling for Highways in the River Environment* (FHWA 2019) provides more extensive information on using 2D models.

This chapter includes information on selecting the most appropriate approach, whether it is 1D, 2D, or 3D numerical modeling, steady or unsteady modeling, or physical modeling. This chapter also provides an overview of developing input data and other considerations common to all bridge hydraulic problems regardless of the approach.

5.2 Hydraulic Analysis Methods

5.2.1 Model Selection

Whether it is numerical or physical, any hydraulic model has assumptions, limitations, data, and other input. The engineer needs to be aware of and understand the assumptions because they form the limitations of that approach. The goal of any hydraulic model study is to accurately simulate the actual flow condition. Violating the assumptions and ignoring the limitations often results in a poor representation of the actual hydraulic condition. Inaccurate and unrealistic results go unnoticed when users treat a model as a “black box;” not knowing how the model produced those results. This is not advisable given the cost of bridges and the potential consequences of failure. Therefore, the approach should be selected based primarily on its advantages and limitations, considering the importance of the structure, potential project impacts, cost, and schedule.

5.2.2 One-Dimensional Modeling

One-dimensional models are useful for analyzing relatively simple hydraulic situations that do not violate the model's internal assumptions. As noted in Chapter 4, 1D models primarily use continuity and the standard-step solution of the energy equation. One-dimensional models represent channels and floodplains with cross sections that describe the geometry and specify hydraulic attributes such as roughness at discrete locations along the reach of interest. The geometry and attributes are assumed to be continuous between sections. Input data and computed model results are associated with these user-defined cross sections.

One-dimensional models "see the world" primarily from a conveyance perspective. A 1D model calculates results based on the conveyance available at each cross section. The model assumes portions of the cross section with larger available area or less flow resistance can convey more flow unless the engineer informs the model otherwise with ineffective flow areas, blocked obstructions, or roadway embankments and bridge or culvert features. Figure 5.1 shows a plan view of a typical 1D model cross section layout and a representative cross section plot.

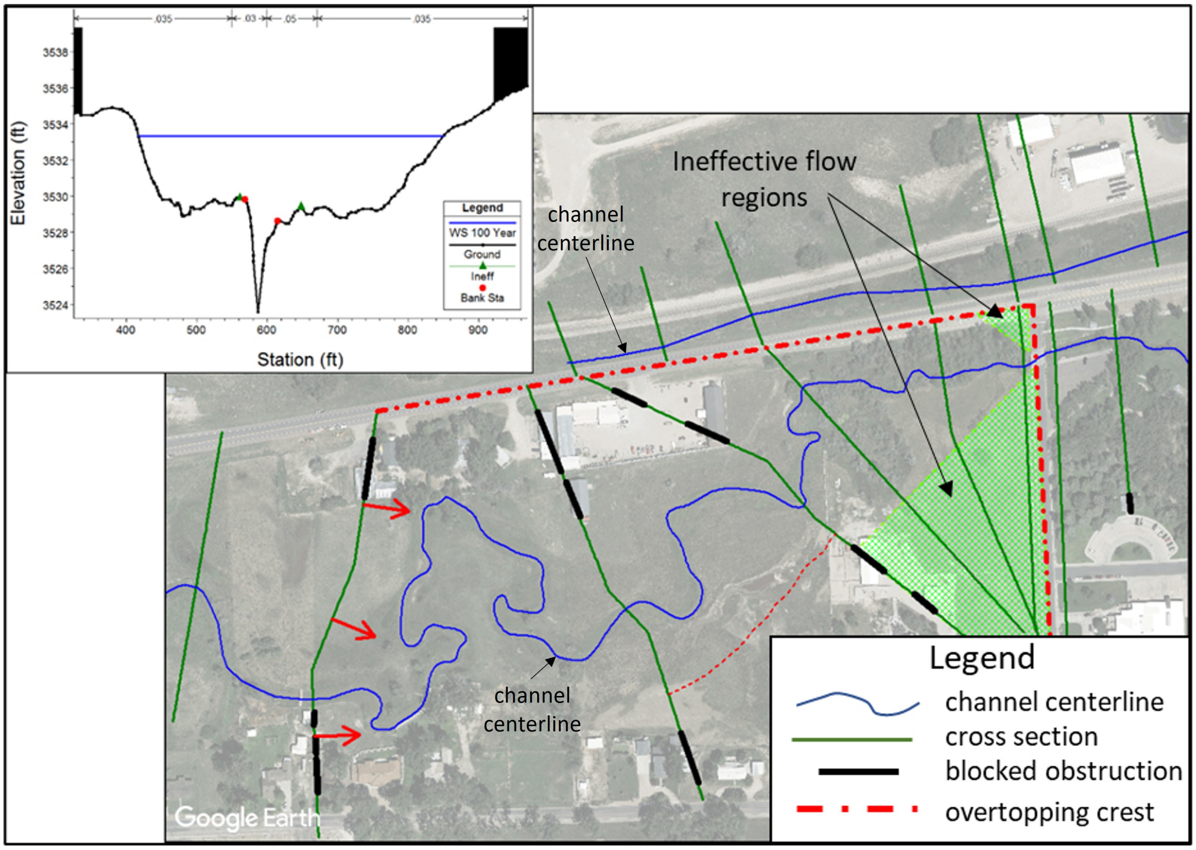

Figure 5.1. Plan view of a typical 1D model and representative cross section plot.

One-dimensional models provide a familiar and computationally efficient method for simple transportation hydraulic analyses involving single-channel streams, simple floodplains, and single bridge or culvert openings. In general, when the lateral movement of flow is small, 1D models provide reasonable results. However, because 1D models provide a simple approach, they also have inherent limitations and involve several assumptions to be made by the engineer. Engineer assumptions and model limitations for 1D hydraulic analysis include, but are not limited to, the items listed in Table 5.1. Because of these assumptions and limitations, 1D models can provide extremely erroneous results as the degree of hydraulic complexity increases.

Table 5.1. Comparison of 1D and 2D modeling approaches.

Hydraulic Variables	1D Modeling	2D Modeling
Flow direction	**Assumed** by user	**Computed**
Flow paths	**Assumed** by user	**Computed**
Channel roughness	**Assumed** constant between cross sections	Roughness values at individual elements used in computations
Ineffective flow areas	**Directed** by user	**Computed**
Flow contraction and expansion through bridges	**Directed** by user	**Computed**
Flow velocity	**Averaged** within main channel at each cross section	**Computed** at each element
Flow distribution	**Approximated** based on conveyance	**Computed** based on continuity and momentum
Water Surface Elevation	**Assumed** constant across entire cross section	**Computed** at each element
Energy Slope	**Single** value for entire cross section	**Varies** throughout 2D model domain
Energy Grade Elevation	**Assumed** as discharge-weighted average for cross section	Not directly computed, but **varies** throughout 2D model domain.

5.2.3 Two-Dimensional Modeling

Two-dimensional modeling represents a significant advancement over 1D modeling. They provide additional insights while having fewer internal limitations and significantly fewer assumptions, and less direction needed by the engineer. Two-dimensional models calculate hydraulic results at all active nodes or cells in a mesh covering the entire geographic extent of a river and floodplain. See the example in Figure 5.2.

Most 2D models used in hydraulic engineering are depth-averaged velocity models. Depth-averaged velocity models neglect vertical variations in the velocity, computing a single velocity vector in the horizontal plane at any point in the model and treating it as if it applies throughout the entire water column. Two-dimensional models also neglect vertical velocity components and accelerations. These simplifications significantly reduce the problem's numerical complexity, producing more manageable calculations. The widely used 2D models all use some variation of the shallow water equations (SWE) presented in Chapter 4 to solve variables at each computation point in the model geometry.

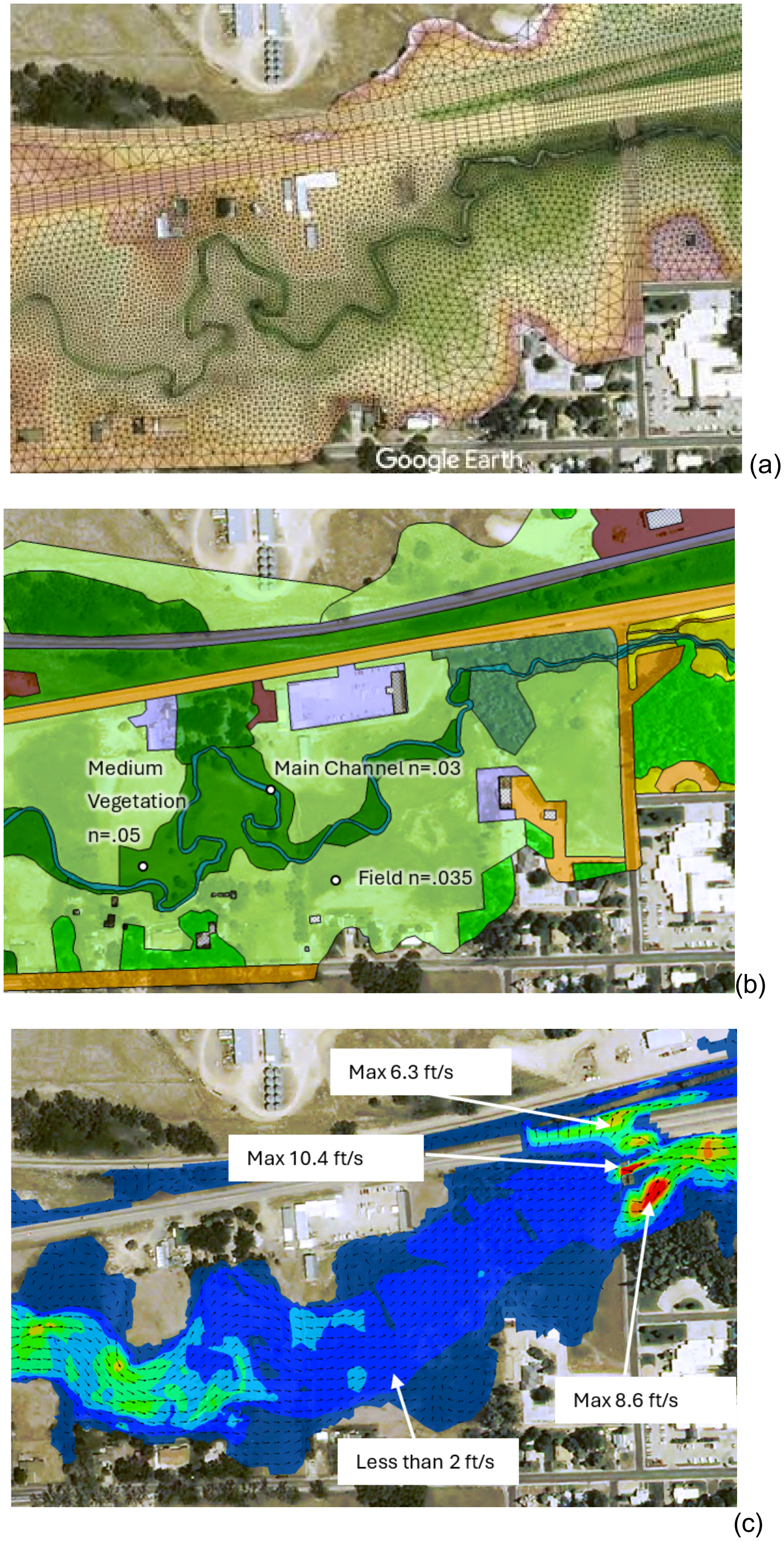

Figure 5.2. Typical 2D model geometry**:** (a) Mesh. (b) Land use types. (c) Velocity contours and vectors.

Many of the advantages of 2D models pertain to flow-related characteristics that are computed by 2D models are user-specified or entirely neglected in 1D models. Table 5.1 illustrates how 2D models handle several hydraulic variables better than 1D models. Two-dimensional models also excel in the visualization and communication of hydraulic results.

5.2.4 Three-Dimensional Modeling

One-dimensional and 2D modeling are sufficient for the needs of many bridge design projects. However, some hydraulic problems are beyond the abilities of 2D models and call for more advanced hydraulic modeling techniques such as 3D modeling or physical modeling, which do not include the simplifications inherent in 1D and 2D modeling. If hydraulic conditions include vertical accelerations or vertical velocity components that are important to the analysis, 3D modeling is a useful tool.

Three-dimensional models do not use depth-averaged velocities and can resolve flow in the z-direction and calculate vertical velocity gradients. Unlike 1D and 2D models, they do not assume a hydrostatic pressure distribution. While they involve more modeling effort and computational resources, 3D models reveal more details of the flow patterns at bridge elements. With the advancement of high-performance computing and computational algorithms, 3D modeling, also known as computational fluid dynamics (CFD), is becoming more feasible for hydraulic engineering applications. Three-dimensional models are suited for localized study areas and are not ideal for analyzing a river and floodplain reach due to the additional amount of input data. Bridge hydraulics researchers commonly use CFD for research related to scour, hydraulic forces on superstructures, and other topics.

5.2.5 Physical Modeling

Physical hydraulic modeling is the oldest form of hydraulic modeling and remains a valuable fluid mechanics tool. Laboratory scale models provide direct experimental data for complex flow fields, flow structure interaction, and erosion processes. Figure 5.3 shows an example of a physical model of a bridge deck and pier used to assess hydraulics associated with pressure flow. Like CFD models, physical models provide insight into vertical velocity profiles and, at larger-scaling factors, quantitative understanding of complex flow patterns necessary for design or forensic analysis scenarios.

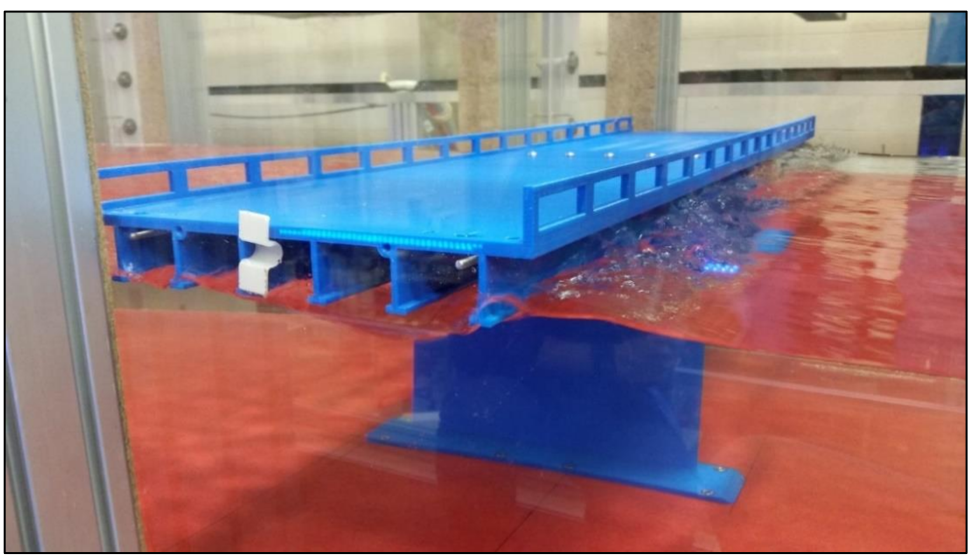

Figure 5.3. Example of a physical model simulating pressure flow hydraulics at a bridge.

Fluid mechanic textbooks (such as Munson et al. 2010) provide in-depth discussions of dimensional analysis and similitude for laboratory scale models. Geometric similarity is primary, although some conditions are evaluated with distorted vertical and horizontal scales. Froude number scaling (the ratio of inertial forces to gravitational force) replicates the dominant hydraulic forces for free-surface flow conditions. When using Froude number scaling, other force ratios, such as the Reynolds number, do not scale. Therefore, physical scale models are not a complete representation of the prototype conditions. Scale models can be either fixed-bed or moveable bed models. Moveable-bed models with individual piers or other structural elements are used to evaluate local scour (TAC 2004). For moveable-bed models, it is necessary, though difficult, to scale the sediment characteristics. ASCE (2008) provides a discussion of sediment transport scaling of physical models.

Physical modeling can be labor-intensive and involves specialized facilities and equipment, including a flume, access to water and sediment, labor, and machine tools to construct a physical representation of the area of interest. Like numerical modeling, physical modeling involves specialized training to develop an appropriate experimental design that captures the data necessary to quantify experimental results. A common analytical approach is to extend physical model study results with calibrated 2D and 3D numerical model analyses.

5.3 Two-Dimensional Bridge Hydraulics Modeling Applications

No numerical or physical model provides an exact representation of the complexities of an actual flow condition. This is true where roadways cross natural watercourses with variable channel and floodplain topography, land use types, and vegetation. The assumptions necessary for 1D models are often violated to a greater degree than is commonly realized. In many cases, experienced engineers can compensate for some of the limitations of 1D models.

Because 2D models avoid many assumptions and simplifications included in 1D models, they better represent the physics of flow and provide more realistic hydraulic results, especially at highway encroachments. Therefore, FHWA strongly prefers using 2D models over 1D models for complex waterways and complex bridge hydraulic analyses. Two-dimensional models provide more accurate representations of:

- Flow distribution
- Velocity distribution
- Water surface elevation
- Backwater
- Velocity magnitude
- Velocity direction
- Flow depth
- Shear stress

These hydraulic variables and properties are essential information for new bridge design, evaluating existing bridges for scour potential, and countermeasure design. The Federal Emergency Management Agency (FEMA) also depends on numerical hydraulic models of extreme events to determine flood hazards. The FEMA and the National Oceanic and Atmospheric Administration (NOAA) commissioned the National Research Council (NRC) to investigate the factors that affect flood map accuracy and identify ways of improving flood mapping (NRC 2009). Among their findings, the NRC recommended increased use of 2D models (NRC 2009).

Two-dimensional models are necessary to sufficiently analyze hydraulics when flow patterns are complex and violate 1D model assumptions. If the engineer has great difficulty visualizing the flow patterns and setting up a 1D model to represent the flow field realistically, that indicates a 2D model is necessary. One study that developed possible criteria for selecting 1D versus 2D models is *Criteria for Selecting Hydraulic Models* (NCHRP 2006). The following section provides a summary of the recommendations from that study.

Two-dimensional models involve less user judgment and are more intuitive to develop. They also provide more accurate results and more informative graphical output than 1D models. With advancements in computing power, software user interface improvements, high-quality data, and training resources, previous barriers to using 2D models have been diminished. For many project applications, experienced engineers can create 2D models faster than 1D models. Numerous conditions call for 2D modeling instead of 1D models when selecting the type of hydraulic model to use for bridge or transportation-related hydraulic problems. Table 5.2 provides hydraulic model selection recommendations for a variety of representative conditions typical for transportation hydraulic analysis. This table does not include 3D modeling or physical modeling because those methods are used primarily to simulate localized bridge elements and are rarely used to analyze entire floodplains and river channels in the vicinity of a bridge.

5.3.1 Multiple Openings

Multiple openings along an embankment are common in rivers with wide floodplains. Rather than using a single bridge, roadway embankments typically include additional floodplain bridges or relief culverts. Although it is possible to configure a 1D model to analyze multiple openings, the model's assumptions and limitations result in an extreme degree of uncertainty in results. The proportion of flow going through a particular bridge and the corresponding flow depth and velocity are crucial for structure design and scour analysis. Because multiple opening bridges represent a large investment, 2D analysis should be used instead of 1D.

Another type of multiple opening is multiple bridges in series. There are conditions when this bridge configuration should be analyzed using 2D models. These include unmatched bridge openings or foundations that do not align. A parallel highway or railroad crossing upstream or downstream may significantly alter the flow conditions and warrant 2D analysis.

Figure 5.4 shows 2D model results (velocity magnitude) for the U.S. Route 1 (US-1) crossing over the Pee Dee River in South Carolina. This model illustrates several reasons for selecting 2D modeling. The floodplain ranges from 4,000 to 8,000 ft and has a highly variable land use and vegetation. The US-1 crossing includes a 2,000 ft main channel bridge and two 500 ft, relief bridges. There is also a railroad crossing downstream. Although the railroad also has three bridge openings, they are shorter and slightly offset from the US-1 bridges. The highest velocity, greater than 8 ft/s occurs in the main channel. However, the center relief bridge has an average velocity of nearly 6 ft/s and the eastern relief bridge has velocities of over 7 ft/s. The floodplain area under the main channel bridge, however, has velocities ranging from 1 to 3.5 ft/s. Therefore, overall conveyance would be improved, and backwater would be reduced by shortening the main channel bridge and lengthening the relief bridges. If changing the bridge lengths would adversely impact the downstream railroad bridges, the 2D model results would also quantify those impacts.

Table 5.2. Hydraulic Modeling Selection.

Bridge Hydraulic Condition	**1D Model**	**2D Model**
Small streams	✅	✅
In-channel flows	✅	✅
Narrow to moderate-width floodplains	✅	✅
Minor floodplain constriction	✅	✅
Wide floodplains	⚠️	✅
Highly variable floodplain roughness	⚠️	✅
Highly sinuous channels	⚠️	✅
Multiple embankment openings	🚫	✅
Unmatched multiple openings in series	🚫	✅
Low skew roadway (<20%)	✅	✅
Moderate skewed roadway (>20% and <30%)	⚠️	✅
Highly skewed roadway (>30%)	🚫	✅
Detailed analysis of bends, confluences, angle of attack	🚫	✅
Multiple channels	🚫	✅
Small tidal streams and rivers	✅	✅
Large tidal waterways and wind influenced conditions	🚫	✅
Detailed flow distribution at bridges	🚫	✅
Significant roadway overtopping	🚫	✅
Upstream controls	🚫	✅
Countermeasure design	🚫	✅

✅ well suited, ideal use

⚠️ possibly suitable, but use with caution

🚫 unsuitable or not recommended

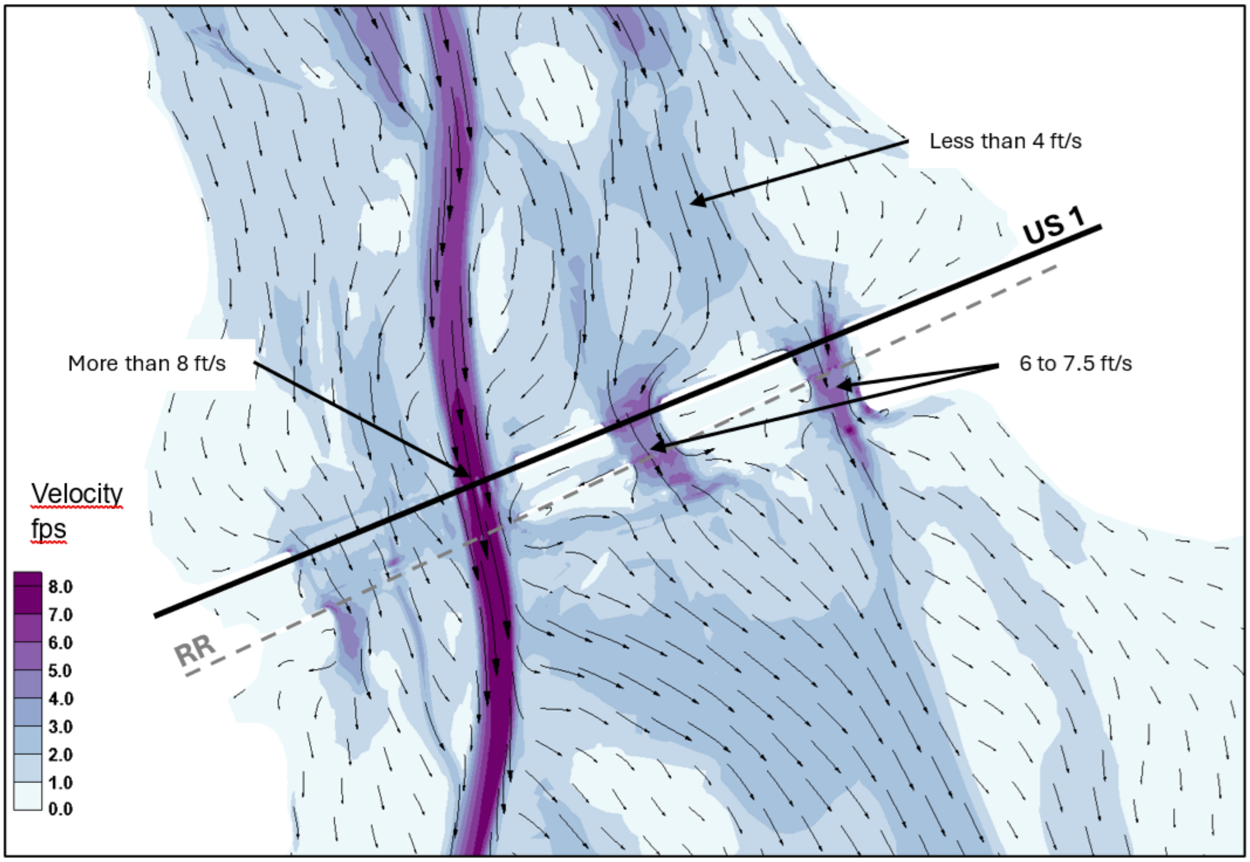

Figure 5.4. Two-dimensional model velocities, US-1 crossing the Pee Dee River.

5.3.2 Wide Floodplains

Floodplains often include features that significantly impact flow conveyance and flow distribution. Abandoned historic channel alignments and changes in land use or vegetation affect floodplain flow distribution. In a 1D model, two cross-sections that are a short distance apart may have significantly different vegetation, such as wooded versus cleared, or may have significantly different topography due to land use activities. If the engineer uses these cross-sections exactly as they exist, the 1D model depicts a sudden change in flow distribution that is not physically possible. To better depict the flow conditions, the engineer can adjust the cross section locations or alter the Manning n values, although this is difficult to implement.

Two-dimensional models avoid these difficulties because, in the simulation, all the flow is interconnected. Therefore, wide and complex floodplains benefit from 2D analysis.

Figure 5.5 shows 2D model results (unit discharge) for the U.S. Highway 641 (US-641) crossing of the Middle Fork of the Clark River near Murray, Kentucky. The floodplain is relatively large, approximately 2,500 ft, compared to the channel width of approximately 50 ft, and has highly variable land use and vegetation. Upstream of US-641 the highest concentration of discharge is in the right floodplain instead of the main channel. This roadway crossing has a relief bridge to help convey the large amount of flow in the floodplain more efficiently than forcing it back to the main channel bridge.

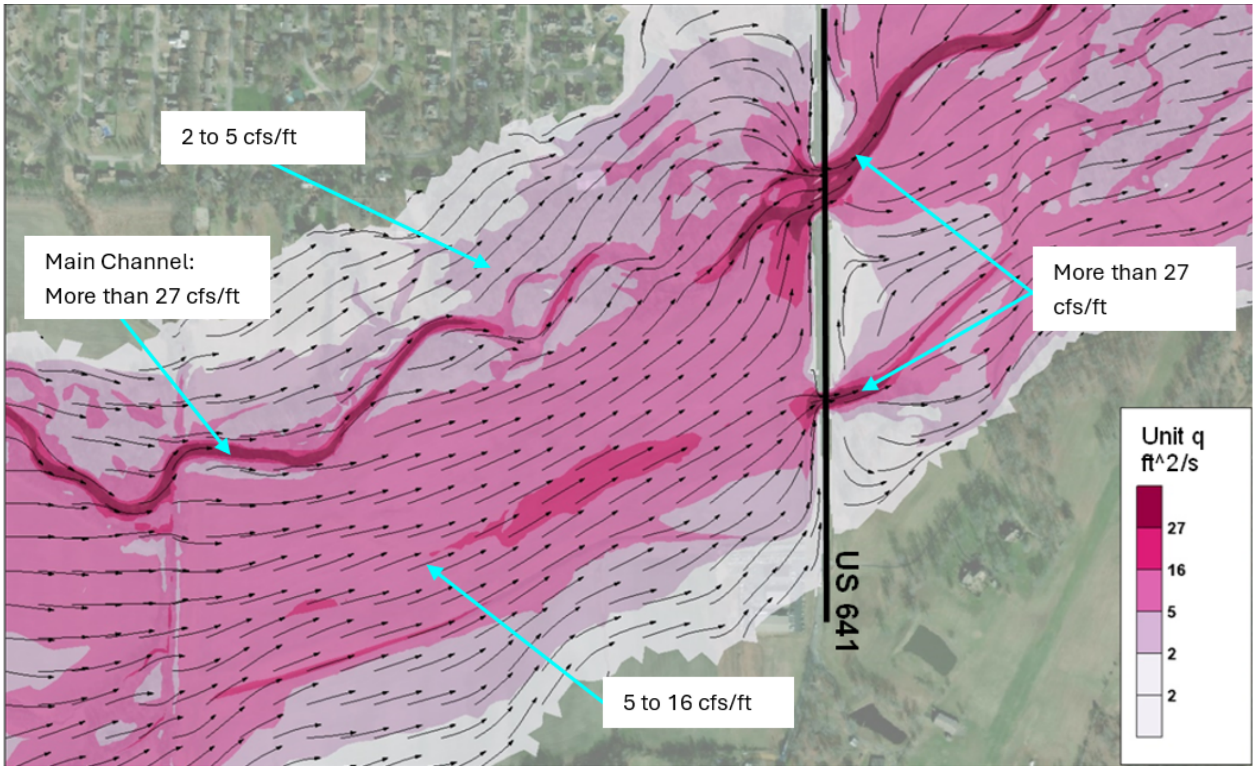

Figure 5.5. Two-dimensional model unit discharge, US-641 crossing the Middle Fork of the Clark River.

5.3.3 Skewed Roadway Alignment

Ideally, roadways are aligned perpendicular to channel and floodplain flows. However, other design constraints often necessitate skewed roadways. FHWA (1978) indicates that skewed crossing with angles of up to 20 degrees produced no objectionable flow patterns. The *HEC-RAS Reference Manual* (USACE 2021a) indicates that using the projected opening in a 1D model is adequate for skew angles of up to 30 degrees for minor flow constrictions.

Two-dimensional modeling is the suggested approach for higher skew angles or moderate skew combined with moderate to high flow contraction. Two-dimensional models better define flow patterns and bridge conveyance and help provide insight into potential backwater problems.

Figure 5.6 Shows a crossing with approximate 25-degree skew to the floodplain with flow from top to bottom. This figure illustrates how floodplain impacts can vary greatly upstream of a skewed crossing. The contours represent the difference in water surface between natural (no bridge crossing) and existing conditions. The darkest color shows the greatest water surface increase and the opposite side of the embankment shows a decrease in water surface compared to natural conditions. The fact that this is also a multiple opening crossing also complicates the hydraulics.

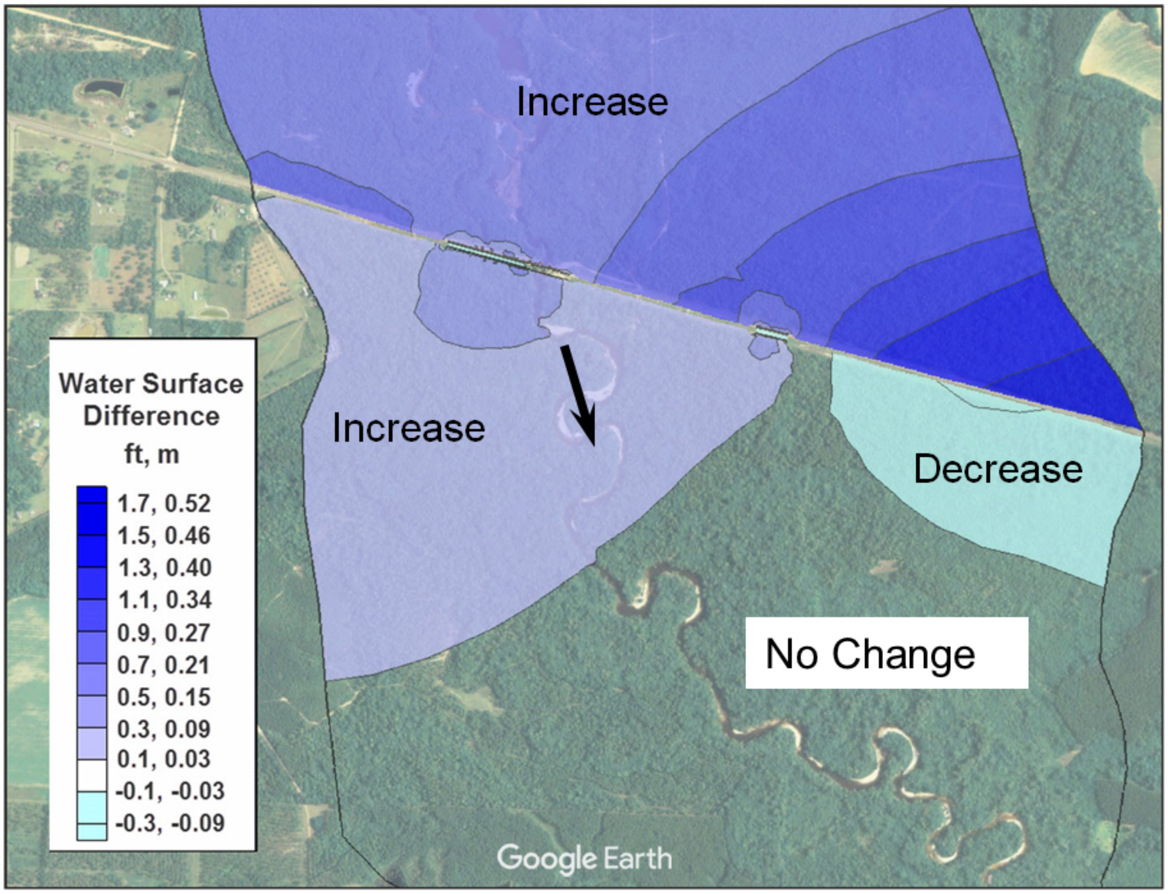

Figure 5.6. Backwater at a skewed crossing.

5.3.4 Road Overtopping

When computing road overtopping, the HEC-RAS 1D model (USACE 2021a) uses the total energy grade line at the cross section upstream of the bridge as the head value in the weir equation. This assumption is reasonable for many conditions. Because standard use of ineffective flow areas can trigger full floodplain flow for any amount of overtopping, The *HEC-RAS Reference Manual* (USACE 2021a) recommends comparing the road overtopping discharge to the floodplain flow and adjusting the Manning n to improve continuity in the lateral flow distribution.

As illustrated in Figure 5.7, for roads crossing wide floodplains or skewed crossings, 2D models offer a better approach. Two-dimensional models can handle the flow types associated with roadway overtopping, such as subcritical, supercritical, and transitional flow. The spatially varying water surface elevation and velocity upstream of the road determine flow over the weir, instead of the single-value energy grade line across the entire upstream cross section. As long as the mesh geometry sufficiently represents the weir crest along element edges, 2D models can better estimate the initiation of overtopping and associated discharges.

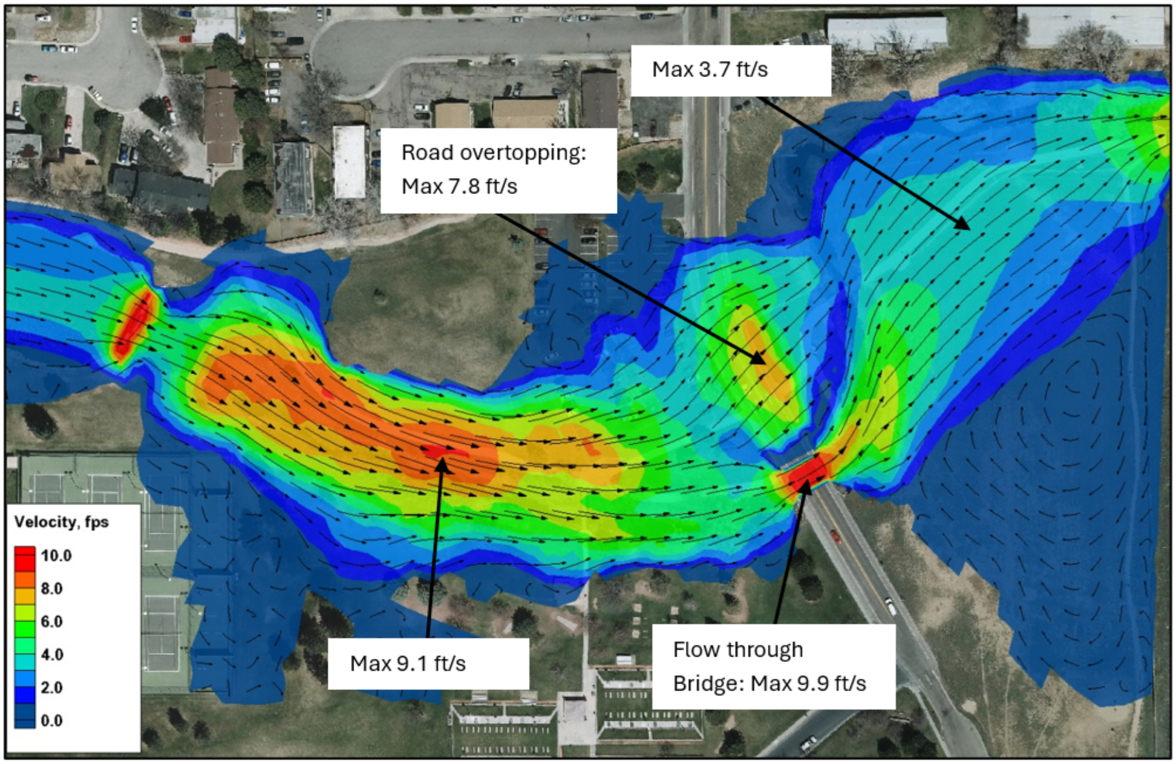

Figure 5.7. Two-dimensional model velocity contours at a bridge crossing with roadway overtopping.

5.3.5 Upstream Controls

For subcritical flow conditions, calculations progress from downstream to upstream. Locally, however, flow distribution, flow depth, velocity magnitude, and velocity direction can be controlled by upstream structures and obstructions. In 1D modeling, the usual approximate approach is to incorporate ineffective flow areas to account for upstream obstructions. While this approach affects the overall flow area and capacity of a cross section, the solution is inaccurate because the conveyance distribution determines the flow distribution. Therefore, 1D models do not fully account for upstream effects.

Figure 5.8 shows velocity conditions at the I-35W crossing of the Mississippi River in Minnesota. This figure illustrates that 2D models can be used to accurately determine whether an upstream condition impacts a downstream structure, even in sub-critical flow conditions. The I-35W Bridge is located downstream of St. Anthony Falls Lock and Dam, which concentrates the approach flow to the I-35W bridge and downstream pier on the 10th Avenue Bridge. During extreme events, the lock and dam could be operated with flow primarily through the three gates (as shown), or additional flow can pass through the lock chambers. For this situation, the conveyance distribution of flow in the downstream channel does not provide an accurate representation of the site hydraulics.

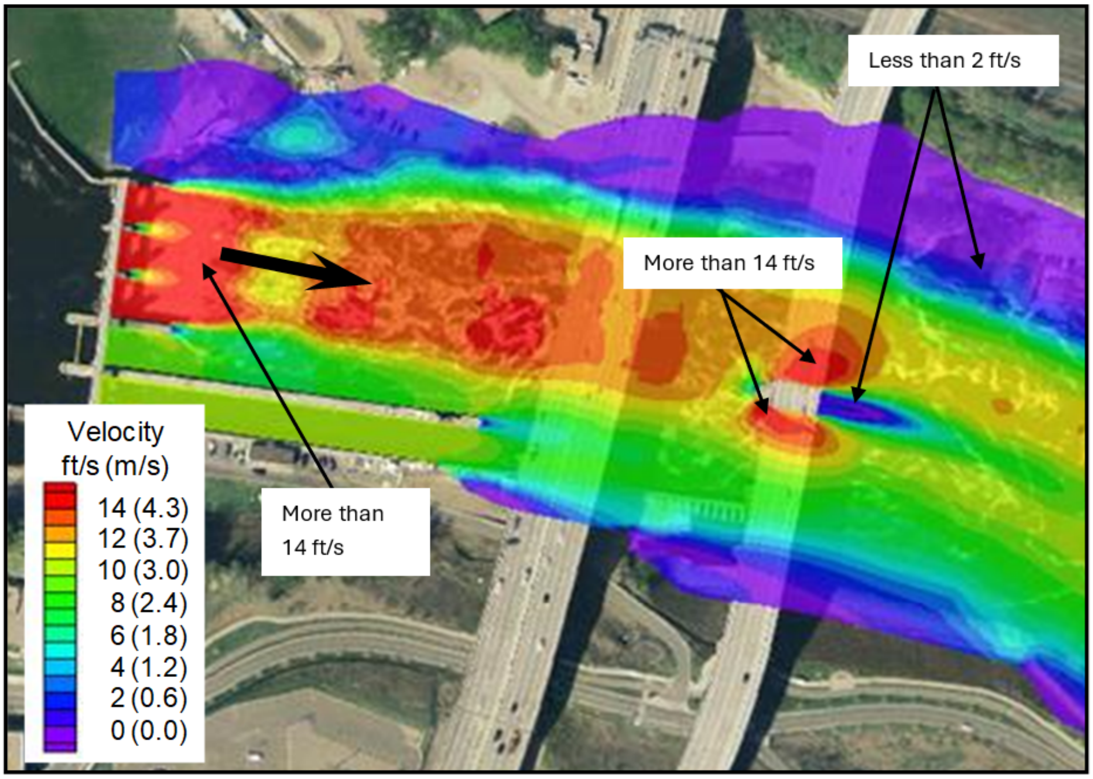

Figure 5.8. Two-dimensional model velocities, I-35W over Mississippi River.

5.3.6 Bends, Confluences, and Angle of Attack

Highly sinuous rivers are, by definition, not 1D, especially during floods when water in the floodplain flows more directly down valley and moves in and out of the channel. One-dimensional models consider different channel and floodplain flow distances between cross sections and compute a discharge-weighted flow length. Two-dimensional models do not make any simplifying assumptions related to channel versus floodplain flow distances because they compute flow paths as part of the solution.

Flow conditions at confluences also vary depending on the main stem's flow versus the tributary. With a 1D model of a bridge at a confluence, determining the attack angle for pier scour calculations is highly subjective. Two-dimensional models provide improved estimates of the attack angle because the model directly computes the velocity direction.

5.3.7 Multiple Channels

Anabranched and braided rivers have multiple channels and flow paths that complicate hydraulic calculations. Figure 5.9 shows an extreme example of multiple channels where Interstate 95 (I-95) and U.S. Highway 17 (US-17) cross the Altamaha River in Georgia. The area is subject to riverine and tidal flooding. Not only are there nine crossings (five on I-95 and four on US-17), but there are more than 20 individual channel segments or reaches that would be necessary if applying a 1D split-flow model. The engineer would also have to decide the amount of adjacent floodplain to assign to each channel segment and may well need to allow for lateral flow between floodplain segments. As a result, 2D models have advantages in this situation, including less effort to set up the model.

Figure 5.9. Channel network, Altamaha Sound, Georgia.

5.3.8 Tidal Conditions and Wind Simulation

Figure 5.10 is an example of the complex channel and hydraulic conditions that occur more frequently in tidal waterways than upland rivers. Tidal waterways include inlets, estuaries, bays, and passages. Causeways cross many bays and estuaries with multiple bridge openings, and the potential exists for overtopping and wave attack. The HEC-25 document (FHWA 2020) includes information and guidance on tidal and coastal conditions, including tides, storm surges, and wind, impacting transportation structures. Wind-driven currents dominate some coastal hydrodynamic conditions. Many 2D models include wind stress acting on the water surface as a boundary condition. Therefore, 2D models are better suited for many coastal bridge hydraulics analyses.

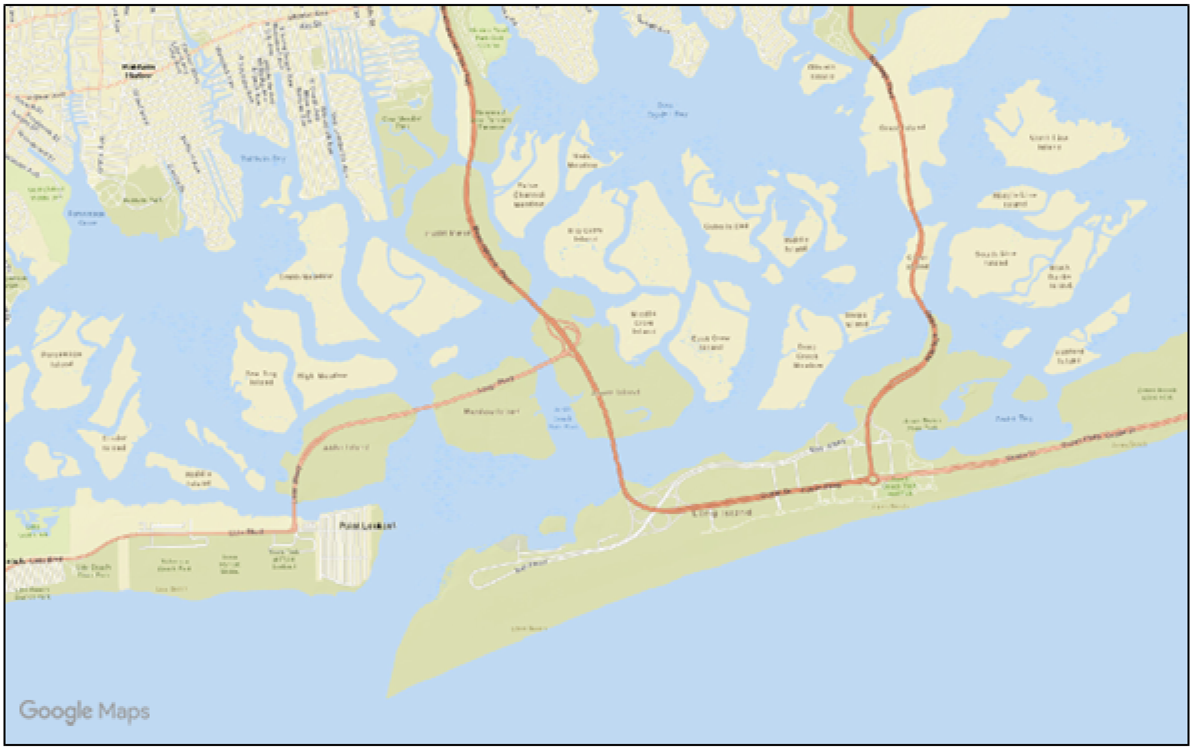

Figure 5.10. Complex tidal area, Long Beach, New York.

5.3.9 Flow Distribution at Bridges

HEC-18 (FHWA 2012b) establishes scour evaluation procedures recommended by FHWA. Flow and velocity distributions within the bridge opening aid in calculating contraction, pier, and abutment scour. One-dimensional models estimate the flow and velocity distribution based on the cross section's conveyance distribution (see Section 6.2). This assumption means that each point in the cross section also has the same water surface elevation and energy slope.

Figure 5.11 shows water surface contours and velocity vectors from a 2D model. The model represents a simple bridge crossing, but the results show that 1D models do not fully represent the hydraulics at even simple bridge crossings. In this figure, the thick lines indicate the channel banks and embankment. The water surface is relatively uniform upstream of the bridge. However, the velocity vectors in the bridge opening's overbank areas are not perpendicular to the bridge face, and the water surface varies by almost 1 foot from the overbank to the channel immediately upstream of the bridge. Although these are indicators that the flow is not 1D, the most significant departure from the 1D model assumptions is the velocity in the overbank areas under the bridge. A 1D model would estimate much lower velocity in the overbanks based on conveyance and equal energy slope at the bridge cross section. The average energy slope in the overbank areas under the bridge is over five times the channel area's energy slope. The resulting velocities are more than twice what a 1D conveyance-weighted calculation would determine.

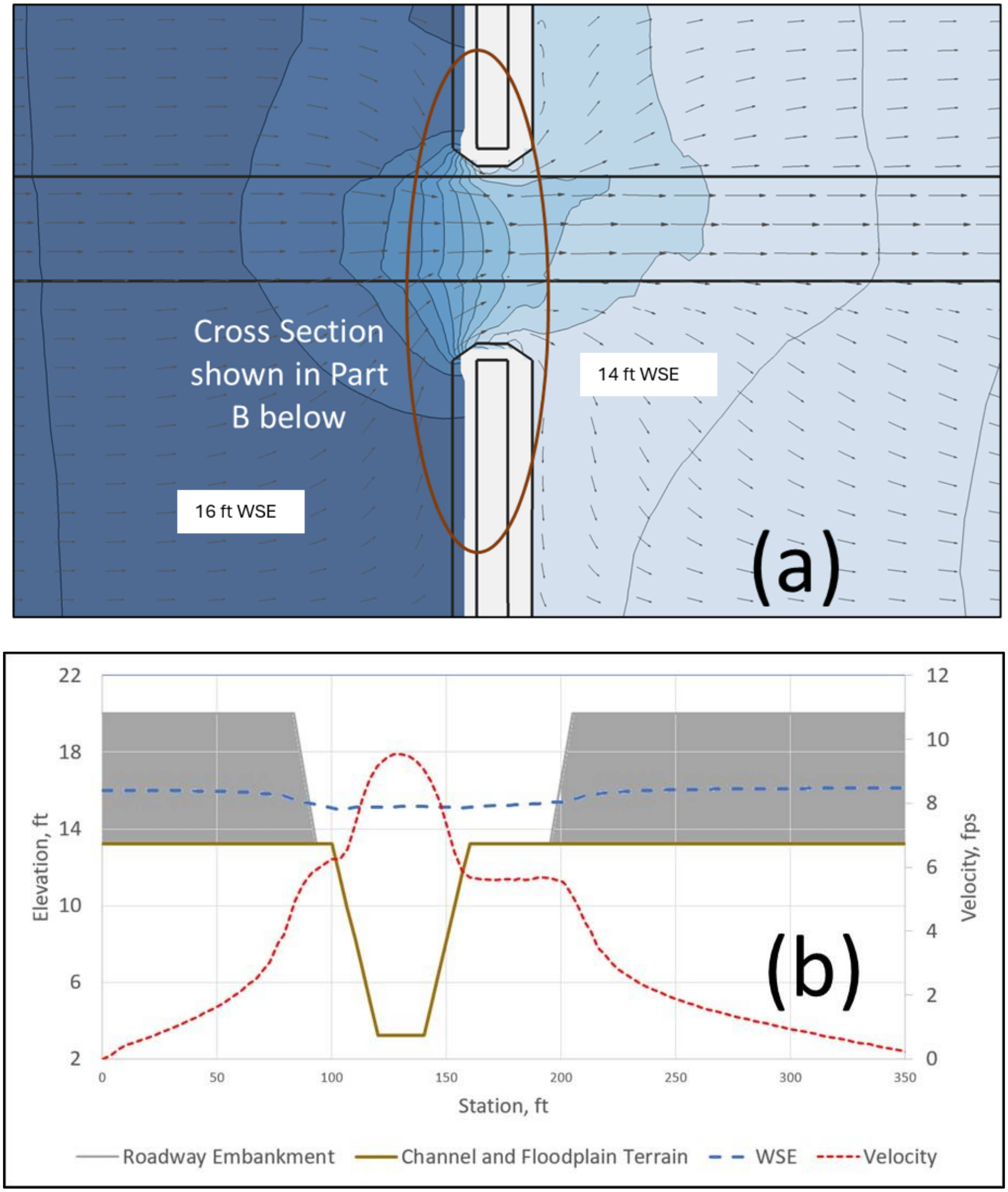

Figure 5.11. Two-dimensional flow within a bridge opening. (a) Plan view showing water surface contours (0.2 ft interval) and velocity vectors. (b) Cross section at upstream edge of bridge opening.

5.3.10 Countermeasure Design

HEC-23 (FHWA 2009a) provides information on designing countermeasures for stream instability and scour. Many countermeasures, including spurs, guide banks, and transverse dikes, significantly alter flow paths and flow distributions. Two-dimensional models set up with a detailed representation of the channel and countermeasure terrain provide an accurate simulation of the flow field in the horizontal plane, including locations of high velocity, flow separation, and flow circulation. Figure 5.12 shows an example of using a 2D model to estimate the size of riprap for a proposed embankment protection countermeasure, using the HEC-23 riprap sizing equations.

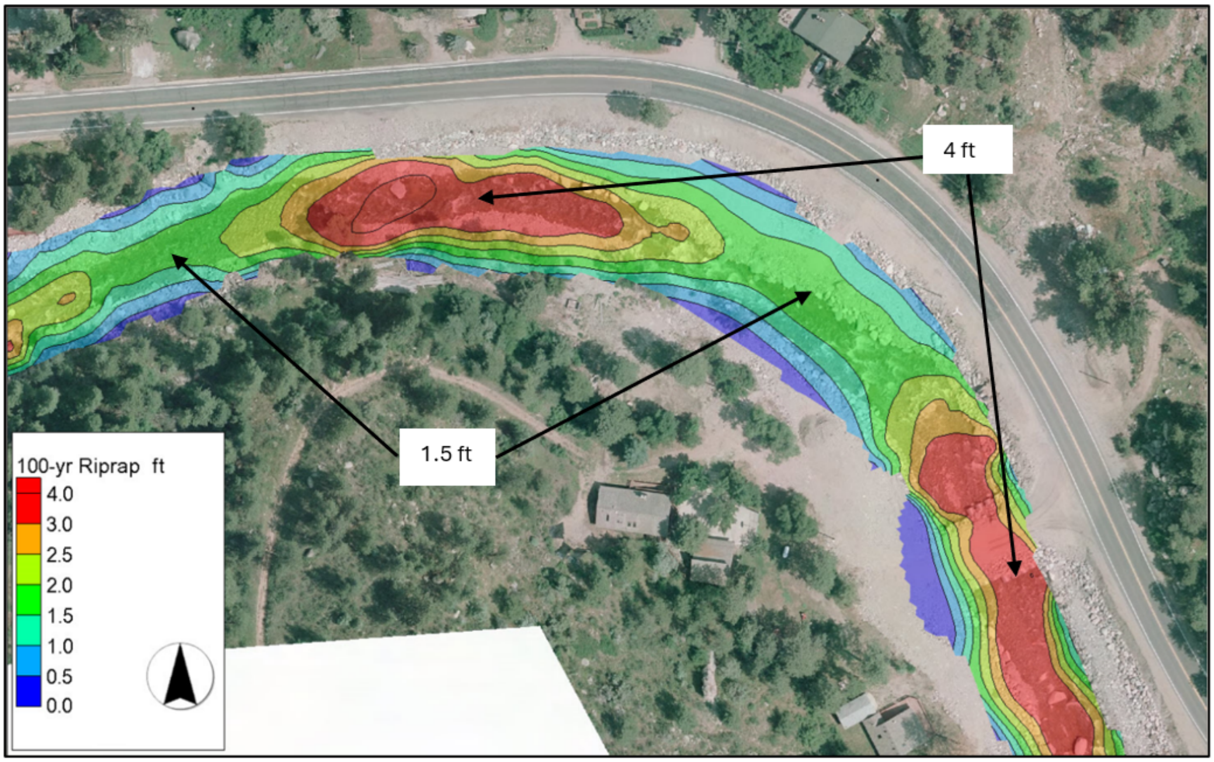

Figure 5.12. Approximate riprap sizing contours based on HEC-23 riprap sizing equations.

5.4 Model Terrain Data & Sources

Regardless of application, all types of hydraulic models require the development of the model geometry from which results are calculated. For all types of models, the terrain data used to create the model geometry has the most significant impact on the quality of the model results and typically involves the most resources to obtain a sufficient representation of a project area. The better a model represents the project area by using high-quality data, the closer the model results emulate actual hydraulic conditions.

Representing hydraulic controls is the most essential aspect of developing hydraulic model geometry. The terrain data should be detailed enough to accurately depict hydraulic controls. The practitioner then creates the model geometry to represent the hydraulic controls. Chapter 7 discusses this concept.

Terrain data needs vary depending on the model application. Multiple data sources are typically necessary to create a digital representation of the entire model area. The following datasets are typically combined to create a sufficient 2D mesh geometry for transportation-related hydraulic analysis.

- Topographic data
- Bathymetric data of river channel

- Supplemental survey along primary roadway including structures

Light Detection and Ranging, or Lidar, is a common type of aerial survey method and is ideal for capturing high-resolution topographic data for large areas. Lidar data is available for most of the continental United States, but at various resolutions and often collected in a “patchwork” manner. More recently, the USGS has engaged in acquisition of nationwide lidar to provide a national baseline of consistent high-resolution topographic elevation data for the United States (USGS 2023).

Lidar systems excel at collecting a detailed representation of a project site, even providing bare-earth surface data through most vegetation. However, lidar data have limitations and often need supplemental survey data. There are situations where lidar typically does not provide sufficient data for bridge hydraulic analysis. For example, many lidar systems cannot collect data underwater or under bridge decks, both of which are needed for hydraulic analysis.

Because of these limitations, it is common to collect supplemental data to create a more thorough representation of the project location's terrain data. In perennial streams, water is typically flowing in the main channel during the lidar data collection. In such cases, additional bathymetric data representing the channel bottom is often necessary to represent the underwater portion of the channel more completely.

In addition to channel data, it is common to collect ground survey data along the primary roadway. This data provides more detail for roadway and structure features such as guardrails, abutments, floodplain under the bridge deck, and bridge/culvert structure data. Additional survey data may also be needed on other structures in the model domain depending on their hydraulic impact on the hydraulic analysis's primary focus. Existing and proposed structure design or as-built plans may provide sufficient structure data without the need for site surveys.

Terrain data is often available from online sources such as those shown in Table 5.3. Similar data sources exist for State, regional, county, and municipal entities. Engineers should become familiar with available local geospatial data resources. Online data sources will likely not provide all the data needed to create and run a 2D model for detailed analysis, but they may be sufficient for planning purposes in many cases.

Table 5.3. Digital terrain data sources (only Federal sites shown).

Agency	Website
NOAA	U.S. Interagency Elevation Inventory
USGS	National Geospatial Program – The National Map
USGS	EarthExplorer
USDA	Geospatial Data Gateway
USGS	NLCD Product Data
NOAA	Bathymetry and Global Relief

Other data types are used to create hydraulic models. Chapter 4 of the *Two-Dimensional Hydraulic Modeling for Highways in the River Environment* (FHWA 2019) provides a more thorough overview of hydraulic modeling data types, additional sources, and other useful information.

5.5 Defining the Model Domain

The domain defines the outer spatial limits of a 2D model. Generally, the domain should fully encompass the potential inundated area and extend some distance upstream and downstream from the location of interest. Model boundary conditions are locations at which flow enters or exits the domain. Boundary condition locations should be far enough away from the area of interest that any errors or deviations from reality introduced at the boundary conditions are computationally insignificant in or near the project area.

Hydraulic models should extend laterally far enough to contain all the water or inundated area within the domain for the flood discharges to be analyzed. If the lateral extents are not sufficient, the edge of the model acts as a vertical wall both in 1D and 2D models. If a 2D model domain is excessively large, model runtime may be longer than necessary, creating inefficiencies in the modeling process. If the project location is tidal, the lateral extents may include the area up to the maximum high-water elevation applied at the downstream boundary condition. Floodplain maps and terrain data are helpful resources to help establish sufficient lateral domain limits. In all cases, a helpful practice is to initially create a coarse, simple model to help determine approximate lateral inundation extents.

5.5.1 Upstream Limit

At a minimum, an appropriate model domain extends far enough upstream to capture the flow contraction region upstream of the bridge crossing. Flow constriction is often the major contributor to backwater, so it is essential to include the complete flow contraction zone. Beyond this consideration, the upstream model limit, or limits if the analysis includes multiple inflows, should be located far enough upstream to allow natural flow distribution before the area of interest. A rule of thumb for initially locating the upstream limit is two to three floodplain widths upstream from the area of interest. This distance allows a 2D model to work out any potential errors associated with the assumed flow distribution at the boundary so that the hydraulic conditions at the location of interest are physics-based. Extending the upstream boundary may be necessary depending on available data for the project, unique topographic features of the location, and assumptions made.

Ideal upstream limits include more constricted locations within the floodplain where the flow is constrained, and hydraulic structures such as weirs and dams. Figure 5.13 shows an example where the floodplain is wide and complex two to three floodplains widths upstream from the project area (dot). However, farther upstream, the floodplain narrows and is less complex, which is a more advantageous upstream limit.

5.5.2 Downstream Limit

The downstream limit of both 1D and 2D models can significantly impact the accuracy of model results, particularly on river systems with relatively flat slopes. When the slope is flatter, the water surface changes can propagate farther upstream. Where uncertainty exists in the downstream boundary condition, extending the model domain farther downstream can reduce or remove the effects of that uncertainty on the results at the project site. As the model domain extends farther from the project area of interest, the accuracy of the terrain data and detailed representation of the terrain is not as critical and approximate data is appropriate if sufficiently far enough away.

Ideal downstream limit locations include sites with known water surface elevations (WSEs) for a given discharge, such as stream gauges, weirs, drop structures, and dams/reservoirs. If no measured or controlled value exists at the downstream limit, approximating a WSE is necessary. As suggested for the upstream limit, positioning the downstream limit using a rule of thumb of 2-3 floodplain widths downstream of the location of interest is a good starting point. Extending the limit farther downstream may be warranted depending on the project needs, unique topographic features of the location, and assumptions made.

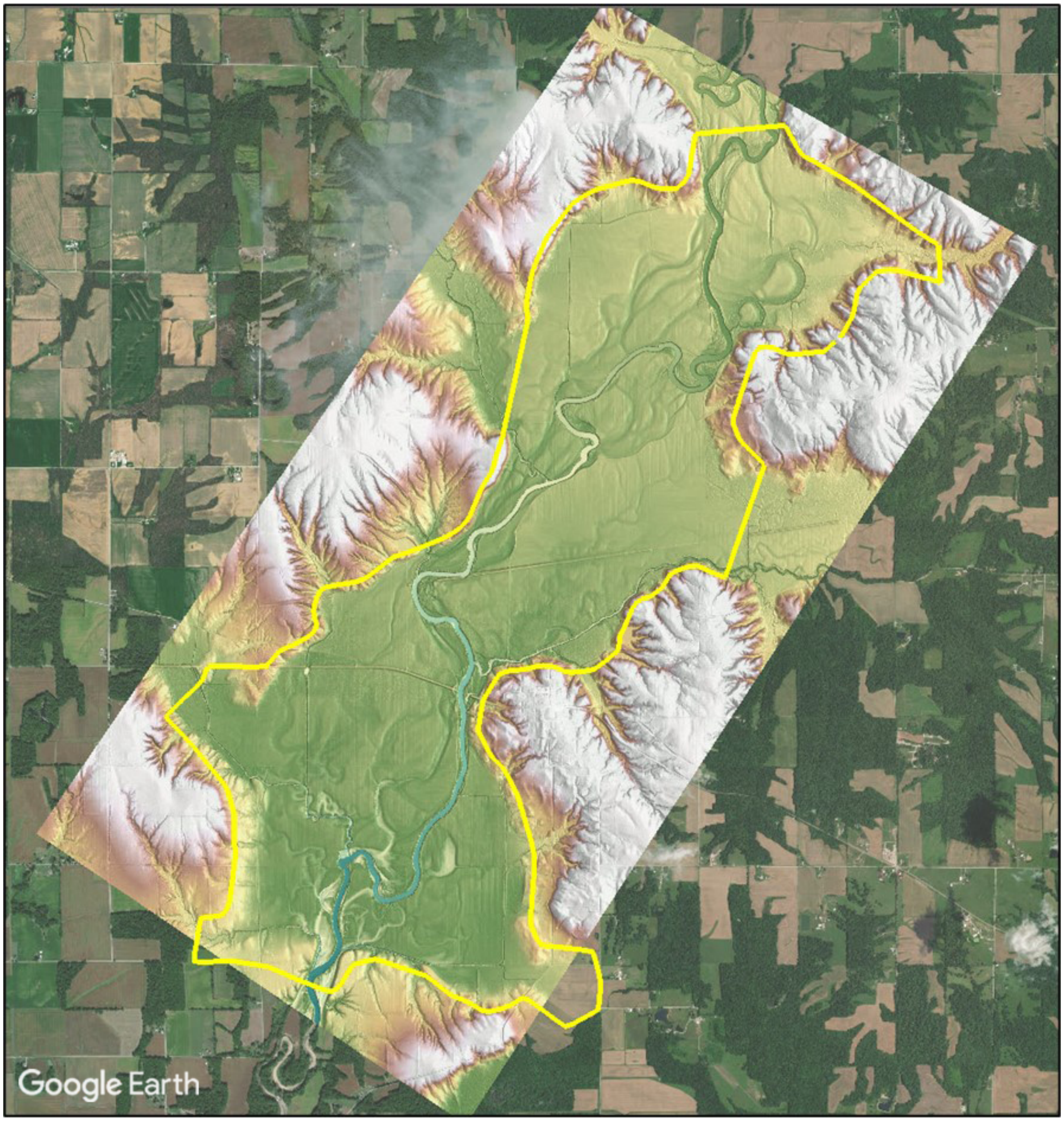

Figure 5.13. Example domain for a 2D model.

As with the upstream limit, it may be advantageous to locate downstream limits at locations with a narrow floodplain and minimal hydraulic complexity (see Figure 5.13). With any downstream limit, it is good practice to conduct sensitivity testing. Section 6.4 in *Two-Dimensional Hydraulic Modeling for Highways in the River Environment* (FHWA 2019) includes more information related to testing model sensitivity to various aspects of hydraulic models. The sensitivity test ensures that variations in the downstream boundary do not change model results at the bridge site or other areas of interest. Figure 5.14 shows how changes in the downstream boundary propagate upstream some distance. The distance of impact varies depending on slope and local terrain features.

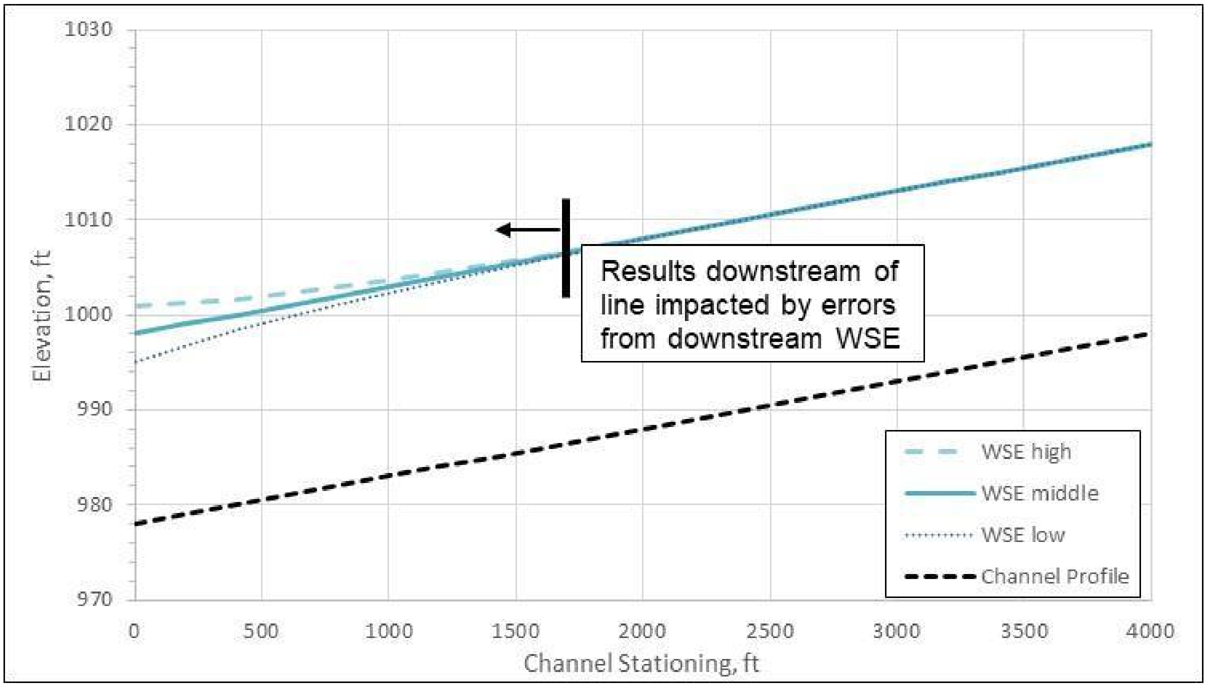

Figure 5.14. Water surface profiles with downstream boundary condition uncertainty.

The specified downstream water level may be transient, as is the case with a tidal boundary. In this situation, flow may enter the domain from the downstream end, and the discharge at the bridge site may change in response to the downstream water surface changes. The engineer can determine the flow conditions at the bridge using unsteady flow modeling in such cases.

5.6 Riverine Hydrologic Information & Analysis

While hydrology has a much broader definition, this section focuses on quantifying the amount of water or discharge that would flow to a highway crossing in an event having a given annual exceedance probability. The engineer can use the determined quantity, as a single constant value or a time series of values, as the upstream boundary input for a hydraulic model. Several methods for determining the discharge have been developed based on physical processes of the hydrologic cycle (deterministic methods) or statistical data analyses that involves various amounts of data and engineering judgment. The method used for a given project depends on the project needs and data availability.

5.6.1 HDS-2 – Highway Hydrology

FHWA published HDS-2 *Highway Hydrology* (FHWA 2023b) to discuss hydrologic approaches, methods, and assumptions pertinent to transportation engineers. HDS-2 provides information, data needs, applications, and limitations for commonly used hydrologic methods to design transportation structures. processes such as rainfall, runoff, storage, and routing are discussed in detail to provide the user with the background necessary to make design decisions.

Deterministic approaches in HDS-2 include the unit hydrograph method, SCS method, and rational method. Statistical theory and flood frequency methods commonly used at riverine crossings, such as regional regressions and log-Pearson Type III, are also provided. HDS-2 includes descriptions of hydrologic data sources and typical practices for collecting and compiling the data used to perform each method. Ultimately, HDS-2 provides insights on method selection based on the project needs and data availability for highway projects.

5.6.2 General Background on Hydrology

The hydrologic cycle describes water movement between the atmosphere, surface, and ground and the transformation of phases (gas, liquid, and solid) as it moves. Major hydrologic processes include precipitation, condensation, evaporation, infiltration, transpiration, percolation, storage, and surface runoff, shown in Figure 5.15. HDS-2 explains each of these processes.

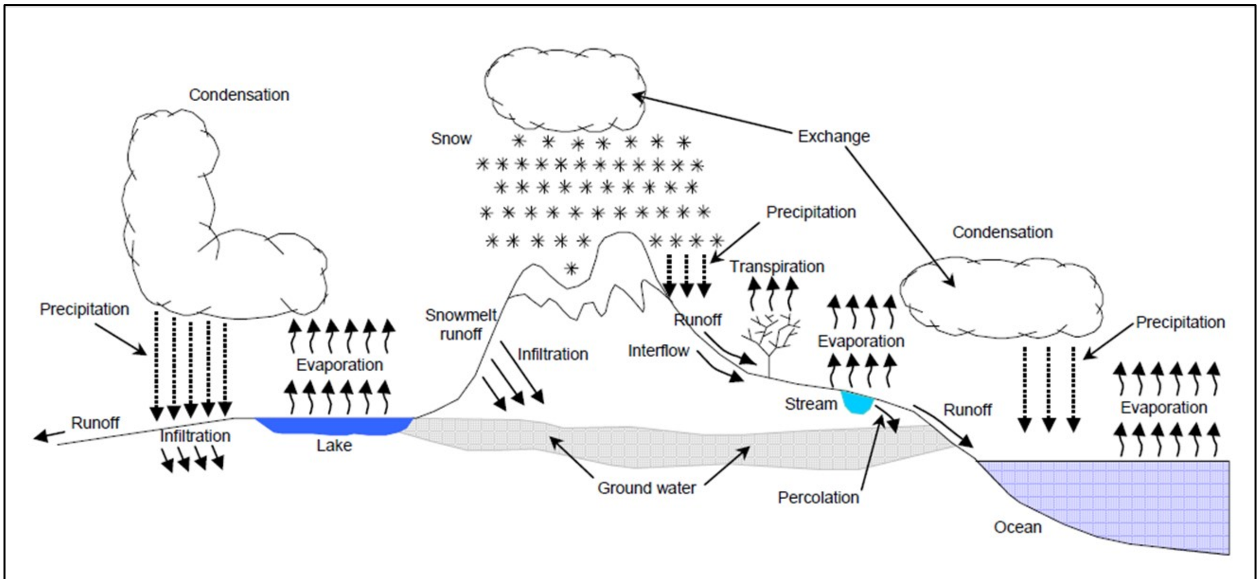

Figure 5.15. An illustration of the hydrologic cycle. Image taken from HDS-2 (FHWA 2023b).

Transportation engineers are primarily concerned with the surface runoff portion of the hydrologic cycle. Runoff is the overland flow of water due to excess rainfall or meltwater. In terms of hydrology, transportation engineers are concerned with the quantity and timing of runoff in rivers and lakes that may damage or impair the highway crossing during large storm events.

Other elements of the hydrologic cycle, including precipitation, infiltration, and storage, all influence runoff and highway design. As precipitation increases, the potential for runoff may also increase depending on the watershed's infiltration and storage capacity. The duration, depth, and intensity of rainfall, combined with surface infiltration in the watershed, determines the amount and timing of excess rainfall, which becomes runoff.

Urban development and other land-use changes can affect runoff by changing the watershed's infiltration and storage capacity. Therefore, it can be important to evaluate the impact of future development, water supply, and flood protection projects on the design of highway crossings. Climate change is another potential source of change in the hydrologic cycle. Chapter 4 of HEC-17 *Highways in the River Environment-Floodplains, Extreme Events, Risk and Resilience* (FHWA 2016) discusses the potential non-stationarity of hydrologic data that can result from many processes, including climate change. The depth of consideration for various elements of the hydrologic cycle and runoff depends on the project location, data availability, and project needs.

5.6.3 Hydrologic Factors and Data

There are three basic elements for hydrologic analyses: (1) measurement and compilation of data, (2) interpretation and analysis of data, and (3) application to design or other practical problems. The extent to which each of these elements need addressing is dependent on the cost of the structure, acceptable risk, data availability, desired accuracy, and time and resource constraints of the project. These factors determine the method to be used and the data used to implement the analysis.

Engineers perform hydrologic analyses at stream crossings to determine either the peak discharge or flood hydrograph. The peak discharge specifies a design flow that the structure is designed to convey with an acceptable level of risk. An annual probability of exceedance (i.e., 100 percent AEP, 10 percent AEP, 1 percent AEP, etc.) or design flood recurrence interval (i.e., 1-year flood, 10-year flood, 100-year flood, etc.) is associated with the peak discharge.

Design flood recurrence intervals for hydraulic structures are often dependent on roadway classification. For example, minor roads (lower classification or traffic volume) may have lower design flood standards than major roads. Therefore, minor roads often experience greater risk than major roads during an equally significant event. The engineer is responsible for determining an acceptable level of risk and associated design discharge for the hydraulic analysis and design of the crossing structure. The design typically intends to prevent road overtopping and provide some freeboard beneath the bridge superstructure at the design peak discharge.

Design flood hydrographs enable the engineer to consider the time variation of runoff discharge or volume in the analysis. Hydrographs are useful in evaluating the impact of urbanization, storage, or other changes in a watershed on travel time, time of concentration, runoff duration, peak flows, and volume of runoff. The design of highway structures crossing streams seldom include flood hydrographs, so transportation engineers typically use peak discharge methods.

Engineers obtain data for peak discharges or flood hydrographs from a variety of sources. Federal agencies that provide public access to data include the United States Geological Survey (USGS), United States Army Corps of Engineers (USACE), National Resource Conservation Service (NRCS), United States Forest Service, United States Bureau of Reclamation (USBR), Tennessee Valley Authority (TVA), Federal Emergency Management Agency (FEMA), and United States Environmental Protection Agency (USEPA). Table 5.4 provides data sources, the associated agency, and links to the data for common data types typically used for hydrologic analyses at highway structures.

The USGS collects and publishes daily streamflow data at over 8,500 locations across the United States. Streamflow gages are vital to conducting flood frequency analyses at gaged locations. Streamflow gage records also allow for the development of regional regression equations that make estimating peak discharge at various return intervals possible at ungaged locations. Precipitation, soils, topography, and land use information are useful for applying the Rational Method, SCS Method, and Unit Hydrograph Method (all described below) at ungaged sites. Aerial imagery can be useful in discerning significant land-use changes over time.

Table 5.4. Hydrology data sources.

Data	Agency	Website
Streamflow	USGS	National Water Information System: Mapper
Precipitation	NOAA	U.S. Hourly Precipitation Data
Soils	NRCS	Web Soil Survey
Topography	USGS	TNM Download
Land Use	USGS	NLCD Product Data
Imagery	USGS	TNM Download

5.6.4 Commonly used Hydrologic Methods for Bridge Design

Multiple methods are available for determining flood discharges on bridge projects. Some methods are more suitable than others for specific sites and watersheds. Most bridge projects only use peak discharges for each relevant flood recurrence interval. The peak-flow methods described below are generally presented in order of preference, subject to the available data and depending on watershed characteristics.

Flood Frequency Analysis Using Gaged Data: The U.S. Government's suggested method for highway structure design is to analyze the annual peak discharge rates from streamflow gage records using the Log-Pearson Type III (LP-III) distribution to develop an estimated relationship between flood recurrence interval and peak discharge. The USGS document *Guidelines for Determining Flood Flow Frequency, Bulletin 17C* (USGS 2019) provides the nationally recognized standard of practice for flood frequency analysis of gaged data. The latest version of the USGS software PeakFQ incorporates the Bulletin 17C methods. Figure 5.16 is an example output plot from PeakFQ.

A streamflow gage should have at least 10 years of annual peak discharge data for use in the Log-Pearson Type III distribution to perform the flood frequency analysis. HDS-2 and Bulletin 17C provide additional guidance on preprocessing data, parameter estimation, and method limitations (FHWA 2023b, USGS 2019). In some States, the USGS has already performed flood frequency analyses at select streamflow gages. Also, many publications exist that provide further guidance for specific States and regions.

Published FEMA Data: If the floodplain has an associated FEMA Flood Insurance Study (FIS), the FIS report includes the peak discharge rates for several recurrence intervals at various locations along the floodplain. These discharge rates can generally be used for bridge design analysis unless other reliable methods provide evidence that the FEMA discharges rates are inaccurate. Whether or not the engineer uses them for design purposes, the FEMA discharge rates from the effective FIS typically are used for floodplain regulation compliance.

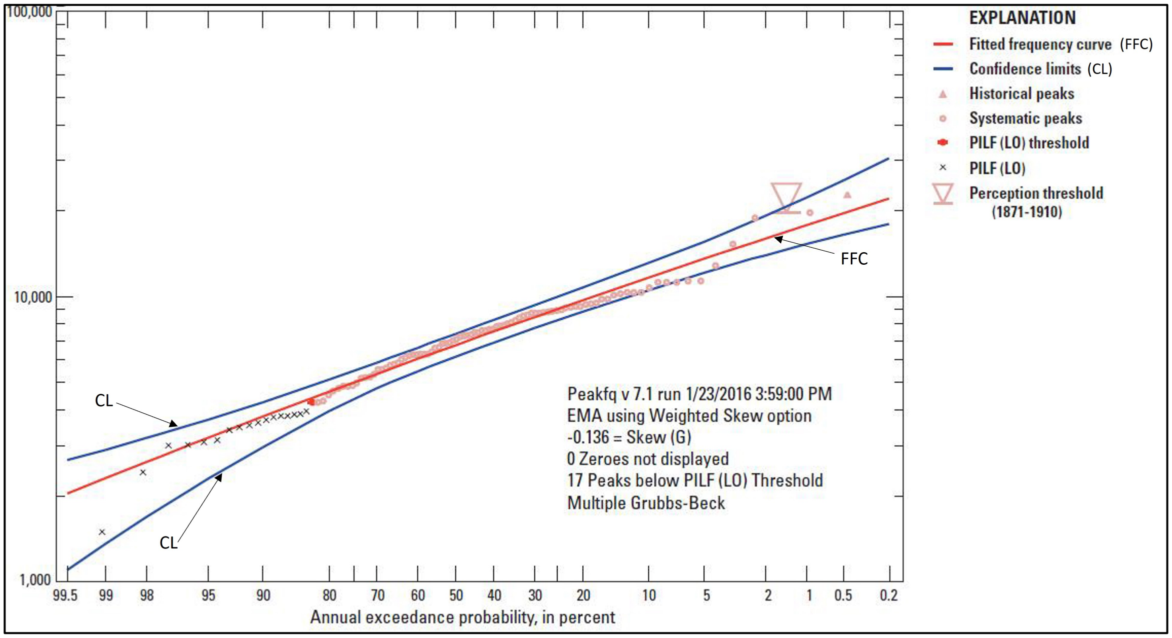

Figure 5.16. Example output plot from PeakFQ, from Estimating Peak-Flow Frequency Statistics for Selected Gaged and Ungaged Sites in Naturally Flowing Streams and Rivers in Idaho (USGS 2017).

Regional Regression Equations: The USGS, FHWA, and State departments of transportation (State DOTs) have developed regional regression equations as a simple method to estimate peak discharge throughout the United States. Flood frequency analyses predicted peak discharges for specific return intervals at gaged stations throughout a State or region. Those peak discharge rates were then regionally correlated with basin characteristics such as size and slope to yield regression equations applicable at ungaged locations.

The USGS web application StreamStats employs published regional regression equations to estimate the peak discharge for recurrence intervals ranging from 2- to 500-years at points selected by users on an interactive map. Note that regional regression equations have a high potential for error at any given location should only be used where gage data is not available. The application can be accessed at StreamStats.

If none of the methods above are practicable because of lack of data or deemed unsuitable for a particular site, HDS-2 describes other available methods (FHWA 2023b). Some other methods that engineers occasionally use include the following.

NRCS TR-55 Method. The NRCS TR-55 peak discharge method is described in detail in the NRCS publication *Urban Hydrology for Small Watersheds* (NRCS 1986) and is consistent with HDS-2. It applies to watershed areas less than 25 square miles. This method uses rainfall, land use, and ground cover as represented by the user's assignment of a Curve Number (CN). It also uses the Time of Concentration (Tc) parameter, which reflects how rapidly runoff can reach the point of interest from the most remote part of the watershed. The 24-hour rainfall volume for the flood recurrence interval of interest is the input for this method. In most States, the engineer obtains this value from the NOAA Atlas 14 Precipitation Frequency Estimates.

Rainfall-Runoff Hydrograph Methods: One of the peak flow methods described above is typically suitable for bridge projects. Some situations, however, may call for a hydrologic analysis using rainfall-runoff methods. Examples of situations calling for rainfall-runoff methods include:

- Large watersheds with no streamflow gage, where more accuracy is needed than can be achieved by the available regional regression equations.
- When a hydrograph is needed rather than just the peak flow, such as for unsteady flow modeling.
- When the duration of flow above a certain threshold is needed (e.g., duration of road overtopping).
- When peak flow attenuation is expected at the project site due to stream routing effects or storage in the watershed (such as wide, low gradient floodplains over a long reach).
- When flood duration is a limiting factor in calculating scour depth.
- When the watershed is complex and includes several subbasins of widely varying characteristics (e.g., rural to urban, mountainous to flat, partly burned watersheds, etc.).
- To aid in estimating the effects of future changes such as land use in the watershed or climate-related changes in rainfall.

Many rainfall-runoff methods are available. The methods differ in several ways, but they have common input data and parameters for the most part.

HEC-HMS, developed and maintained by the USACE Hydrologic Engineering Center, is a public-domain tool that is useful for performing rainfall-runoff analysis using various methods. It can be downloaded at HEC-HMS.

5.7 Defining & Assigning Inflow & Outflow Boundary Conditions

5.7.1 Inflow Discharge

A peak discharge value is the most common type of inflow boundary condition input for steady flow riverine hydraulic models. Multiple peak flow discharges may be necessary if the model domain includes tributary flow. Section 5.7.3 below provides an overview of how to determine what flow combinations to analyze when the project site is near a confluence. Unsteady flow models use inflow discharge hydrographs for upstream boundary condition input.

5.7.2 Outflow Water Surface Elevation (WSE)

The downstream boundary condition, where the water exits the model, commonly uses a WSE. A single WSE across the floodplain at the downstream end of the model does not typically represent reality in natural river systems. However, it is usually an adequate approximation if the boundary condition is far enough away from the location of interest. The WSE can be a single constant value or represented by a rating curve relating WSE to discharge. Common sources of known WSE’s include FEMA FIS, gage data from USGS, and observed high-water marks when the corresponding discharge is known or can be estimated. Previous 1D and 2D models can be sources of WSEs if available.

One can also use normal depth to approximate the downstream WSE if known values are not available. In natural channels, flow conditions are unlikely to satisfy normal depth criteria due to longitudinal topographic and roughness variations. However, normal depth is a reasonable approximation if more representative data is not available. Engineers often perform a sensitivity analysis to evaluate the potential hydraulic impacts of this assumption at the project location by varying the input energy slope. Extending the model farther downstream is another strategy to decrease uncertainty. Many hydraulic modeling software applications compute normal depth using an energy slope and discharge provided by the engineer.

Critical depth is an option for determining the WSE boundary condition value if the downstream boundary is known to be at a control section, such as an abrupt grade break or drop in the channel. Still, critical depth is rare in most natural alluvial channels.

One can develop rating curves from the sources mentioned above for a range of discharges. Some software applications provide additional capabilities in developing rating curves from normal depth and critical depth calculations. Occasionally a FEMA FIS only includes the 100-year profile or only includes flood discharges that do not match the desired discharge for the project. When confronted with this situation, the energy slope can be computed from the discharge with a FIS profile value and applied to other discharges for normal depth calculations at the boundary condition. It is common to use this approach in conjunction with extending the model farther downstream or performing a sensitivity analysis.

5.7.3 Coincident Flows at Confluences

When a bridge location over a stream is near a confluence with another stream, the engineer considers the other stream's potential influence on the hydraulics at the crossing. Questions to consider include:

- If the bridge is upstream of the confluence: How will the other stream affect the water surface profile through the bridge waterway for various flood recurrence intervals?
- If the bridge is within or near the floodplain confluence zone: How does the interaction between the flows from the two streams affect the distribution and direction of flow throughout the confluence area?

To appropriately consider the confluence effects, the two streams' coincident flow probabilities should be estimated. Consider a bridge crossing a minor tributary stream a short distance upstream from a major river, as illustrated in Figure 5.17. The tributary would likely have a much smaller contributing drainage area than the river. Different factors may produce severe flooding on the river than those that cause severe flooding on the tributary. Floods on large rivers, for instance, are often driven by spring runoff supplemented by long-duration spring rainfall and may last for weeks. Intense thunderstorms often drive floods on smaller tributaries at other times of the year and typically last a few hours.

Many hydraulic engineers intuitively recognize that the 100-year flood on a small tributary is not likely to coincide with 100-year flood levels on a large receiving river. However, it is necessary to determine the possible combinations of flow in the tributary and receiving river with a 100-year recurrence interval. Overestimating the coincident-probability discharge rates can lead to the engineer adopting a design condition with a greater recurrence interval than intended. Underestimating the values could lead to negative consequences, such as a bridge low chord profile that provides less freeboard than intended for the design event.

Available information on coincident flow frequencies at confluences has been scarce. NCHRP conducted a research project (Project 15-36) (NCHRP 2010c) intending to develop practical, reliable procedures for estimating coincident flow probabilities at confluences. This research provides event combinations with equal joint probabilities based on total drainage area and the ratio of the two basin drainage areas. For example, the engineer could analyze two basins with a total combined area greater than 350 square-miles and basin area ratio greater than seven (7) by combining the 2- with 100-year, 50- with 79-year, and 66- with 66-year, 79- with 50-year, and 100- with 2-year events on the two streams. The five combinations all have joint AEPs of 0.01

(100-year recurrence intervals). Although the combinations are equally likely to occur, any could produce the highest flooding at the bridge, produce the greatest scour potential, or control some other design aspect.

Figure 5.17. Highway crossing near a confluence of a smaller stream into a larger river.

5.7.4 Steady Flow vs. Unsteady Flow Modeling

A majority of riverine bridge hydraulic analyses use steady-state boundary conditions, where the peak flow conditions for various design events are used for hydraulic design and scour conditions. Chapter 8 provides information on modeling unsteady flows. Unsteady flow modeling is applicable for several conditions, including nearly all tidal applications. An exception for tidal models is when a peak discharge and the corresponding water surface elevation are reliably established by other means. On rare occasions, the engineer may need to conduct unsteady flow modeling to evaluate storage effects or for regulatory compliance when the jurisdictional floodplain regulation model is an unsteady flow model.

Some people often make a conjecture that increasing the size of a bridge increases downstream flooding by decreasing the amount of water stored upstream of the existing structure. Investigations into this topic by McEnroe (2006) concluded that structure-induced detention storage affects few highway culverts and even fewer bridges. Therefore, enlarging bridges and culverts seldom increases downstream flooding, with the benefits of providing greater capacity typically outweighing other concerns. Roads that overtop are unlikely to have increased downstream flow when constructing larger structures. Unsteady flow modeling is an ideal option to assess the potential for increased downstream flooding. However, the model extent is set to capture upstream available storage, and the downstream extent is set to account for dynamic routing effects. The downstream boundary condition is specified to apply to the full range of flows (rating curve and normal depth).

Although the potential downstream impacts to bridge enlargement are generally minor or negligible, bride enlargement benefits are often considerable. McEnroe (2006) indicates that benefits include reductions in backwater, flooding, overtopping, and scour. He also indicates that even if the downstream flows increase, the flow hydrograph will more closely resemble the natural conditions existing before constructing the roadway. Other benefits often include greater resilience and reliability, and improved aquatic and terrestrial organism passage.

CHAPTER 6
ONE-DIMENSIONAL BRIDGE HYDRAULIC ANALYSIS

6.1 Introduction

This chapter provides information on one-dimensional (1D) modeling techniques for bridge hydraulic analysis. Chapter 5 extensively describes the approximations, assumptions and limitations of 1D modeling. This chapter describes appropriate engineering practices one can employ when 1D modeling is to be performed. These practices, while improving the quality of 1D models, do not remove their underlying assumptions and limitations. The model yields accurate results only to the extent that the physical conditions of the river and bridge crossing fit within those limitations.

One-dimensional analysis can range from approximate methods requiring just a single waterway cross section to detailed water surface profile calculations involving many cross sections and multiple stream reaches. Approximate methods typically assume uniform flow (see Chapter 4). The uniform-flow assumption allows the use of Manning's Equation to calculate flow depth and elevation at a particular cross-section (Equation 4.10). However, assuming uniform flow in bridge hydraulics problems yields inaccurate results. Bridge-constricted floodplains and stream reaches usually exhibit non-uniform flow. Therefore, the broadly accepted practice employs water surface profile calculations when performing 1D modeling.

6.2 Water Surface Profile Computations with 1D Modeling

This section describes the most widely used approach to analyzing water surface profiles of natural streams with non-uniform flow conditions. This technique computes the water surface profile as a variable function along the stream's length. The continuity and energy equations discussed in Chapter 4 govern all the methods discussed in this section. One-dimensional modeling programs use the standard-step method to solve the governing equations along a stream reach. Other numerical approaches exist, such as the direct-step method. However, the standard-step method is better suited for cross sections with irregular geometry placed at known locations.

6.2.1 Cross Sections and Conveyance

Cross sections are the computational points in 1D hydraulic analysis. The standard-step calculations progress along the stream reach from one cross section to the next until each cross section has a calculated water surface elevation. The standard-step solution of the energy equation uses the concept of cross-section conveyance extensively. Conveyance is the hydraulic capacity of a cross-section deriving from its geometric shape, size, and roughness before considering the slope. Chapter 4 presents the expression for computing the conveyance of a cross section or a portion of a cross-section (Equation 4.12).

Cross-Section Location and Orientation: The engineer chooses the cross-section locations to capture transition points in the slope, width, geometry, flow rate, and roughness in the stream channel and floodplain. All cross sections are oriented perpendicular to the flow direction and have sufficient extent to contain the entire flow width.

Figure 6.1 depicts the cross-section layout for a floodplain model computed by the standard-step approach. The figure shows that angle points in the sections help to better represent the variations in flow direction across a floodplain. Calculations progress along the stream segment, one cross-section at a time. The hydraulic solution from the previously solved cross section and the user-specified information about the cross-section currently under consideration both contribute to the determination of the energy loss between the two cross-sections. Knowing the energy loss allows the computation of the energy grade line and water surface elevation at the current cross section.

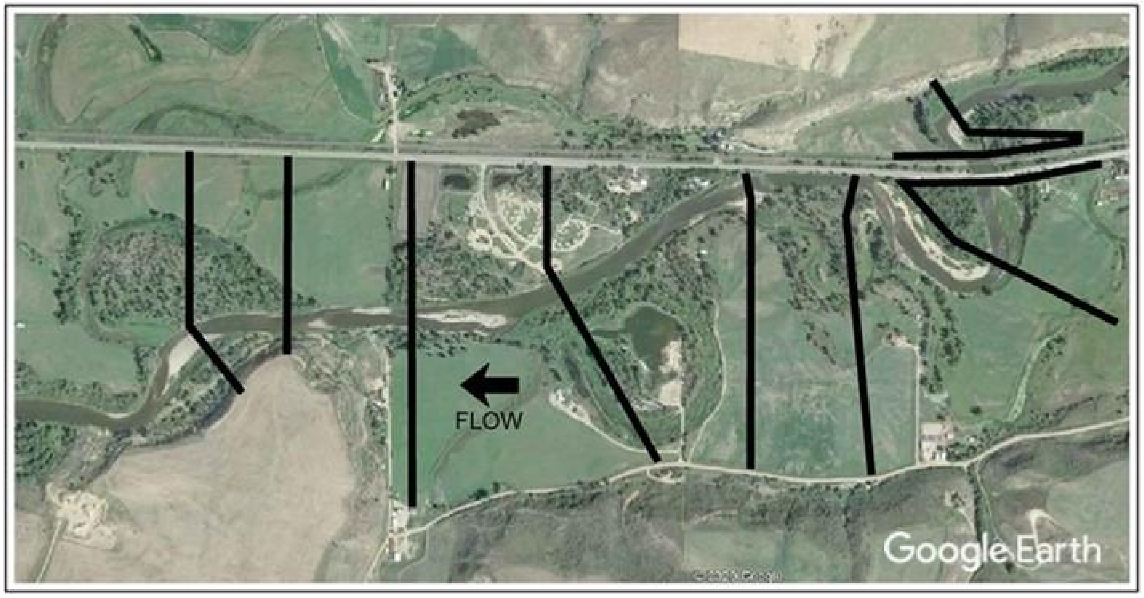

Figure 6.1. Cross-section layout for a standard-step floodplain model.

Equation 4.13 is the expression for determining a cross section's energy slope. The friction loss between two adjacent cross sections is the integral of the friction loss function. A simplified numerical integration is achieved by multiplying the average slope by the distance between the cross sections. Several methods are available to approximate the average energy slope between two cross sections (defined in Chapter 4 as the friction slope). The Average Conveyance Equation, presented below, is the most common method:

$$S_f = \left(\frac{Q_1 + Q_2}{K_1 + K_2}\right)^2 \tag{6.1}$$

where subscript 1 designates the downstream cross section, subscript 2 designates the upstream cross section, and the variables are:

S_f	=	friction slope (average energy slope) between two cross sections, ft/ft
Q	=	total cross-section discharge, cfs
K	=	total cross section conveyance, cfs

One can expect variation in conveyance when modeling natural streams and floodplains. Two cross sections near each other typically have more similar conveyance values than cross sections farther apart. A large difference in conveyance between two adjacent cross sections leads to a large difference between their energy slopes, as illustrated in Figure 6.2. Therefore, as the reach length increases, so does the potential error in the computed friction slope average between two cross sections. Additionally, as the reach length increases, the friction slope's error is applied over a longer distance, thus affecting the total friction loss. Keeping the reach lengths short from one cross section to the next minimizes the potential inaccuracy stemming from the variation of the friction slope. Thus, the engineer inserts additional cross sections to keep the reach lengths short enough to avoid significant error in the friction slope and total friction loss calculations (USACE 1986). The engineer strives for a balance between accuracy and economy in establishing the number of cross sections. One can use successive modeling trials to determine how many additional cross sections are needed. The number of cross sections is sufficient when inserting more cross sections does not significantly change the results.

Within the United States, a widely used 1D modeling program is the USACE's HEC-RAS software product. HEC-RAS generates warnings to alert users to potential model shortcomings. Several of the warnings within HEC-RAS provide checks on cross-section spacing and reach lengths. For example, when the conveyance ratio between two adjacent cross sections is less than 0.7 or greater than 1.4, HEC-RAS provides a warning. These warnings provide another guide for the appropriate distance between cross sections in a model. One can add cross sections where the warnings appear, with a goal of eliminating individual warning messages. However, one should not expect to eliminate all warning messages in most models.

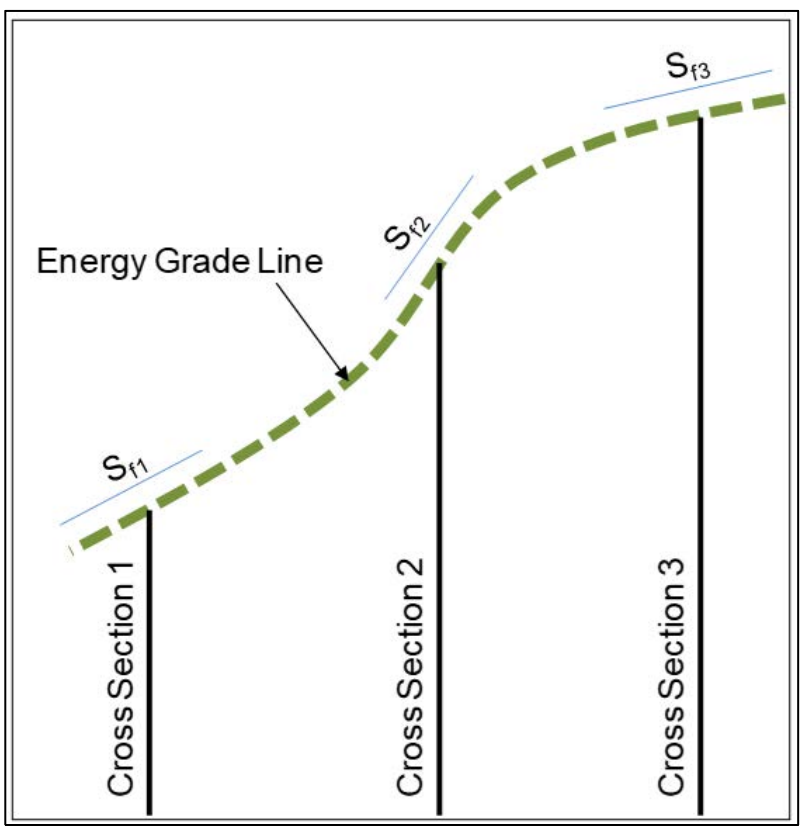

Figure 6.2. Profile-view illustration of the energy slope at each cross-section in a stream segment. Flow direction is right to left.

Cross-Section Subdivision: In a natural floodplain, the flow depth, roughness, and velocity usually vary throughout the width of the cross section. As shown in Figure 6.3, accurate conveyance calculations depend on subdividing cross sections into regions of similar flow properties.

As an illustration, consider the cross-section in Figure 6.3 (but without the indicated subdivisions) for a condition in which the water surface is just below the top of the left bank of the main channel. The conveyance would reflect only the area and hydraulic radius of the main channel. For a water surface elevation just above the top of the left bank, the water surface would extend out onto the left overbank. If the cross section were not subdivided, the wetted perimeter might increase by hundreds of feet, but the area would increase very little because of the small depth of flow on the overbank. The much-increased wetted perimeter divided into the little-increased area would lead to a decrease in the hydraulic radius, which would lead to a decrease in the conveyance compared to the water surface just below the channel bank. This condition is not physically realistic and causes a discontinuity in the calculated conveyance as a function of water surface elevation (see Figure 6.4). In physical reality, the small increase in water surface elevation would lead to a small increase in conveyance because the channel conveyance increases, and some floodplain conveyance is added. In a conveyance calculation without subdivision, the conveyance appears to decrease in response to the increased water surface. Cross-section subdivision avoids this error.

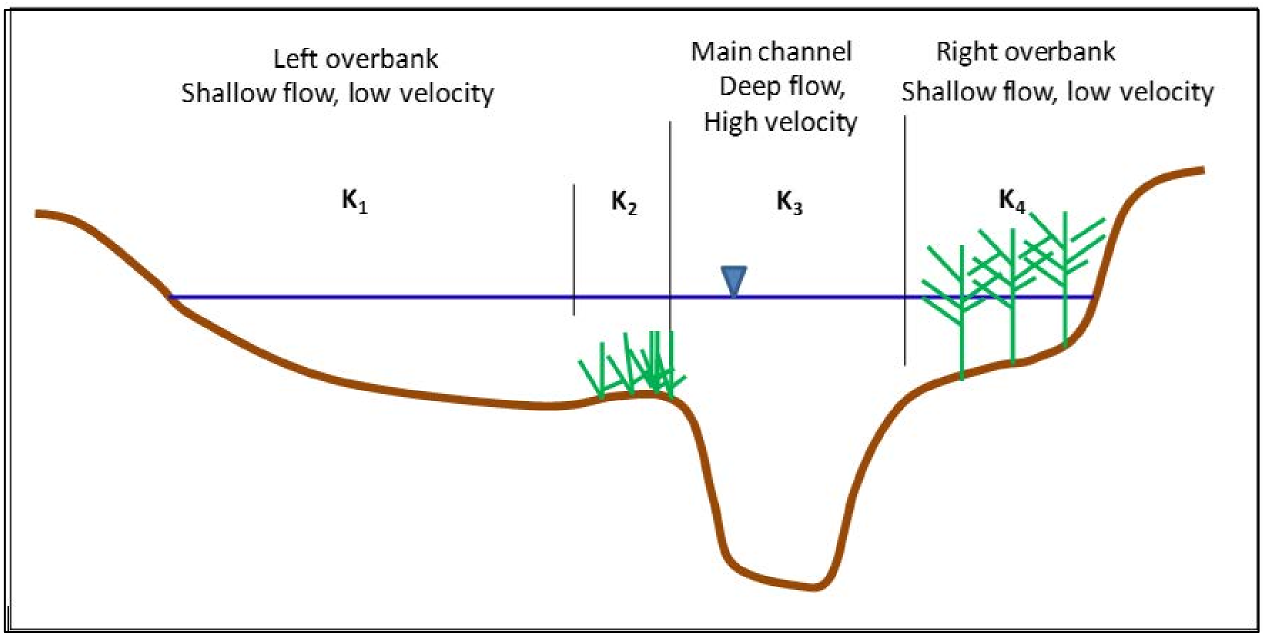

Figure 6.3. Example of an appropriately subdivided cross-section.

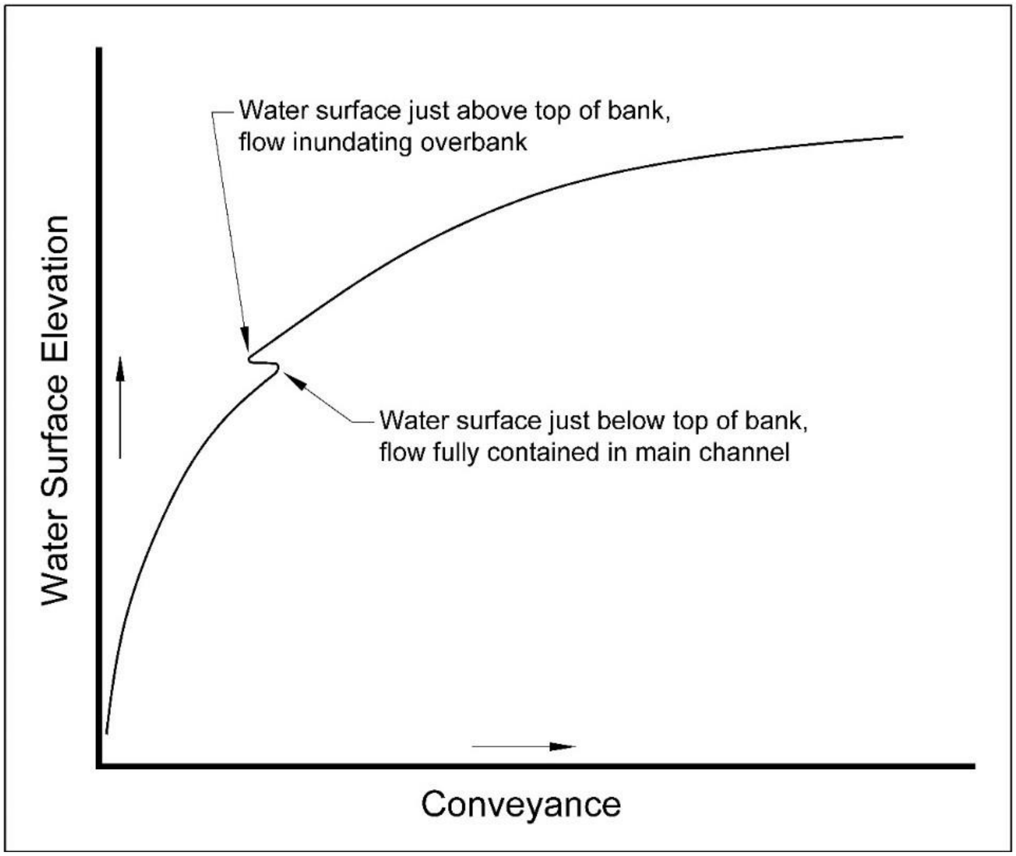

Figure 6.4. Discontinuity of computed conveyance for a non-subdivided cross-section.

When a cross section is subdivided appropriately, the velocity inside any subdivision is roughly uniform, even if the average velocities in two adjacent subdivisions are significantly different. The total conveyance of a cross section is the summation of each subdivision's conveyance, as presented in the equation below.

$$K = \sum_{1}^{N} K_i \tag{6.2}$$

where:

K_i	=	conveyance of a subdivision, cfs
N	=	number of subdivisions in the cross-section

The conveyance of a subdivision is:

$$K_i = \left(\frac{1.49}{n_i}\right) A_i R_i^{2/3} \tag{6.3}$$

where:

n_i	=	Manning's roughness coefficient for the subdivision
A_i	=	effective flow area of the subdivision, ft^2
R_i	=	hydraulic radius of the subdivision, ft

The example cross section shown in Figure 6.3 has four subdivisions, two in the left overbank and one each in the main channel and the right overbank. Overbank areas are subdivided at changes in roughness. However, in the main channel, it is more common to treat variable roughness (e.g., willows on the upper channel banks with an unvegetated channel bottom) by calculating a single composite Manning's n value that applies to the entire channel width. The subdivision of conveyance refines the accuracy of the total cross-section conveyance. It also provides a rational, though not always accurate, means of distributing the total discharge within the cross section. One-dimensional analysis programs, including HEC-RAS, distribute discharge within a cross section in proportion to the conveyance. If the left overbank has one-sixth of the total conveyance, for instance, the program assigns it one-sixth of the total discharge.

The calculations of the energy and momentum correction coefficients, the representative reach length between cross sections, and the average velocity in each subdivision are based on the distribution of flow between the main channel and overbanks. An approximate determination of the velocity distribution across a channel or overbank area is possible by further dividing these regions into smaller subdivisions and distributing the discharge to each subdivision in proportion to conveyance.

Ineffective Flow: A 1D modeling program assumes that any area in a cross section below the water surface elevation is available for conveyance and uses that conveyance unless the user specifies otherwise. The engineer may want to exclude a portion of a cross section from the conveyance for several reasons, including:

- That portion of the cross-section is in the stagnant or eddying wake area downstream of a large obstruction, such as a building
- It is immediately upstream of an obstruction such that any water moving in the area is in a lateral direction rather than in the downstream direction
- It is an area where water can pond but cannot effectively convey flow from upstream to downstream, such as an area behind a levee that is connected to the flowing water downstream but not upstream
- It is outside of the effective contraction zone upstream of a constriction or the effective expansion zone downstream of a constriction, such as a road crossing (discussed in more detail in the next section)

Engineers have used various approaches to exclude portions of cross sections from the conveyance, including raising the ground elevation value or artificially increasing the roughness coefficient. In the HEC-RAS program, the user can specify areas within a cross section where the flow is ineffective (the Ineffective Flow setting) up to a user-designated water surface elevation. If the water surface in the cross section reaches the designated elevation, the ineffective flow specification is nullified. The use of ineffective flow specifications plays an essential role in modeling bridge crossings with HEC-RAS, as discussed in later sections. Figure 6.5 is an example of ineffective flow specifications at a bridge crossing.

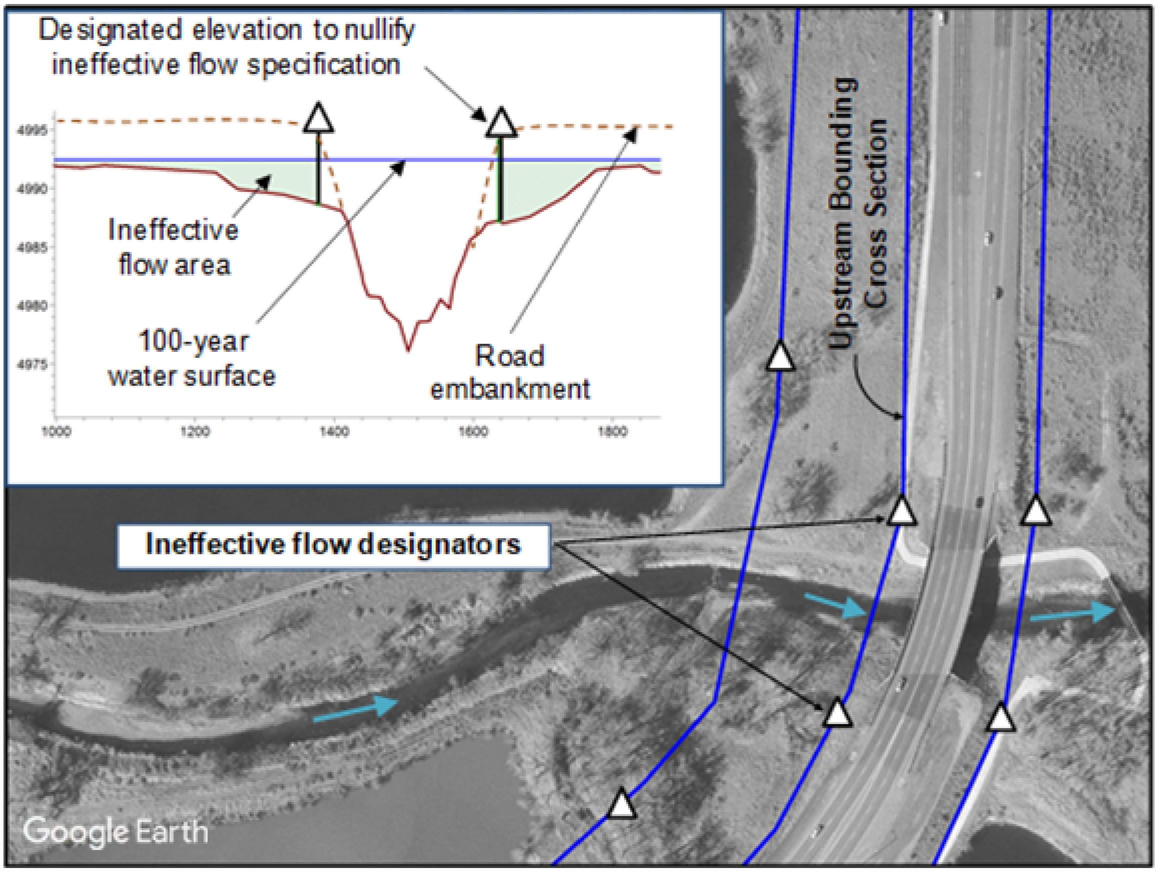

Figure 6.5. The use of ineffective flow at a bridge crossing.

6.2.2 Subcritical and Supercritical Flow Regimes

Natural streams and floodplains flow predominantly in the subcritical flow regime. Therefore, most bridge hydraulic analyses are concerned with subcritical flow. Occasionally, however, supercritical flows are present in segments of the stream reach being analyzed. A water surface profile model that includes both subcritical and supercritical flow segments is said to have a mixed-flow regime. A mixed-flow analysis computes a subcritical water surface profile, beginning at the downstream-most cross section and proceeding to the upstream end of the reach. Then the analysis works in the downstream direction to calculate a supercritical profile. The analysis yields a subcritical and supercritical solution at each cross section. In the case of HEC-RAS, the program calculates the specific force (the sum of the inertial and pressure force) for each solution to determine which controls the water surface profile at each cross section. The solution with the greater specific force controls. The specific force is computed by the following expression:

$$SF = \frac{Q^2 \beta}{g\ A_m} + A_t\ \bar{Y} \tag{6.4}$$

where:

SF	=	specific force, ft^3
β	=	momentum distribution coefficient

A_m	=	effective flow area in the cross-section, ft^2
A_t	=	total inundated cross-sectional area, including areas of ineffective flow, ft^2
$\overline{Y}$	=	depth from the water surface to the centroid of the total inundated area, ft

The program completes the mixed-regime profile after identifying the controlling solution at each cross-section. Open-channel flow passes through an abrupt transition called a hydraulic jump when it goes from subcritical upstream to supercritical downstream. The hydraulic jump's upstream end can be located through mixed-regime water surface profile calculations.

Figure 6.6 is an example of a mixed-flow water surface profile. A mild slope flows into a steep downstream slope, passing through critical depth at the slope break. Flow is subcritical at both ends, but there is an internal segment of supercritical flow from the slope break to the hydraulic jump. There is a subcritical profile at the downstream end of the steep slope controlled by a high-water surface elevation at the downstream boundary. The flow upstream of the slope break is subcritical.

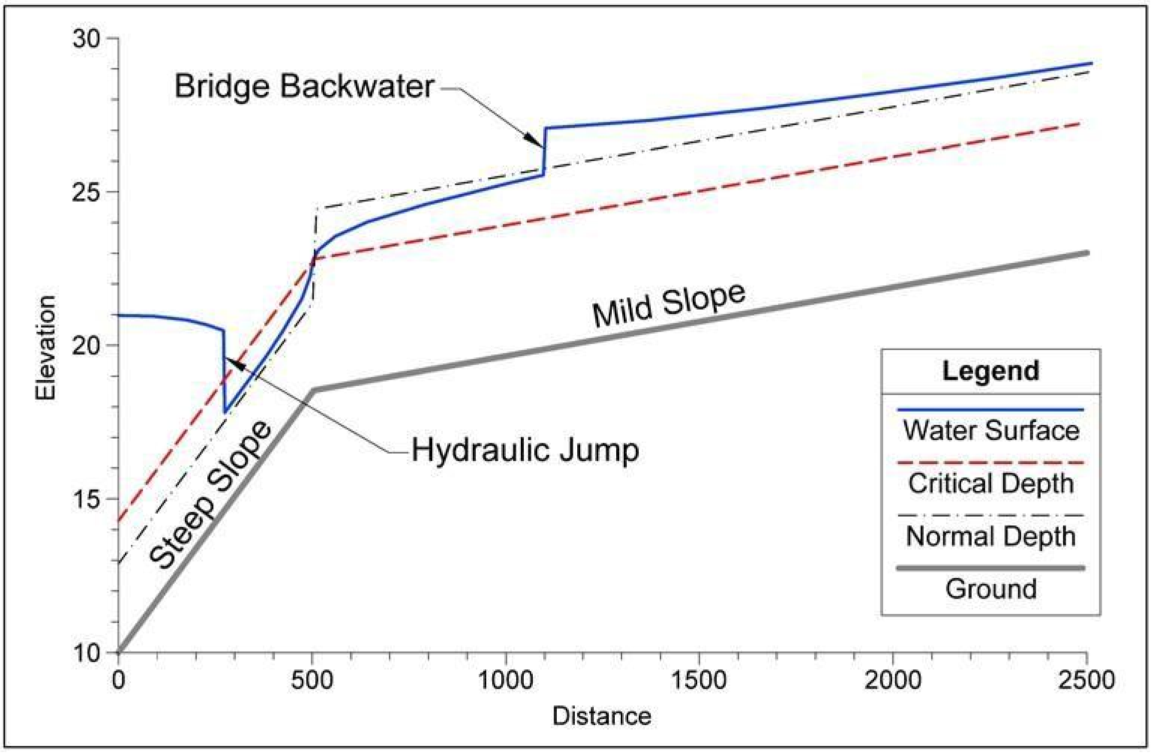

Figure 6.6. Example of a mixed-flow regime profile.

6.3 One-Dimensional Hydraulic Analysis of Highway Crossings

6.3.1 Effects of Highway Crossings

By economic necessity, a typical highway crossing consists of a bridge together with earth-fill embankments encroaching into the floodplain from one or both sides. The encroachment forms a hydraulic constriction. Most bridge crossings are constructed with the abutments at the channel banks or set back from the banks, thereby avoiding significant constriction of the main channel flows. At such crossings, constriction is not appreciable until the flood discharges are high enough to exceed the main channel banks and inundate the overbank areas.

When a bridge crossing constricts flow, the energy losses in the regions upstream and downstream of the crossing are greater than they would be without the constriction. The flow upstream of the bridge contracts from the full floodplain width to the structure opening width. The flow re-expands to occupy the full floodplain width on the downstream side. Both the contraction and expansion processes require some longitudinal distance from the crossing for fully established flow. This document refers to the establishment zones as the contraction and expansion reaches, or collectively as the transition reaches. Increased friction losses (due to decreased conveyance) and transition losses characterize both the contraction reach and the expansion reach.

6.3.2 Locating Cross Sections to Model Highway Crossings

The engineer's placement of cross sections and the modification of individual cross sections' conveyance properties drive the analysis of the transition reaches. Figure 6.7 illustrates the typical 1D modeling framework for analyzing a bridge crossing. The narrowing and widening of the effective flow width in the contraction and expansion reaches are the dominant factors in modeling the transitions. The outer streamlines in the transition reaches would naturally follow curvilinear flow paths. In 1D modeling, however, the engineer simplifies the problem by assuming linear transition tapers, as shown in Figure 6.7. The hydraulic model's computation of the excess loss is directly related to the length of the contraction reach (the distance from the approach section to the upstream bounding section) and the expansion reach (the distance from the downstream bounding section to the exit section). The longer the transition reach, the more excess loss is expected. The engineer assigns the approach and exit sections' locations, which directly determine the contraction and expansion reach lengths.

This situation illuminates one of the limitations of 1D analysis in modeling constrictions. A 2D model's calculations determine the length and configuration of the transition flow regions. In 1D modeling, the engineer imposes the configuration of the transitions on the model by locating the cross sections and modifying the conveyance properties. The placement of the approach and exit sections depends on the engineer's assessment of the appropriate taper rates for the contracting and expanding flow. The taper rates, i.e., contraction rate (CR) and expansion rate (ER) in Figure 6.7 vary depending on many site-specific factors. Chapter 5 of the *HEC-RAS Hydraulic Reference Manual* provides guidance on assigning CR and ER (USACE 2021a).

Expansion Reach: Table 6.1 below, taken from the *HEC-RAS Hydraulic Reference Manual* (USACE 2021a), summarizes the results of research carried out by the USACE Hydrologic Engineering Center and documented in *Research Document 42* (USACE 1995). It gives the ranges of expected values of ER for different combinations of three factors. Each cell in the table combines the degree of constriction (the bridge opening width divided by the floodplain width), the stream's slope, and the roughness ratio (the average Manning's n in the overbanks divided by the Manning's n in the main channel). The engineer selects a cell based on the value of each factor that is closest to the subject stream reach. The selected cell gives the range of appropriate ER values.

Table 6.1. Ranges of expansions rates, ER (after USACE 2021a).

b/B ratio	Slope	$n_{ob}/n_c = 1$	$n_{ob}/n_c = 2$	$n_{ob}/n_c = 4$
0.10	0.0002	1.4 – 3.6	1.3 – 3.0	1.2 – 2.1
0.10	0.001	1.0 – 2.5	0.8 – 2.0	0.8 – 2.0
0.10	0.0.002	1.0 – 2.2	0.8 – 2.0	0.8 – 2.0
0.25	0.0002	1.6 – 3.0	1.4 – 2.5	1.2 – 2.0
0.25	0.001	1.5 – 2.5	1.3 – 2.0	1.3 – 2.0
0.25	0.002	1.5 – 2.0	1.3 – 2.0	1.3 – 2.0
0.50	0.0002	1.4 – 2.6	1.3 – 1.9	1.2 – 1.4
0.50	0.001	1.3 – 2.1	1.2 – 1.6	1.0 – 1.4
0.50	0.002	1.3 – 2.0	1.2 – 1.5	1.0 – 1.4

Variables: b = bridge length, B = expanded flow width, S = slope, n_c = channel Manning's n, n_{ob} = overbank Manning's n.

The engineer can decide on a value within the cell's range and use that value to estimate the length of the expansion reach. The expansion reach length is the distance required for the effective flow to expand to the edges of the floodplain at the ER taper rate. For example, an ER value of 2 with a floodplain encroachment distance of 100 feet on one side of the floodplain means the flow takes 200 feet to expand fully on that side. For asymmetric encroachments, where the constriction is more pronounced on one side of the floodplain than the other, the expansion reach length can be based on the average encroachment distance. Once the expansion reach length has been estimated, the engineer locates the exit cross section and plots it on a topographic map or aerial photograph. Often the expansion reach is so long that the engineer wants to insert intermediate cross sections between the downstream bounding section and the exit section. Inserting intermediate cross sections is encouraged as long as one uses ineffective flow specifications to represent the expansion taper, as discussed later in this section.

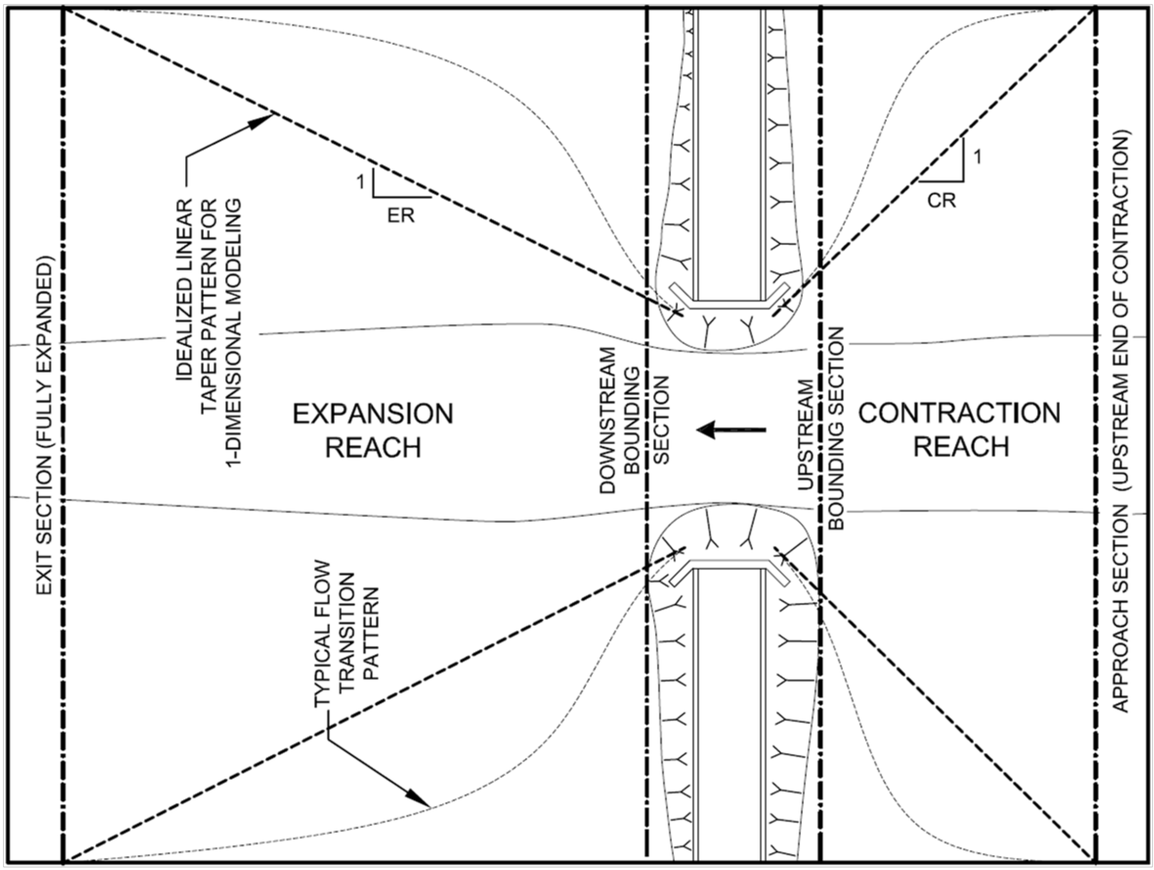

Figure 6.7. Illustration of flow transitions upstream and downstream of a bridge crossing (after USACE 2021a).

Contraction Reach: Table 6.2 below, taken from Appendix B of the *Hydraulic Reference Manual*, summarizes the research on the contraction rate published in *Research Document 42* (USACE 1995). It is similar to Table 6.1, but involves fewer factors. A cell in the table is selected based on the stream slope and roughness ratio. The values in the cell represent the appropriate range of CR values. The engineer selects a value within the range and determines the length of the contraction reach. Every cell in the table includes a CR value of 1 within its range. The overall data set had both a mean and median value of 1. Common practice, therefore, is to use a value of 1 for the CR. This practice is usually appropriate, but for cases in which the bridge design is highly sensitive to the amount of backwater (e.g., to comply with FEMA floodway regulations), it may be advisable to use the table to select CR.

Table 6.2. Ranges of Contraction Rates, CR (after USACE 2021a)

Slope	n_{ob}/n_c = 1	n_{ob}/n_c = 2	n_{ob}/n_c = 4
0.0002	1.4 – 3.6	1.3 – 3.0	1.2 – 2.1
0.001	1.0 – 2.5	0.8 – 2.0	0.8 – 2.0
0.002	1.0 – 2.2	0.8 – 2.0	0.8 – 2.0

Variables: S = slope, n_c = channel Manning's n, n_{ob} = overbank Manning's n

Tidal Bridges: Cross-section locations ideally accommodate flow in both directions for 1D unsteady-flow analysis of bridges over tidal streams. The approach section during ebb tide conditions, in which the flow direction is toward the ocean, is the exit section during flood tide conditions, in which the flow direction is away from the ocean. In this case, it is logical for the CR and ER values to be the same, with the value set at a compromise between the ER and CR values that would have been selected if the bridge were not tidal. It is common to use a value of 1.5 for both CR and ER.

6.3.3 Applying Ineffective Flow at Bridge Crossings

Section 6.2.1 describes the Ineffective Flow feature in HEC-RAS. This feature is convenient in modeling bridge crossings. Figure 6.7 shows the upstream and downstream bounding sections appropriately located beyond the embankment fills' side slopes. Therefore, the cross-section geometry reflects the floodplain, not the roadway. Ineffective flow areas on the upstream and downstream bounding sections represent the presence of the highway embankments. The inset in Figure 6.5 shows an example of an upstream bounding section at a bridge. The ineffective flow setting is set back laterally from the abutment station by a distance equal to the CR or ER value multiplied by the cross-section's distance upstream or downstream from the bridge. The engineer assigns the elevation setting on the ineffective flow setting based on the water surface elevation at which a significant amount of discharge would flow over the top of the road embankment.

The engineer enters ineffective flow specifications for any cross sections inserted within the transition reaches, between the approach section and the bridge or between the bridge and the expansion section, to reflect the taper of the contracting or expanding flow. A helpful practice is to plot the cross-section lines and the taper lines on a topographic map or aerial photograph to guide the placement of the ineffective flow specifications.

6.4 Bridge Hydraulic Conditions

6.4.1 Free-Surface Bridge Flow

Free-surface bridge flow refers to the range of flow conditions that maintain a free water surface under the bridge, with no low chord submergence. The *HEC-RAS Hydraulic Reference Manual* refers to free surface bridge flow as “low flow” and classifies this type of flow into Class A, Class B, or Class C, depending on the flow regime in the stream reach and in the bridge waterway itself.

Class A is the most prevalent free-surface bridge flow condition. Class A flow conditions are subcritical upstream of the bridge, downstream of the bridge, and throughout the bridge waterway. Class A flow generally satisfies the constraints of gradually varied flow throughout the reach of interest. HEC-RAS provides four available approaches to modeling Class A free-surface bridge flow, described further in the next section.

Class B flow passes through critical depth within the bridge waterway, which implies that supercritical flow exists at least for a short distance downstream of the critical depth control section. The potential for Class B flow to occur inside a bridge waterway stems from two causes. First, the elevation of critical depth is often higher in the constriction than upstream or downstream. Second, the water surface within the constriction draws down. The flow conditions upstream and downstream of a bridge in Class B flow are most commonly subcritical. A hydraulic jump often exists either within the bridge waterway or a short distance downstream of the bridge. Class B flow can sometimes occur in conjunction with a supercritical stream profile. In this case, the bridge waterway is a control section with subcritical flow upstream and a hydraulic jump occurring some distance upstream of the bridge.

In Class C flow, the regime is supercritical upstream and downstream of the bridge and through the bridge waterway. Class C flow is rare because natural channels, even on steep grades, such as mountain streams, rarely support uninterrupted supercritical flow over long reaches (Jarrett 1984). Class C flow, therefore, would typically be expected only in engineered flood control channels on a steep slope. Figure 6.8 illustrates Class A, B, and C flow conditions.

6.4.2 Overtopping Flow

Overtopping flow refers to flow crossing over the roadway approaches or the bridge deck. A broad-crested weir appropriately represents overtopping flow conditions, since the road embankment is elevated above the floodplain grade, the dimension of the crest in the direction of flow (across the road) is broad, and the overtopping depth is comparatively shallow. Chapter 4 discusses broad-crested weirs and the equations for analyzing weir flow conditions. The quantity of flow going over the road instead of through the bridge can be considerable. For instance, the weir flow could easily exceed 25 cfs for every 10 feet of weir length with just one foot of overtopping depth. The overtopping flow potential is high for crossings with relatively low road profiles and wide floodplains.

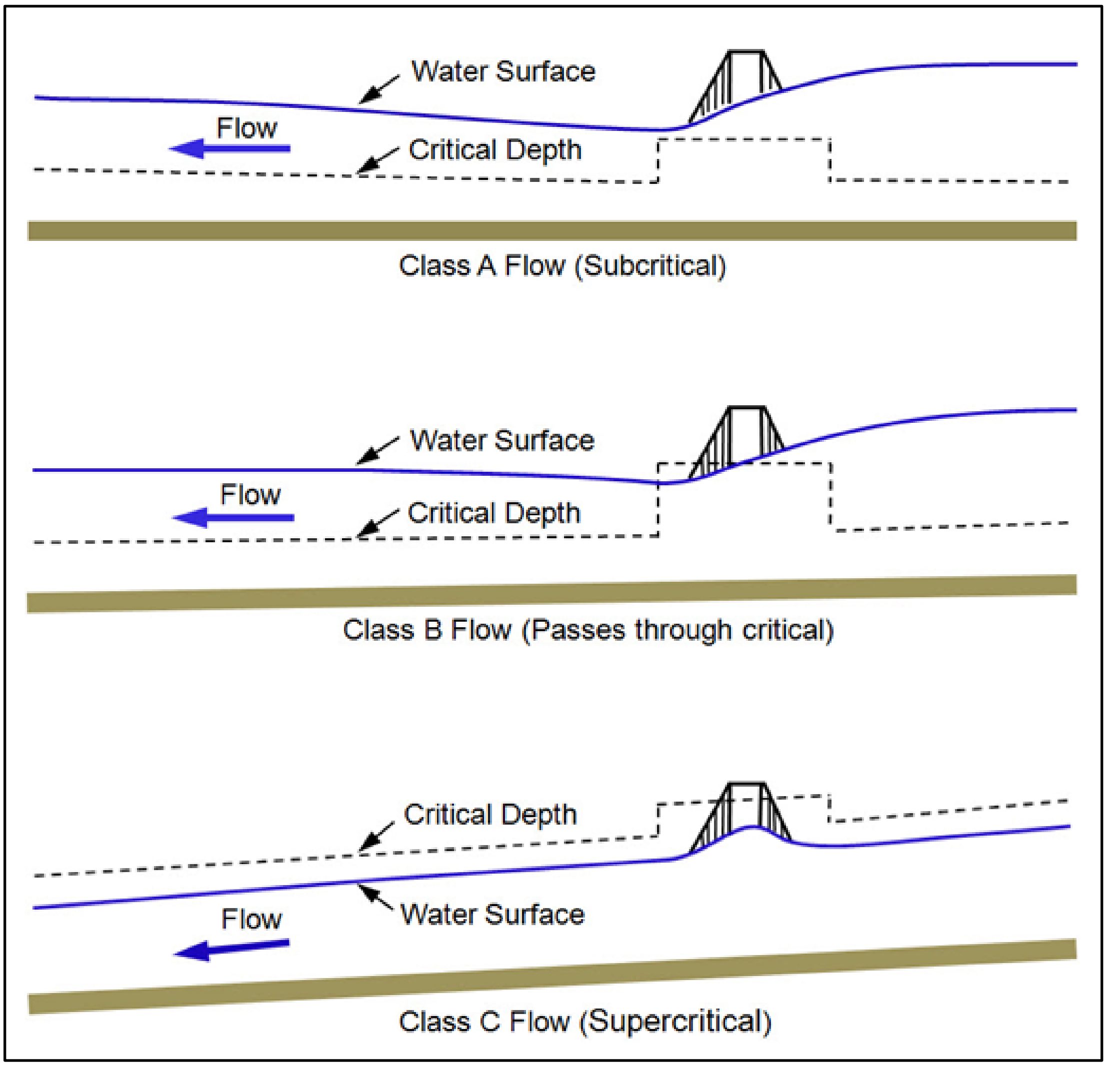

Figure 6.8. Illustration of free-surface bridge flow Classes A, B and C.

Overtopping flow at bridge crossings is usually combined with either free-surface bridge flow or submerged-deck flow in the bridge waterway. The engineer can determine how much flow is going under the bridge and how much over the bridge deck or roadway by the 1D modeling principle that all flow paths from the upstream bounding section to the downstream bounding section are to result in the same energy loss. There is only one flow distribution between overtopping and bridge flow that results in equal energy loss.

In some cases, a weir does not accurately represent the flow over the roadway approaches. This occurs either because the road is at or very near floodplain grade (in other words, there is little or no embankment fill) or because the downstream water surface elevation is so high above the weir crest elevation that it drowns out the weir. An imbalance in the lateral flow distribution may exist between the weir flow and the corresponding overbank flow at the upstream and downstream bounding sections. For instance, the weir flow reported in the model results may be much less than the flow reported in the overbanks at one or both of the bounding sections. The reverse situation is also possible. To correct this imbalance in the flow distribution, the engineer can determine the controlling component of the overtopping flow. If the reported weir flow is the smaller quantity, then the scenario is weir-capacity limited. One can correct the discontinuity by increasing the Manning's n values in the overbanks of the bounding sections, reducing the overbank flow until it roughly matches the roadway overtopping flow. If the overbank flow is the smaller quantity, then the situation is overbank-supply limited. In this case, the engineer can adjust the weir coefficient downward, within reason, reducing the weir flow until it roughly matches the supplying overbank flow. The *HEC-RAS Applications Guide* (USACE, 2021b) describes this process in the context of an example problem. The approach described here is intended to generally reduce discontinuities in the flow distribution and should not be expected to result in a highly accurate flow balance. The limitations of 1D modeling cause the potential discontinuity described in this paragraph. An engineer performing a 2D model of a similar situation would not need to make any adjustments because the program accurately accounts for the continuity of flow distribution within its normal calculations. Therefore, 2D modeling has clear advantages over 1D modeling for crossings with potential road overtopping.

6.4.3 Flow Submerging the Bridge Low Chord

Orifice flow usually occurs when the water surface is above the highest point of the bridge's low chord. When the low chord is submerged only at the superstructure's upstream edge, the orifice is considered free-flowing and not affected by tailwater. This document refers to such a condition as “orifice bridge flow.” Just as the headwater upstream of an inlet-control culvert is not affected by conditions downstream of the culvert entrance, the backwater upstream of a bridge operating under orifice bridge flow is not affected by conditions downstream of the upstream edge of the superstructure.

Another type of orifice flow exists when the low chord's highest point is submerged at both the upstream and downstream edges of the superstructure. This type of flow approximates a tailwater-controlled orifice. Just as the headwater upstream of an outlet-control culvert is sensitive to conditions within and downstream of the culvert barrel, the backwater upstream of a bridge operating under full-flowing or tailwater submerged orifice conditions is affected by conditions within and downstream of the bridge waterway. For this document's purposes, this condition is termed "submerged-orifice bridge flow.” The *Handbook of Hydraulics* (Brater & King, 1976) and many other hydraulic references discuss and derive the equations used for analyzing orifice flow conditions.

6.5 One-Dimensional Bridge Modeling Approaches

6.5.1 Modeling Approaches for Free-Surface Bridge Flow Conditions

This section explains the approaches used to analyze the various types of flow conditions that can exist at a bridge. The *HEC-RAS Hydraulic Reference Manual* explains the approaches in greater detail (USACE 2021a). The information presented in this section is predominantly taken from that source but with most of the detail omitted. Except in the case of the WSPRO method, the discussion below applies specifically to the region between the upstream bounding cross-section and the downstream bounding cross-section. Upstream and downstream of this region, the energy equation governs.

The HEC-RAS program makes four modeling approaches for Class A flow available to users: The Energy Method, the Momentum Balance Method, the Yarnell Equation, and the WSPRO Method, all described below. Figure 6.9 shows the cross-section identifiers for reference to the bridge flow equations.

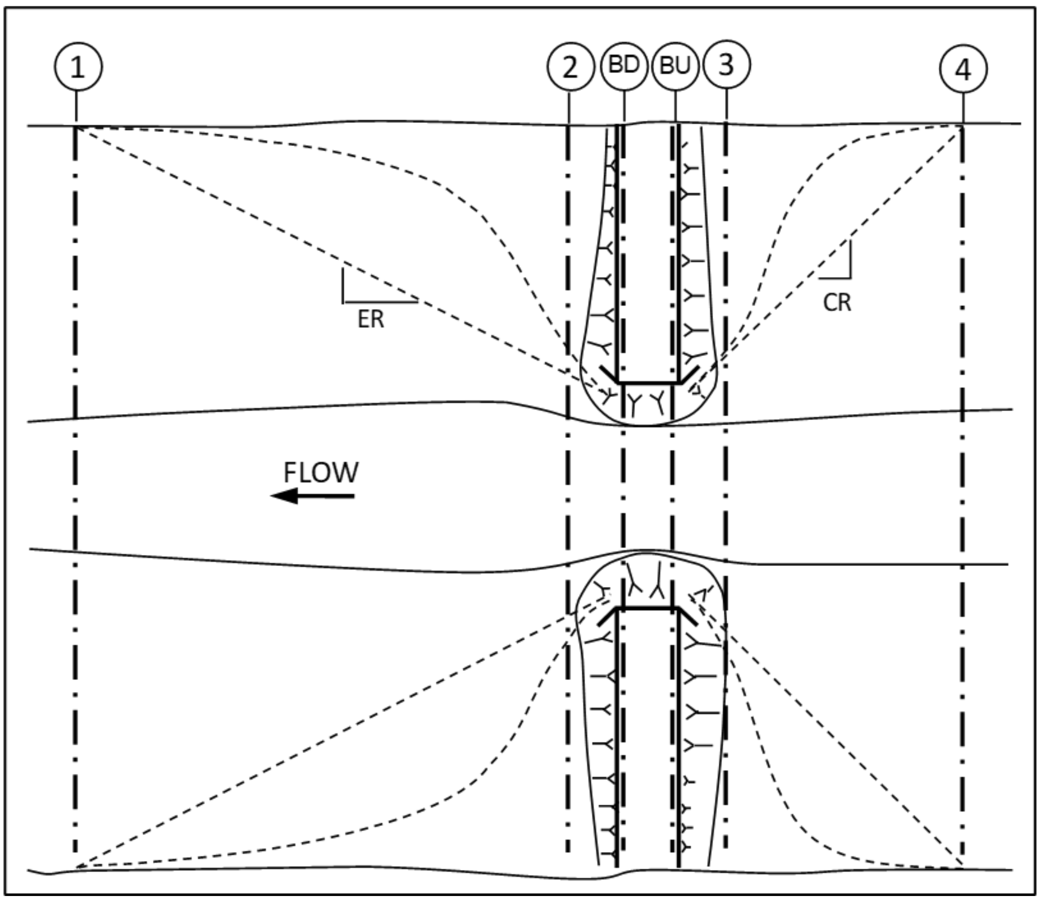

Figure 6.9. Plan-view layout showing cross-section identifiers referenced by the bridge hydraulics equations.

Energy Method: Chapter 4 describes the energy equation in detail. Section 6.2 describes its general application to 1D water surface profile calculations. At bridges, the energy method subtracts the road embankments, abutments, bridge deck, and piers from the effective flow area. The method increases the wetted perimeter to account for the sides of piers, the abutments, and the bridge's low chord if it is in contact with the flow. The low chord and pier sides can significantly affect the wetted perimeter. Decreasing the area and increasing the wetted perimeter reduces the conveyance. The degree of reduction depends on the obstruction imposed by the bridge and road embankments. The reduced conveyance increases the friction slope, increasing the friction loss through the bridge waterway.

Momentum Balance Method: The Momentum Balance Method operates on the principle that the sum of forces acting in a given direction on a control volume is equal to the mass of the water in the control volume multiplied by its acceleration. Section 4.3.3 describes the momentum equation. Unlike the Energy Method, the Momentum Balance Method provides a way to account for a hydraulically streamlined pier design. The equations include a term for pier drag force. The user input includes the drag coefficient (see Table 4.3), which can vary from 0.29 (for elliptical shaped piers with 8:1 length to width ratio) to 2.0 (for square-nosed piers). These large differences in the drag coefficient can affect the results, especially when piers are the primary cause of energy loss at the bridge crossing. Like the energy method, the momentum method includes pier sides in the wetted perimeter. Shear stress is multiplied by the wetted perimeter to determine this resisting force. Therefore, in the momentum method, piers contribute to two resisting forces: pier drag and friction along the pier sides.

HEC-RAS provides an option to include or exclude the weight component and/or the friction component from the force balance. Excluding either of these forces results in an incomplete momentum solution. The friction component is always present as a resisting force, and the weight component is typically a driving force since channel slopes tend to be positive at bridges. Excluding a driving force results in overestimating the upstream water surface and excluding the friction force results in underestimating the upstream water surface.

Yarnell Equation: The Yarnell Equation is strictly empirical, in contrast to the Energy Method and the Momentum Balance Method, which are both theoretically derived. Yarnell performed roughly 2600 flume experiments, observing the change in water surface elevation caused by inserted piers of varying size, shape, and configuration. The Yarnell Equation, described in detail in the *HEC-RAS Hydraulic Reference Manual* (USACE 2021a), is based on those experiments. The Yarnell K coefficient accounts for various pier shapes.

WSPRO Method: The FHWA developed and supported a water-surface-profile computer program called WSPRO beginning in the 1980s. WSPRO became the standard bridge hydraulic analysis software for many State DOTs. HEC-RAS includes the bridge hydraulics approach from that program as an available method. The WSPRO Method uses a standard-step solution of the energy equation and is similar to the Energy Method in most respects. Unlike the other three free-surface bridge flow methods discussed here, the WSPRO Method works from the exit section to the approach section and not just between the upstream and downstream bounding sections. The WSPRO Method's calculation of total energy loss in the region between the upstream bounding cross section and the approach section (Cross Sections 3 and 4 in Figure 6.9) differs notably from the Energy Method. The WSPRO method uses an effective average flow length in this segment. The effective average flow length is the average flow path length for 20 equal-conveyance idealized stream tubes that flow from Cross Section 4 to Cross Section 3 following theoretical curvilinear paths. The *WSPRO User's Manual* (FHWA 1986) explains the details of the assumed stream tube flow paths.

6.5.2 Selection of Free-Surface Modeling Approach

The Energy, Momentum Balance, and WSPRO Methods are all suitable to most bridge crossing sites under free-surface conditions. Among these three, the Momentum Balance Method is unique in accounting for the pier drag as a function of pier shape. Therefore, the Momentum Balance Method is a good choice when piers are the predominant energy loss factor, especially when the pier geometry is somewhat streamlined. The Momentum Balance Method realizes the beneficial effects of the streamlined design.

The Energy and WSPRO Methods are both applicable when friction loss and constriction effects are predominant. In most cases, the results of the two methods, when applied correctly under the same conditions, are very similar in terms of the energy grade line and water surface elevation upstream of the bridge. However, only the WSPRO Method accounts for different types of abutments geometries (for instance, spill-through abutments vs. vertical abutments with wing walls). The Energy Method is often preferred because it has less extensive input requirements than the WSPRO Method and, in most cases, produces roughly the same results. The Momentum Method also typically performs well in situations where constriction is the predominant loss factor. It also has the advantage that it can better accommodate rapidly varied flow, a characteristic of Class B and Class C free-surface bridge flow.

The Yarnell Equation is suitable only for generally uniform and regular waterways, and with little or no constriction, because of its empirical development. It can perform well in analyzing bridges over human-made channels such as irrigation canals or engineered flood control channels.

The Momentum Method is ideal for Class B and Class C flow conditions, for the reasons mentioned earlier. The Energy Method is also an acceptable approach, although less ideal.

6.5.3 Modeling Approaches for Overtopping Flow and Flow Submerging the Low Chord

The HEC-RAS program makes two approaches available for modeling overtopping and orifice bridge flow conditions: The Energy Method and the Pressure and Weir Method. Note that in HEC-RAS terminology, orifice and submerged-orifice bridge flow conditions are termed "pressure flow."

Energy Method: The Energy Method simply continues the standard-step solution of the energy equation through the bridge structure and vicinity. It accounts for the blockage caused by the road embankments, abutments, bridge deck, and piers by reducing the conveyance. If the water surface is high enough to overtop the road, the program treats the flow area above the road as conveyance area, not as a weir. The Energy Method does not compute or report the quantity of overtopping flow. If the low chord is submerged, the added wetted perimeter negatively affects conveyance, but the program does not attempt to compute orifice conditions.

Pressure and Weir Method: The Pressure and Weir Method uses the broad-crested weir equation to compute the quantity of any overtopping flow. The method chooses between two different orifice equations when orifice or submerged-orifice bridge flow is detected.

The technique for computing weir hydraulics in the case of flow over the road or bridge deck is similar to the approach described in FHWA document HDS 1 *Hydraulics of Bridge Waterways* (FHWA 1978). Figure 6.10 depicts the condition of flow overtopping a roadway embankment.

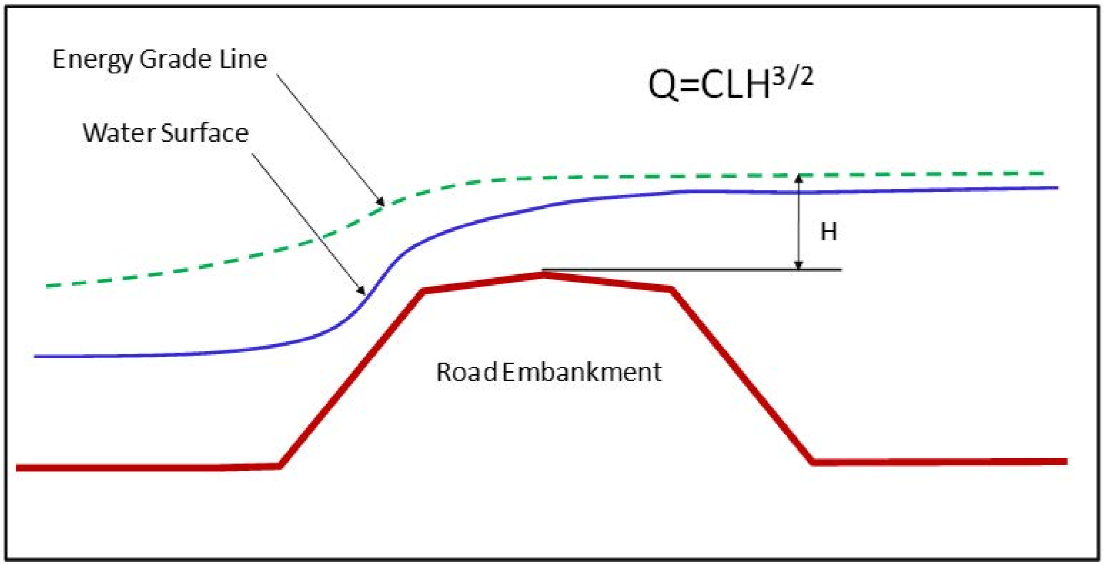

Figure 6.10. Illustration of flow overtopping a roadway embankment.

Equation 4.19 is the broad-crested weir equation. The flow over the weir is proportional to the discharge coefficient. The discharge coefficient for a broad-crested weir generally ranges between 2.6 and the theoretical maximum of 3.1. Road embankment overtopping is typically modeled with a discharge coefficient of 3.0 to 3.1. A bridge deck is not an ideal broad-crested weir, and a discharge coefficient between 2.6 and 3.0 is appropriate. HDS 1 (FHWA 1976) provides additional information on assigning the discharge coefficient value.

Figure 6.10 depicts an example of unsubmerged weir flow because the water surface downstream of the bridge is lower than the weir crest elevation (the crown of the road). In cases where the downstream water surface elevation is above the weir crest elevation, the weir flow is submerged. The submergence does not start to affect the weir capacity unless the submergence ratio is above roughly 80 percent. The submergence ratio is the tailwater depth divided by the head, and the tailwater depth is the downstream water surface elevation minus the weir crest elevation. It is appropriate for a submergence ratio greater than 80 percent to reduce the discharge coefficient C. The relationship between the discharge coefficient and the degree of submergence is provided graphically in HDS 1 (FHWA 1978).

Overtopping flow at a bridge crossing coexists with free-surface bridge flow or orifice flow in the bridge opening. The program determines the discharge through the bridge waterway versus that over the road or bridge deck, using an iterative approach to find the flow distribution. The solution finds the amount of weir flow such that the head elevation driving the weir flow is the same as the energy grade line elevation upstream of the bridge resulting from the losses experienced by the non-overtopping flow passing through the bridge waterway. This iterative approach can sometimes lead to numerical instability in the model, especially when the tailwater submergence is high.

Figure 6.11 illustrates the non-submerged-orifice pressure-flow condition. The bridge opening acts as an orifice control section, with no influence from downstream conditions. Equation 4.20 is the general orifice equation. In non-submerged orifice flow, the head forcing the flow through the orifice (the head differential) is the height from the energy grade line upstream of the bridge to roughly the vertical center of the bridge opening height (the elevation halfway up the Z dimension). The value of the discharge coefficient varies, as stated in Section 4.5.2. Its value is a function of the ratio of the upstream flow depth to the height of the bridge opening (Y_3/Z) and is calculated automatically in the HEC-RAS model.

Figure 6.12 shows the case of submerged-orifice pressure flow. The head differential is measured from the upstream energy grade line to the downstream water surface elevation, reflecting that the downstream conditions directly affect the backwater. In this condition, the discharge coefficient is not a function of the flow depth. Field data recorded in HDS 1 (FHWA 1978) indicated that the values of C for submerged-orifice bridge flow range from 0.7 to 0.9. HEC-RAS uses a default value of 0.8 for C in submerged-orifice pressure flow.

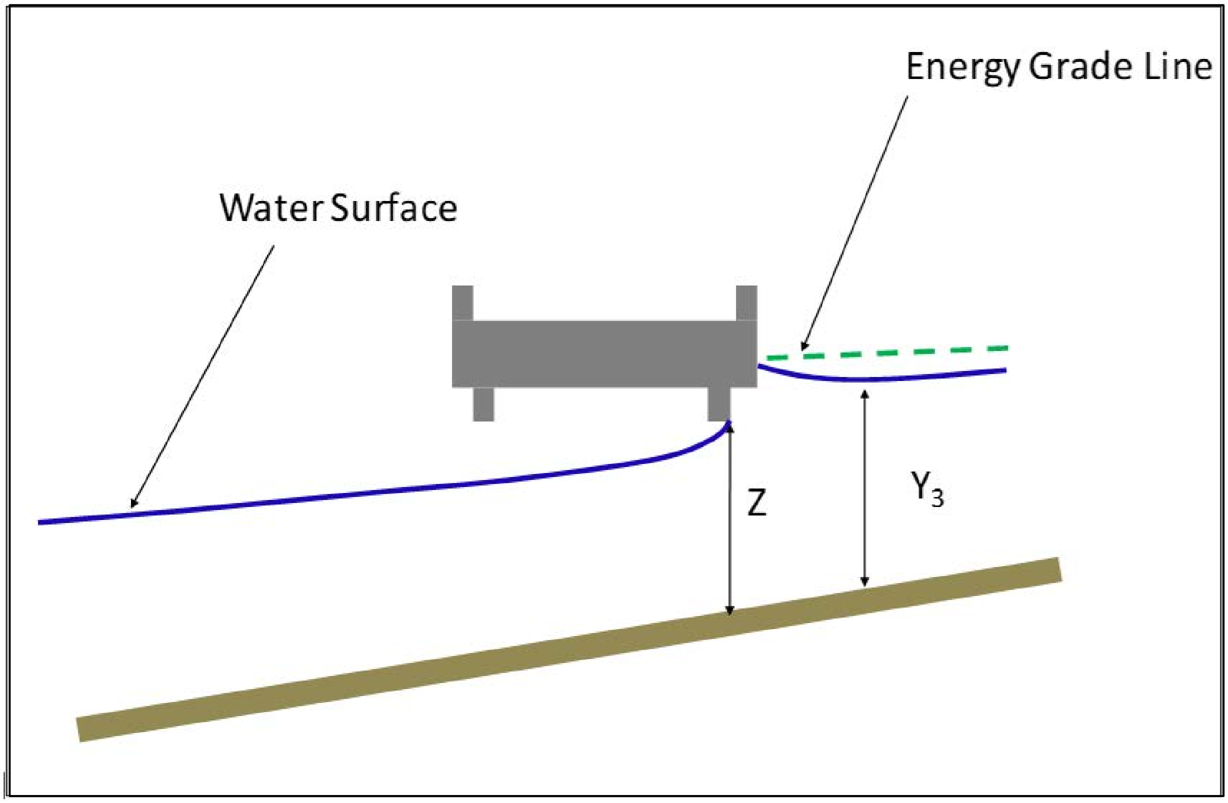

Figure 6.11. Sketch of orifice pressure flow.

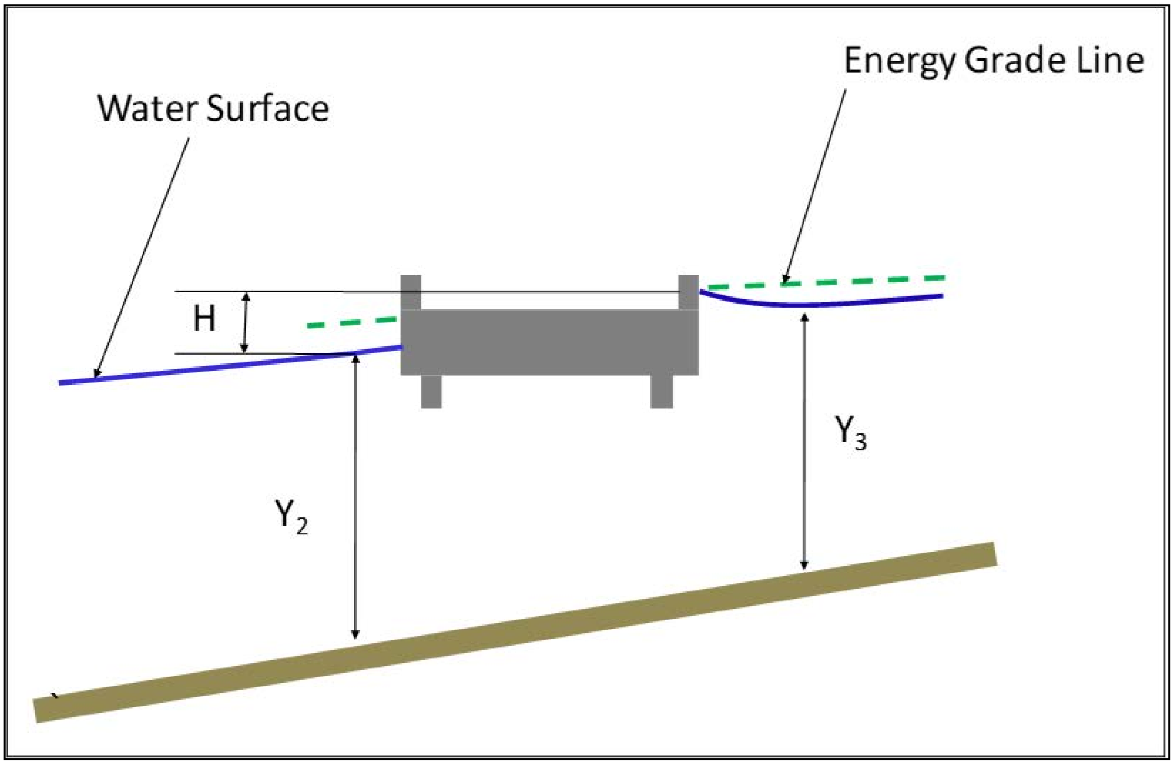

Figure 6.12. Sketch of submerged-orifice pressure flow.

6.5.4 Selection of Overtopping and Submerged-Low-Chord Modeling Approach

The selected modeling approach in HEC-RAS for overtopping or submerged-low-chord conditions usually has more significant consequences than the free-surface modeling approach. The Energy Method results can be far different from those of the Pressure and Weir Method, and the differences can affect the bridge's design. This section identifies the more accurate modeling approach for various situations.

The Pressure and Weir Method is the more accurate approach for many scenarios, including the following:

Overtopping with Little or No Tailwater Submergence: Flow over an elevated embankment truly functions as weir flow if there is no tailwater submergence effect. Weir flow conditions prevail even for a reasonably large tailwater depth above the weir crest. Submergence does not affect the weir capacity unless the submergence ratio exceeds 0.80. The Energy Method typically overestimates the backwater caused by the crossing in such cases because it fails to acknowledge the weir's high flow capacity. This effect increases with greater lengths of road overtopping. The Energy Method also overestimates the discharge and velocity inside the bridge waterway, leading to overestimated scour depths.

Overtopping with a Significant Change in Water Surface Elevation: If overtopping occurs and the water surface difference is substantial across the road alignment, then true weir flow conditions are likely. Again, the Energy Method overestimates the backwater caused by such cases.

Orifice or Submerged-Orifice Bridge flow with $Y_3/Z \geq 1.1$: The bridge waterway is functioning as an orifice when the flow depth just upstream of the bridge (Y_3 in Figure 6.11) is more than 1.1

times the bridge opening height (Z in Figure 6.11). In this case, the Pressure and Weir Method yields accurate results. The Energy Method tends to underestimate the head required to force the flow through the bridge waterway in such cases. Consequently, the Energy Method underestimates the backwater caused by the crossing. If the Pressure and Wier Method is selected, the HEC-RAS program automatically determines whether the correct equation is orifice or submerged-orifice.

Some scenarios are better suited to the Energy Method, including:

Overtopping of a Low or At-Grade Roadway: The road does not act as a weir unless it is elevated above the floodplain grade on an embankment of significant height to act as a weir during overtopping flow. Therefore, flow over an at-grade road or a low embankment does not behave as weir flow. The Pressure and Weir Method could potentially underestimate the backwater by attributing too much capacity to the road overtopping segment in such cases. The Pressure and Weir Method could also underestimate the flow through the bridge waterway, leading to underestimated velocity and scour potential.

Overtopping Flow with Minimal Water Surface Change: A minimal drop in the water surface from one side of the road embankment to the other suggests that the weir crest is highly submerged by the tailwater, which drowns out the weir. The Energy Method is appropriate in such cases. The weir equation would likely underestimate the backwater like the previous scenario. The HEC-RAS user can select the Pressure and Weir Method but specify that the program reverts to the Energy Method if the submergence ratio exceeds a threshold amount (often entered at 98 percent).

Engineers occasionally encounter borderline cases where the decision between the two methods is not clear cut. One such example is when the water surface submerges the bridge's low chord but with a low degree of submergence ($Y3/Z < 1.1$), and there is no overtopping flow. The flow conditions are not firmly in the realm of orifice flow at this degree of submergence, and the Energy Method may be more appropriate. In this case, a reasonable approach is to conduct the analysis with both the Pressure and Weir and the Energy Methods and use the more conservative result.

The scenarios above call for making decisions after evaluating the flow conditions. The engineer cannot usually anticipate at the outset what types of flow conditions will be observed in the model results. Therefore, the modeling process requires some iteration before arriving at a final analysis model. The engineer makes an initial model run with the Pressure and Weir Method selected. If the model results show overtopping, the engineer identifies whether the weir crest is submerged and by how much, and then determines whether the overtopping flow is functioning as weir flow. If so, then the Pressure and Weir Method is appropriate. If not, the Energy Method is more suitable. The model results may show submergence of the low chord but no overtopping. In this case, the engineer decides between the Pressure and Weir and Energy Methods based on the degree of low chord submergence.

6.6 Special Cases in One-Dimensional Bridge Hydraulic Analysis

6.6.1 Skewed Crossing Alignment

One-dimensional modeling assumes the flow direction to be perpendicular to the cross section. A highway crossing is skewed to the flow direction when it crosses the floodplain or main channel on an alignment that is not perpendicular to flow (see Figure 6.13). When a cross section has a skewed orientation, the cross-sectional flow area is exaggerated, and the conveyance is overestimated. Therefore, a skewed road crossing requires adjusting the geometric input to avoid overestimating the crossing's capacity. The need for the adjustments described in this section arises because of the limitations of 1D modeling. Furthermore, even with these adjustments, 1D modeling of skewed crossings is less accurate than 2D modeling. One-dimensional modeling is not suitable for skew angles exceeding 20 degrees.

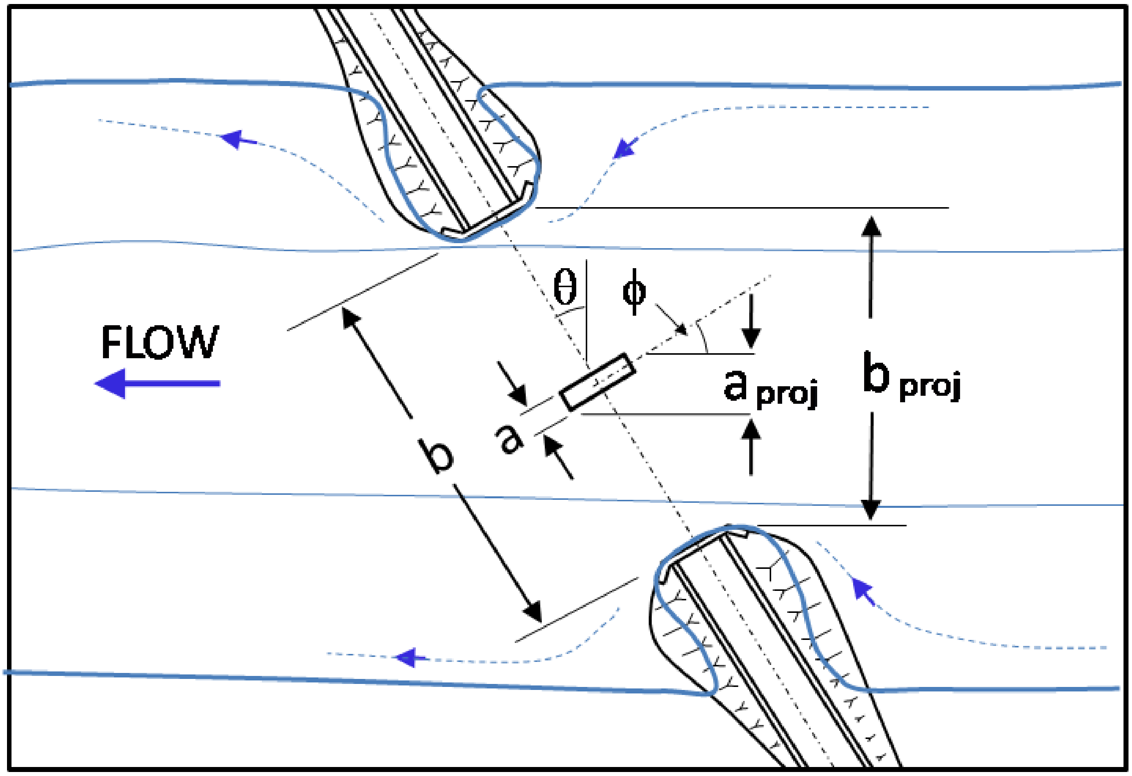

Figure 6.13. Illustration of a skewed bridge crossing.

The engineer can incorporate the effects of skew by first entering the bounding cross sections, road embankment and bridge data as if there is no skew. One can then adjust the station values of the bounding cross section points, bridge abutments, and pier centerlines by a factor equal to the cosine of the skew angle (see Equation 6.5). After the adjustment, the resulting hydraulic capacity reflects the cross-sectional width and area that are projected to a line perpendicular to the flow.

$$X_{adjusted} = X_{unadjusted} \cos\theta \tag{6.5}$$

where:

X	=	station value of a cross section point, abutment, or pier centerline, ft
θ	=	skew angle, degrees

In some situations, a significant portion of the total discharge (say more than 50 percent) is overtopping the road embankments. In these cases, the engineer may decide to leave the overbank portions of the bounding cross sections unadjusted since weir flow tends to orient itself to be perpendicular to the weir crest. When the piers are skewed to the flow direction the projected pier width is calculated as follows.

$$a_{proj} = (L \times Sin\,\phi) + (a \times Cos\,\phi) \tag{6.6}$$

where:

a_{proj}	=	pier width projected to the direction of flow, ft
L	=	pier length, ft
ϕ	=	skew angle of the pier axis from the flow direction, degrees
a	=	actual pier nose, width, ft

6.6.2 Crossings with Parallel Bridges

Parallel bridges are found where divided highways cross streams. Figure 6.14 shows an example of a parallel bridge crossing. Hydraulically the two bridges are in series. Physical modeling results reported in HDS 1 (FHWA 1978) show that two identical bridges in series and close to each other produce about 1.3 to 1.5 times the backwater caused by one bridge alone, depending on the distance between the two bridges. The maximum clear distance between the bridges in the study cited was 9 bridge deck widths. The total backwater is less than twice the single-bridge backwater because the full contraction and re-expansion of the flow occur only once (contracting upstream of the upstream bridge and re-expanding downstream of the downstream bridge).

Figure 6.14. Example of a parallel bridge crossing. Flow direction is right to left.

Ideally, the engineer models the two bridges as separate structures in series, but this approach requires additional effort that may not be necessary. Modeling two parallel, identical bridges as a single bridge may be acceptable, depending on the purpose of the analysis. When using this approach, it is appropriate to enter the bridge deck width as the sum of the two bridges plus the gap in between.

In many scenarios, it is necessary to model the two bridges as separate structures in series, each with its own bounding cross sections and bridge data specifications. Such cases occur:

- When the purpose of the model is to develop the hydraulic design of the two structures and the engineer needs refined hydraulic information to apply to each structure independently for freeboard determination and scour evaluation.
- When the flow can re-expand and consequently re-contract between the two bridges, as could occur if there is a substantial distance between the bridges and the ground between the divided roadways is low enough to allow it.
- When submerged-orifice bridge flow is a possibility, since the backwater of the upstream bridge is very sensitive to conditions downstream in a submerged pressure-flow condition.
- When the two bridge structures do not have identical span lengths, pier geometry, or deck profile.

6.6.3 Crossings with Multiple Openings in the Embankment

Some crossings require relief bridge openings or culverts through the embankment in addition to the main bridge opening. Wide floodplains and those with separate side channels are examples of sites that might require multiple openings. Figure 6.15 shows an example of a multiple-opening crossing. Engineers can carry out rough 1D analyses of multiple-opening crossings by employing the energy conservation principle. In past decades, the lack of other options justified the use of 1D models for this purpose. Accurate modeling of the multiple-opening scenario, however, is beyond the limitations of 1D modeling. In the current era, bridge analysis studies involving multiple openings always call for 2D modeling.

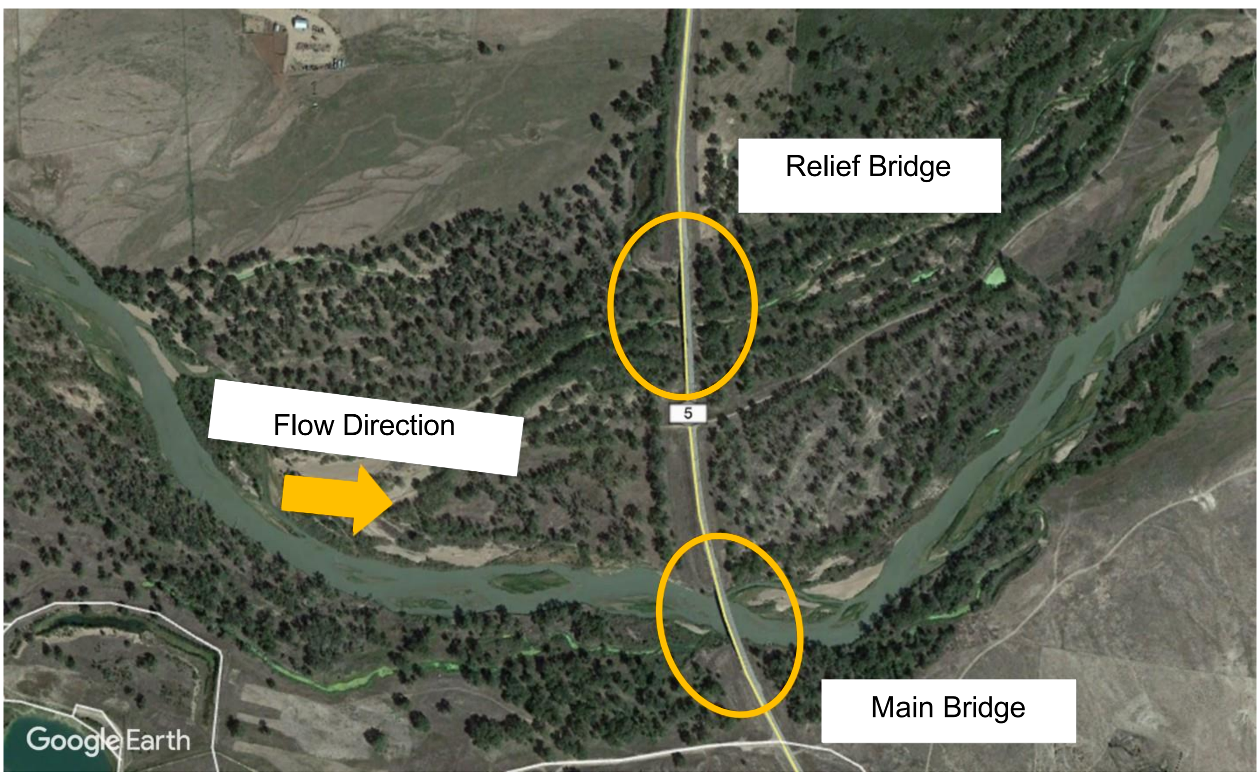

Figure 6.15. Example of a multiple-opening crossing.

CHAPTER 7
TWO-DIMENSIONAL BRIDGE HYDRAULIC ANALYSIS

7.1 Introduction

The interface of rivers and highways presents many design challenges that favor 2D hydraulic modeling. FHWA published the *Two-Dimensional Hydraulic Modeling for Highways in the River Environment Reference Document* (FHWA, 2019) to assist hydraulic engineers in becoming proficient in applying 2D models to transportation projects. Along with model-specific users documents, the reference document is the recommended resource for engineers applying 2D models to transportation-related projects in the river environment.

Chapter 4 provides principles of the fundamental concepts for most hydraulic modeling calculations encountered in open channel flow and bridge hydraulic analysis. Chapter 5 provides information on selecting the most appropriate hydraulic modeling approach and an overview of developing boundary condition input data and other considerations common to all bridge hydraulic problems. This chapter provides an overview of topics related to developing the 2D model geometry specifically for bridge hydraulic modeling.

7.2 Two-Dimensional Modeling Numerical Methods

The three most common numerical methods used for solving the 2D partial differential equations are finite difference, finite element, and finite volume. Each method subdivides (discretizes) the model domain into small shapes, referred to as elements or cells. The collection of elements constitutes a mesh or grid that represents the model domain. If the elements are uniform shapes and sizes (i.e., squares and rectangles), common terminology refers to it as a structured grid. An unstructured grid applies when the elements are variable shapes (i.e., triangles, quadrilaterals, and other polygons) and sizes. The term mesh is another name for an unstructured grid.

7.2.1 Finite Difference

The finite difference method is the most direct approach to mapping differential equations to individual cells. Instead of representing equations as a continuum, single point values of key variables are used to estimate the terms in the governing equations. Computations are typically made at the center of each cell within a regularly-spaced (structured) grid. This structured grid type makes solving the equations more efficient and stable, but it is challenging to represent irregularly-shaped terrain features within the domain, such as river channels or roadway embankments.

7.2.2 Finite Element

The finite-element method uses an unstructured grid of variously-shaped and -sized elements. This method integrates a form of the governing equations over each element while accounting for values at neighboring elements to develop a solution. This approach produces a continuous or smooth solution across each element. The use of an unstructured mesh is more applicable to hydraulic analysis involving transportation infrastructure compared to a structured grid. However, the finite element method can have stability limitations and potential mass conservation issues compared to finite volume or finite difference solvers.

7.2.3 Finite Volume

The finite-volume method also uses an unstructured grid. However, the numerical basis of finite volume recognizes that many of the processes at work can be represented by the conservation of mass and conservation of momentum laws of physics. Each element is treated as a unique control volume, and the rate of change of variables is calculated at each element edge. The finite volume method only needs to perform flux evaluations across the element boundaries. This method forces conservation of mass in all grid cells, which conserves mass locally and globally in the grid. The finite volume approach results in more stable solutions than finite element methods and performs better in mixed flow regime situations common in constricted floodplain situations like bridge crossings.

7.2.4 Implicit vs. Explicit

Numerical solution schemes can be either explicit or implicit. An explicit scheme directly solves the next time step using known variable values at the current solution time step. An implicit scheme uses values from both the current and the next time step and employs matrix or iterative techniques to obtain the solution. The FHWA document *Two Dimensional Hydraulic Modeling for Highways in the River Environment* (FHWA 2019) provides detailed explanations and examples of implicit and explicit solution schemes. Models that employ explicit solution schemes require consideration of the Courant condition. Section 8.4.4 of this HDS-7 document explains this consideration in detail.

7.3 Geometric Requirements for Unstructured Grid Models

7.3.1 Meshes

Each numerical model imposes limits on the type of mesh used and the type of elements that make up the mesh. Structured grids are more computationally efficient but have limitations on how they represent topographic features and the level of hydraulic detail they can provide due to limitations on mesh resolution. The most structured format is a pure Cartesian grid with similar-sized elements (Figure 7.1). While still maintaining the simplicity of a grid, some models allow variable resolution by allowing elements to be rectangular or vary in size. Quad-tree grids provide the greatest flexibility for structured grids. This grid type allows elements to be sub-divided into smaller elements to provide higher resolution in areas of complex geometry. However, the orientation of the elements remains constrained to the orientation of the grid.

The most flexible mesh type consists of a broadly variable network of triangles and quadrilaterals, also known as an unstructured grid. General guidelines on element shape are followed to maximize numerical stability and performance; however, no absolute constraints are imposed on shape and size variation. This type of mesh is a faceted surface representing the ground. As facets get smaller (points closer together), the representation improves. Some models support the option of additional element shapes. One example is adaptive elements, which do not require transition elements such as triangles. Another example is the use of Voronoi polygon elements, which allow for more edges.

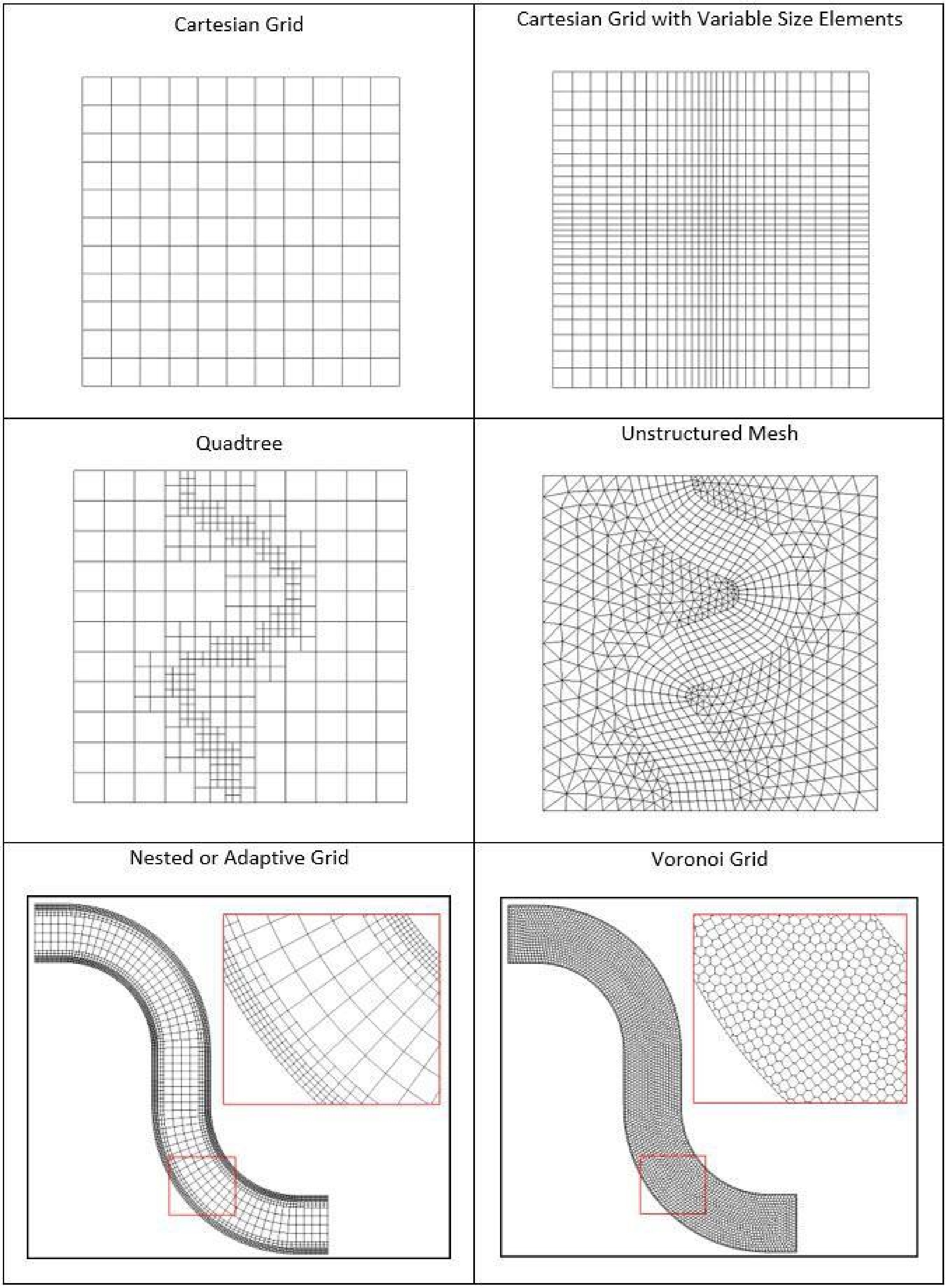

Figure 7.1. Grid and mesh type example.

7.3.2 Mesh Resolution

A mesh typically consists of thousands to tens of thousands of elements. Due to the potentially large number of elements, the generation of a mesh has traditionally occupied a significant portion of the time required to perform a 2D model simulation. Today, improved mesh creation tools and utilities are available, which make the mesh development process less laborious.

The fidelity with which the mesh represents key geometric and geographic features in the domain significantly impacts the validity of the results computed by a numeric model. The omission of a key feature such as a channel that conveys flow can result in higher water levels in the model compared to what would actually occur (see Figure 7.2). Conversely, failing to represent an obstruction, such as an embankment across part of the floodplain, can result in lower water levels or incorrect distribution of flow. The accurate representation of geometric features is the primary aspect of a well-constructed mesh. Additional refinement, or increased mesh resolution, may be necessary to reliably represent hydraulic conditions.

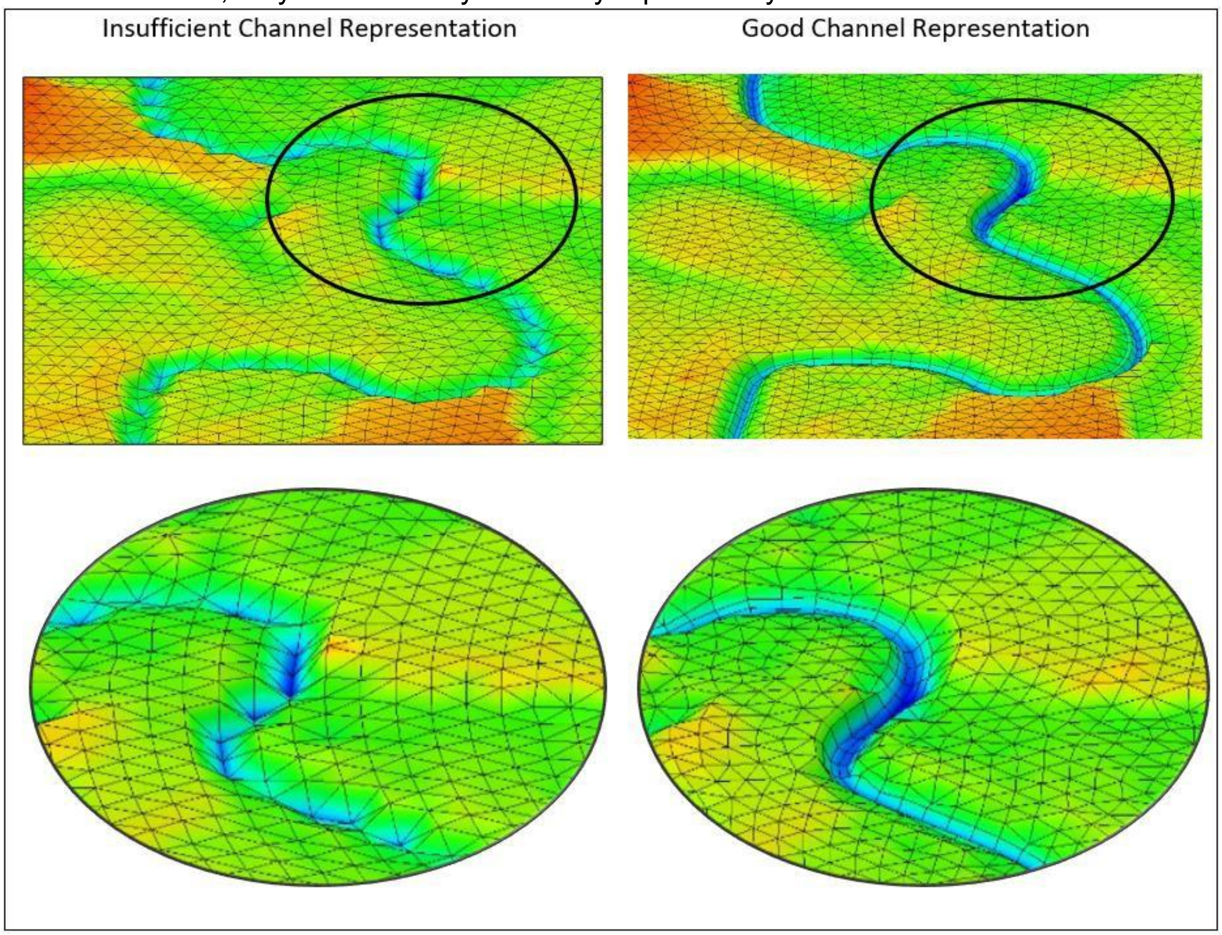

Figure 7.2. Insufficient versus good channel representation of a channel in a mesh.

When adequate terrain data is available, the engineer addresses a few questions during the mesh development. These include: how many elements are required and how best to distribute points to represent the surface? The answers to these questions depend on the features and structures present in the model domain, flow pattern variations in the domain, and the purpose of the model. Flow fields can vary significantly in a small area, such as at a contraction through a bridge and around piers. An adequate mesh includes enough resolution to capture the significant geometric features that impact conveyance and the computed flow field and water surface variations.

A mesh with very small elements would capture all significant features but could result in a larger number of elements, which may consequently lead to longer computing and post-processing time than a more efficient mesh. The other extreme is to generate a mesh with enough resolution to represent the significant features and no more. Starting with this minimum-resolution mesh, it is a good practice to refine areas with substantial geometric or hydraulic change until additional refinement does not change the results of interest, such as the velocity field through a bridge opening. This approach balances computational efficiency with the quality of the results.

Models that use sub-grid resolution, described in Section 7.4, sometimes allow for coarser mesh resolution However, in areas of high hydraulic variability, even models with sub-grid resolution would need smaller-sized elements to better represent the rapid change in hydraulic conditions. With all mesh and element types, it is essential to align element edges along controlling hydraulic features, such as the toe and top of a roadway embankment, channel bank, or levee.

7.3.3 Mesh Quality

Historically, 2D models exhibited numeric sensitivity to element shape. If elements were not close to equilateral triangles or quadrilaterals, the models would diverge and not arrive at a solution. The latest generation of widely-used 2D models are not nearly as sensitive to these shapes as older models. However, it is good practice to review mesh quality to improve model performance and results.

Models that use regularly sized, adaptive, or Voronoi elements inherently have good shape and mesh quality. Unstructured meshes require some care to avoid element shapes that cause model instability. Element shapes that typically cause issues include overly small and wide interior angles.

Since all numerical methods use some type of matrix theory, a single instability prevents the creation of any solution. Therefore, reviewing mesh quality is important. Some modeling software applications include tools to help review mesh quality issues. One can view mesh quality similarly to errors, warnings, and notes in HEC-RAS 1D modeling. It is standard practice to eliminate critical errors and warnings. However, typically not all warnings and notes are significant, and some may not require adjustments to the model.

7.4 Representing Terrain

The terrain data and geometry developed for the model have the most significant impact on the accuracy of model results. Hydraulic controls are the portions of the terrain having an impact on hydraulics. Hydraulic controls include conveyance features such as channels and high ground such as roadway embankments. Developing a 2D model includes the necessary step of verifying the digital terrain surface represents hydraulic controls as they relate to the purpose of the model. Starting with an accurate terrain surface, an engineer can develop a mesh geometry representing the hydraulic controls within the model domain. Terrain features controlling hydraulics at low discharges may not have any hydraulic impact for high discharges. Similarly, some terrain features controlling hydraulics at high discharges have no hydraulic impact for low discharges, such as overbank topographic features outside of the water surface extents at lower discharges.

At any element size, the representation of the terrain surface includes some errors. Some models (i.e., HEC-RAS 2D) store additional information along the edge of an element to account for the approximation in the mesh representation of the terrain. This is commonly referred to as sub-grid scale feature representation. Instead of storing a single elevation, sub-grid elements store a minimum elevation and a curve representing the elevation versus area for an edge or elevation versus volume for the element. The sub-grid scale approach alone does not, however, represent the hydraulic controls within the element. Therefore, the model does not account for impacts the sub-grid features have on flow direction within the element. This is why using breaklines, that force element edges, to represent hydraulic controls for all element types is of primary importance when developing model geometry.

Figure 7.3 illustrates two methods of representing an embankment in a mesh. On the left, a single line of element edges represents the crest of the embankment. On the right, the embankment has two rows of elements along the crest. The bottom graphics illustrate a cross section cut through the embankment. The dashed line represents the actual shape of the embankment, the solid line represents the cross section cut from the mesh, and the shaded blocks represent the element elevations used by the model.

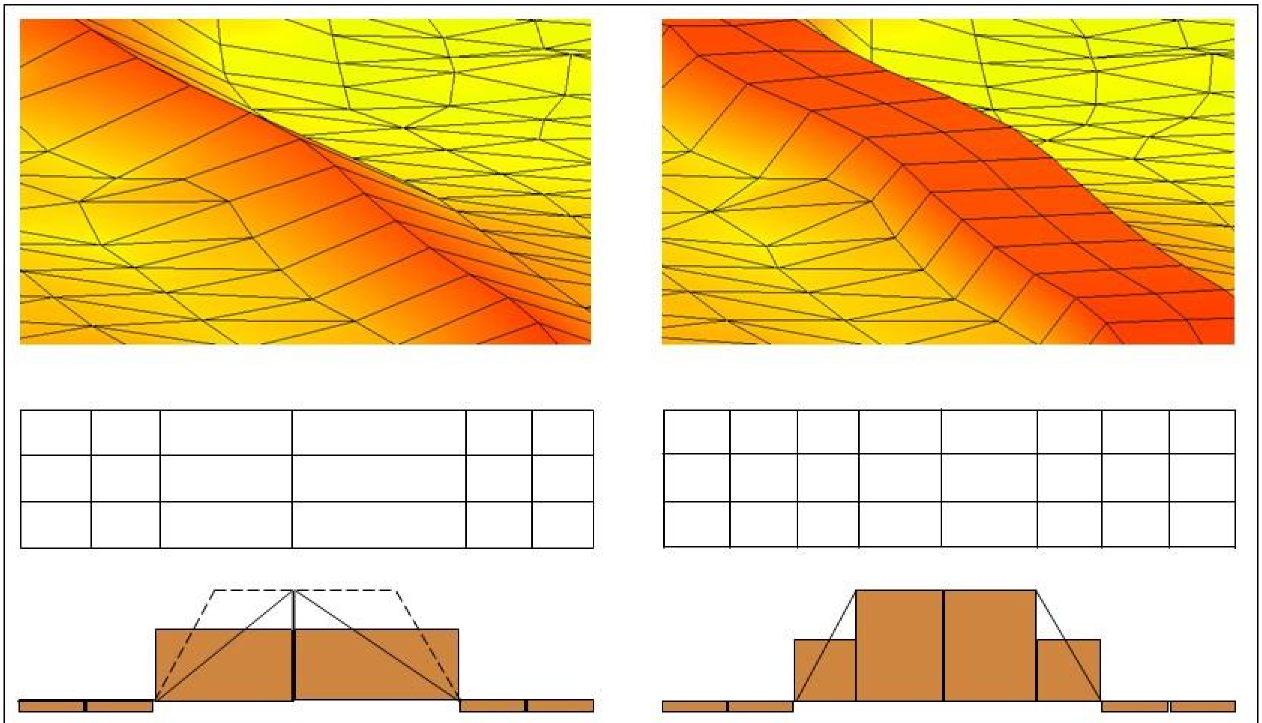

Figure 7.3. Representation of embankment at nodes and elements.

Most models evaluate flow across the element edges. Some models (i.e., SRH-2D) use elevations at the element centers while also evaluating elevations specified at edges for hydraulic barrier definition. The black lines between blocks represent the barrier portion of the embankment. Increasing the mesh resolution is warranted if more detailed hydraulic results are needed that accurately represent the water surface profile and overtopping velocity and depths.

The mesh should incorporate significant hydraulic control features such as channels and embankments. This does not necessitate that the resolution increase everywhere, or for that matter at all. It does involve the placement of breaklines consisting of a row of nodes where the control features exist. Informed selection of the resolution to represent these features, and achieve the desired resolution of hydraulic results, allows for efficient meshes that adequately represent the terrain.

7.5 Representing Flow Resistance

Manning’s n values, discussed in Chapter 4, are the most common method used to account for the flow resistance of vegetation and other surface features. To assign roughness parameters to each element, engineers develop a polygon dataset representing various land use types for the entire project area (Figure 7.4) and then assign appropriate Manning’s n values to the designated polygons.

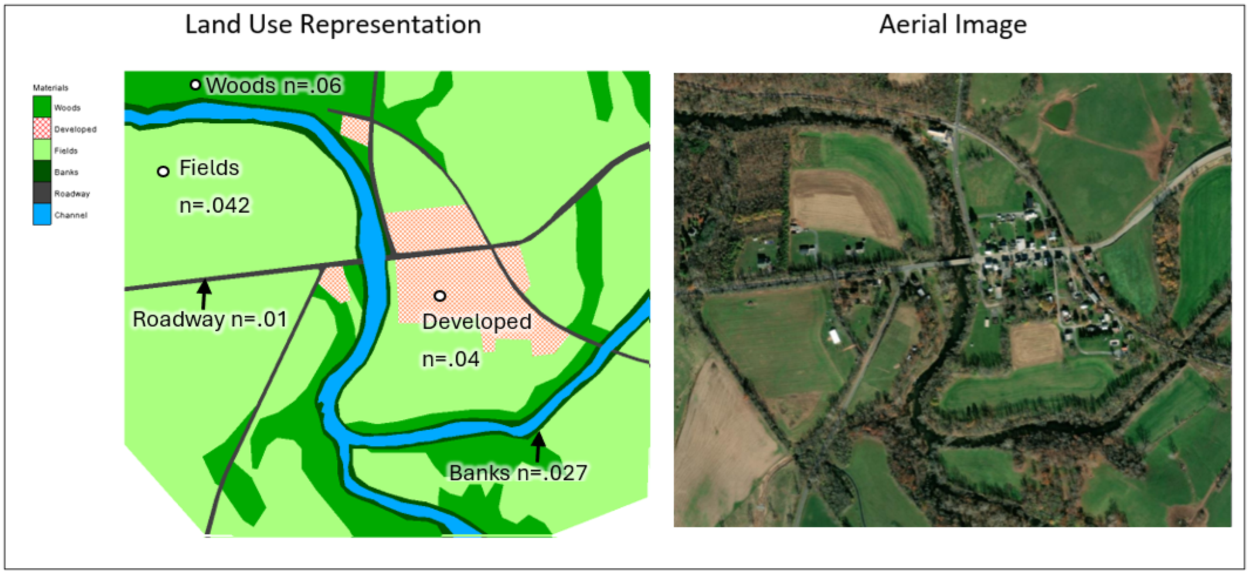

Figure 7.4. Spatial representation of land use types and associated aerial image.

Existing Manning’s n references were developed for application to analytical and 1D models. Two-dimensional models inherently account for some of the assumptions built into the existing n value guidance. It is therefore logical to conclude that n values for 2D models ought to be lower than n values for 1D models. Calibrated 2D models have demonstrated that n values can be approximately 10 percent less than values used for 1D models. However, there have not yet been sufficient studies published on this topic to make definitive conclusions. In addition, the current standard of determining n values is not accurate enough to warrant adjustments to account for the differences between 2D and1D modeling unless calibration data demonstrates it is warranted. Therefore, existing Manning's n value references listed in Chapter 4 are appropriate for determining values for both 1D and 2D models.

Most 2D models allow for depth varied Manning's n values, similar to 1D models. Many land-use types can result in a variation of roughness values depending on flow depth. However, there is currently very little reference information available. Application of depth varied n values requires engineering judgment.

Manning’s n values are sufficient for representing flow resistance for most of the model domain. However, it is often necessary to represent some features in more detail. Two-dimensional models include additional methods for representing various structures. One method for representing structures is to create a hole in the mesh that acts as a wall and does not allow flow through that area of the model domain. This is similar to using blocked obstructions in 1D models. Some models also allow for an unassigned land-use type that functions similar to a hole. Either method is appropriate for buildings when model application requires more granular flow information around structures.

Applying obstructions is another option some models provide for representing flow resistance. Obstructions include point and line features located in single or multiple elements. Input parameters include length, width, and height. The model then computes the drag associated with the obstruction and reduces the conveyance through the elements containing the obstruction. This represents the impact to conveyance through the element caused by the obstruction but does not simulate the detailed flow patterns around the specific obstruction.

7.6 Modeling Highway Crossings

One of the major benefits 2D modeling provides vs. 1D or simple analytical methods is an improved characterization of flow patterns through bridges, near culverts, and over weirs. Most structures consist of complex shapes with abrupt features. They also influence flow patterns and can induce complex and rapidly varying flow conditions. Complex flow conditions include flow regime transitions, rapid spatial changes in flow direction, formation of eddy flow patterns, flow splits, pressure flow, orifice flow, weir flow, and others.

While representing natural features involves care when building a mesh, hydraulic structures present additional challenges. Such structures include vertical walls, enclosed structures such as culverts and bridges, and complex piers. Two-dimensional model meshes cannot directly represent all of these complexities explicitly within the mesh geometry, but methods are available in some models for incorporating structural features.

Typically, models employ three methods to represent hydraulic structures. These are:

1. Building the structure into the mesh (explicit representation).
2. Characterizing the effects of the structure as an attribute of the element that contains the structure (examples of this include additional drag added by an obstruction inside an element, sub-grid scale feature attributes, or pressure flow zones that enforce a maximum water surface elevation based on a user entered low chord).
3. Representing the structure with a separate (normally 1D) model, or calculation, and coupling that model, or results, with the 2D model calculations.

7.6.1 Representing Embankments and Bridge Abutments

Representing structures explicitly with 2D mesh elements (method 1) is preferable when it is practicable. A common example is overtopping flow at an approach roadway embankment near a bridge. Although the profile along the top of the embankment could be represented as a 1D weir structure (method 3) within a 2D model domain, representing the embankment geometry with multiple mesh elements, laterally and longitudinally, results in a better representation of the geometric feature within the model and yields more realistic results.

Similarly, representing abutments in the mesh is a common approach, as illustrated in Figure 7.5. As with the area under the bridge, the 2D flow computations include the flow around the abutments. Including all areas of the roadway embankment and abutment is reasonable if the water level in the simulation exceeds the approach elevations.

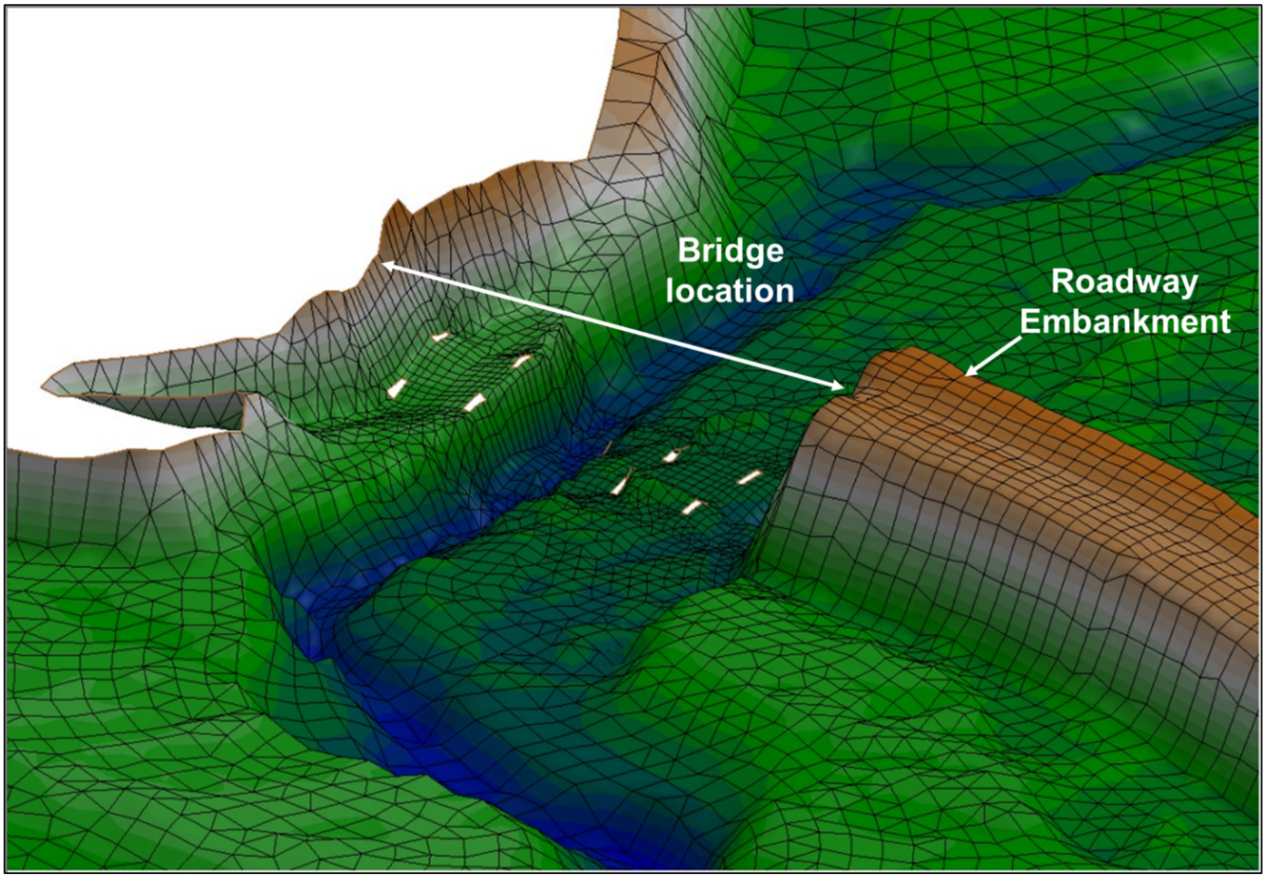

Figure 7.5. Oblique view of mesh geometry representing the channel, floodplain, roadway embankment, and abutments at a bridge.

7.6.2 Representing Bridge Piers

Incorporating piers as holes in the mesh (method 1), as shown in Figure 7.5, or by accounting for the drag force within the element containing the pier (method 2) are two common approaches to pier modeling. Increasing the mesh resolution to incorporate the pier in the mesh definition makes sense if defining flow patterns around piers or defining the flow distribution through the bridge opening is necessary. Treating the pier as an additional drag force to the containing element can be used if the element size is larger than the pier dimensions, or if the pier is complex and representing it with a single element shape is not sufficient. appropriate. Because of the 2D hydraulics assumptions and the 3D nature of physical flow around piers, resolving the hydraulics beyond the point of two to three elements across a pier is not justified nor recommended.

Some models do not allow holes in the mesh. In that case, mesh resolution around the pier can be increased, and the elevation of nodes inside the pier increased to create a geometric representation of the obstruction to flow.

Method 2 is appropriate for analyzing debris on a pier, if necessary. By approximating the size and shape of a debris jam, engineers, or some modeling software, can determine the approximate drag force on the debris and apply an equivalent increase in n value at the impacted elements.

7.6.3 Representing Bridge Decks, Pressure Flow, and Deck Overtopping

The bridge deck affects the hydraulics when the water level in a model reaches the lowest structural member. Even though the mesh in this region of the model already represents flow under the deck, another method is necessary to represent the deck itself. This is traditionally accomplished in one of two ways. The first is to specify additional attributes to the nodes or cells in the area of the bridge. The second is to extract the bridge area from the model and treat it as a separate flow model.

The first approach splits the influence of a bridge deck into two components, pressure flow under the bridge and flow over the bridge. Several models allow the specification of a ceiling elevation for each node or element under the bridge. Figure 7.6 illustrates a bridge structure over a mesh. The engineer specifies the elevation of the bottom of the deck for each node under that structure and the model incorporates pressure terms into the partial differential equations for shallow flow. Some models include functionality to simulate the orifice effects at the entrance to the pressure zone.

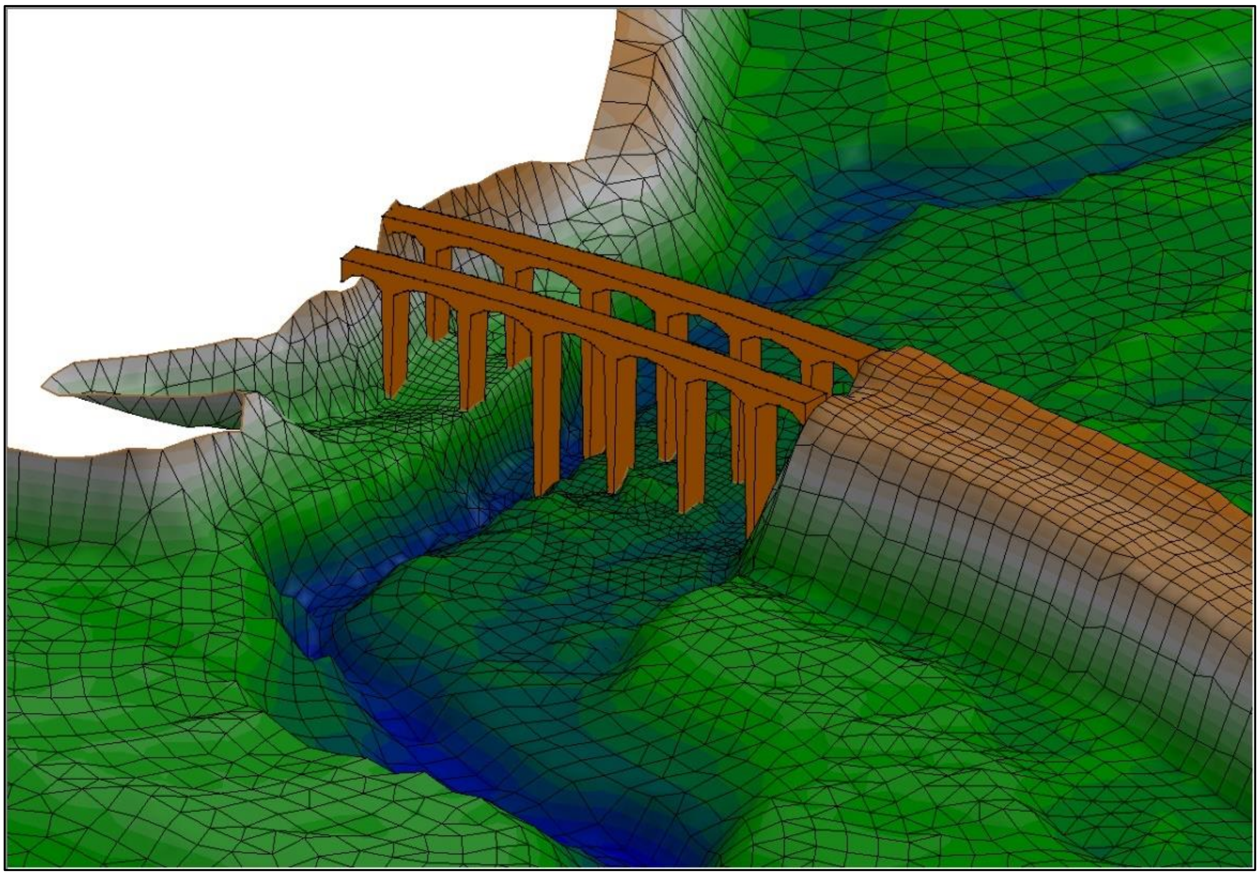

Figure 7.6. Oblique view of a mesh geometry and a bridge deck.

If flow for the simulation reaches the bottom of the bridge deck but does not exceed the elevation needed to overtop, this approach suffices to represent the effects of the deck. Often, superelevation of the bridge or the height of the superstructure on the bridge (railing and barriers) result in all overtopping flow occurring over the road instead of the bridge. In these cases, the model simulation does not require changes to represent flow over the bridge. To represent expected flow over the bridge deck, the user needs to specify the type of weir and the elevation of the crest to allow for a 1D computation of flow (using a weir equation) that leaves the 2D model domain at the upstream face of the bridge and re-enters the 2D model domain at the downstream face.

Another option in some models is to represent the bridge deck as an obstruction. This approach can effectively generate backwater attributed to a bridge but generally needs calibration and provides no additional information about flow over versus flow through the bridge opening. The approach results in reduced velocities at the location of the obstruction because it includes the effect of the obstruction on backwater but does not account for the effect of the reduced opening on the velocity. This means that the engineer is left to determine the true velocity through the opening via separate calculations.

There are important reasons for including a bridge in a hydraulic model. The first is to simulate the detailed flow and velocity distribution near the bridge. The second is understanding the flow paths and patterns. This is the case for designing a new bridge or analyzing an existing bridge, for both hydraulic design and scour evaluation. In these situations, the mesh represents the physical properties of the bridge. Not all modeling programs support all these features. For most transportation-related hydraulic analyses, it is ideal to represent the geometry of the bridge components and their influence on the flow as close to reality as possible. The selection of a modeling program that has the appropriate features and capabilities for a given project is very important. It is equally important to understand how to represent bridge components in the selected model correctly.

7.6.4 Culverts

The three methods described above for representing hydraulics structures also apply to culverts. Culverts may (1) be the primary focus of a 2D model, (2) be a hydraulic feature included in a model, or (3) be so insignificant they do not warrant inclusion in a model. Representing culverts as 1D features within a 2D model has been common practice. In this approach, the 2D model computes WSEs at the upstream and downstream end of a culvert. Combining these water levels or energy grades with the properties of the culvert (number of barrels, barrel size and shape, entrance type, material type, etc.) the program computes the flow through the culvert. This result becomes a local boundary condition in the 2D model with flow leaving the 2D model domain at the upstream end of the culvert and re-entering at the downstream end of the culvert. Because the culvert may impact the upstream and downstream WSE, the calculation is typically iterative. This approach is, in effect, a 1D or analytical model of the culvert that communicates with, or is integrated into, the 2D model. The accuracy is limited by the method that the model employs to compute flow through a culvert. For example, the FHWA program HY-8 assumes still pool conditions upstream of the culvert and thus omits the velocity head of flow entering the culvert.

Another option for modeling culverts is representing them similar to a bridge, as described in the section above. In this case, the user incorporates the lower portion or floor of the culvert geometry into the mesh and represents the top of the culvert as a bridge deck (Figure 7.7). The vertical sides of the culvert can be represented as very steep banks in the mesh or as holes, depending on the model used. This approach involves more work in developing the mesh but has the advantage of representing the 2D flow field through the culvert and accounting for upstream velocity.

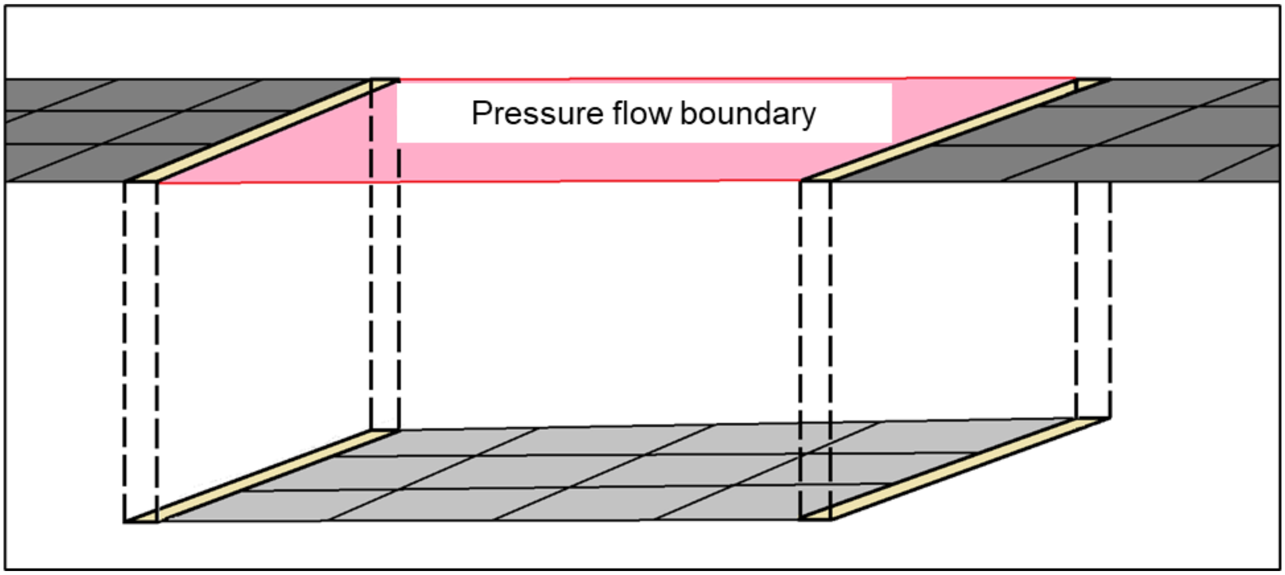

Figure 7.7. Culvert represented as a pressure zone with vertical walls.

CHAPTER 8
UNSTEADY-FLOW HYDRAULIC ANALYSIS & BRIDGES OVER TIDAL WATERWAYS

8.1 Riverine Unsteady Flow

Almost all flow in rivers and streams changes with time and is therefore unsteady to some extent. During a river flood, for instance, the discharge rate at a given location along the river rises to a peak and then gradually decreases, eventually returning to a non-flood condition. A flood hydrograph depicts the relationship of discharge versus time for a particular location. Flood hydrographs in natural river systems often attenuate in the downstream direction due to floodplain storage. Figure 8.1 illustrates the concept of flood hydrograph routing. The figure shows an inflow hydrograph at the upstream end of a river reach and another hydrograph at the downstream end of the reach. The downstream hydrograph exhibits a delay (travel time) and a reduced peak discharge (attenuation). Storage within the reach causes the attenuation of the peak discharge. The amount of attenuation is a function of the storage volume available and occupied. Wide floodplains with deep flow attenuate a hydrograph more than narrow floodplains or shallow flow.

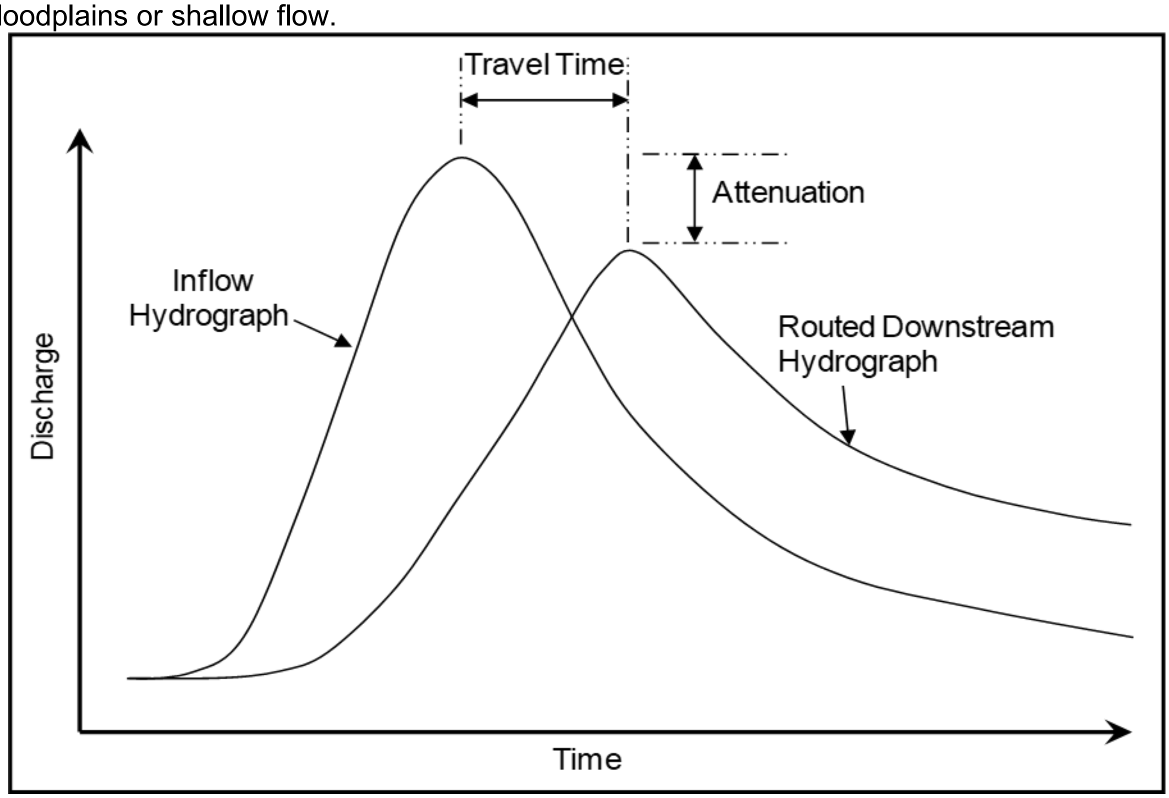

Figure 8.1. Illustration of flood hydrograph routing.

Figure 8.2 schematically represents a riverine unsteady-flow problem in a three-dimensional coordinate space. The horizontal axis represents distance, x, the vertical axis represents discharge, Q (a function of distance and time), and the third axis represents time, t. The figure shows the effects of unsteady flow on the discharge through a river reach over time. As the flood wave travels downstream, the hydrograph is attenuated and lagged in time. Figure 8.2 also illustrates that the conditions at a location x>0 do not change for some amount of time (lag time) until the initial upstream change in flow propagates downstream.

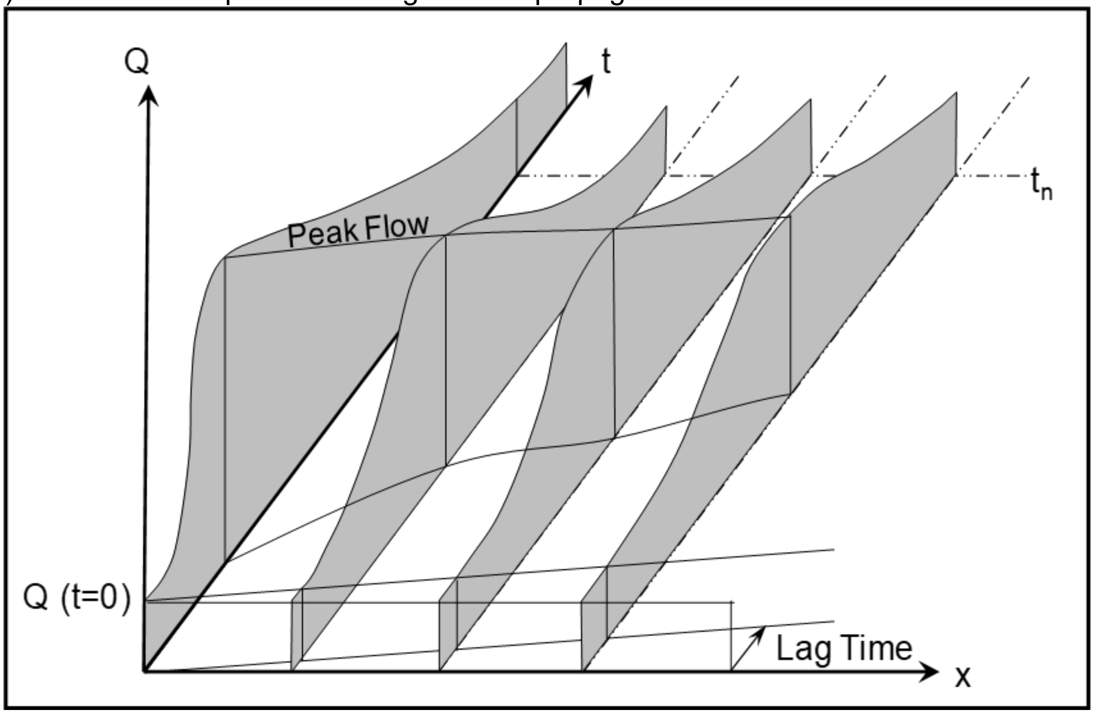

Figure 8.2. Unsteady solution of discharge versus distance and time.

The initial conditions for an unsteady flow model consist of a solution of the steady-state flow, Q, at time t = 0 (a steady-state water surface profile computation), and the inflow hydrograph at the model inflow boundary. As represented in Figure 8.2, the flow is known all along the x axis (throughout the model reach) for time t = 0, and the flow is known for all time steps at the upstream inflow boundary where x = 0.

8.2 Applications of Unsteady Flow in Riverine Bridge Hydraulics

Engineers typically conduct riverine bridge hydraulic studies using steady-flow hydraulic modeling of peak flow conditions at the bridge for a given flood condition. The hydrologic analysis that precedes the hydraulic modeling sometimes incorporates hydrologic routing methods to route the flood hydrograph from upstream to the bridge location, yielding a routed peak discharge for the steady-flow hydraulic analysis. Hydrologic routing is a simplified approach that is not as accurate as unsteady-flow hydraulic modeling but is typically acceptable for most riverine bridge hydraulic studies. In rare cases, a riverine bridge design study or an evaluation of an existing bridge may call for unsteady-flow modeling. Some examples include:

1. To determine whether a bridge project could impact flood peak discharges by altering the routing and attenuation within a stream reach.

2. To estimate the time of flood wave arrival and the duration of flooded conditions at a roadway crossing for flood warning systems and highway operation management.
3. To recreate an actual flood that occurred for forensic analysis purposes.
4. To incorporate the bridge project into an existing unsteady-flow floodplain model (as may be required by a city, county, levee district, or other jurisdictional authority).
5. To simulate the subject reach under various release operation scenarios from an upstream dam (for flood control, power generation, recreation, wildlife management, etc.).
6. To model a dam-break flood through the bridge reach.
7. To allow for a combined hydrologic/hydraulic analysis through rain-on-grid modeling.

Some of these listed scenarios merit further discussion. Occasionally a concern arises that a bridge replacement project may cause increased peak discharge rates downstream by increasing the hydraulic capacity of the bridge opening. The engineer can use unsteady-flow modeling to determine whether the project has any appreciable effect on flood hydrograph routing to the reach downstream of the bridge. Only rarely does a bridge replacement project result in increased peak discharges downstream. Exceptions may consist of more extreme cases, such as replacing a small culvert opening with a bridge that provides much greater hydraulic capacity.

Many communities, regional authorities, and transportation departments have begun to examine the performance of their transportation networks during floods from the standpoint of continued mobility, resilience and the safety of the traveling public. Accurate flood wave modeling of hypothetical and actual flood hydrographs is helpful for flood warning systems, evacuation plans, road closures, detour planning, and road maintenance operations.

Rain-on-grid modeling is an increasing practice, especially in flood inundation modeling. For example, many current studies to develop approximate floodplain maps involve coarse 2D hydraulic models of the entire watershed and the stream reaches of interest using rain-on-grid techniques to determine the approximate flood hydrology. Such practices are appropriate for their intended purpose of approximate flood inundation mapping. However, approximate models do not accurately resolve bridge hydraulics concerns such as backwater, freeboard, roadway overtopping, or scour variables. Rain-on-grid hydrologic analysis requires the model to cover a much larger domain than a model developed solely for hydraulic analysis, creating a need to use a coarse mesh resolution. Accurate bridge modeling requires a highly detailed mesh with relatively small elements (see Chapter 7). In model studies using rain-on-grid hydrology, bridge hydraulic analysis may warrant a separate model with appropriate detail and mesh resolution. The peak flow values would be extracted from the coarser rain-on-grid model and used as boundary conditions for the smaller, more detailed model.

The scenarios described above are examples that require unsteady-flow modeling of a bridge reach. However, steady-flow analysis is adequate for a large majority of riverine bridge hydraulic design studies.

8.3 Unsteady-Flow Phenomena in Rivers And Floodplains

Steady-flow analysis assumes that the discharge does not vary at a given location but allows the flow to vary with respect to location along the river reach. As stated earlier, most bridge hydraulic studies are, appropriately, performed using steady-flow analysis to calculate the water surface, flow depth, and velocity throughout the model domain for peak flood flow conditions. Some important assumptions inherent in this approach include:

- Peak flow coincides with the peak stage in the reach.
- The peak flow can be adequately estimated at all locations along the channel reach.
- Peak stages and flows occur simultaneously throughout the model domain (typically a short reach of channel).

The unsteady nature of flood flows can cause actual behaviors that do not agree with those assumptions. For instance, in streams with small bed slopes (i.e., slopes less than 0.0004 ft/ft) or highly transient flows, such as tidal influences or dam breach flood waves, the peak stages do not necessarily coincide with the peak discharges. The rating curves of stage versus discharge are not single valued for rapidly rising or falling flood hydrographs. Actual rating curves can plot as a loop for flood conditions due to changing the energy slope throughout the flood event. The looped rating curve means that two discharges are possible at one stage depending on whether the stage occurs on the hydrograph's rising or falling limb. Several hydraulic parameters affect the magnitude of the loop. Downstream backwater effects may be the most impactful. Each flood follows a different loop. Figure 8.3 illustrates looped rating curves for two floods with the arrows designating the rise and recession limbs of the discharge hydrographs. The inner loop is for a slower rate of rise and fall, which creates a narrower loop.

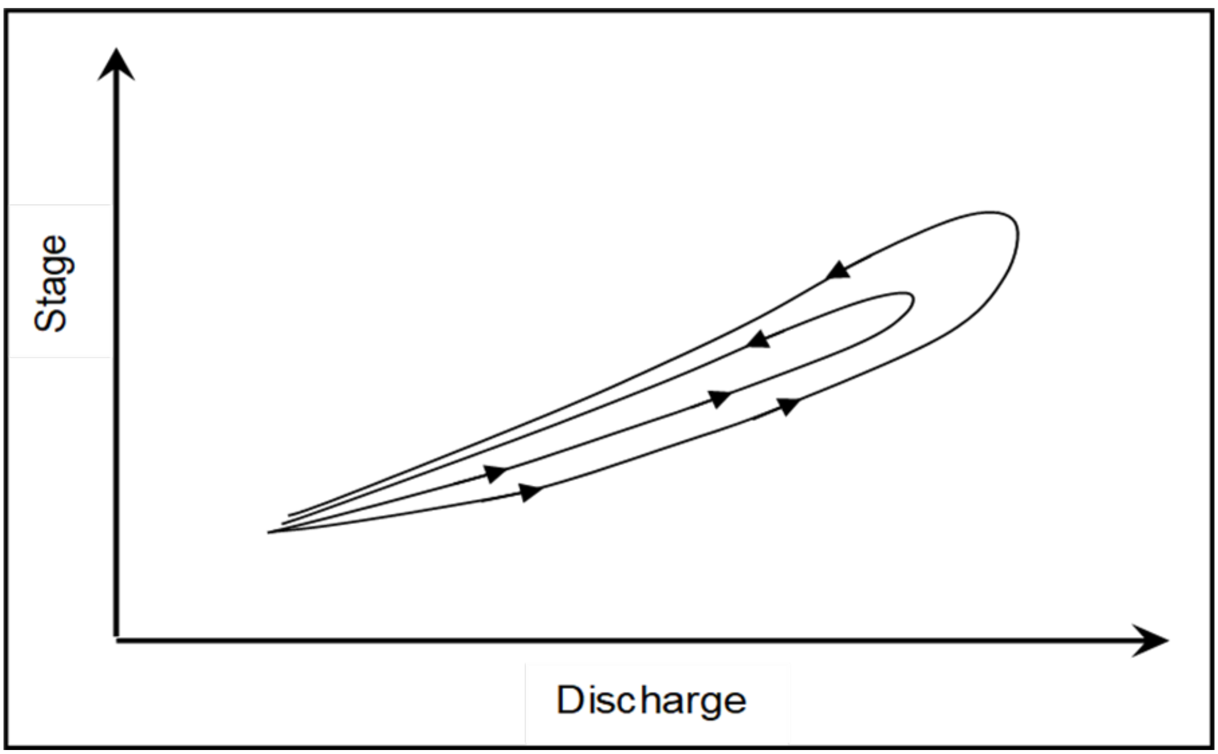

Figure 8.3. Looped rating curves showing the rising and recession limbs of a hydrograph.

The total flow downstream from a junction of two tributaries is not necessarily the combination of the two flows. Backwater from the flow at the junction can cause storage in upstream areas, reducing the flow combinations. Tributary flows entering the main stem channel may experience a flow reversal caused by flow in the main stem backing up into the tributary or vice versa, for instance, when a large tributary flood enters the main stem during a period of low flow.

If the discharge or stage at the upstream model boundary is changing rapidly, the acceleration terms in the momentum equation are substantial, and thus unsteady flow is a more appropriate computation. Examples of this phenomenon are dam-break analyses and rapid upstream gate openings and closures. Regardless of the slope of the channel, an unsteady-flow analysis produces different and more accurate results than a steady-flow analysis for very rapid changes in flow conditions.

8.3.1 Floodplain Storage and Routing

Section 8.1 describes the routing of flood hydrographs due to storage. The overbanks in a river reach provide most of the storage that causes routing. When the river is rising, flow moves laterally away from the channel, inundating the overbanks and filling available storage areas. As the depth increases, the overbanks begin to convey water downstream. When the river stage is falling, water in the overbanks moves back toward the channel from the overbanks supplementing the main flow in the channel.

The flow patterns in occupying and retreating from the overbanks are 2D in nature. Two-dimensional unsteady-flow models accurately simulate flood routing. One-dimensional models can approximate routing effects because the primary direction of the flow in the reach is downstream. Steady-flow models do not consider storage volume in the calculations or results but rather focus strictly on flow moving downstream through the model domain. Unsteady-flow models by necessity account for both storage and conveyance. In 1D unsteady-flow modeling, there are various ways to account for storage. Program options are available to restrict flow to effective areas such as ineffective flow specifications and levees. The engineer decides the most effective way to incorporate the available storage into a 1D model from various approximate approaches.

Chapter 5 explains the differences between the assumptions and limitations of 1D and 2D models. Those same differences make 2D modeling more accurate in occupying available storage and correctly simulating flood routing. Simply by correctly representing the overbank terrain with sufficient detail, the engineer can ensure accurate routing computations by the 2D model without needing to incorporate additional features or assumptions to represent ineffective flow.

8.4 Modeling Considerations for Unsteady-Flow Analysis

8.4.1 Model Domain Requirements

Chapter 5 explains the requirements for adequate upstream and downstream model extents in steady-flow modeling. All the factors described in Chapter 5 apply to unsteady-flow analysis for both 1D and 2D modeling. However, an additional consideration for unsteady-flow modeling indicates a need for potentially much greater upstream and downstream extents. The engineer needs to extend the limits of an unsteady-flow model to include all potential storage upstream and downstream that could affect the results of interest at the bridge. Keeping the model extents to the minimum required for steady-flow modeling results in an inaccurate simulation that underestimates the storage and routing effects. The need to account for storage in unsteady-flow modeling also dictates that the lateral extents of the mesh (in 2D modeling) or the cross sections (in 1D modeling) need to be enough to capture all the storage volume, including ponded water that may not be flowing downstream at all.

8.4.2 Boundary Condition Development

Unsteady-flow models compute time-varying simulations. The engineer enters the start time, computational time step, and simulation duration as model-control input. A time-varying simulation calls for time-varying boundary conditions. Typically, in riverine unsteady-flow modeling, a discharge hydrograph is entered for each inflow boundary condition. Therefore, the engineer needs to develop a full flood hydrograph for each event of interest. The flow conditions at the outflow boundary location also vary with time throughout the simulation. Thus, a constant specified water surface is not an appropriate downstream boundary condition entry. It is appropriate to enter a boundary condition specification that allows the water surface to change as the outflow changes, such as normal depth or a rating curve.

The fluctuating tide levels drive or strongly influence the flow directions, discharge rates, and velocities throughout the model domain in tidal settings. The engineer develops downstream boundary conditions in the form of stage hydrographs representing the normal tide cycle with potential storm surge effects, if any, superimposed on the tides.

8.4.3 Time-Specific Model Results and Output

Steady-flow modeling assumes a constant flow rate and boundary conditions that persist indefinitely. The model produces one set of results throughout the model domain representing that constant condition. Some programs, such as the 2D modeling program SRH-2D, conduct all model runs as unsteady-flow simulations. The engineer achieves a steady-flow solution by ramping up the flow boundary conditions over time to the desired peak conditions and then holding those boundary conditions constant over a long time until the complete solution is steady over several time steps. The model computes a solution at every time step, but only the results at the last time step (after the establishment of steady flow) are of interest to the engineer in such a simulation.

In unsteady-flow applications, the engineer is interested in the results at many computed time steps. The peak discharge and other results needed for design occur at different time steps in different locations. Full hydrographs of discharge, water surface, and velocity can be determined and plotted at locations throughout the model. The engineer selects the time interval for the model to produce output files of the simulation results. The output time interval is typically much longer than the computation time step. For example, a model may have a duration of 12 hours with a computational time step of a few seconds. The engineer may instruct the program to produce output files every 15 or 30 minutes. When setting up an unsteady 2D model with tens of thousands of elements, one should expect a substantial disk storage requirement for the output files.

8.4.4 Numerical Stability in Unsteady-Flow Modeling

A model has to be numerically stable to achieve an accurate simulation. Model stability is often more challenging for unsteady-flow models than for steady flow. A numerical model is unstable when numerical errors grow to the extent that the solution begins to oscillate, or the errors become so large that the computations diverge, and the model fails to complete the simulation. Factors affecting stability for 1D models include cross-section spacing, computational time step, theta weighting factor, solution iteration, and solution tolerances. Chapter 6 explains the requirements for locating cross sections in 1D steady-flow models. Unsteady-flow models may require more cross sections (shorter cross-section intervals) to maintain numerical stability.

The numerical method applies the theta weighting factor when solving the unsteady-flow equations in a 1D model. Theoretically, theta can vary from 0.5 to 1.0. However, a practical limit is from 0.6 to 1.0. A theta of 1.0 provides the most stability, while a value of 0.6 provides the most accuracy. The engineer seeks a balance between accuracy and computational robustness when selecting theta.

One-dimensional models are vulnerable to instability when the computed water surface elevation crosses certain thresholds from one time step to the next. Take as just one example a time step when roadway overtopping begins at a bridge crossing. The ineffective flow specification may deactivate when the water surface reaches the overtopping elevation so that suddenly a large portion of the cross-section conveyance becomes available. This sudden change can cause a shock that leads the entire model solution to diverge. The engineer can employ specific workarounds when such a problem arises, but the process can be tedious and may compromise the simulation quality.

Two-dimensional models can also become unstable in some unsteady-flow simulations. However, 2D models are typically more robust and stable than 1D unsteady-flow models. This is because the overall 2D modeling approach is free from many of the approximating assumptions of 1D models. Mesh quality and resolution, combined with the computational time step, factor into the stability of 2D models.

An appropriate time step promotes stability in both 1D and 2D modeling. The Courant-Friedrichs-Lewy (CFL) condition (commonly referred to as the Courant condition) can guide the time step setting. The dimensionless Courant number compares the flood wave celerity, V_w (the rate at which information can pass through the solution area), to a representative length of a computational element, ΔX. The ΔX may be a representative distance between cross sections in a 1D model or the dimension of a 2D model element in the direction of flow. The flood wave speed usually is greater than the average velocity. A reasonable approximate value is 1.5 times the average velocity. Under the Courant condition, the Courant number is ideally less than or equal to 1, as expressed below.

$$C_r = V_w\left(\frac{\Delta t}{\Delta x}\right) \leq 1.0 \text{ or } \Delta t \leq \frac{\Delta x}{V_w} \tag{8.1}$$

Modeling methods that employ explicit methods require a time step that limits the advancement of flow through the domain to less than one computational element per time step, thus satisfying the Courant condition. Some of the 2D modeling programs currently in widespread use employ implicit numerical methods. Implicit schemes link all the elements together through an iterative solution that allows the transmission of signals throughout the model mesh. The Courant condition is not inviolable in an implicit scheme model but selecting a time step that generally satisfies the condition can aid in model stability.

8.5 Dynamic Equations for Unsteady Flow

8.5.1 One-Dimensional Saint-Venant Equations

The equations governing the flow of water, in general, derive from known physical principles, the conservation of mass and momentum. One-dimensional unsteady-flow models employ numerical techniques to solve the Saint-Venant Equations, which consist of the unsteady-flow continuity equation and the dynamic momentum equation. This section presents and explains the equations but not their derivations. Multiple textbooks present derivations of the equations, including *Open-Channel Flow by Chaudry* (2008).

Unsteady Flow Continuity Equation:

The continuity equation for a control volume (the reach between two cross sections) is:

$$A\frac{\partial V}{\partial x}+VT\frac{\partial y}{\partial x}+T\frac{\partial y}{\partial t}=q \tag{8.2}$$

where:

A	=	Total inundated area of a cross section (active flow and inactive area), ft^2
q	=	Lateral inflow per unit length in a reach, ft^2/s
T	=	Top width at the water surface, ft
V	=	Cross-sectional average flow velocity, ft/s
y	=	flow depth, ft
x	=	distance (with respect to)
t	=	time (with respect to)

The physical meanings of the terms in the continuity equation are:

$A\frac{\partial V}{\partial x}\cdot$	=	prism storage
$VT\frac{\partial y}{\partial x}\cdot$	=	wedge storage
$T\frac{\partial y}{\partial t}$	=	temporal rate of rise

Dynamic Momentum Equation:

One expression of the dynamic momentum equation presents each term as a gradient or slope:

$$S_f=S_0-\frac{\partial y}{\partial x}-\frac{V}{g}\frac{\partial V}{\partial x}-\frac{1}{g}\frac{\partial V}{\partial t} \tag{8.3}$$

The physical meanings of the terms in the dynamic momentum equation are:

S_f	=	friction slope (frictional forces)
$\frac{\partial z}{\partial x}$	=	= bed slope (gravitational effects)

$\frac{\partial y}{\partial x}$ = pressure differential across the control volume

$\frac{V}{g}\frac{\partial V}{\partial x}$ = convective acceleration across the control volume

$\frac{1}{g}\frac{\partial v}{\partial t}$ = local temporal acceleration within the control volume

Solved together with the boundary conditions, Equations 8.2 and 8.3 are the complete dynamic wave equations for one-dimensional unsteady flow.

8.5.2 Two-Dimensional Shallow Water Equations

Variations of the shallow water equation (SWE), derived from the Navier-Stokes Equation, are the governing equations for most 2D models. Chapter 4 presents the 2D unsteady-flow continuity equation (Equation 4.4) and the 2D unsteady-flow momentum equations (Equations 4.8 and 4.9). FHWA (2019) provides a detailed description of the development of these equations and illustrates their terms' physical meanings.

Examination of the SWE reveals that the equations account for forces, gradients, and velocities with components in the x and y directions at every element. In solving the governing equations, 2D models move flow laterally into and out of storage through time following fluid mechanics principles. Unlike a 1D model, a 2D model does not rely on ineffective flow specifications by the user to determine how to occupy the floodplain and is not constrained to a single water surface across the floodplain or a single direction of action for velocities and forces.

8.5.3 Simplifications of the Dynamic Wave and Shallow Water Equations

Some modeling programs simplify the 1D and 2D unsteady-flow momentum equations for various applications. Approximations to the full dynamic wave equations combine the unsteady-flow continuity equation with reduced versions of the momentum equations. The most simplified version is the kinematic wave approximation, which eliminates the temporal and convective acceleration terms and the pressure gradient terms. This reduces the 1D momentum equation to simply:

$$S_f = S_o \tag{8.4}$$

One common application of the kinematic wave approximation is shallow overland flow computations over steep terrain for hydrologic analysis.

The diffusive wave approximation adds the pressure differential terms to the kinematic wave approximation. It omits temporal and convective acceleration terms. This approximation applies to all situations suitable to the kinematic wave equations and is generally more accurate. However, the omission of the acceleration terms means that momentum forces are neglected. Accurate modeling of most bridge crossings requires the momentum force terms. Therefore, models using the diffusive wave approximation are not suitable for bridge hydraulic studies.

The full dynamic wave equations incorporate all terms of Equations 8.2 and 8.3 for 1D modeling and all terms of Equations 4.4, 4.8, and 4.9 for 2D modeling. Unsteady flow, 1D or 2D analysis of bridge crossings is best served by models using the full dynamic wave equations.

8.6 Bridges Crossing Tidal Waterways

Flow in tidal waterways is inherently unsteady at all times due to the perpetual fluctuation of the tides. The stage hydrograph (water surface elevation versus time) shifts in time and attenuates, or in some cases, amplifies with distance away from the ocean. The flow can reverse directions multiple times a day at a particular location, flowing inland on the flood tide and flowing back out toward the ocean on the ebb tide. Heavy sustained winds exert an additional unsteady flow effect on extensive, open tidal waterways. Bridges in coastal settings can also be affected by waves.

The remainder of this chapter explains the additional hydraulic concerns for bridges over tidal waterways. The two primary FHWA references relevant to this topic are *HEC-25 Highways in the Coastal Environment, Third Edition* (FHWA 2020), and *A Primer on Modeling in the Coastal Environment* (FHWA 2017). The following sections summarize essential information from these two documents. Chapter 9 of this document discusses scour considerations at tidal and coastal bridges.

8.7 Types of Tidal Waterways

Tidal waterways can include any body of water whose hydraulic behavior is either dominated or influenced by tides and storm surges. Tidal fluctuations are the drivers of the flows and water levels in bays, lagoons, inlets, and tidal connecting channels. These water bodies experience flow reversals with every tide cycle. Estuaries can include reaches that are tidally dominated and others that are tidally influenced.

8.7.1 Estuaries

As described in HEC-25, an estuary is the part of a river that is affected by tides (FHWA 2020). The tides modulate the flow and stage in tidally influenced river reaches, but the flow normally does not reverse under normal tide conditions. The tidal fluctuations result in a cyclic variation in the downstream control of the tailwater. The degree to which tidal fluctuations influence the discharge at a particular river crossing location depends on such factors as the relative distance from the ocean to the crossing, the riverbed slope, the cross-sectional area, available storage volume, and hydraulic resistance. Although other factors are involved, the relative distance of the river crossing from the ocean is a reasonable qualitative indicator of tidal influence. For crossings located far upstream, tidal fluctuations are minimal and have only a minor influence on the depth, velocity, and discharge through the bridge. At locations close to the ocean, tidal fluctuations are large and dramatically affect the discharge, stage, and velocity of the estuary flows. In reaches near the ocean, the magnitude of the tidal fluctuations is large enough to cause regular flow reversals connected to the tide cycle.

The controlling hydraulic events for the design of bridge crossings over estuaries can include:

- Riverine flood events in the tidally influenced region, especially for larger drainage areas. Combining the riverine design flood with low tide levels would yield a lower water surface, but higher velocity for scour evaluation. Combining the riverine flood with high tide levels would yield a lower velocity but higher water surface elevation for freeboard and backwater analysis.
- Storm surge events or extreme astronomical tides in the tidally dominated region, especially when the drainage basin is small. The worst-case conditions for design could occur during flood tide (flows in the upstream direction) or during the ebb tide (flows in the downstream direction).

- Either type of event in the transition region between tidally influenced and tidally dominated. It is usually necessary to model both riverine design floods and storm surge design events to determine which conditions control at the site.

8.7.2 Bays, Lagoons, and Tidal Connecting Channels

HEC-25 describes a bay as a body of water almost surrounded by land but open to some tidal flow communications with the ocean (FHWA 2020). Bays are connected to the ocean through inlets but can also connect to other bays or lagoons through tidal connecting channels. Complex tidal systems consisting of bays and lagoons connected by channels often exist between barrier islands and the mainland shore.

Storm surges or extreme astronomical tides are the controlling hydraulic events for the design of bridge crossings over bays, lagoons, or connecting channels. Riverine flows into bays are usually negligible compared to tidal flows. The water levels in bays respond to the ocean tides. The degree of responsiveness depends on the bay's size and the hydraulic capacities of the inlets connecting the bay to the ocean. A large bay may experience tide level ranges that are only a small fraction of the ocean tide range. Sustained winds can significantly affect the water levels and currents within bays and lagoons and consequently their connecting channels.

Determining the controlling hydraulic event is challenging and often requires modeling multiple scenarios when a crossing is part of a complex system behind a barrier island. Figure 8.4 shows multiple highways crossing over a system of bays, lagoons, and inlets. When a location is nearly equidistant between two inlets of substantial size, the controlling event could be a storm that causes a high surge at one inlet and a much lower one at the other inlet, or a storm that causes the reverse. Another possible design storm is one that causes a high sustained wind along the axis of the bay in either direction. Waves are a consideration in the design of bridges over bays, lagoons, or connecting channels because of their potential to damage the bridge superstructure, cause scour at piers and abutments, and compromise scour countermeasures.

8.7.3 Inlets

In HEC-25 an inlet is a short, narrow waterway connecting a bay, lagoon, or similar body of water with a large parent body of water (FHWA 2020). Tidal inlets convey flow back and forth between the ocean and the connected body of water. The width, depth, orientation, and stability of an inlet can significantly influence the hydraulics of the bay and the beaches along the shoreline adjacent to it.

The difference in water level between the ocean and the connected water body determines the flow direction, discharge, and velocity through an inlet. If the ocean level is higher, the flow is into the bay (flood tide). If the bay level is higher, the flow is toward the ocean (ebb tide). The water level in a large bay can lag significantly behind the ocean levels. Therefore, when the ocean tide is rising or falling rapidly, the difference in water level can become substantial. The large difference and short length of an inlet result in a steep slope and, consequently, a high peak discharge and velocity. Notably, a tidal inlet can reach a high peak flood discharge and ebb discharge twice every 25 hours. Strong sustained winds can boost or impede the flow through an inlet if the wind direction aligns with its axis.

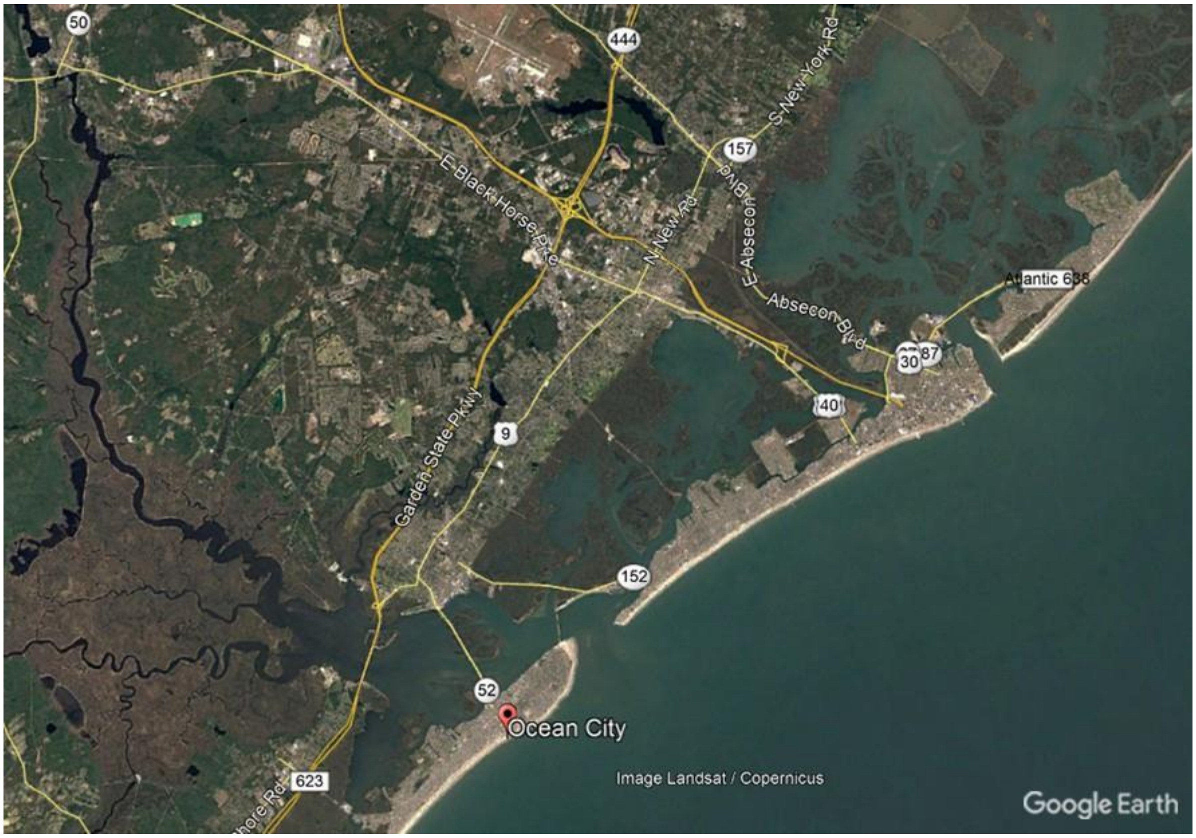

Figure 8.4. Multiple highway crossings within a complex tidal system behind barrier islands in New Jersey.

Tidal inlets can be geomorphically unstable. They tend to migrate along the shore unless they are locked into place by jetties, bulkheads, or revetments. Some inlets experience severe long-term degradation under the action of the astronomical tides. The vertical stability of an inlet is a complicated function of the tide range, the size of the bay, the amount of sediment reaching the inlet via littoral drift, whether the littoral drift is interrupted by jetties or other works, along with other factors.

The controlling hydraulic event for design of a bridge over a tidal inlet is usually a storm causing a surge. The maximum rate of the rise or fall of the storm tide (astronomical surge plus storm surge) yields the peak flow and velocity through the inlet. The maximum height of the storm tide combined with computed wave heights determines the appropriate elevation of the lowest structural member of the superstructure. If the inlet is experiencing long-term degradation, the role of the astronomical tides in the process may be significant.

8.7.4 Other Tidal Waterways

Passages between an island and mainland or between two islands are another type of tidal waterway. Significant tidal flow can occur in a passage if there is even a small shift in the timing of the tide from one side of the passage to the other. Flow through passages may not be significantly influenced by tides but can be dominated by ocean currents and wind.

The Intracoastal Waterway is a series of natural and dredged channels intended for small craft navigation along the U.S. Atlantic and Gulf Coasts. The dredged channels may be within shallow bays or canals cut to connect interior waterways. The hydraulics in the canal sections are controlled by tidal action at the ends of the sections and may be influenced by wind and upland runoff.

The HEC-25 document (FHWA 2020) presents extensive information on tidal waterways and the hydraulic processes to consider in designing and evaluating bridge crossings.

8.8 Astronomical Tides & Tide Cycles

The tide the cyclical rise and fall of ocean level caused by the gravitational pull of the moon and sun. Accurate astronomic tide predictions are available at many locations along the U.S. coast. Weather phenomena such as sustained winds or storm surges can cause differences between the actual tide and the predicted tide. The daily tide range varies with time at a given location, and the average tide range varies with location along the coast.

8.8.1 Tide Characteristics

A tidal day is roughly 24.8 hours. The tidal period at a given location is the time between successive high tides or successive low tides. Locations with semidiurnal tides see two high tides and two low tides every tidal day. In other words, the tidal period is half of a tidal day. If one of the high tides each day is significantly higher than the other, the tide is mixed. Mixed-tide locations experience a higher high water, lower high water, higher low water, and lower low water every tidal day. Semidiurnal or mixed tides prevail over most of the U.S. coast. Diurnal tides have a tidal period of one tidal day and thus produce one high tide and one low tide every tidal day. Figure 8.5 illustrates the types of astronomical tides.

The tidal range is the difference in height between consecutive high and low waters or between higher high and lower low waters. The tidal range varies over time at a given location. Spring tides have the greatest range and occur twice in a lunar month. Neap tides also occur twice in a lunar month and have the smallest tidal range. A lunar month is approximately 29.5 days. The tidal range also varies by location. The mean range can be less than a foot in some locations and over 30 feet in others.

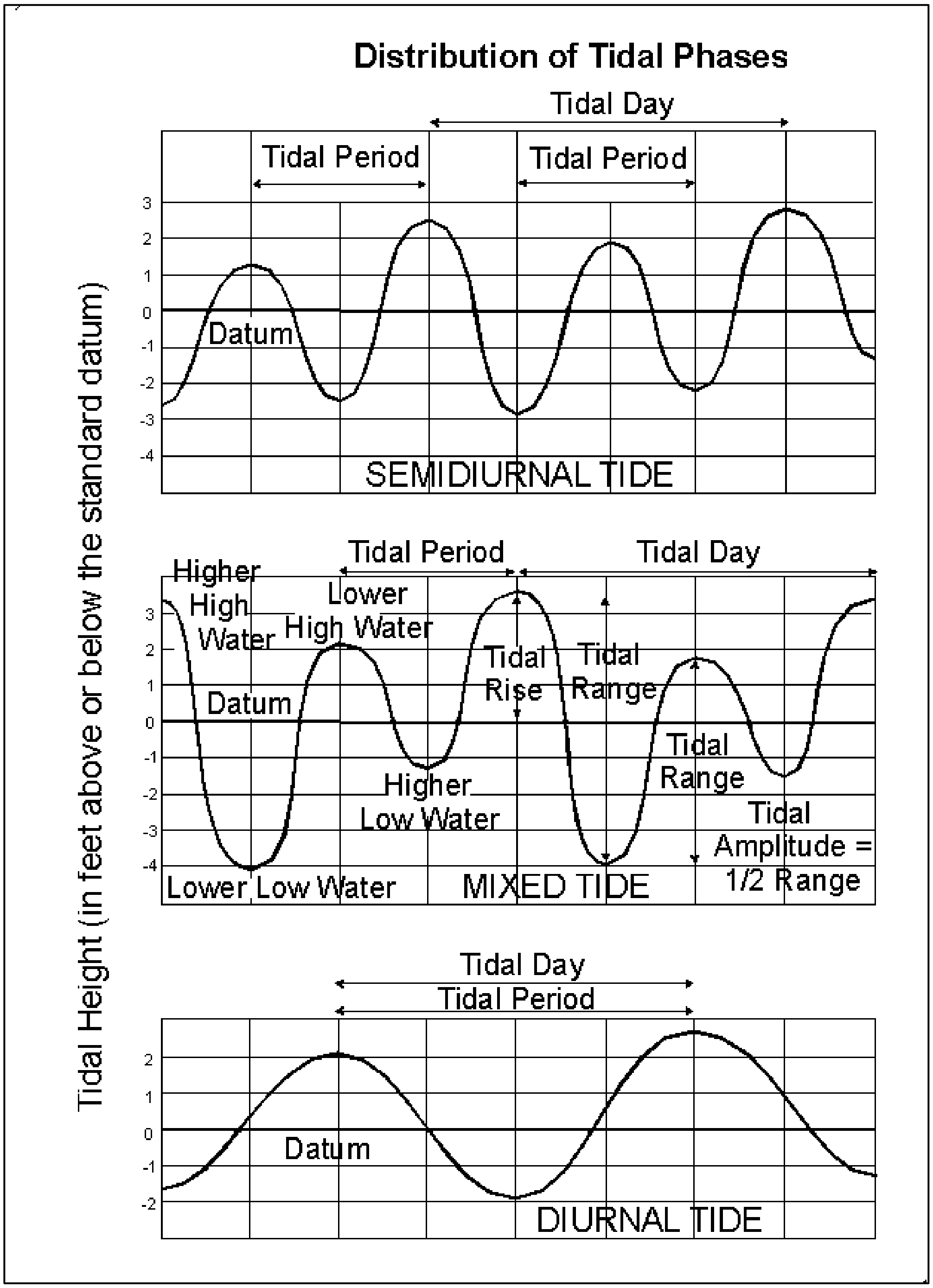

Figure 8.5. Basic definitions of astronomical tides (image from the National Oceanic and Atmospheric Administration's online document *Our Restless Tides*.

8.8.2 Tidal Datums and Benchmarks

Engineers performing design projects in tidal settings need to determine the relationship between the standard vertical survey datum adopted for the project and the tidal datums at the site of interest. Tidal datums refer to the tide levels averaged over an 18.6-year tidal epoch at a specific location. There are several tidal datums at each location. The mean high-water datum (MHW) is the average of the high-water elevations. The mean higher high water (MHHW) is the average of the higher high-water elevations. The mean low water datum (MLW) is the average of the low water elevations, and the mean lower low water datum (MLLW) is the average of the lower low water elevations. Navigation charts typically provide depths below MLLW because actual depths encountered will usually be greater than the chart depths, even near low tide. The local mean sea level datum (MSL) is the average of all water level observations over a tidal epoch.

Fixed vertical survey datums are specified for geodetic surveying and set by the NOAA's National Geodetic Survey. The National Geodetic Vertical Datum of 1929 (NGVD 1929) and the North American Vertical Datum of 1988 (NAVD 88) are two primary vertical survey datums used in the United States. The relationship between the survey datum, NAVD 88, and the tidal datums, e.g., MSL and MLLW, has been calculated by the NOAA National Ocean Service for many of the tide gages around the United States. Figure 8.6 illustrates the survey and tidal datum values for Charleston, South Carolina. The relationship between datums is site-specific and is not the same at other locations along the coast.

The information presented in Figure 8.6 enables the engineer to assign fixed-datum elevations to the tidal datums for sites near the Charleston tide station. For instance:

MSL is 0.22 feet below 0.00 NAVD 88: Elevation **-0.22 feet NAVD 88**

MLLW is 2.92 feet below MSL: Elevation -2.92-0.22 = **-3.14 feet NAVD 88**

MHW is 5.40 feet above MLLW: Elevation 5.40-3.14 = **2.26 feet NAVD 88**

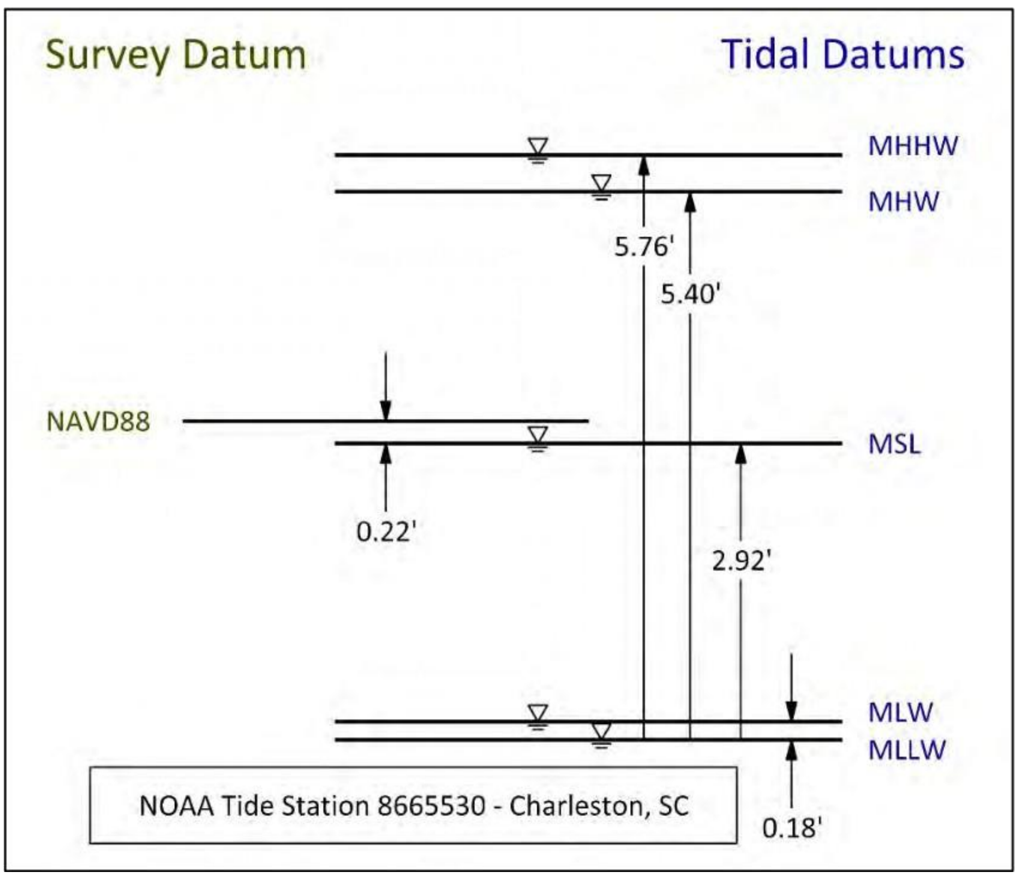

Figure 8.6. An example of the relationship between the survey and tidal datums (image from HEC-25).

8.9 Currents & Discharge in Tidal Waterways

Depending on the coastal location, normal U.S. tidal ranges can vary from less than 1 foot to over 30 feet. A tidal rise or fall of several feet over a typical semidiurnal half-period of about 6.2 hours can result in a significant water surface gradient, especially in an inlet connecting the ocean to a bay. This gradient generates a current in the same direction.

Figure 8.7 is a plot of unsteady-flow model results showing the relationship of tidal stage and discharge (labeled flow) near the mouth of an estuary in South Carolina. In this hydrograph plot, negative discharge values denote flow moving upstream in the estuary (flood tide), and positive values indicate flow moving downstream (ebb tide). The discharge reverses direction in response to the stage fluctuations. Each peak flood tide discharge occurs between a low and the next high tide, and each peak ebb tide discharge occurs between a high and the next low tide. Slack tide periods (discharge at zero) occur close to but are not necessarily coincident with high and low tides. The scenario depicted in Figure 8.7 is a storm surge that causes the tide to reach more than 8 feet elevation. The highest flow magnitude is nearly 40,000 cfs, reaching a flood tide peak roughly one hour before the highest tide level at that same location.

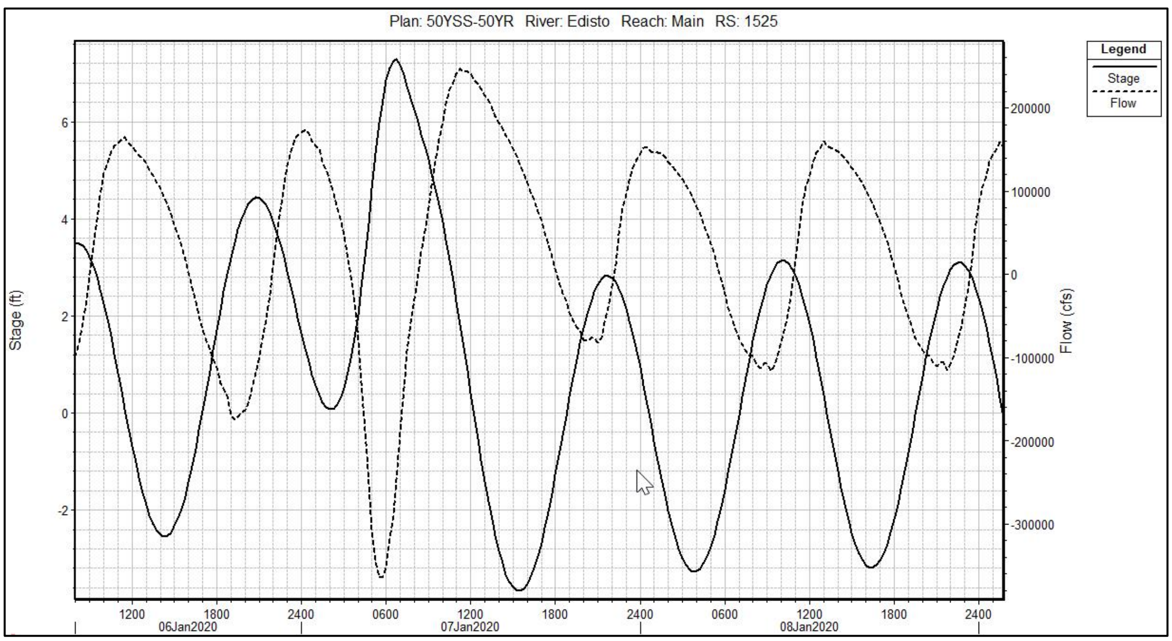

Figure 8.7. Stage and discharge hydrographs for a storm surge scenario near the mouth of an estuary.

The peak velocity magnitude for each tidal swing is a function of:

- The rate of rise or fall of the ocean level.
- The hydraulic capacity of the waterway (depth, width, Manning's n value).
- The inland storage volume between the low and high water level (termed the tidal prism).

The ocean water seeks to fill the tidal prism to the same level. A greater prism volume demands more flow through any connecting waterway. Consider an inlet connecting the ocean to a small bay. The tidal prism volume is small. A given rate of tidal rise at the ocean may generate modest flood tide velocities. If that same inlet were connected to a much larger bay and had the same rate of tidal rise, the flood tide velocities would be much higher because the larger bay demands more flow through the inlet.

8.10 Waves

Waves can exert strong forces on coastal infrastructure. They can damage bridge superstructures, wash away roadway embankments, scour abutment foundations, and destroy scour countermeasures. Waves are usually generated by wind and can propagate across the ocean for thousands of miles. Figure 8.8, taken from HEC-25, illustrates the basic parameter definitions of water waves. The wavelength (L) is the distance between wave crests; the wave height (H) is the vertical distance from the crest to the trough of an individual wave; water depth (d) is the vertical distance from the bed to the still water level (SWL), the level of water if waves were not present. The wave height is the main parameter of interest in assessing and designing for wave-related risks at a bridge. Wave heights are a function of the sustained wind speed, the fetch length (the distance of open water over which wind can blow in a specific direction to generate waves), and the depth of the water body. The *Coastal Engineering Manual* (USACE 2002) provides empirical methods for calculating wave heights. However, many design situations justify a numerical modeling approach that couples a hydrodynamic 2D model with a wave model. Typically, the result of interest from a wave height calculation or a wave model is the significant wave height, H_s, which is the average height of the one-third highest waves. The *AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms* (AASHTO 2008) suggests using the maximum wave height in determining the bridge clearance and the wave forces acting on a bridge. It describes the maximum wave height as:

$$H_{max} = 1.80H_s \tag{8.5}$$

The wave height is often depth limited because a wave breaks when the water depth is too shallow to support its height. For a given water depth at the bridge, the nominal maximum wave height is 0.8 times the depth.

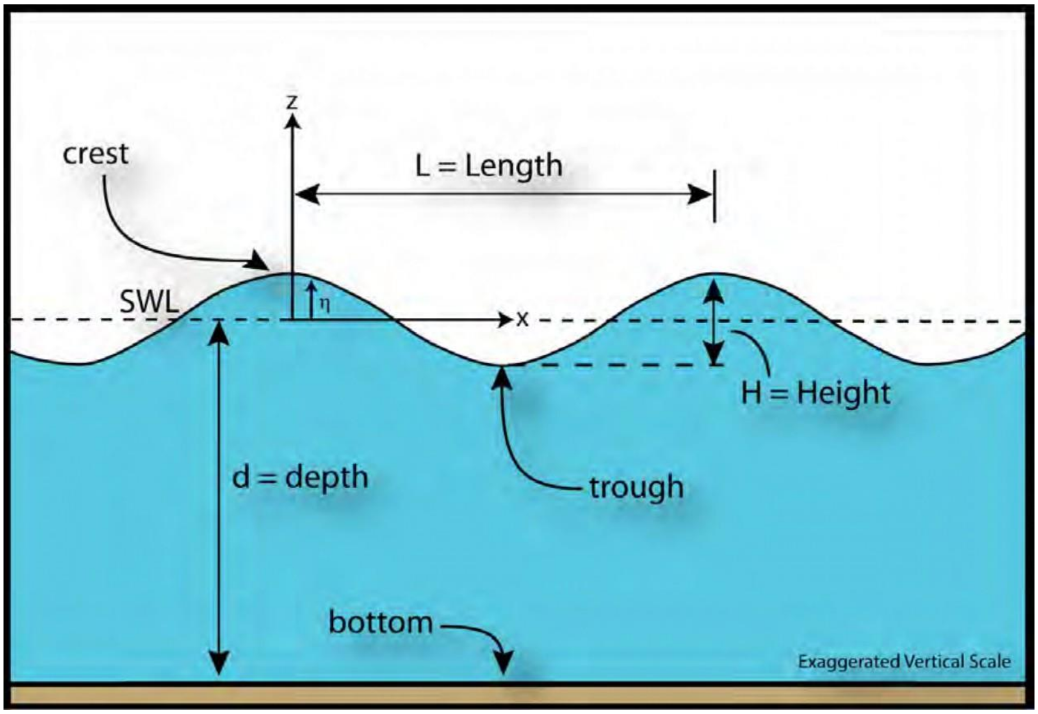

Figure 8.8. Wave parameter definitions (from HEC-25).

8.10.1 Waves Considerations in Bridge Design

Waves riding on top of storm surges can impact bridges with tremendous force. For example, HEC-25 describes how waves from the Gulf Coast hurricanes of 2004 and 2005 caused billions of dollars in damage to bridges and moved bridge deck spans that weighed over 340,000 pounds each (see Figure 8.9) (FHWA 2020). Bridge superstructures are especially vulnerable when a storm surge is high enough that large waves can reach them. Waves exert both an uplift force and horizontal force on the superstructure. Chapter 11 discusses waves as they relate to hydraulic forces on bridge elements. Engineers can find information on designing bridges to avoid or accommodate wave loading in the *AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms* (AASHTO 2008). Waves also add to the scour potential at bridge piers and abutments. Waves are a driving consideration in designing scour countermeasures to protect bridge abutments and approach roadways. HEC-25 (FHWA 2020) includes information on scour countermeasure design in the coastal environment.

Figure 8.9. U.S. Highway 90 bridge across Biloxi Bay, Mississippi, after Hurricane Katrina (from HEC-25).

8.10.2 Tsunamis

Tsunamis usually result from disturbances on the seabed, such as earthquakes or volcanic eruptions. Tsunamis can hit any U.S. coast, but 95 percent of them hit the Pacific States and territories (NTHMP 2018). Tsunamis can impart massive forces on bridge structures and cause scour at bridge piers and abutments. HEC-25 (FHWA 2020) includes extensive discussion on assessing a site's vulnerability to tsunamis and summarizes several references providing information on the forces and potential scour a tsunami can generate.

8.11 Hydraulic Effects of Hurricanes

A hurricane is a tropical cyclone that produces sustained wind speeds greater than 74 mph. The term "sustained wind" refers to wind speeds (10 m above the surface) that persist for durations of one minute. The Saffir-Simpson scale categorizes hurricanes according to wind speed, however there is little correlation between the scale and surge and waves. Two events of the same category may result in quite different storm surge still water elevations and wave conditions at different locations. Therefore, the Saffir-Simpson category of a storm is not generally useful in estimating the risk it poses to the hydraulic performance of bridges the storm may affect.

In regions vulnerable to hurricanes, they are usually the basis of the design events for bridges over tidal waterways. Hurricanes cause storm surges, partly because of the associated central pressure deficit and partly because of the action of high onshore winds. Hurricanes cause increased current velocities in tidal waterways because of the surge. High wind shear stresses on the water surface potentially boost these velocities if the wind blows in the flow direction. A bridge in a hurricane is subject to high wave attack potential because of the extreme winds and the deep water.

Hurricanes can also generate record rainfall intensities in coastal regions. Hurricane rainfall has led to the flood of record at many river streamflow gages. However, riverine flood peaks from hurricane rainfall typically occur hours after the storm surge peak. The South Carolina Department of Transportation (SCDOT) examined 29 USGS streamflow gages that had collected data during at least one of the 14 hurricanes that affected South Carolina since 1989, providing 186 total events at the gages. The gage records showed that the greatest AEP associated with any of these events in a timeframe relevant to storm surge (within a few hours of landfall) was12 percent AEP, which equates to an 8.4-year return period, and 95 percent of the events were smaller than a 50 percent AEP (2-year return period) event (SCDOT 2021). Therefore, it is unrealistic to combine design-level peak riverine flooding with design-level storm surges in a single hydraulic analysis. The peak discharge at a bridge site generated by a storm surge is often in the upstream direction (during the flood tide). Any riverine discharge would offset this peak. Therefore, it is good practice to run at least one storm surge simulation with only base riverine flows.

8.12 Hydraulic Effects of Nor'easters

Nor'easters are extratropical storms that have caused many historically significant coastal flood events on the Mid-Atlantic and New England coasts. These events rarely obtain hurricane-level wind speeds, but they can cause significant coastal surge effects and wave damage. Nor'easters do not typically produce the high surge-driven currents that hurricanes can cause because the rate of ocean level rise is typically slower than hurricanes. However, the elevated water surface and the large waves caused by the sustained high winds can pose a threat to bridge structures.

8.13 Sea Level Rise

Sea levels along the U.S. coast are slowly rising, with projections for the rate of rise to increase in this century (FHWA 2020). HEC-25 provides an extensive discussion on the historical trends and projected increases in the rate of rise. Important terminology for discussion of the topic includes relative sea level rise (RSLR) and global mean sea level rise (GMSLR). RSLR is the combined effect of the changes in ocean water elevation and ground elevation at a given location. Ground elevation can change due to subsidence, post-glacial rebound, and tectonics. For instance, coastal Louisiana is experiencing subsidence due to oil and gas extraction. GMSLR is the average sea level rise across all the ocean basins. RSLR is the sea level rise that coastal highway experience and will continue to experience.

HEC-25 references NCHRP Project 15-61 (NCHRP 2019), which suggested that the planning and design of coastal transportation infrastructure consider future RSLR projections. The NCHRP report presents minimum RSLR projections for design consideration throughout the remainder of this century. It presents higher projections for consideration on projects whose performance is sensitive to design sea levels and/or when designing highly valued infrastructure with a long design life. The NCHRP report's suggestions are not legally binding.

The effects of higher sea levels on bridge hydraulics will include:

- The need for higher superstructure elevations to provide clearance above higher still water levels and greater wave heights.
- Greater wave heights leading to increased wave impact forces and wave-induced scour.

- Greater current velocities from tides and storm surges due to deeper flow and increased tidal prism volume
- Increased potential for wave overtopping of roads and protective revetments.

8.14 Analysis Approaches for the Hydraulic Design of Bridges over Tidal Waterways

HEC-25's Chapter 14 provides an extensive discussion of analysis methods for assessing the vulnerability of transportation facilities to extreme coastal events (FHWA 2020).

8.14.1 Level of Effort 1: Use of Existing Data and Resources

Many data resources exist that can provide helpful information about a project site's exposure to risk from severe coastal events. Level 1 studies typically include a field visit and an examination of several online data resources. Commonly used data sources at this level of effort include:

- FEMA Flood Insurance Rate Maps and Flood Insurance Studies.
- Any location-specific storm surge information developed by the USACE, NOAA, USGS, or other agencies.
- Streamflow gages at or near the site.
- The USACE Coastal Hazards System.
- The USACE Wave Information Studies (an online database of hindcast, nearshore wave conditions).
- Bridge inspection reports for existing bridges at or near the crossing site, including underwater inspections.
- Aerial imagery, both recent and historic.
- Historical data and projections of sea level rise, from NCHRP 15-61 (2019), for example.

In the Level 1 study, the engineer can assess whether a crossing site is tidally controlled or tidally influenced, estimate the approximate design high water level, and assess the potential for wave attack. Aerial imagery enables the engineer to examine the lateral stability of the waterway being crossed and determine the maximum fetch length, keeping in mind that strong hurricane winds can blow from any direction. Considerable uncertainty remains about the exposure of the bridge to various coastal hazards after the Level 1 study is completed.

8.14.2 Level of Effort 2: Modeling of Storm Surge and Waves

Studies at this level provide detailed information about exposure under extreme events. Level 2 requires much more time, effort, and cost than Level 1. Level 2 activities include, but are not limited to:

- Selection of extreme event and sea level rise scenarios appropriate for the region.
- Development of suitable hydrodynamic models.
- Validation of the hydrodynamic models.
- Mapping of hazards such as inundation, waves, and wave runup) as assessments of exposure under each scenario.

The extreme event scenario(s) selected may be associated with a design event, having an associated AEP, or may be a recreation of one or more historical storms or a modified version of a historical event. An appropriate Level 2 study decreases the uncertainty to a degree that may be acceptable for most bridge design projects.

8.14.3 Level of Effort 3: Modeling in a Probabilistic Framework

Studies at this level characterize exposure in terms of probability and risk. The general methodology is similar to Level 2. However, a Level 3 study includes a much larger number of scenario simulations than Level 2, possibly on the order of one hundred simulations. Level 3 studies are appropriate for major regional evaluations and specific projects involving a high level of sensitivity and the potential for extreme risk of economic loss or loss of life. HEC-25 presents a case study of the Boston Central Artery Project as an example of a project that justified a Level 3 study. Ideally, the Level 3 study yields a robust simulation matrix to support the assignment of probabilities to potential tides and waves and storm conditions.

8.14.4 Modeling

A Primer on Modeling in the Coastal Environment (FHWA 2017) provides detailed information to aid in selecting and developing models for transportation projects in tidal waterways. It refers to the three-level approach described above and explains the most suitable modeling approaches for Level 2 and Level 3 studies.

With few exceptions, the complexity of bridge designs over tidal waterways justifies 2D, unsteady-flow modeling. The *Primer* makes a distinction between hydraulic and hydrodynamic models. The document includes HEC-RAS 1D and SRH-2D as examples of hydraulic models. Both hydraulic and hydrodynamic models:

- Solve equations of fluid motion in space and time.
- Provide some method for the treatment of turbulence.
- Simulate wetting and drying.
- Have some limitations or restrictions on numerical stability.

Hydrodynamic models differ from hydraulic models in that they include:

- Influence of the Coriolis force (the acceleration on a fluid imparted by Earth's rotation).
- Wind shear stress on the water surface.
- Tides.
- Wave generation and propagation.

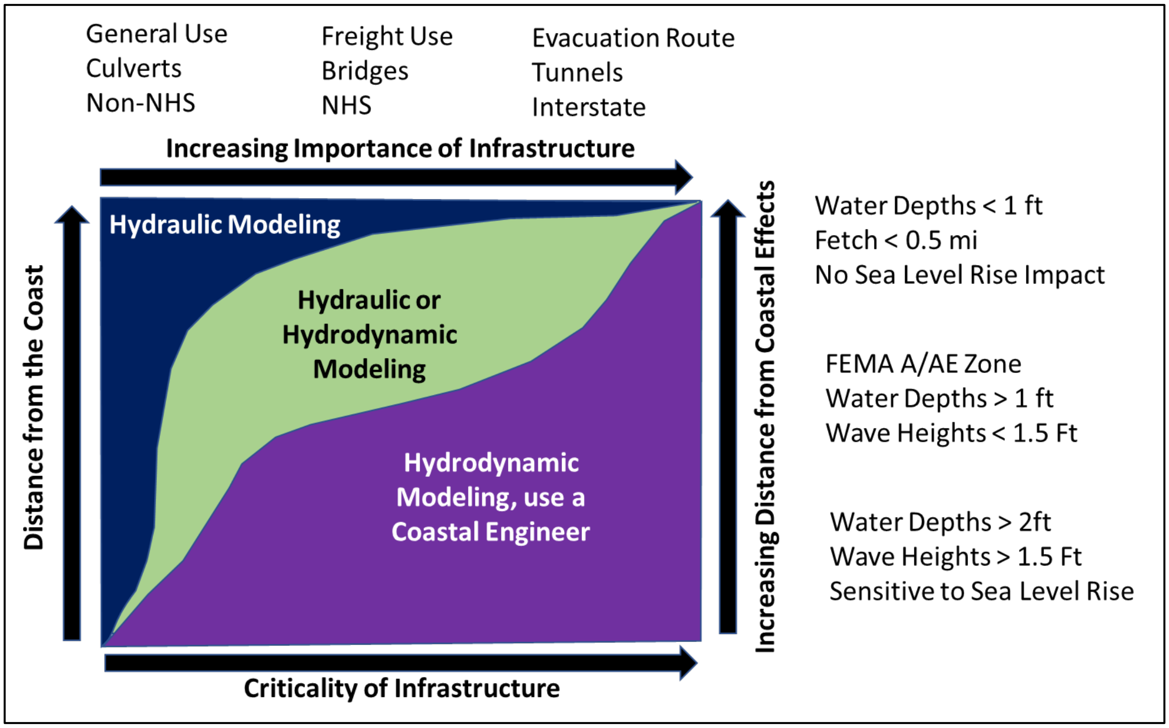

Figure 8.10. Qualitative guide for model selection in the coastal environment, after *A Primer on Modeling in the Coastal Environment* (FHWA 2017).

CHAPTER 9
BRIDGE SCOUR CONSIDERATIONS & SCOUR COUNTERMEASURES

9.1 Introduction

Scour is the engineering term for flowing water's erosion of soil, alluvium or other materials surrounding bridge foundations (piers and abutments). Safe bridge design accounts for scour and stream instability conditions that might occur over the life of the bridge. This chapter primarily describes scour considerations and approaches for bridges within the riverine context. The HEC-18 and HEC-20 documents (FHWA 2012b, 2012c) are the primary FHWA references on evaluating and designing for scour and stream instability at bridge crossings. HEC-18 presents FHWA guidance for scour prediction and scour design methods. HEC-20 provides guidance for evaluating stream stability problems at highway crossings.

Scour is often greatest during extreme floods when flood flows that are spread across the channel and overbanks converge into a narrower bridge opening. Event-driven scour increases with increasing flow velocity and depth. Scour is additive to the long-term stream instability components of channel shifting and degradation. Scour and channel instability are the most common causes of bridge failures.

HEC-18 defines total scour at a highway crossing as the sum of long-term degradation of the channel bed (neglecting any aggradation potential for foundation design), contraction scour at the bridge, and local scour at the piers or abutments, considering lateral channel instability potential over the service life of the bridge (FHWA 2012c). This chapter describes these components and related bridge design considerations in more detail.

The 9th edition of the American Association of State Highway and Transportation Officials (AASHTO) *Load and Resistance Factor Design (LRFD) Bridge Design Specifications* includes the following provisions for scour and stream stability design factors in the LRFD context (AASHTO 2020).

- The *LRFD Bridge Design Specifications* states that the *"The design flood storm surge, tide, or mixed population flood shall be the more severe of the 100-year events or an overtopping flood of lesser recurrence interval,"* per Section 2.6.4.4.2. The 2021 FHWA document *Scour Conditions within AASHTO LRFD Design Specifications* states that the *"Scour Design Flood [defined in HEC-18] is equivalent to AASHTO LRFD definition of Design Flood for Bridge Scour."* (FHWA 2021). Designers should examine floods up to and including the appropriate scour design flood that generates the worst-case scour depths (FHWA 2021).
- For the Scour Design Flood, *"the streambed material in the scour prism above the total scour line shall be assumed to have been removed for (foundation) design conditions"* per Section 2.6.4.4.2. This loss of streambed materials in the total scour prism applies for bridge abutment and pier foundation design at both the Strength and Service Limit States. See Article 3.7.5, Section 10.5.2.1, and Section 10.5.3.1 (AASHTO 2020) for more information on these limit states.
- For the Scour Check Flood, Section 2.6.4.4.2 states, *"The stability of bridge foundation[s] shall be investigated for scour conditions resulting from a designated flood storm surge, tied or mixed population flood not to exceed the 500-year event or from an overtopping flood of lesser recurrence interval…. The Extreme Event Limit State shall apply."* The 2021 FHWA document *Scour Conditions within AASHTO LRFD Design*

Specifications states that the *"Scour Check Flood [defined in HEC-18] is equivalent to AASHTO LRFD definition of Check Flood for Bridge Scour"* (FHWA 2021).

- *"Spread Footings on soil or erodible rock shall be located so that the bottom of the footing is below scour depths determined for the check flood for scour,"* per Section 2.6.4.4.2.

23 CFR Part 625 – Design Standards requires that *"a. National Highway System (NHS) projects must follow certain hydrologic, hydraulic, and scour related sections of the AASHTO LRFD Bridge Design Specifications 8th edition*[1] *[§625.3(a)(1) and §625.4(b)(3)]. B. Non-NHS projects must follow State DOT drainage and/or bridge standard(s) and specifications [§625.3(a)(1)]."* (FHWA 2021).

The 9th edition of the AASHTO *LRFD Bridge Design Specifications* (AASHTO 2020) also includes design provisions in Section 2.6 related to scour and stream instability. AASHTO's specifications and commentary in this section include the following statements (all references AASHTO 2020; section numbers as noted):

- *"The added cost of making a bridge less vulnerable to damage from scour is small in comparison to the total cost of a bridge failure."* (2.6.4.4.2)
- Consider *"stream instability, backwater, flow distribution, stream velocities, scour potential, flood hazards, tidal dynamics (where appropriate), and consistency with established criteria for the National Flood Insurance Program"* during bridge design alternative evaluation (2.6.1).
- *"Evaluate the stability of the waterway and assess the impact of construction on the waterway."* (2.6.4.2).
- Consider *"the effect of natural geomorphic stream pattern changes on the proposed structure."* (2.6.4.2)
- Consider *"whether the stream reach is degrading, aggrading, or in equilibrium."* (2.6.4.2)
- For unstable streams or flow conditions, *"assess the probable future changes to the plan form and profile of the stream,"* and *"determine countermeasures to be incorporated in the design, or at a future time, for the safety of the bridge and approach roadways."* (2.6.4.2)
- *"Locate abutments back from the channel banks where significant problems with ice/debris buildup, scour, or channel stability are anticipated…"* (2.6.4.4.1)
- *"Design piers on floodplains (overbanks) as river piers. Locate their foundations at the appropriate depth if there is a likelihood that the stream channel will shift during the life of the structure or (if) channel cutoffs are likely to occur."* (2.6.4.4.1)

Scour and stream instability are significant aspects of safe bridge design and are a complex combination of hydrologic, hydraulic, fluvial-geomorphic, erosion, scour, sediment transport, geotechnical, and structural considerations. The following sections describe scour and stream instability processes, how to obtain data from hydraulic models for computing scour, and hydraulic modeling of scour countermeasures.

[1] Per FHWA Memorandum dated April 11, 2022, the FHWA approved the use of AASHTO LRFD Bridge Design Specification, 9th Edition, 2020, pursuant to 23 CFR 625.3(f)(2) (https://www.fhwa.dot.gov/bridge/structures/policy. cfm).

9.2 Scour Concepts for Bridge Design

Floods convey water in the river channel and the overbanks adjacent to the channel. Figure 9.1 illustrates a bridge crossing and the flow characteristics for a flood condition and includes streamlines and velocity contours. The streamlines in the overbanks divide the flow into 5 percent increments. The closely spaced streamlines in the channel divide the flow into 10 percent increments of flow. The road embankments constrict the flow into the bridge opening. Upstream of the bridge constriction, where flow fully occupies the floodplain, approximately 65 percent of the flow is in the channel and 35 percent is in the overbanks. In the bridge opening, approximately 90 percent of the flow is in the channel and 10 percent is in the overbank areas between the channel banks and abutments (setback area). The velocity is less than 6 ft/s in the upstream channel and 1 ft/s in the upstream overbanks. This compares with velocities in bridge opening as high as 9 ft/s in the channel and 4 ft/s in the setback areas. The higher velocity in the bridge opening is more erosive than the upstream flow velocity. Beside the increased velocity, bridge piers and abutments locally obstruct flow and increase scour at these locations. The sections below describe the types of scour that can affect bridges.

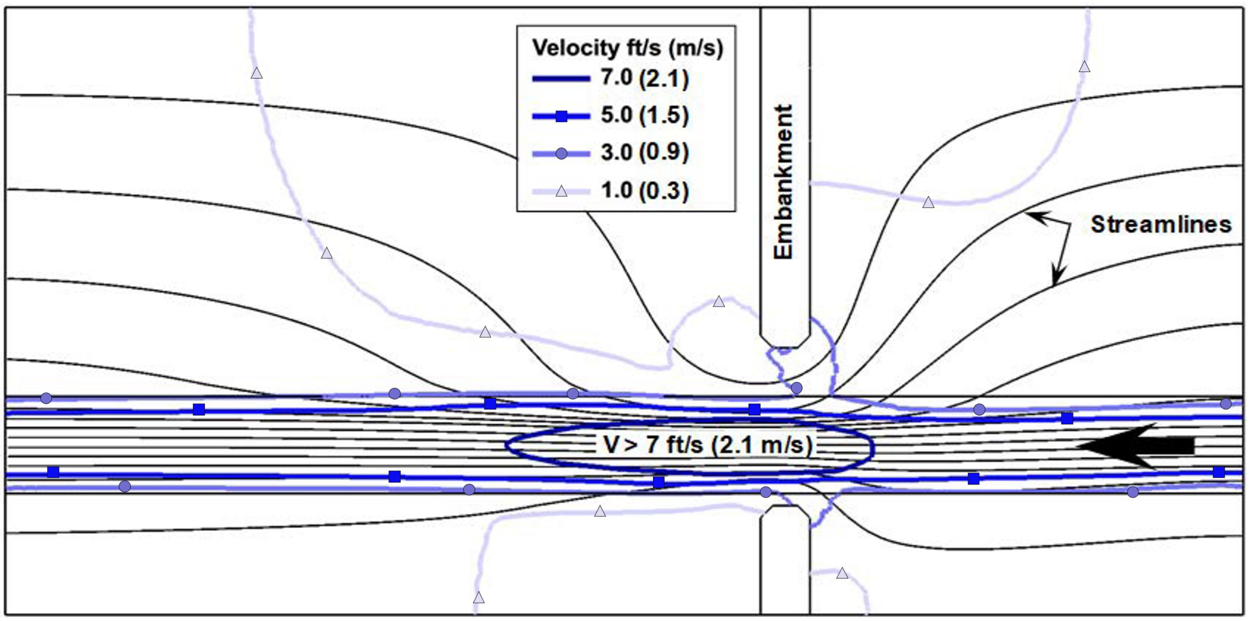

Figure 9.1. Velocity and streamlines at a bridge constriction.

9.2.1 Contraction Scour

Contraction scour is caused by a sediment imbalance process that occurs during floods when the sediment supply from upstream is less than the sediment transport capacity in the bridge opening. As illustrated in Figure 9.1, flow velocities increase throughout the contracted section. Since shear stress is generally proportional to velocity squared, shears stresses increase even more dramatically. There are two sediment supply conditions for contraction scour: clear-water and live-bed. Clear-water contraction scour occurs when the upstream flow velocity is insufficient to transport bed material. The HEC-18 document (FHWA 2012b) includes equations for determining the critical velocity when bed material movement is initiated, which depends on flow depth and particle size. Clear-water conditions occur for fine sediment sizes (sands and fine gravel) only when flow velocity is small and for coarse sediment sizes (coarse gravel and cobbles) even for relatively high velocity. Clear-water conditions typically occur in overbanks when the soils are protected by vegetation. Live-bed conditions occur when there is sufficient flow velocity to transport bed material upstream of the bridge. Very fine sediment (clay and silt) is not often found in channel beds in significant amounts and does not generally play a role in either clear-water or live-bed contraction scour. The water may be turbid due to suspended transport of silt and clay, but the scour condition may still be clear-water from the standpoint of bed material transport.

For clear-water contraction scour, the flow velocity in the bridge opening is sufficient to move bed material even though the upstream flow velocity is too low for bed material movement. For live-bed contraction scour, the higher flow velocity in the bridge opening has a greater capacity for transporting sediment than the lower upstream flow velocity. In either case, there is an imbalance between sediment supply and sediment transport capacity, and contraction scour occurs. As contraction scour progresses, the channel bed erodes and lowers, thereby increasing the flow depth and decreasing the flow velocity until the bed material transport capacity equals the supply from upstream. This equilibrium condition is considered the ultimate scour for the flow conditions under consideration. Contraction scour takes time to develop and, depending on the duration of the flood, the scour may stop short of the ultimate scour depth.

Accurate contraction scour calculations depend on accurate estimates of flow distribution upstream and within the bridge opening. Two-dimensional models provide more reliable and more informative hydraulic results than 1D models, and therefore 2D hydraulic modeling can be expected to lead to more accurate scour estimates. For contraction scour calculations, the flow is divided into channel, left overbank, and right overbank in the unconstricted flow upstream of the bridge, and divided into channel, left setback, and right setback areas under the bridge. These subarea discharges control the contraction scour process.

Figure 9.2 includes a plan and profile sketch to illustrate the flow conditions for contraction scour. The designer selects the approach cross-section location (Cross Section 1) at a point upstream of the flow contraction region caused by the bridge and roadway embankments. This location should also be placed so that the hydraulic conditions are generally representative of upstream sediment transport conditions. Velocity field visualization using 2D model results can directly assist in locating Cross Section 1 (FHWA 2019) for a given simulation. When using a 1D model, best practice guidance can be applied using HEC-RAS Hydraulic Reference Manual Appendix B methods to locate Cross Section 1. Designers commonly locate Cross Section 2 at the point of maximum contraction or constriction in the bridge opening. The automated scour variable extraction tools provided with SMS assume this cross-section is located at the centerline of the bridge alignment.

Live-Bed Contraction Scour: Live-bed scour almost always occurs in river channels during design-level flood events because an alluvial channel can fully mobilize and transport its bed materials. Exceptions to this are boulder-bed and bedrock channels that are not alluvial. Channels that have significant levels of diversion and/or flood control are other examples of potentially non-alluvial channels. Non-alluvial channels are more likely to experience clear-water scour for design scour and check scour events. The total sediment transport in the approach main channel depends on the channel flow depth, velocity, discharge, width, and sediment size. Floodplain areas under the bridge (setback areas) can be live-bed when velocity, shear stress, floodplain sediments, and vegetation conditions produce mobile bed sediment transport in the upstream floodplain, although this is not common.

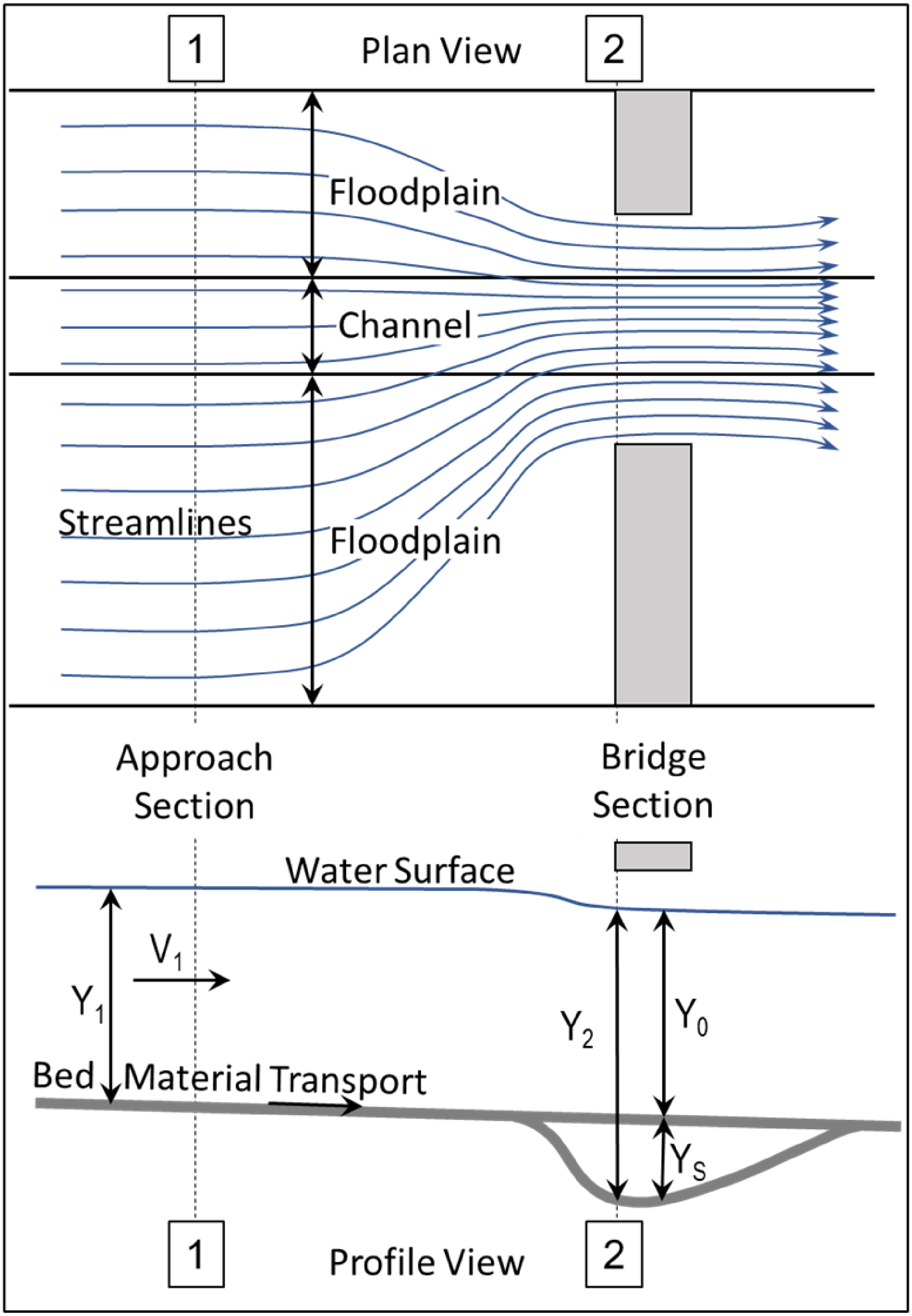

Figure 9.2. Live-bed contraction scour variables.

For live-bed contraction scour, the flow velocity in the river channel is high enough to transport bed material at the approach section (Cross Section 1). At the bridge section (Cross Section 2), overbank flow has entered the channel, so the channel discharge, velocity, and sediment transport capacity are greater than in the channel at the approach section. Rigid bed hydraulic models, whether 1D or 2D, incorporate pre-scour bed elevations so the flow depths and velocities do not incorporate scour. From the pre-scour hydraulic conditions, the contraction scour calculation predicts the flow depth including scour.

Assuming the flood conditions persist long enough, live-bed contraction scour progresses until the sediment transport capacity in the bridge channel (contracted section) matches the sediment transport supply from the upstream channel. HEC-18 (FHWA 2012b) presents the live-bed contraction scour equation. The equation yields the total flow depth including scour in the main channel under the bridge (Y_2), and the scour depth is calculated as the difference between this depth and the pre-scour flow depth. Because the calculation approach assumes that the bed material size is consistent along the channel reach, the bed material size is only used to determine whether or not live-bed conditions exist and does not actually appear in the live-bed contraction scour equation.

The following variable definitions are specific to live-bed scour in the main river channel. Live-bed scour is the most common channel scour during a flood event. The variables pertain to the area of active sediment transport. Therefore, the widths, depths, and discharges are most appropriately determined for the main channel bottom width. However, top widths can also be used as long as the variables are determined consistently at the approach and within the bridge opening. When the bridge includes setback areas the values are determined for the main channel portion of the bridge opening. If mobile bed conditions exist in upstream floodplain areas, the live-bed contraction scour equation can be applied using upstream variables that are determined for the areas of active sediment at the approach section. The HEC-18 live-bed scour equation includes the following variables:

- Y_1, the average flow depth (hydraulic depth) in the main channel upstream of the bridge (or area of active sediment transport).
- Q_1, the discharge in the main channel upstream of the bridge (or area of active sediment transport).
- W_1, the width of the main channel upstream of the bridge (or area of active sediment transport).
- Y_0, the average flow depth (hydraulic depth) in the main channel within the bridge opening (or contracted section) before contraction scour.
- Q_2, the discharge in the main channel within the bridge opening (or contracted area).
- W_2, the width of the main channel within the bridge opening (or contracted area).

An important distinction for live-bed scour calculations is that the variables pertain to the main channel, which is transporting channel bed material, and not the overbanks, which typically do not transport bed material load. If an overbank area is also transporting bed material load (i.e., it is not vegetated and the predicted overbank velocities exceed the critical velocity for the overbank materials), the live-bed contraction scour calculation would involve the upstream overbanks (left, right or both) and the corresponding setback area rather than combining the overbanks and channel into a single calculation.

Another distinction is that the channel width used at the approach section in the equation is technically the width of active bed material transport. A 2D model can directly visualize this region based on a calculation of the critical velocity. In practice, most designers simply establish the bank locations including the total unvegetated channel width. It is essential to use a consistent definition of width and corresponding discharges and depths when applying the equation. It is often much easier to isolate the sediment-transporting widths and determine the appropriate input values in 2D models than in 1D models because in 1D models the channel definition and output extends to the tops of the bank slopes on both sides.

Larger amounts of contraction scour occur for greater differences between channel discharge at the approach and bridge sections. Scour also increases for narrower channel widths at the bridge section. The worst-case bridge opening condition for live-bed contraction scour is expected when the bridge abutments and road embankments encroach into the channel and the entire flow is conveyed in the constricted channel at the bridge opening. Live-bed contraction scour decreases as the abutments are set back farther from the channel banks and when fewer or narrower piers are located in the channel.

Clear-Water Contraction Scour: Clear-water contraction scour typically occurs in the setback areas under a bridge and in relief bridges located in the floodplains. The unconstricted floodplain flow upstream of the bridge typically has low velocities in the overbanks and would not be expected to mobilize the granular overbank sediments. Overbank soils often consist of cohesive materials covered with vegetation. Very fine particles (silts and clays) may move downstream in suspension, but there is usually little potential for bed material transport or live-bed scour in the overbanks. During floods, the flow velocity in the setback area under the bridge is, however, often high enough to cause erosion. Clear-water scour is, therefore, an erosion process based on excessive flow velocity and shear stress under the bridge rather than sediment transport process of matching upstream sediment supply.

Figure 9.3 includes a plan and profile sketch of clear-water contraction scour conditions. The important variable at the approach section (Cross Section 1) is the average velocity (V_1), but strictly to compare it to the critical velocity for bed material transport. This comparison is made if there is any uncertainty about whether the upstream flow is transporting bed material (e.g., the overbank is largely or completely unvegetated). The channel could have clear-water contraction scour, but it most often only occurs in the setback areas. As shown in

Figure 9.3, if there is a relief bridge through the embankment on the overbank, this opening also typically experiences clear-water contraction scour.

The clear-water contraction scour equation considers only the hydraulic conditions in a particular subarea under the bridge, not upstream conditions. The calculations assume that clear-water contraction scour occurs until erosion of the ground surface, which increases depth and decreases flow velocity, produces a non-eroding velocity. The non-eroding velocity is a function of grain size for non-cohesive soils and is a function of critical shear stress for cohesive soils. HEC-18 (FHWA 2012b) includes equations for clear-water contraction scour. As with the live-bed contraction scour equation, the clear-water contraction scour equation yields a total depth including scour (Y_2) under the bridge, and the predicted scour is the difference between this depth and the pre-scour depth.

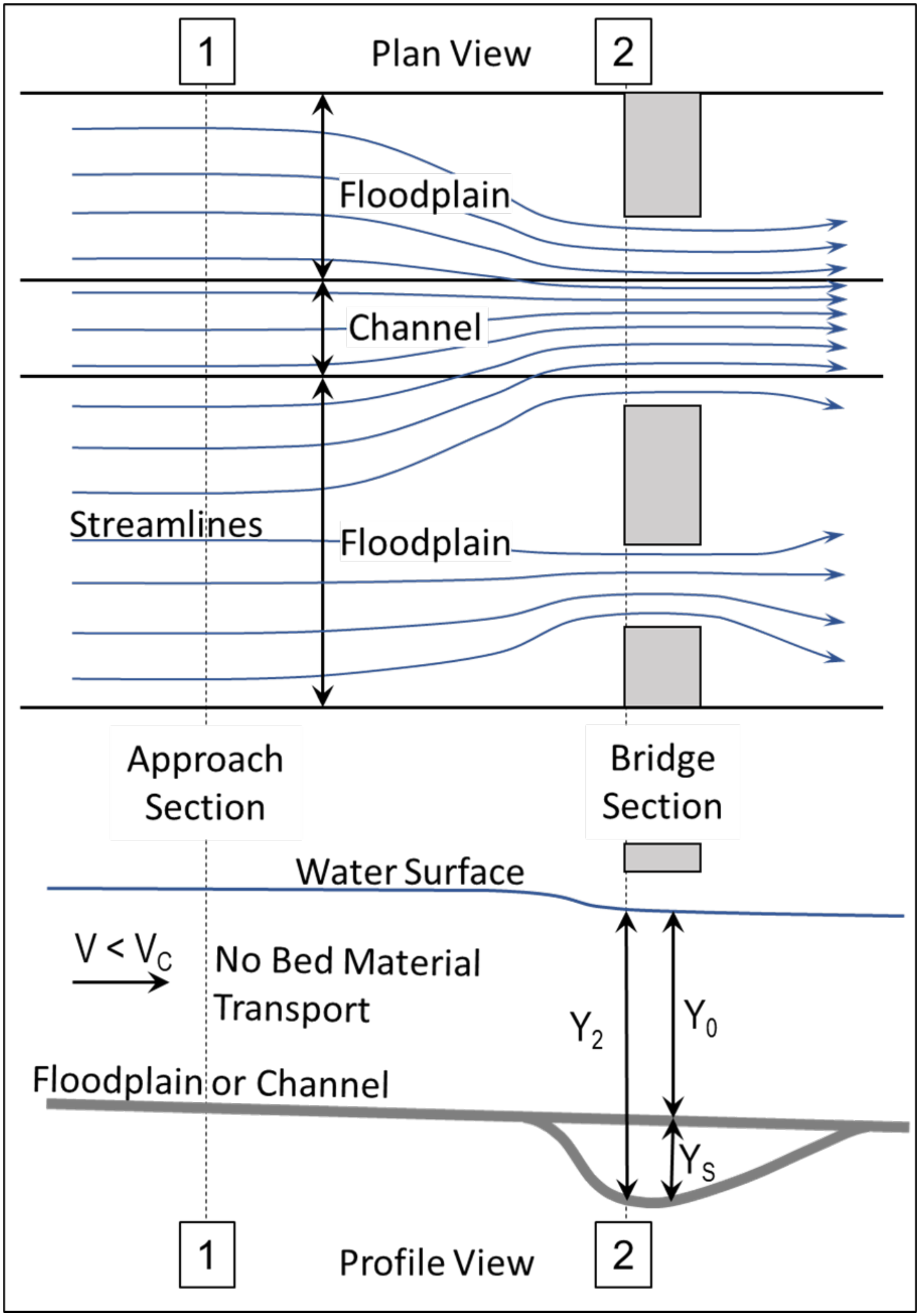

Figure 9.3. Clear-water contraction scour variables.

The clear-water contraction scour equation incorporates the following variables:

- Q, discharge in the subarea within the bridge opening.
- W, discharge in the subarea within the bridge opening.
- Y_0, flow depth in the subarea within the bridge opening before contraction scour.
- D_{50}, median grain size of the soil or bed material in the subarea within the bridge opening or contracted section.
- Critical shear stress or critical stream power of cohesive or erosion-resistant layers in the subarea within the bridge opening or contracted area.

The clear-water contraction scour is **individually** calculated for each subarea. Subareas include setback areas between the channel and abutments, relief bridge openings, and, occasionally, channel areas.

Vertical Contraction Scour (Pressure Flow Scour): Pressure flow scour is another type of contraction scour. It is created by a vertical constriction and can occur whether or not a horizontal constriction is present. Pressure flow scour may be either live-bed or clear-water depending on the upstream flow and sediment characteristics. Prediction of pressure scour for flow conditions submerging the lowest structural member may be needed for safe bridge design and for evaluation of scour at existing bridges. HEC-18 (FHWA 2012b) presents a prediction procedure for pressure scour. For the contraction scour component of total scour when pressure flow conditions exist use the greater of the vertical contraction scour, which includes horizontal contraction scour, or long-term degradation plus the horizontal contraction scour excluding the vertical component (FHWA 2018).

Figure 9.4 illustrates the flow characteristics at a fully submerged bridge deck. See HEC-18 for detailed descriptions of the variables and equations for calculating pressure flow scour. Pressure flow conditions can significantly increase total scour at a bridge. The contraction scour depth (Y_2 from live-bed or clear-water contraction scour calculations) plus the thickness of the separation zone (dimension t) in Figure 9.4 is subtracted from the bottom of the bridge deck to determine the scour elevation. The scour calculation excludes the weir flow.

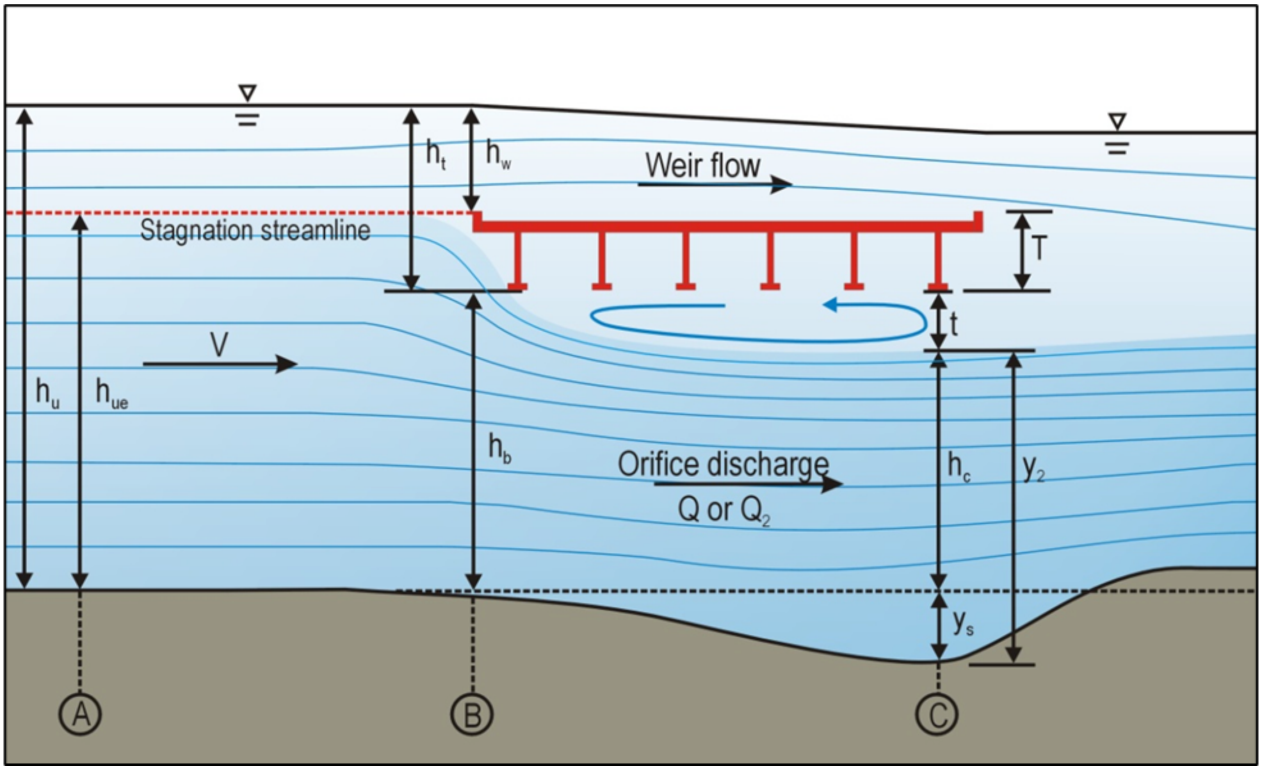

Figure 9.4. Vertical contraction scour.

9.2.2 Local Scour

Local scour occurs where the flow field is disrupted by an obstruction. The name of this component is "local" scour because it occurs in the vicinity of the obstruction, not across the entire channel or bridge section. The obstruction redirects and locally accelerates the flow, vortices and flow separation regions form, and there is increased turbulence. The two most common types of local scour at bridges are pier scour and abutment scour. Ice and debris can also impact local scour.

Pier Scour: Figure 9.5 illustrates pier scour. The velocity upstream of the pier accelerates around the pier and flow turns sharply downward along the front face of the pier. A "horseshoe" vortex forms where the downward flow reaches the bed, and the size of the vortex increases as the scour hole enlarges. The flow around the pier sheds vortices on the sides of the pier. Sediment deposition often occurs in the wake area downstream of the pier.

Pier scour can occur in clear-water or live-bed conditions. There are many factors that influence the magnitude of pier scour. The main hydraulic factors are the velocity, depth, and the flow's angle of attack to the pier alignment. The hydraulic conditions driving pier scour are a short distance upstream of the pier, outside the influence of the pier itself, but within the effects of the bridge constriction. The pier shape (including circular, square, sharp, rounded-, or rectangular-nosed), pier width, and pier length also contribute significantly to the amount of pier scour. Pier scour calculations also take into account complex pier geometries that include pile groups, pile caps, and footings. Some pier scour equations include the sediment size, density, and gradation. Although pier scour may appear to be a relatively simple process, the calculations are often cumbersome for all but the simplest cases.

HEC-18 (FHWA 2012b) presents several pier scour equations for various conditions. In summary, pier scour calculations can incorporate the following variables:

- Flow velocity directed at the pier, but outside the local hydraulic effects of the pier.
- Flow depth at the same location.
- Angle of attack of the velocity relative to the pier alignment.
- Pier width, length, shape.
- Complex pier geometries including footings, pile caps, and pile groups.
- Bed condition (i.e., clear-water, live-bed, presence of bed forms).
- Bed material size, density, and gradation.
- Bed layers critical shear stress or critical unit stream power.

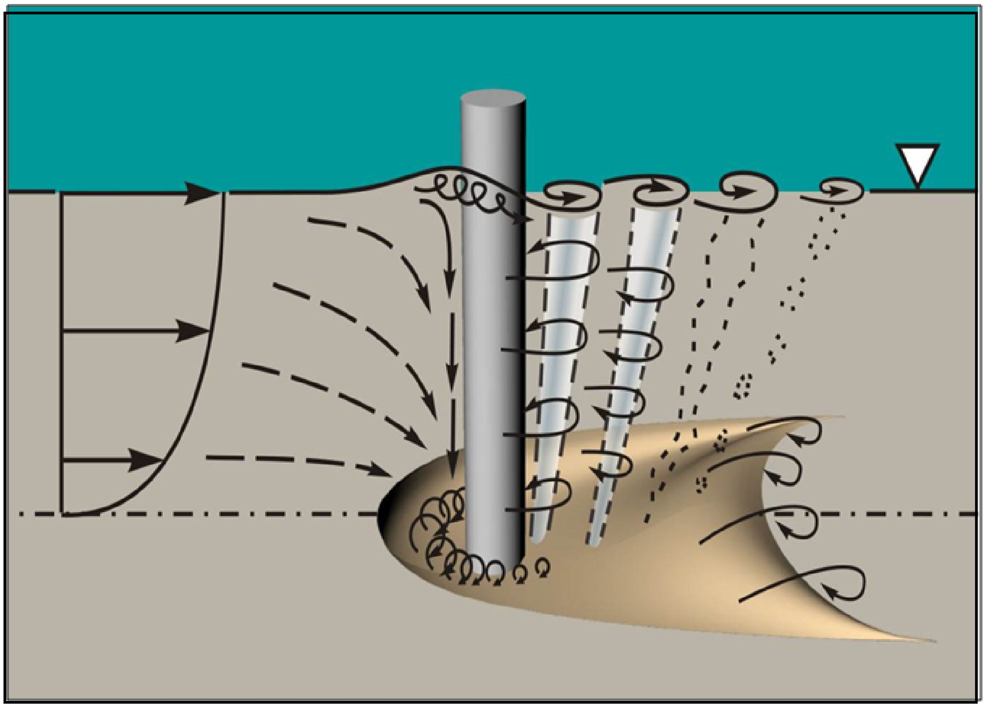

Figure 9.5. Flow features at a cylindrical pier (NCHRP 2011a).

Abutment Scour: Scour occurs at abutments when the roadway embankment and abutment obstruct the flow during floods. Abutment scour is a type of local scour with a close connection to contraction scour because the embankment is the primary cause of flow constriction.

NCHRP (2011b) conducted an evaluation of abutment scour processes and prediction methods. The conclusions and recommendations that pertain to abutment scour evaluations and safe bridge design include:

- Contraction scour should be viewed as the reference scour depth for calculating abutment scour. Abutment scour should be taken as the product of the contraction scour caused by flow acceleration through the constricted opening multiplied by a factor accounting for large-scale turbulence. This approach would replace the current approach of adding contraction scour to a separately computed abutment scour.
- Abutments should be designed to have a minimum setback distance from the channel bank of the main channel with riprap protection of the embankment and a riprap apron to protect against scour. The setback distance should accommodate the apron width recommended in HEC-23 (FHWA 2009a).

- Two-dimensional models should be used on all but the simplest bridge crossings as a matter of course.

Abutment foundation designs also consider total scour, including potential impacts from long-term degradation, lateral migration, contraction scour, and local scour. Some State DOTs may allow designers to neglect the local scour component of abutment scour for design. In this case, the abutment foundation designer considers potential impacts from long-term degradation, lateral migration, and contraction scour. Considering these impacts, they design a scour countermeasure for check scour conditions using the methods presented in HEC-23 as modified by FHWA *Hydraulic Considerations for Shallow Foundations* (FHWA 2018).

NCHRP (2010b) developed abutment scour equations that account for a range of abutment types, abutment locations, flow conditions and sediment transport conditions. HEC-18 (FHWA 2012b) incorporates these relationships and *Hydraulic Considerations for Shallow Foundations* (FHWA 2018) adopts their application. Figure 9.6 illustrates abutment scour conditions. For designs placing the abutments well back from the channel, abutment scour occurs entirely on the overbank. HEC-18 refers to abutment scour under these conditions as "Scour Condition B." Abutment scour can also occur entirely in the channel or in the overbank and channel when the abutments are close to the channel. HEC-18 refers to abutment scour under these conditions as "Scour Condition A." In the latter case the channel sediment and overbank soil characteristics, including grain size and cohesion, factor into the proportion of scour that occurs in the overbank versus channel. The NCHRP equations calculate the contraction scour (either Scour Condition A or Scour Condition B, as appropriate) and apply an amplification factor depending on the abutment shape and flow conditions.

Abutment scour can result in geotechnical failures of the embankment or channel bank materials. Once the geotechnical failure depth is reached, scour does not increase in depth but progresses laterally, potentially creating a free-standing abutment foundation that would act more as a pier from the standpoint of scour. Although an abutment set well back from the channel is likely to have Scour Condition B in a design condition, if the channel could migrate into the abutment area over the bridge service life, then that potential future condition (Scour Condition A) should also be considered during design (FHWA, 2018).

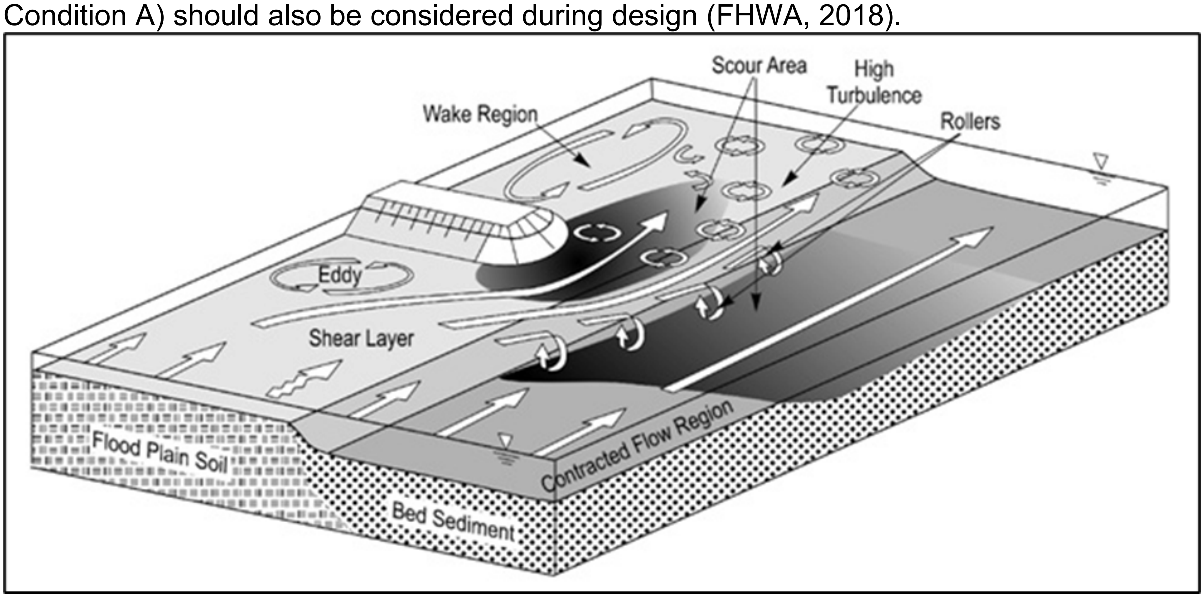

Figure 9.6. Illustration of abutment scour conditions (NCHRP 2011b).

9.2.3 Effects of Debris on Scour

Debris accumulation is a common problem at bridges, especially during floods. Debris loading and impact forces can damage piers, decks, and girders. Debris accumulation can reduce the waterway opening, thereby increasing upstream flooding. Debris accumulation increases all types of scour potential. For example, contraction scour potential increases when debris narrows the flow width by blocking a portion of the bridge opening. Similarly, pressure scour potential increases when debris accumulates on the bridge deck and girders, causing greater vertical constriction.

Increased pier scour is the most common type of debris scour problem. Figure 9.7 shows an example of pier scour increased by debris. Debris clusters are highly variable from one bridge location to another and at the same bridge from one flood to the next (NCHRP 2010c). HEC-20 (FHWA 2012c) provides information on identifying upstream debris production potential and the potential for debris collection depending on the pier location in the channel. HEC-18 (FHWA 2012b) provides scour relationships for debris clusters on piers. Figure 9.8 illustrates that debris scour at a pier depends on the flow impacting the pier and the flow plunging below the debris blockage. The plunging flow creates a scour hole just downstream of the leading edge of the debris cluster and the pier obstruction creates a local scour hole. The maximum scour depth at a pier is expected to occur when the debris cluster size and the flow depth cause these two scour holes to coincide.

One-dimensional hydraulic models such as HEC-RAS can hydraulically simulate debris blockage at individual piers by reducing flow area in the bridge opening. Chapter 7 describes multiple approaches to representing piers in a 2D model, all of which are adjustable to allow for the inclusion of debris collection. For debris collected at the bridge deck, the low chord of the bridge can be adjusted in both 1D and 2D models.

Figure 9.7. Looking down at a debris and scour hole at the upstream end of a pier.

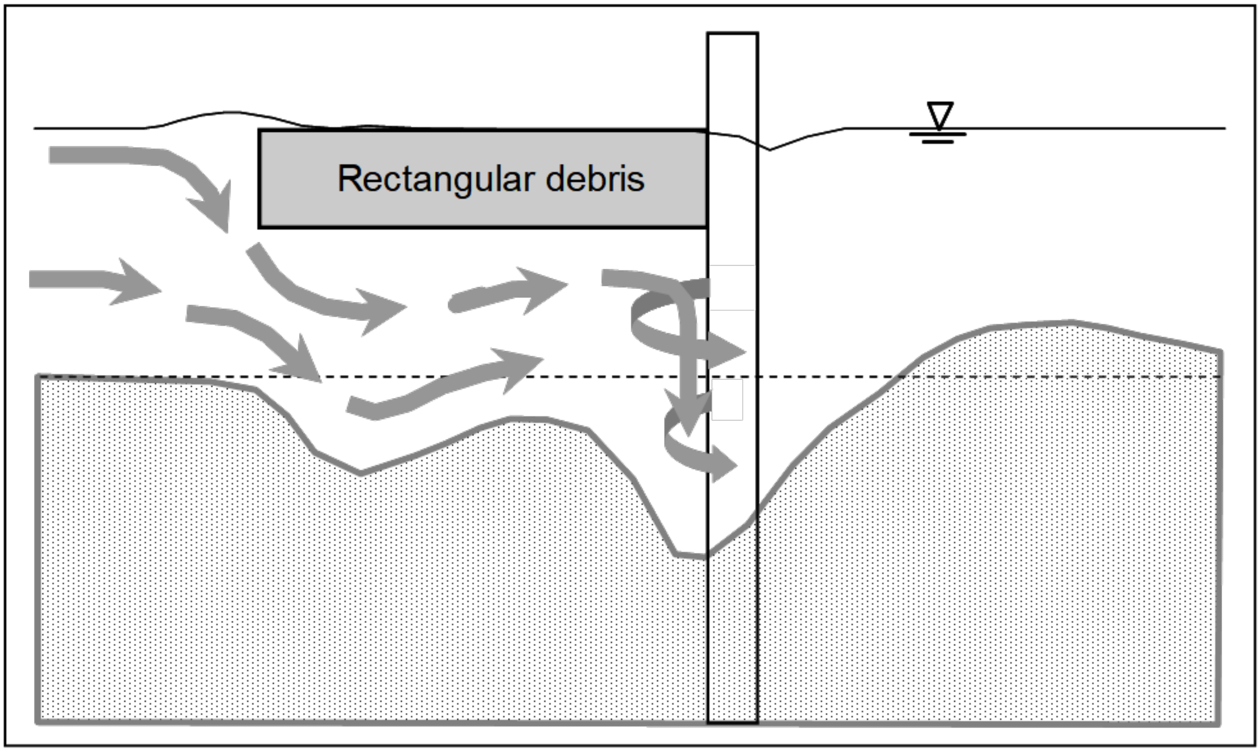

Figure 9.8. Idealized flow pattern and scour at pier with debris.

9.2.4 Channel Instability

In the context of safe bridge design, channel instability includes any channel change that can threaten a bridge foundation. Whether the changes are natural or result from human activities, they can expose foundations and increase scour during floods. HEC-20 and HEC-16 (FHWA 2012c and 2023) provide information on evaluating channel instability at bridges. Even though channel changes may be gradual or episodic, they are usually cumulative and have a long-term effect on the channel over the life of the bridge. Therefore, the potential for vertical and horizontal change is a consideration in safe bridge design.

Channel instability evaluations consider existing and potential future conditions. Factors warranting consideration when assessing potential channel instability include:

- Channel size and form.
- Flow and flood history.
- Valley and floodplain setting.
- Geologic and other vertical or horizontal controls.
- Channel and overbank materials.
- Vegetation and other biological influences.
- Land-use.
- Sediment sources and supply.

Vertical Instability: Vertical channel change includes aggradation and degradation resulting from a long-term excess or deficit in sediment supply, and degradation caused by headcutting. Long-term trends in discharge also impact channel geometry because increased flows tend to enlarge channels. This can occur due to increased runoff from urbanization, from climate change, and many other causes. Bridge inspection files that include repeat cross section measurements over time are useful in identifying aggradation and degradation problems and trends. The sediment transport chapter (Chapter 10) includes the discussion of sediment continuity and how sediment transport concepts can be used to analyze aggradation and degradation potential when there is an imbalance of sediment supply and transport capacity.

Headcuts occur when channel degradation progresses up the channel from downstream. They are often caused when the downstream base level of a channel is lowered. Figure 9.9 shows a headcut that could migrate upstream and through the bridge crossing during future runoff events. Features of a headcut that threaten bridges include long-term degradation that persists after the headcut has migrated upstream of the bridge, the plunge pool when the headcut is under the bridge, and channel widening that occurs because bed lowering often destabilizes channel banks.

Lateral Instability: Figure 9.10 shows progressive channel migration over a 72-year period at a highway crossing. The channel banks were identified and traced from historical and recent aerial photography. These bank lines show not only trends of channel migration down valley and across valley, but also variability in channel width through time. The channel migration process includes erosion of the bank materials, bank geotechnical failures, transport of the eroded and failed materials, and sediment accretion on the insides of bends (point bars). Reviewing historical aerial photography is not only useful for identifying the potential for lateral instability problems at a bridge but also facilitates predictions of channel location during the life of the bridge. HEC-20 (FHWA 2012c) presents photo-comparison techniques. Channel migration is typically a gradual long-term process, but single flood events can also cause extreme channel migration and widening, which can present significant challenges for bridge design.

Figure 9.9. Headcut downstream of a bridge (permission for use by Don Lozano).

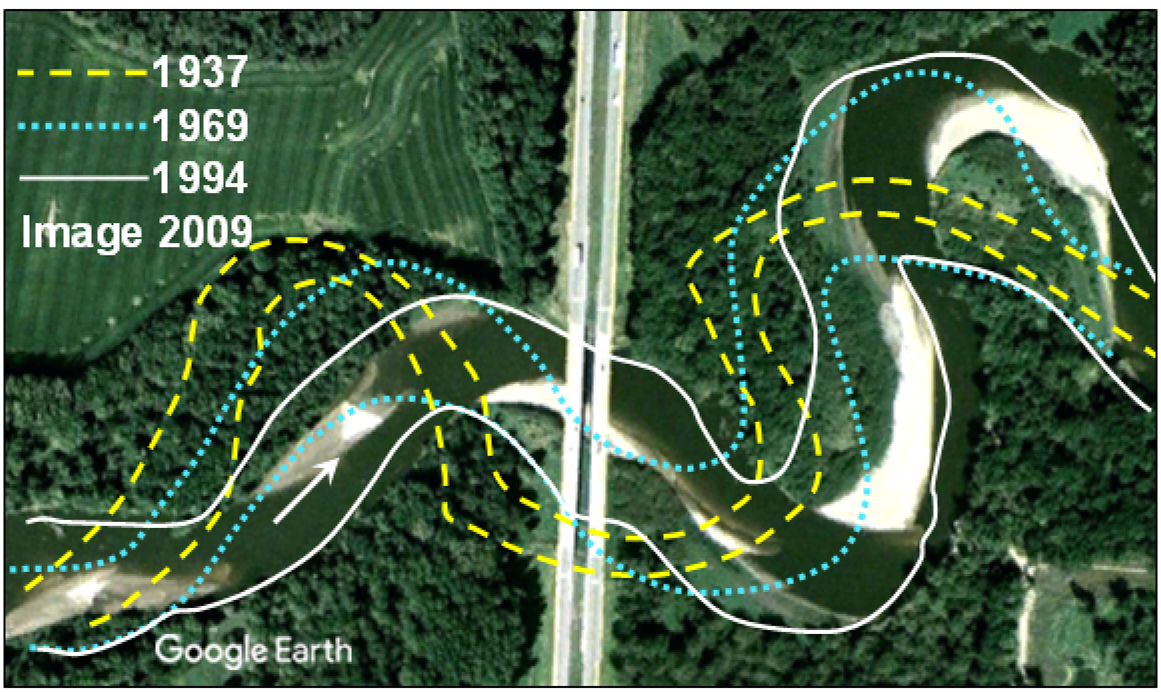

Figure 9.10. Meander migration on Wapsipinicon River near De Witt, Iowa.

9.2.5 Scour in Tidal Waterways

This chapter primarily describes scour approaches within the riverine context. Bridges over tidal waterways present somewhat different scour-related challenges. Tidal currents and waves affect bridges and roads near the coast continuously, or occasionally during storms. HEC-25 (FHWA 2020) is the primary FHWA reference for information on design of highways in the coastal environment. Coastal bridges differ from riverine bridges in both the geographic setting and the types of hydraulic conditions. Many of the scour concepts and equations transfer from river to coastal applications, but coastal conditions present additional considerations affecting hydraulic analysis and scour calculations. Chapter 8 provides more detailed information about coastal processes and tidal waterways. This section summarizes information found in the Coastal Scour chapter of HEC-25.

In coastal environments, bridges range in complexity from small, local roads crossing tidal creeks to limited access highways along the open coast. It is good design practice to account for the potential range of hydraulic conditions at a bridge crossing to identify the conditions that govern scour. In the riverine environment, the engineer determines the discharge prior to the hydraulic analysis. Riverine hydraulic models for scour calculation purposes often only extend a relatively short distance upstream and downstream of the crossing. In the coastal environment, unsteady-flow models determine the discharge hydrograph at the crossing. Such models simulate the effects of time-varying water elevations from tides and storm surges, and incorporate the full inundated area, often extending long distances upstream and downstream of the crossing. Coastal hydraulic conditions also include waves that transfer shear stress to the bed, transport sediment, and generate scour.

Depending on the region of the United States and the coastal setting, tides, tsunamis, hurricanes, nor'easters, or strong frontal systems drive the hydraulic design conditions. The basic mechanisms driving scour are similar in both the riverine and coastal environments: flow velocity and depth. However, the coastal scour response to design events differs from riverine for the following reasons:

- No distinct flow from an "upstream" source of sediment exists.
- Flows are unsteady and multi-directional.
- The presence of waves complicates the scour processes.
- Large surge/stage events do not always produce large velocities.
- The highest velocity during a tide or storm surge can occur well before or after the peak stage.

The National Bridge Inspection Standards (NBIS) is the Federal regulatory framework for the inspection of bridges and the appraisal of scour. [23 CFR 655 Subpart C]. The NBIS makes no distinctions between riverine and coastal scour situations. However, the FHWA cautions that in the coastal environment, the hydraulic expertise brought to bear on interdisciplinary teams should consider the inclusion of coastal engineering expertise, where appropriate. The AASHTO *Guide Specifications for Bridges Vulnerable to Coastal Storms* (AASHTO 2008) proposes a statement of qualifications for a coastal engineer and presents a list of conditions that require attention by a coastal engineer. That list includes prediction of potential wave scour at bridges and seawalls, along with design of countermeasures for wave-induced scour at bridge abutments and approaches.

HEC-25 includes example design-event scour analysis workflows for tidally-influenced, tidally-dominated, and tidal bridge crossings. These workflows include discharge and water surface boundary conditions, hydraulic and hydrodynamic model options, and whether wave-induced scour or other considerations are included. Existing engineering practice supports the concept of maximum scour depth for pier scour based on foundation parameters (i.e., flow velocity and depth from currents and pier geometry) with both currents and wave parameters playing a large role at abutments. As with the riverine environment, pile bents near abutments may experience additional local scour due to the abutment conditions.

In addition to other physical processes, the short duration of the typical design storm can also be considered for tidal scour design. All scour, whether local pier and abutment scour or contraction scour, takes time to reach an ultimate amount based on the maximum hydraulic condition. In some cases, especially for contraction scour, a storm surge would not be expected to produce ultimate scour during the relatively short duration of the highest velocity portion of the event. Although research is needed on this topic for local scour, sediment transport analyses can be applied to better estimate contraction scour during short periods.

9.3 Computing Scour

The quality and accuracy of hydraulic modeling directly impact the accuracy of scour calculations. The scour calculation variables include velocity, depth, discharge, flow width, unit discharge, and flow direction. If the model geometry or the representation of terrain is inaccurate, bank stations are not correctly or consistently defined and hydraulic controls may be missed. If Manning's n values are not accurate, or model assumptions are violated, then the poor quality of the hydraulic results used in scour calculations can result in unreasonable and incorrect scour estimates. The variables all depend on the suitability of the hydraulic model to define flow distribution, which affects the calculations of contraction, pier, and abutment scour.

This section discusses extraction of the necessary hydraulic information from 1D and 2D models. Recognizing that 2D models provide more accurate representations of the flow field and flow distribution, FHWA encourages the use of 2D modeling for all but the most straightforward bridge crossings. Two dimensional models are also much better at simulating the effects of hydraulic controls due to variability in channel geometry and floodplain terrain.

9.3.1 Events Analyzed

HEC-18 (FHWA 2012b) provides considerations on selecting flood events for analyzing scour and designing foundations to withstand scour. HEC-18 states "Bridge foundation for new bridges should be designed to withstand the effects of scour caused by hydraulic conditions from floods larger than the design flood" (meaning the hydraulic design flood). 23 CFR § 650.105 (d) states: *"Design Flood shall mean the peak discharge, volume if appropriate, stage or wave crest elevation of the flood associated with the probability of exceedance selected for the design of a highway encroachment. By definition, the highway will not be inundated from the stage of the design flood."*

This definition recognizes that there is a range of AEP events that could apply to the hydraulic design of a structure. Exceeding the hydraulic design flood could result in road overtopping and may necessitate temporary road closure or repair, but does not mean that the structure has collapsed, failed, or has even been damaged.

HEC-18 identifies scour-related design floods as the "Scour Design Flood" and "Scour Design Check Flood." The flood discharges for the scour-related design floods exceed the Hydraulic Design Flood because the consequences of exceeding a scour design are far greater. These consequences include potential structure failure, extended loss of service, and loss of life. In the Scour Design Flood, bridge foundations and countermeasures (if applicable) are designed to withstand the computed total scour including applicable factors of safety, load factors, and resistance factors at the relevant (service and strength) limit states. In the Scour Design Check flood, bridge foundations are designed to provide a minimum factor of safety for the computed total scour at the extreme event limit state. When scour countermeasures are used to protect bridge foundations from scour, they are also designed to be stable for the extreme event limit state.

For NHS bridges, FHWA requires application of certain AASHTO LRFD standards per 23 CFR § 625. 4. AASHTO LRFD Bridge Design Specifications identifies relevant scour design flood and scour design check flood event frequencies, their corresponding load factors, and corresponding limit states as described in Section 9.1. For non-NHS bridges, State DOTs establish the relevant event frequencies and corresponding load factors and limit states per 23 CFR § 625.3(a)(2). HEC-18 identifies example risk-based minimum scour design flood frequencies related to the hydraulic design frequency.

Other events are also considered; a more-probable (lower recurrence interval) event than a given design recurrence interval event may cause more severe (deeper) scour elevations and should be evaluated for design. For example, HEC-18 Table 2.1 pairs 50-year hydraulic design frequency with 100-year and 200-year scour design flood and scour design check flood frequencies, respectively. However, if the roadway overtopping flood is between any of these events, it is also analyzed because it could cause the most severe scour condition at the bridge.

9.3.2 Using One-Dimensional Models

Figure 9.11 shows the cross sections in the vicinity of a bridge crossing for a 1D hydraulic model. The Approach section represents conditions just upstream of the point where flow begins to contract into the bridge constriction and the Exit section represents where the flow is fully re-expanded. The contraction and abutment scour calculations include channel and overbank discharges at the Approach section and Crossing section. In a HEC-RAS model, the Crossing section includes bridge and roadway geometry data placed between the two bounding cross sections that are adjacent to the bridge and roadway (see Chapter 6). One-dimensional models provide estimates of hydraulic variables by computing incremental conveyance throughout each cross section and distributing flow in proportion to conveyance.

Figure 9.12 is a graphical representation of flow distribution at the bridge crossing from a HEC-RAS model. The results are also available as tabular output from the HEC-RAS program. The cross section shown is the upstream bounding cross section. This cross section is used in scour calculations to avoid pier influence. The diagonally hatched areas are ineffective flow areas created by the embankment blockages. This figure shows low velocity in the overbank areas under the bridge where flow depth is low and Manning's n value is high. Velocity is high in the channel where flow depth is high, and Manning's n is low.

Pier scour calculations use the velocity and depth upstream of the pier. Maximum velocity and depth, highlighted in Figure 9.12, are often used for pier scour calculations because they produce the most conservative results and because a shift in the thalweg could direct this velocity at any pier in the channel.

It is important to remember that the flow distribution in HEC-RAS is approximate because it is based on several assumptions, including:

- Flow is gradually varied.
- Flow is distributed relative to incremental conveyance.
- There is a level water surface across the entire cross section.
- There is a single value of energy slope across the entire cross section.

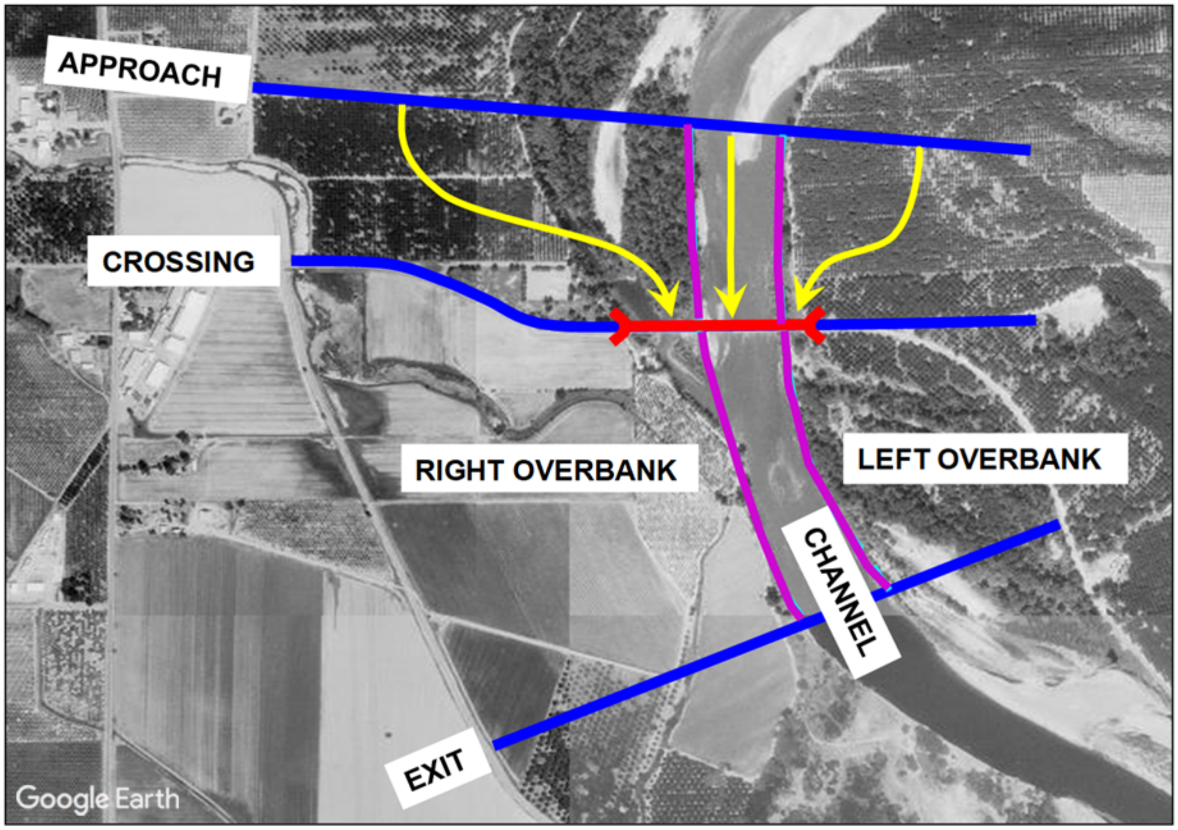

Figure 9.11. Cross section locations at bridge crossings in 1D models.

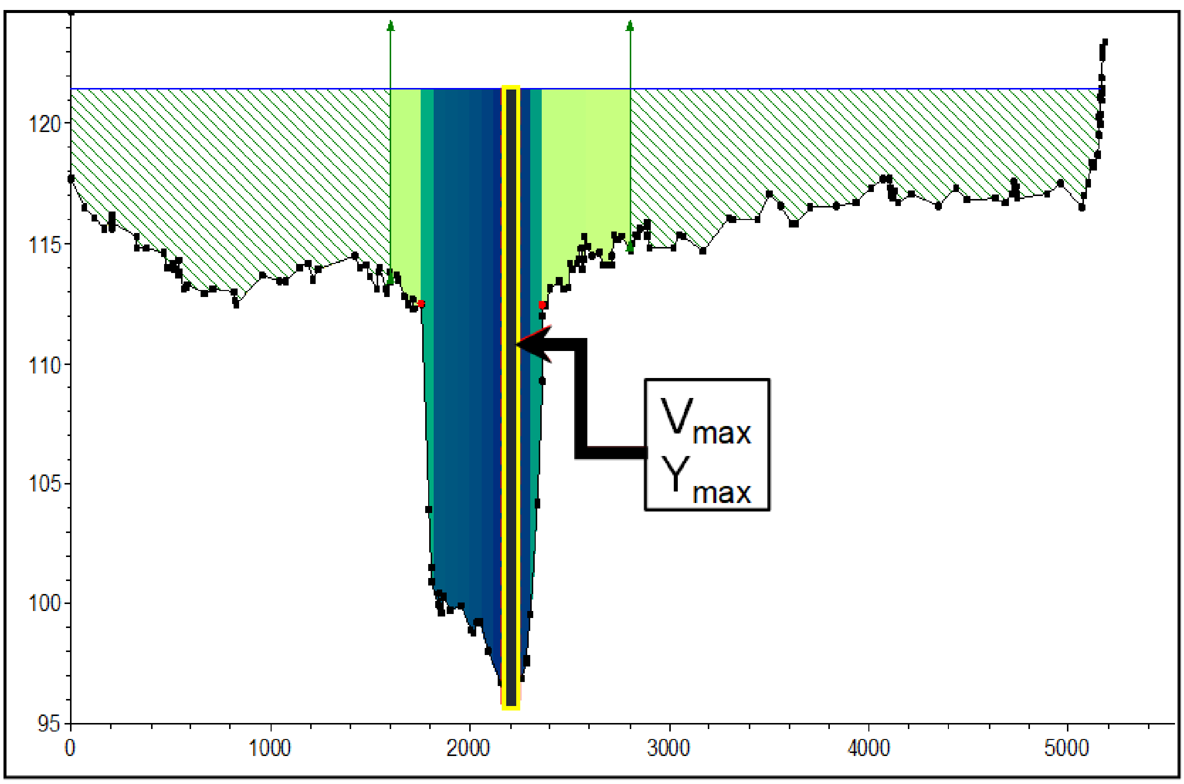

Figure 9.12. Flow distribution from 1D models.

Although these assumptions affect the entire flow distribution to some extent, the greatest error is near the abutment where much higher velocity and flow concentration (unit discharge) are typically expected. As discussed in the next section, the only assumption from the list above that carries into 2D models is that flow is gradually varied.

9.3.3 Using Two-Dimensional Models

Figure 9.13 shows 2D model results graphically as velocity contours and vectors. Contours of depth and water surface elevation are also available as graphical output, and some 2D modeling post-processors accommodate calculation and display of new data sets using the model results. The figure depicts a complex flow situation where a highway crosses a channel and wide overbanks. There is a long, main channel bridge, a shorter relief bridge on the overbank in the upper right corner of the figure. There is also a narrow railroad embankment, which has a main channel bridge and relief bridge, downstream of the wide highway embankment.

For pier scour calculations, designers can obtain point values of velocity, depth, and angle of attach directly from model results at any location within the model domain. The FHWA document *Two-Dimensional Hydraulic Modeling for Highways in the River Environment* (FHWA 2019) presents additional information on selecting the location from which to obtain the velocity, depth, and flow direction for a given pier. Figure 9.13 also shows four flow lines (also known as flow observation or monitoring lines). A 2D model can compute the discharge and hydraulic variables through the flow area defined by these flow lines. The flow lines in this figure are positioned to compute channel discharge at the bridge opening and the approach section, overbank flow in the wide setback area under the main channel bridge, and total flow in the relief bridge. The flow line output can also include the area, length, average velocity, and average depth. These variables provide the input data for contraction and abutment scour calculations.

Figure 9.13 also shows the flow concentration (high velocity) at the two abutments of the main channel bridge. One-dimensional models are not capable of computing this type of flow concentration. Multiplying velocity by depth at any point in the 2D model domain yields the unit discharge at that point. Dividing the discharge by the flow line length yields the average unit discharge across the flow line.

The FHWA-adopted 2D modeling suite includes the Surface Water Modeling System (SMS) pre- and post- processor and the United States Bureau of Reclamation Sedimentation and River Hydraulics 2D numerical model (SRH2D) (FHWA 2019). SMS includes FHWA-supported scour-variable extraction tools to aid scour calculation using SRH2D hydraulic results. Once the user completes and reviews a model simulation, they can develop a scour coverage in SMS. The user then exports a *. HYD file that includes approach section channel gradation (bed material gradation), along with hydraulic variables for contraction scour, pier local scour, and abutment combined contraction and local scour estimation. This file is compatible with the FHWA software package Hydraulic Toolbox (v4.2.2 and later), expediting hydraulic variable and soil gradation export from a 2D model. Figure 9.14 presents an example 2D model making use of the scour coverage. The figure shows the approach, contracted, and bank arcs. The model also includes abutment and pier arcs. The tools in the scour coverage extract the required variables for contraction scour, abutment scour, and pier scour at the locations indicated by the arcs.

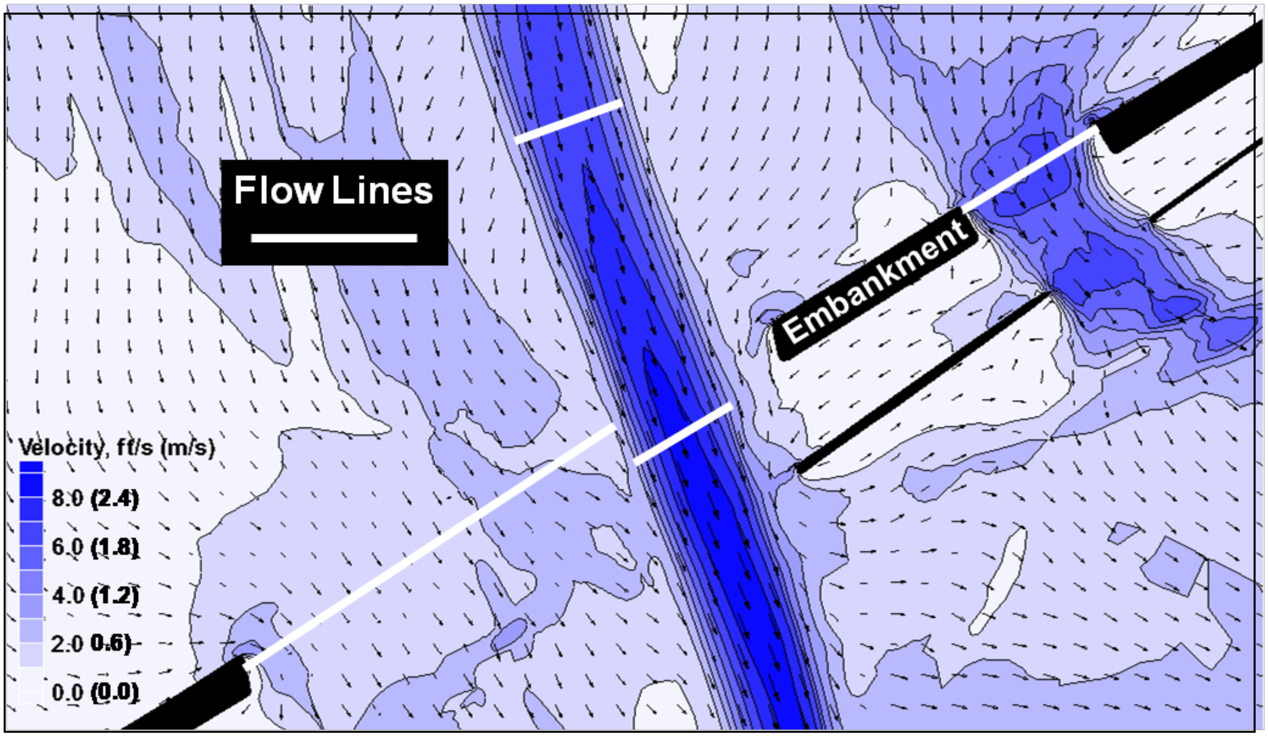

Figure 9.13. Velocity and flow lines in 2D models.

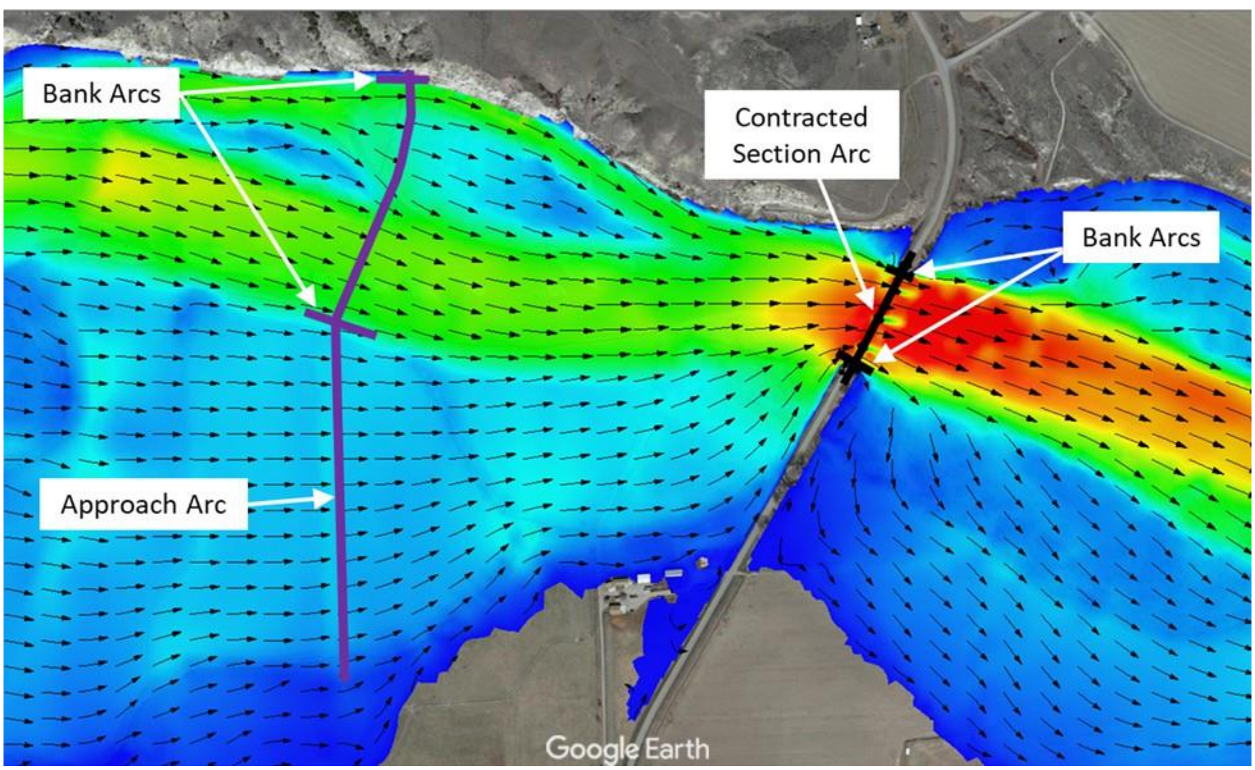

Figure 9.14. Example SMS Scour Coverage.

The Hydraulic Toolbox applies HEC-18 scour prediction methods, provides a convenient tabular summary table, and can plot a graphical depiction of the resulting scour prism. Both the scour coverage extraction process and the Hydraulic Toolbox make assumptions concerning the scour reference elevation and may combine local and cross-section maximum scour input variables for local scour computations. The Hydraulic Toolbox also requires additional scour inputs, including pier shapes and refined dimensions, bridge structure information, abutment configurations, channel bedform conditions, and long-term degradation potential. Prior to finalizing scour design elevations, the user reviews and updates each input variable, ensuring that the tools have reasonably combined and applied the extracted values. For example, in a laterally unstable reach, a designer would likely choose the thalweg as the design reference elevation for all substructure elements vs the exported (rigid bed) cross-section elevations from the hydraulic model. They would also likely evaluate abutment scour for Scour Condition A for main-channel hydraulic conditions. The resulting post-scour elevations at each substructure element are then used in the structural design of the pier and abutment foundations. Figure 9.15 presents an example Hydraulic Toolbox scour prism plot.

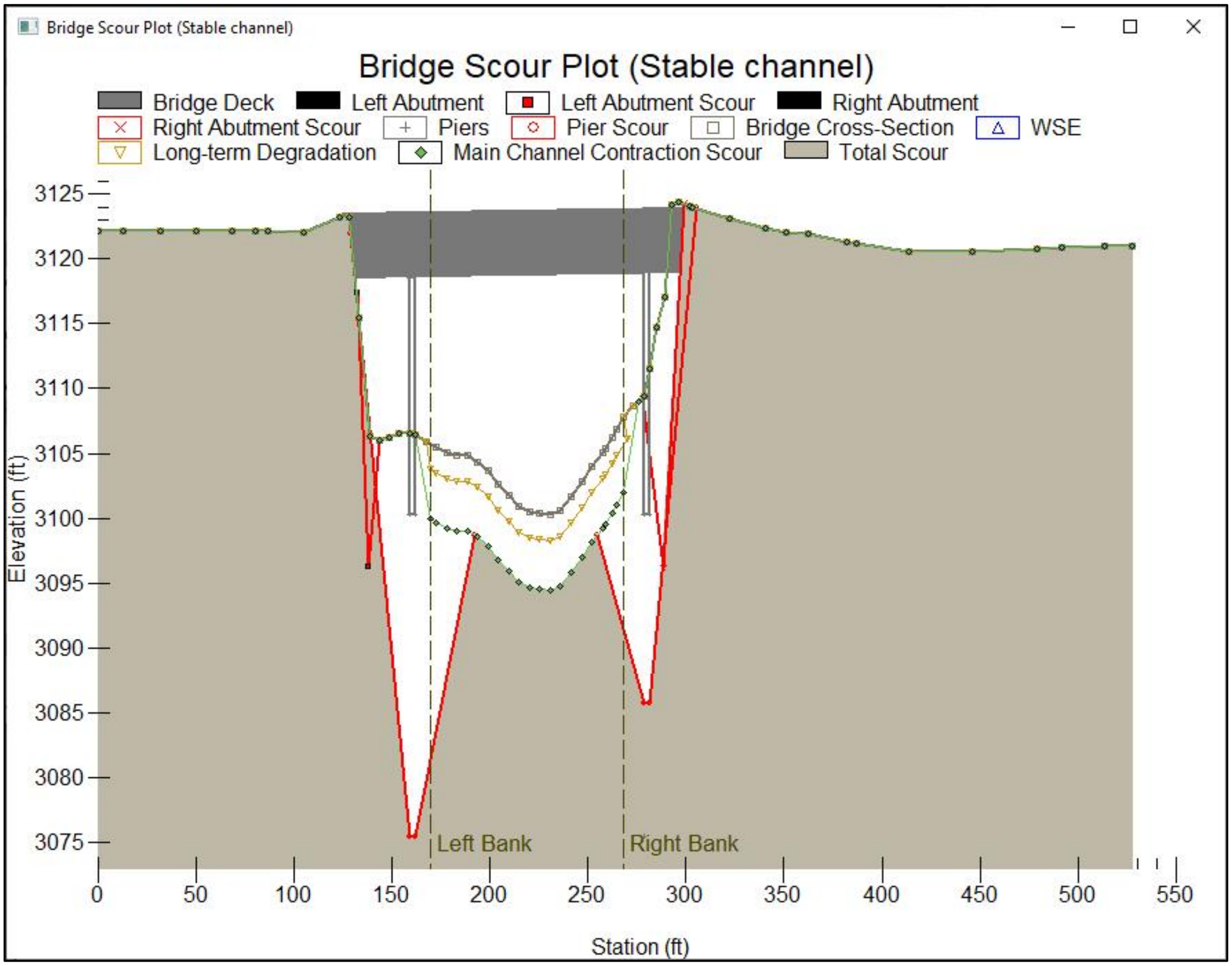

Figure 9.15. Example Hydraulic Toolbox scour plot depicting long-term degradation, contraction scour, and local scour for a laterally stable channel.

9.4 Scour Countermeasures

Scour and stream instability threaten the safety of bridges over water. Scour and stream instability countermeasures control, inhibit, change, delay, or minimize these threats. HEC-23

(FHWA 2009a) provides guidance and resources to aid countermeasure selection and design for various types of threats considering the range of river characteristics that are encountered. In addition to countering erosion and scour, with few exceptions hydraulic countermeasures also alter flow patterns and need to be included in hydraulic models. This section describes hydraulic modeling and design considerations for several countermeasures.

The structural designs of bridge piers and abutments must consider total scour potential (long-term degradation, contraction scour, local scour) and account for lateral instability over the service life of the bridge (per 23 CFR § 650.313(o)) as described in Chapter 3 and Section 9.1. HEC-18 describes design considerations for limiting potential stream stability and scour problems for new bridge construction (FHWA 2012b).

As with scour calculations, reliable scour countermeasure designs depend on the quality and accuracy of the hydraulic modeling. The design variables primarily include velocity, depth, and shear stress, each of which is best determined from 2D models. Inaccuracies in model geometric representation of the channel, floodplains, bridge elements, and countermeasures can produce poor quality hydraulic results and over- or under-designed countermeasures. A range of flow conditions is simulated to determine the controlling design condition.

9.4.1 Abutment Scour Countermeasures

As described in the 2021 FHWA document *Scour Conditions within AASHTO LRFD Design Specifications,* FHWA and AASHTO require roadway embankment protection at bridge abutments to improve the resiliency of the roadway network against flood-related service interruptions and to reduce maintenance (FHWA 2021). When the abutment foundations have been designed for total scour, including abutment scour, AASHTO still requires states to protect the abutment fill slopes from erosion (AASHTO 2020). Neither AASHTO nor FHWA specify design flood frequencies or design criteria for abutment slope protection. States may select to design such protection for the Scour Design Flood and use HEC-23 (FHWA 2009a) methods or may take a risk-based approach. Section 9.2.2 of this HDS-7 document presents additional considerations for abutment countermeasure design if a State DOT allows a designer to neglect the local scour component of total scour (only at abutments; total scour includes local scour without exception for piers). In that case, foundation stability is dependent upon the countermeasure, and it is appropriate to design the abutment scour countermeasure to protect the abutment and embankment fills for the scour check flood event and the associated scour elevations (FHWA, 2018). HEC-23 (2009a), supplemented with *Hydraulic Considerations for Shallow Abutment Foundations* (FHWA 2018), presents additional design, cross-section, and layout information for the designer.

HEC-23, supplemented with *Hydraulic Considerations for Shallow Abutment Foundations* (FHWA 2018), presents information and resources for designing abutment scour countermeasures. Key points include:

- The flow field near an abutment violates the 1D modeling assumptions. Abutment scour countermeasure material type and size are more accurately determined using 2D model results for the relevant design event.
- The abutment horizontal apron resists local scour effects at the toe of abutment walls or fill slopes. Slope protection alone is inadequate.
- The abutment horizontal apron may be undermined unless the apron top elevation is buried below long-term degradation plus contraction scour (or vertical contraction scour if applicable), considering lateral instability. If an abutment could experience main-channel conditions over its service life, the pre-scour reference elevation is the channel thalweg for design.

- Abutment scour protection designs wrap around the bridge embankment fills on the upstream and downstream embankment faces to resist erosion caused by flow separation and wake eddy effects.

Figure 9.16 presents a cross-section sketch of a bridge with abutment scour countermeasures. The abutment spill-through slopes are set back from the main channel, and the bridge has deep foundations. The left side depicts scour elevations for a bridge crossing a laterally stable channel, and the right side depicts scour elevations for a bridge crossing a laterally unstable channel and scour elevations are referenced to the main channel thalweg.

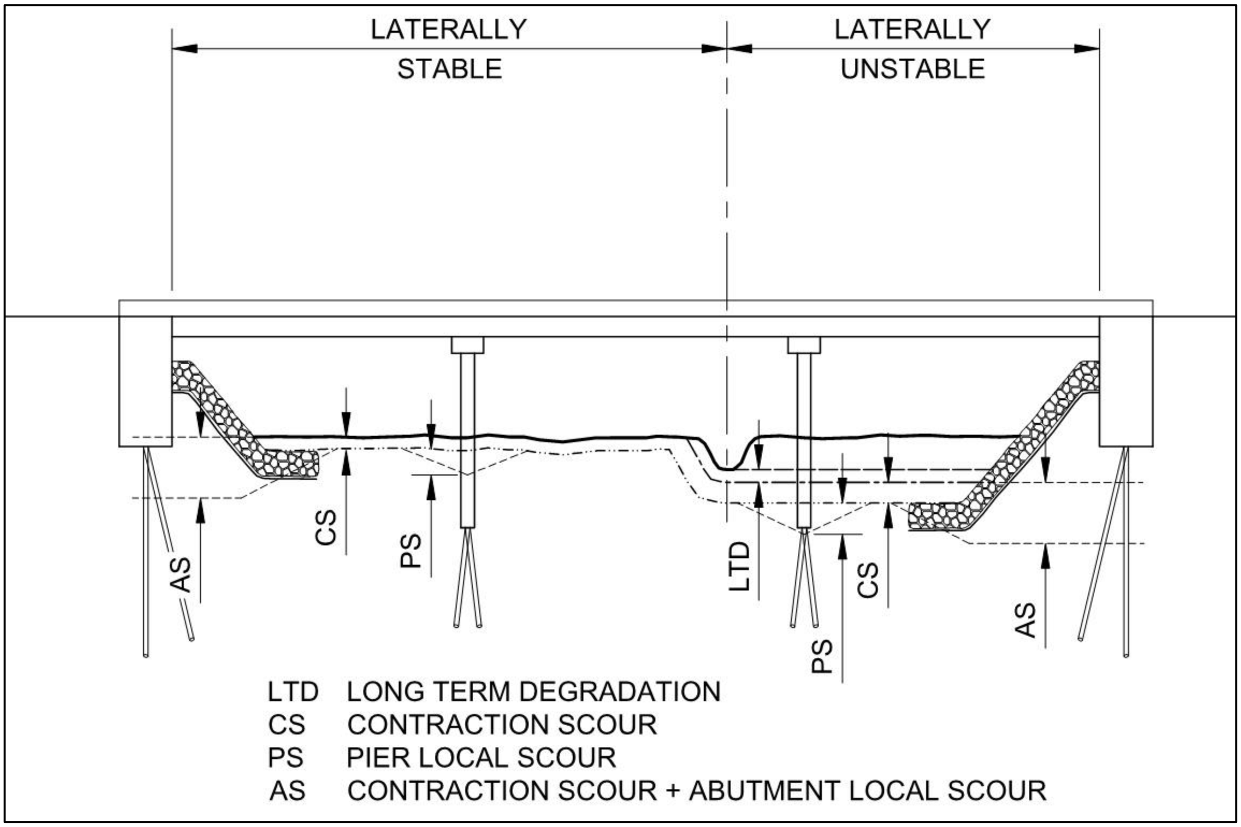

Figure 9.16. Example bridge opening cross-section with abutment scour countermeasures protecting spill through slopes. (Figure after Idaho Transportation Department, 2021).

Revetments and Vegetation: Channel bank revetments and vegetation are the most common type of countermeasure against lateral stream instability and bank erosion. Revetments are placed directly on the channel bank and include riprap, articulating concrete blocks, various types of mattresses, engineered log jams, and may be used in combination with vegetation as hybrid nature-based solutions. Hydraulic modeling of revetments and vegetation includes adjusting geometry to represent design grading and assigning representative values of Manning’s n for the countermeasure material.

Guide Banks: When embankments encroach on wide floodplains, significant amounts of flow may approach the bridge opening parallel to the roadway embankment. The resulting flow concentration and large-scale turbulence can generate large amounts of scour at the abutment. Flow separation can also reduce the effective bridge opening, potentially increasing contraction scour. Guide banks (as shown in Figure 9.17) can prevent severe abutment scour and reduce flow separation. Some local scour will still occur, but typically only at the upstream end of the guide bank, well away from abutment foundation elements.

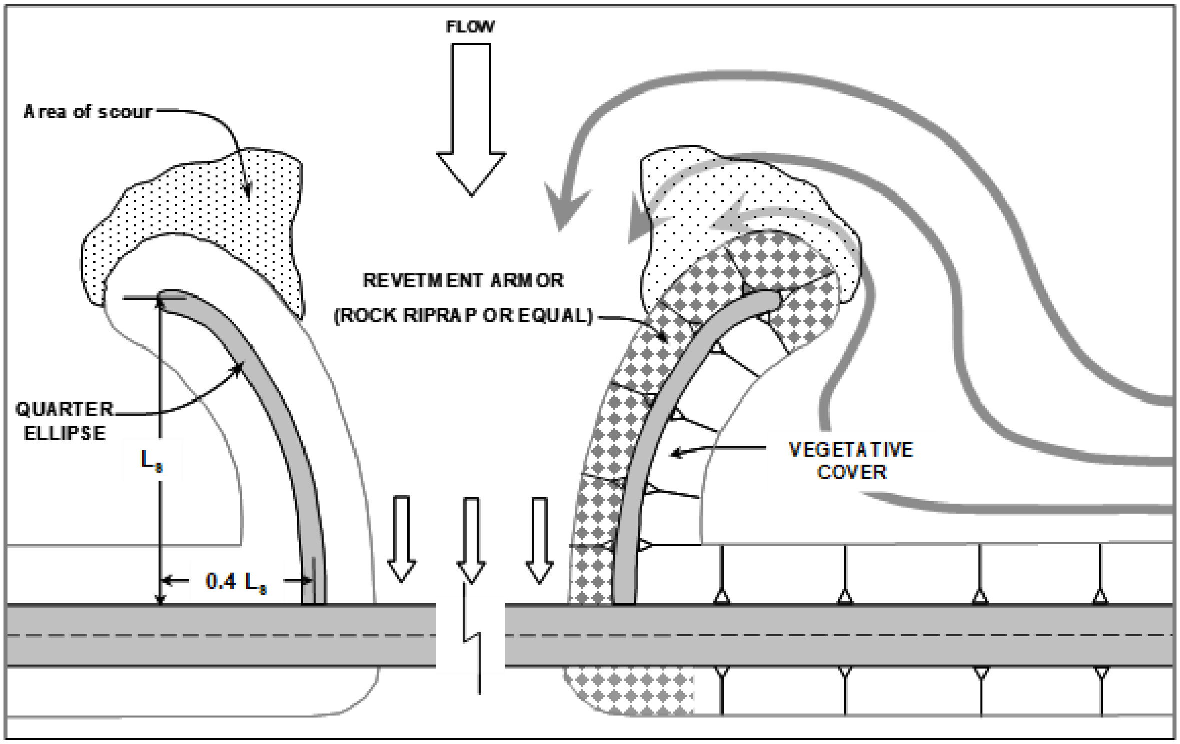

Figure 9.17. Typical guide bank (modified from FHWA 1978).

As shown in Figure 9.18, 2D models can directly model the geometry of guide banks. As described in *Two-Dimensional Hydraulic Modeling for Highways in the River Environment: Reference Document* (FHWA 2019), the unstructured mesh shown in this figure demonstrates that areas of rapid change in velocity magnitude or direction require a more refined network of elements. An unstructured mesh also allows for detailed assignment of Manning's n values. Figure 9.19 shows the flow field around this guide bank and the flow around the abutment at the other end of the bridge. There is flow separation under the bridge right abutment (left side of figure) but not on the guide bank side. Maximum flow velocities are also much lower at the guide bank protected side.

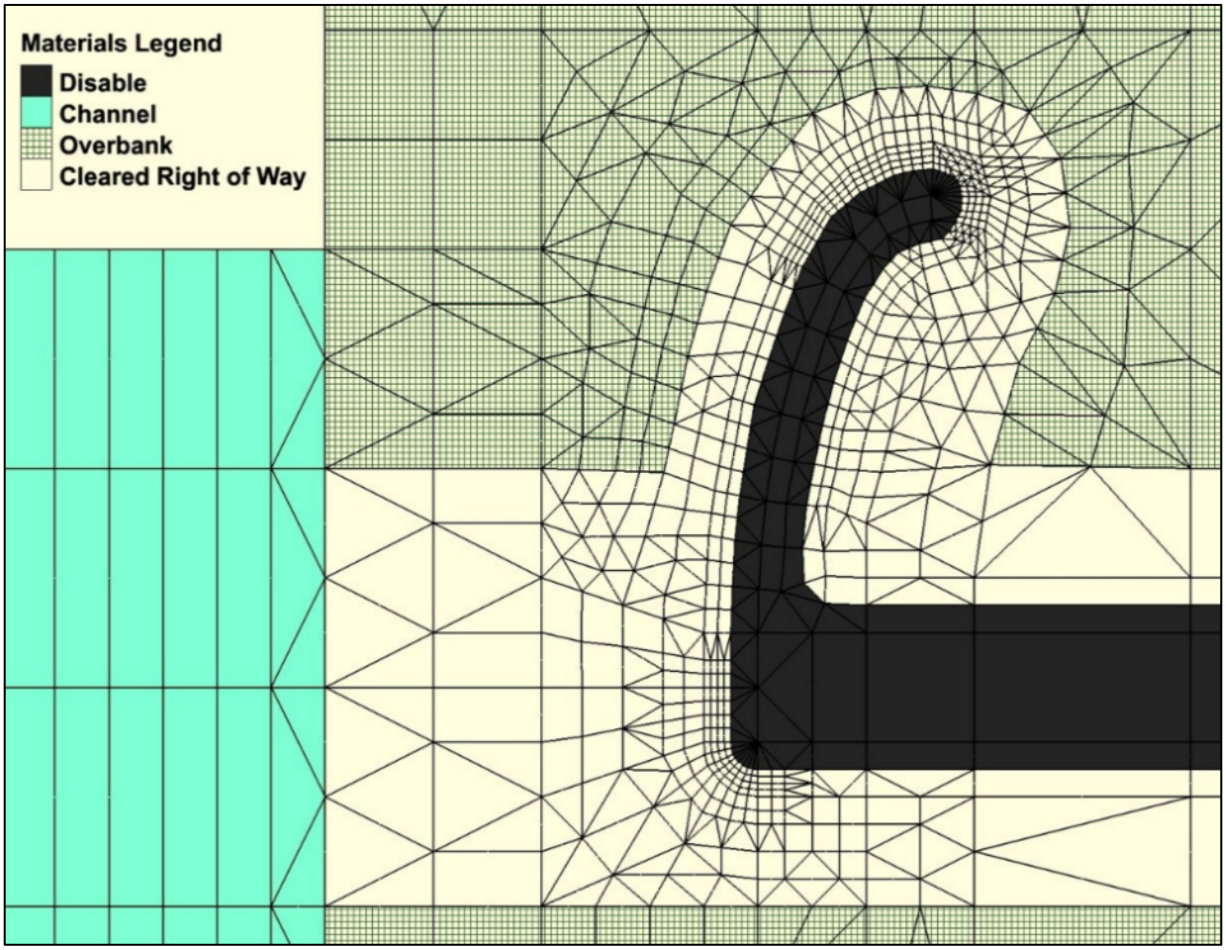

Figure 9.18. Guide bank in an unstructured 2D model grid.

An engineer can model a guide bank with a 1D model, but only approximately. A 1D model is not capable of simulating and representing the beneficial scour effects of the guide bank. Figure 9.19 clearly illustrates some of the benefits of 2D modeling for hydraulic countermeasure analysis and design. Two-dimensional models simulate the true flow field, water surface elevations, and velocities much more accurately and thoroughly than 1D models. Because 2D models conserve momentum in the X and Y directions, flow direction is directly computed throughout the model domain, allowing accurate determination of the angle of attack, though potential for future change may alter flow patterns over the service life of the bridge. The 2D model also shows that the right abutment, which does not include a guide bank, has flow separation and a portion of the bridge opening is not effective for conveying flow, thereby potentially increasing contraction scour. The maximum velocity at the guide bank is also much lower than at the opposite abutment. Therefore, the required riprap size is much smaller for the guide bank. HEC-23 presents information and resources for guide bank design.

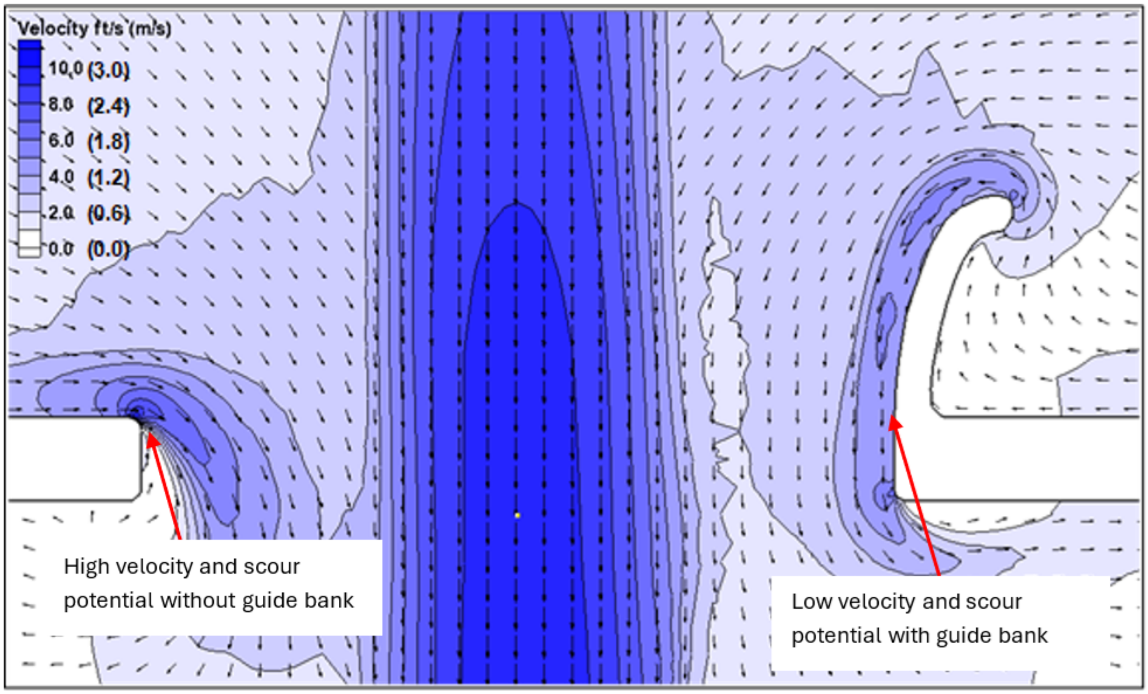

Figure 9.19. Velocity magnitude and vectors at a bridge opening with one guide bank.

Spurs and Bendway Weirs: Spurs and bendway weirs can protect channel banks from erosion and improve flow alignment at bridges where channel migration has occurred. Figure 9.20 and Figure 9.21 show 2D modeling results of bankfull flow at an unprotected bend and the same bend protected with spurs. As discussed in HEC-23 (FHWA 2009a), the hydraulic function and design of these two countermeasures are significantly different in that in-channel flows overtop bendway weirs, by design, and do not overtop spurs. The primary similarities of these structures are that they extend from the bankline and usually have unprotected channel bank between structures. In the models shown in Figure 9.20 and Figure 9.21, high velocities reach the toe of bank in the unprotected bank (Figure 9.20) but low velocity circulation occurs along the bank when protected by spurs (Figure 9.21). Shear stress (or any other hydraulic parameter) can also be computed, contoured, and compared between existing and proposed conditions using the 2D model interface.

These models also illustrate that the upstream spur is subjected to the highest flow velocity and that the spurs are likely to shift the thalweg and may erode and shift the opposite bankline because of the increased flow velocities away from the spurs. One-dimensional modeling could also be used to simulate these conditions. However, the results would be more indicative of average conditions at a cross section rather than the detailed distributed results of the 2D flow field. However, even a very refined 2D model network is not a complete representation of the flow characteristics since there are significant and consequential three-dimensional flow features associated with bendway weirs and spurs, and especially when structure overtopping occurs.

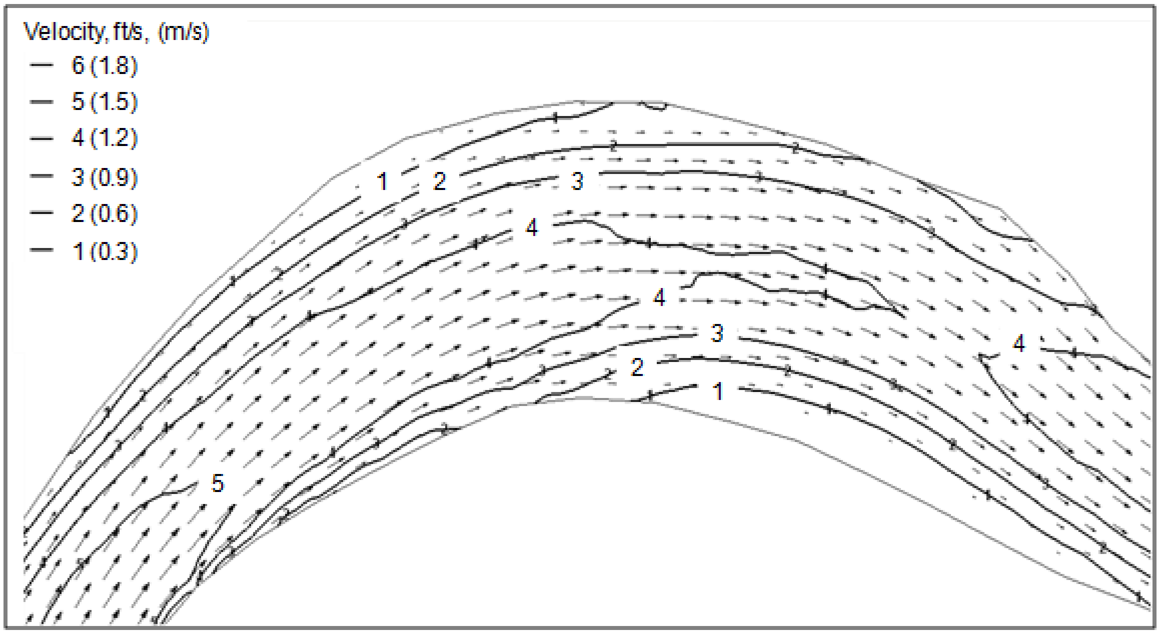

Figure 9.20. 2D analysis of flow field without spurs.

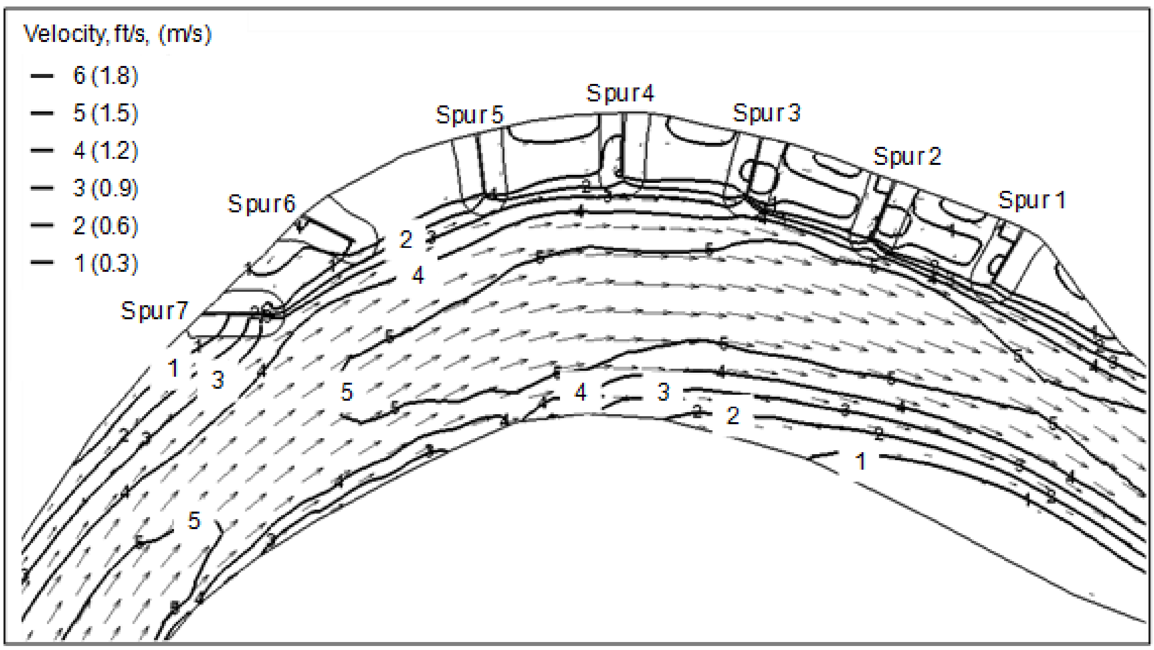

Figure 9.21. 2D analysis of flow field with spurs.

CHAPTER 10
SEDIMENT TRANSPORT & ALLUVIAL CHANNEL CONCEPTS

10.1 Introduction

Flowing water forms alluvial channels and floodplains by the transport and deposit of sediments. The alluvial sediments present in a stream reach are available for erosion and transport in the future, leading to potential channel adjustments. Aggradation and degradation are the overall raising or lowering of a channel bed over time from sediment accumulation or erosion. Channel widening and shifting result from bank erosion due to hydraulic forces or by mass failure of the bank.

Most channels are not stationary but adjust their bed and banks during the bridge's life, and safe bridge design accounts for potential channel adjustment. FHWA defines resilience as the *"… the ability to anticipate, prepare for, or adapt to conditions or withstand, respond to, or recover rapidly from disruptions …"* [23 U.S.C. § 101(a)(24)]. Reliability is tied to resilience because a resilient transportation network is safer and less susceptible to delays and failures. Identifying and accommodating potential future river conditions in the design process makes highway facilities are more resilient and reliable. Therefore, the moveable boundary of the alluvial river adds another aspect of consideration to bridge design. In addition to accommodating potential future conditions, bridge designs that appropriately convey the sediment coming from upstream involve less maintenance and are less disruptive to aquatic habitats.

Channel stability and sediment transport are complex processes that interact to produce the existing channel form and future channel adjustments. HEC-20 (FHWA 2012c) calls for qualitative evaluation (Level 1) and standard engineering analyses (Level 2) even when performing an advanced numerical sediment transport modeling (Level 3). The information and understanding of dominant processes gained in Level 1 underpin Level 2 and 3 analyses. Level 2 analyses are warranted in new bridge designs and many scour evaluations of existing bridges. Sediment transport studies conducted in Level 3 are infrequently performed and reserved for addressing complex situations.

Several factors influence sediment transport, as described in HEC-16 (FHWA 2023a). The factors include:

- Sediment properties.
- Hydrology.
- Watershed and land-use conditions.
- Channel geometry (shape and slope).
- Vegetation.
- Other biological factors.

Channels respond and adjust to changes in flow and sediment supply. Therefore, changing watershed conditions often cause channel geometry adjustments. Channel geometry, bed material, and vegetation determine hydraulic variables (such as velocity and depth for a given discharge), thereby affecting sediment transport capacity. Consequently, sediment transport and channel stability depend not only on the specific physical processes but also on the history of natural and human-induced factors in the watershed.

The HEC-20 (FHWA 2012c) and HEC-16 documents (FHWA 2023a) are the primary FHWA references related to stream instability and sediment transport topics. Other resources include

Sedimentation Engineering (ASCE 2008), textbooks such as Simons and Senturk 1992, Yang 2003, and Julien 2010, and the documents for specific numerical computer programs that incorporate sediment transport.

10.2 Application of Sediment Transport to Bridge Projects

Sediment transport analyses can play a role in several aspects of safe bridge design. Of primary concern is whether the channel will experience long-term aggradation or degradation. Aggradation decreases flow capacity and may increase the frequency and magnitude of flooding, road overtopping, and loss of service. Degradation threatens bridge foundations by removing support and making the bridge more vulnerable to scour during floods. A related concern is that the bridge could alter the prevailing flow conditions and cause aggradation or degradation. Transportation agencies may also conduct channel restoration as part of a bridge replacement. Sediment transport analysis can help to determine the potential impacts of the restoration to avoid creating a channel that does not adequately convey sediment supplied from upstream. The engineer can perform a more detailed evaluation of contraction scour using a sediment transport model instead of the standard contraction scour equations found in HEC-18. In summary, the following reasons may justify sediment transport analysis as part of a bridge design project.

- More detailed evaluation of potential contraction scour.
- Evaluation of long-term degradation or aggradation potential.
- Concerns over a bridge or culvert replacement impacting future channel stability.
- Maintaining sediment conveyance through a bridge, culvert, or river reach.
- Evaluation of channel restoration project impacts on sediment transport and vertical channel stability.

10.3 Sediment Continuity

The amount of material transported, eroded, or deposited in an alluvial channel is a function of sediment supply and channel transport capacity. The tributary watershed and erosion occurring in the upstream channel bed and banks provide the sediment supply. Sediment transport capacity is primarily a function of sediment size and the hydraulic properties of the channel. When the transport capacity of the flow equals the sediment supply from upstream, a state of equilibrium exists.

Natural rivers often exhibit substantial variability of velocity, depth, and width along the watercourse. This implies that sediment transport capacity also varies, which results in aggradation and degradation. A channel that adjusts to transient sediment transport imbalances but maintains a long-term balance of sediment supply and transport capacity is considered as dynamically stable or in dynamic equilibrium. In a dynamically stable reach, adjustments to the bed and banks allow the channel to maintain long-term sediment continuity with upstream sediment supply. See HEC-16 for additional information on dynamically stable channels.

Applying the sediment continuity concept to a channel reach illustrates the relationship between sediment supply and transport capacity. During a given time period, the amount of sediment coming into the reach minus the amount leaving the downstream end of the reach equals the change in the amount of sediment stored in that reach (Figure 10.1). The sediment supply from the watershed and channel (upstream of the study reach plus lateral input directly to the study reach) defines the sediment inflow to a given reach. The transport capacity of the channel within the given reach defines the sediment outflow. Changes in the sediment volume within the reach occur when the total input to the reach (sediment supply) is not equal to the downstream output (sediment transport capacity). When the sediment supply is less than the transport capacity, erosion (degradation) occurs in the reach to satisfy transport capacity at the outlet, unless controls exist that limit erosion. Conversely, when the sediment supply is greater than the transport capacity, deposition (aggradation) occurs in the reach.

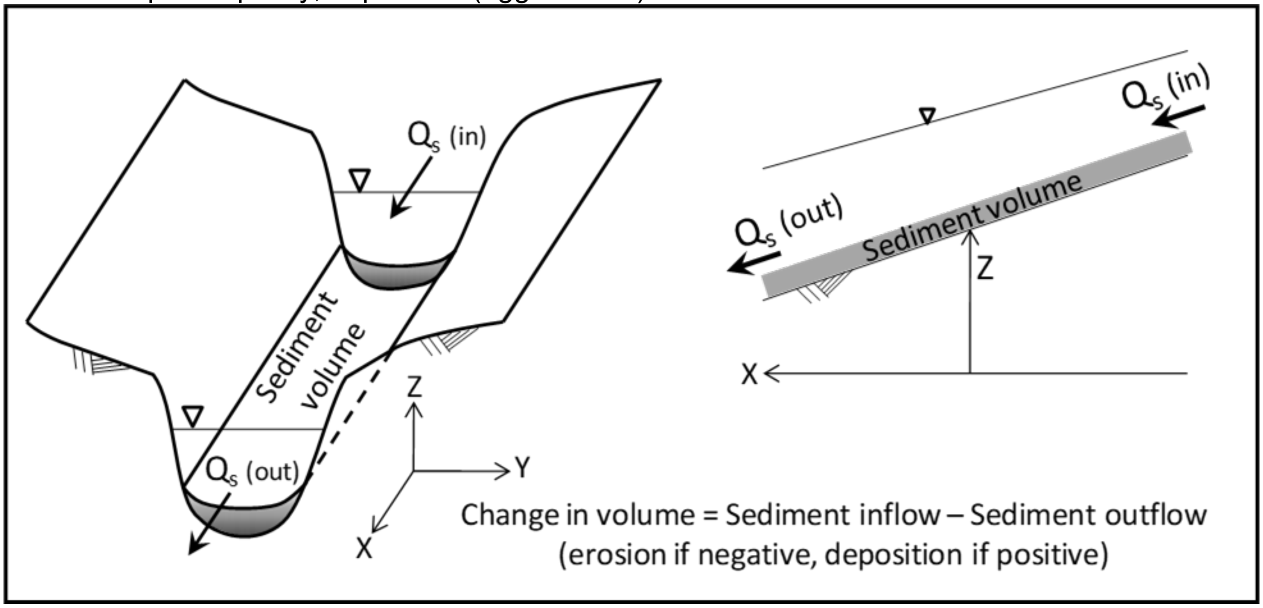

Figure 10.1. Definition sketch of the sediment continuity concept.

Controls that limit erosion may either be human-induced or natural. Human-induced controls include bank protection works, grade control structures, and stabilized bridge or culvert crossings. A channel may contain natural geologic controls, such as bedrock outcrops. Coarse material in the channel bed is another type of potential natural control, as it can result in the formation of a surface armor layer of larger sediments.

The Exner Equation describes the sediment continuity mathematically. The one-dimensional differential form of the equation is:

$$(1-\eta)W\frac{\partial Z}{\partial t} = -\frac{\partial Q_s}{\partial X} \tag{10.1}$$

where:

W	=	Active width of the channel, ft
Z	=	Channel bed elevation, ft
t	=	Time, s
η	=	Bed material porosity (volume of voids/total volume)
Q_s	=	Sediment transport rate, cfs
X	=	Distance along the channel, ft

Applied to a channel reach, the sediment continuity equation is:

$$\Delta Z = \frac{\Delta t\left(Q_{s(in)} - Q_{s(out)}\right)}{WL(1-\eta)} \tag{10.2}$$

where:

L	=	Reach length, ft

10.4 Sediment Transport Concepts

10.4.1 Initiation of Motion

Flowing water subjects exposed bed material particles to drag and lift forces. The flow field near the boundary, turbulence fluctuations, particle size, particle shape and relative position with respect to other particles all contribute to or affect these forces. Gravitational and external support forces (friction and other point contacts between grains) act on the sediment particles to resist motion. The resisting forces are functions of the particle density, size, shape, and relative position to other particles. This problem has been simplified and studied experimentally by many scientists for laboratory conditions, including Shields (1935). Detailed discussions are available from many sources. Shields related the beginning of motion to particle size, particle submerged unit weight, and flow shear stress to predict the initiation of motion.

Standing water exerts hydrostatic pressure on the channel bed. For uniform flow with small slopes, the flowing water exerts a time-averaged shear stress in the direction of flow equal to the hydrostatic pressure times the channel slope:

$$\tau_0 = \gamma\, y\, S_0 \tag{10.3}$$

where:

τ_0	=	Shear stress, lb/ft^2
γ	=	Unit weight of water, lb/ft^3
y	=	Flow depth (hydraulic radius, hydraulic depth, or local depth), ft
S_0	=	Bed slope (or energy slope for gradually varied flow)

For non-uniform, gradually varied flow conditions, one can calculate the shear stress acting on the bed surface using the same equations by substituting the energy slope for the bed slope. Another useful formula for estimating average shear stress for uniform or gradually varied flow conditions is:

$$\tau_0 = \frac{\gamma}{y^{1/3}}\left(\frac{Vn}{1.486}\right)^2 \tag{10.4}$$

where:

n	=	Manning roughness coefficient
V	=	Flow velocity, ft/s

Equation 10.4 shows the relationship between velocity and shear stress; shear stress is proportional to velocity squared. The Shields parameter relates critical shear stress to particle size and unit weight by the following relationship:

$$\tau_c = k_s D_s (\gamma_s - \gamma) \tag{10.5}$$

where:

τ_c	=	Critical shear stress for the beginning of motion, lb/ft^2
k_s	=	Shields parameter
D_s	=	Particle size, ft
γ_s	=	Unit weight of the particle, lb/ft^3

Shields parameter generally ranges from 0.03 to 0.10 for natural sediments and depends on particle shape, angularity, gradation, and imbrication. Engineers commonly use a Shields parameter value of 0.047 for sand sizes. When the shear stress exerted by the flow exceeds the critical shear stress of the particle, the channel bed begins to mobilize, transporting bed material downstream. Because smaller, more easily moved particles are shielded by larger particles and larger, harder to move particles project further into the flow field, the concepts of hiding and equal mobility (threshold) of sediment bed load have been developed (see ASCE 2008 and Julien 2010 for further information on this topic). While the particles are not precisely equally mobile, it is useful to consider the bed as mobile when the median particle size, D_{50}, is used to evaluate bed mobilization.

Particle motion begins as sliding and rolling of individual particles along the bed. Shields equation is not a sediment transport equation because it does not provide any estimate of the amount of sediment in motion. It is also important to note that only the shear stress acting on the particles, or grain friction, should be used in applying this relationship. This means that shear stress from other sources, including bed forms or vegetation, are excluded from the grain friction. From the standpoint of Equation 10.4, the Manning's n value of the grain friction would be used.

10.4.2 Modes of Sediment Transport

Once the exerted shear stress exceeds the critical shear stress, bed material begins to move (roll, slide, and saltate) along the bed surface. This material is referred to as bed load or contact load because it is in near-continuous contact with the bed. For small amounts of excess shear stress (defined as exerted shear stress minus critical shear stress), this is the only mode of bed material transport. As excess shear stress increases, turbulence begins to entrain some of the particles into the flow. The turbulence acts to mix the particles in the water column while at the same time, gravity causes the particles to settle. Therefore, bed material can also be transported downstream as suspended bed material load. Figure 10.2 illustrates the two types of bed material load.

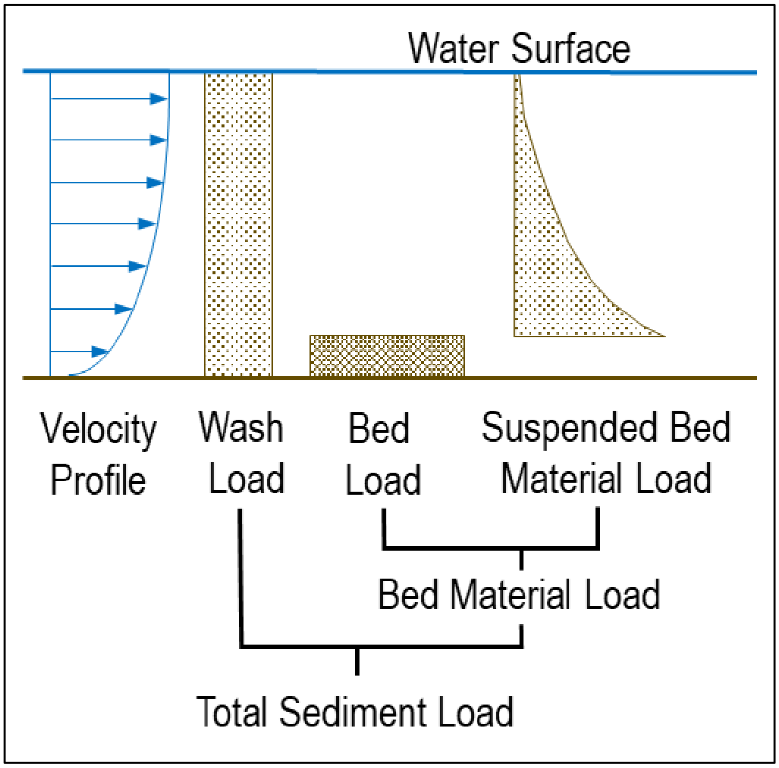

Figure 10.2. Definitions of sediment load components.

The suspended bed material load shown in Figure 10.2 results from the interaction between gravity and turbulence. Because gravity is causing particles to settle, the concentration is highest near the bed. Turbulence mixes the particles in the water column and, depending on the size and density of the particles, relatively few particles may reach the surface. The suspension of particles is illustrated in Figure 10.3, which shows the concentration profile for a range of particle sizes in a turbulent flow field. An equation that describes the concentration profiles is:

$$\frac{c}{c_a} = \left[\left(\frac{y_0 - y}{y}\right)\left(\frac{a}{y_0 - a}\right)\right]^z \quad (10.6)$$

where:

c	=	Sediment concentration at height y from the bed
c_a	=	Reference sediment concentration at height a above the bed
a	=	Reference height above the bed, ft
y_0	=	Total flow depth, ft
z	=	Rouse number = $\omega/\beta kv^*$
ω	=	Fall velocity of the particle in quiescent water, ft/s
β	=	Parameter relating particle and momentum transfer due to turbulence
k	=	Von Karman's constant of 0.4
v*	=	Shear velocity = $(\tau_0/\rho)^{0.5} = (gRS_0)^{0.5}$, ft/s
ρ	=	Water density, slugs/ft^3

g	=	Acceleration due to gravity, ft/s^2
R	=	Hydraulic radius, ft

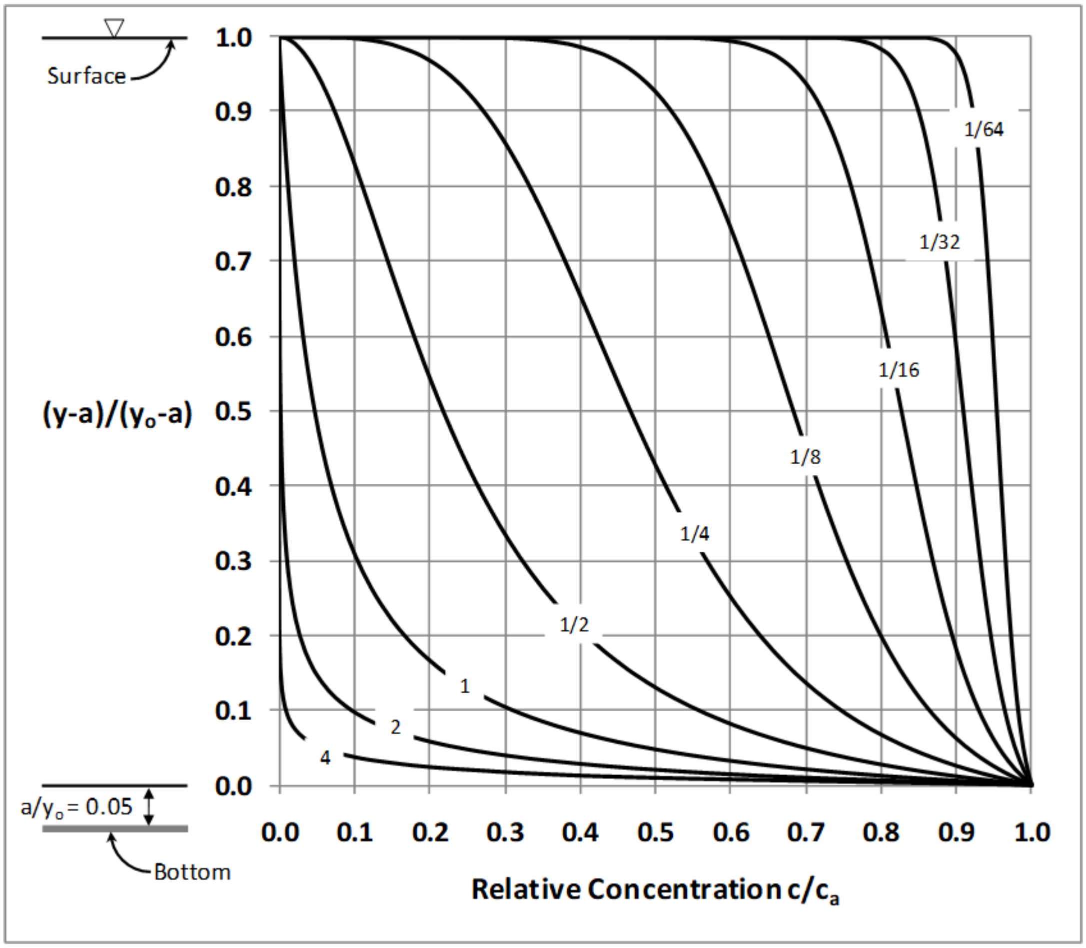

Figure 10.3. Suspended sediment concentration profiles. Each profile curve is for a different value of Rouse number.

Larger particles have greater fall velocities and higher Rouse numbers. Therefore, Figure 10.3 shows that large particles remain close to the bed for a given level of turbulence (as represented by the shear velocity). Finer particles have smaller Rouse numbers, are mixed higher into the flow, and have higher relative concentrations. Julien (2010) indicates that particle sizes with Rouse numbers less than 0.025 (1/40) have essentially uniform concentration profiles throughout the water column. These particles are extremely fine, primarily silts and clays. They have very small fall velocities and make up the wash load portion of the total sediment load. Wash load results primarily from upland erosion and bank erosion of floodplain materials. Wash load material is not found in appreciable quantities in the channel bed and can include coarser sizes such as sand.

In summary, the total sediment load transported by a channel reach consists of bed material load and wash load. A portion of the bed material load moves along the bed (bed load), and the remainder moves in suspension (suspended bed material load). The wash load is supplied to the reach from upstream, not derived from the channel bed. Sand may act as wash load In coarse-bed channels, such as cobble-bed and boulder-bed streams. This is because such channels have negligible quantities of sand in the bed and because the supply is far less than the channel's capacity to transport this size.

10.4.3 Bed Forms

The bed material moves easily in sand-bed streams, and the flow continually reshapes the bed. The interactions between the movement of the water-sediment mixture and the sand-bed create a range of bed configurations which in turn alter the resistance to flow, velocity, water surface elevation, and sediment transport. Consequently, understanding the different types of bed forms that may occur, their geometry, flow resistance, and sediment transport associated with each bed form aids in analyzing hydraulic conditions in an alluvial channel.

Flow Regime. Bed forms in alluvial sand-bed channels are divided into lower and upper regimes separated by a transition zone. Figure 10.4 illustrates the regimes and transition zone. The numbers in the figure indicate the order that bedforms occur as flows increase. Although flow velocity is higher for the upper regime, there is no specific relationship between the classification of bed-form flow regime and Froude Number (supercritical versus subcritical conditions). The flow regimes are:

- The lower flow regime, where resistance to flow is large, and sediment transport is small. The bed form is either ripples or dunes or some combination of the two. Water surface undulations are out of phase with the bed surface, and there is a separation zone downstream from the crest of each ripple or dune. The downstream movement velocity of the ripples or dunes depends on their height and the velocity of the grains moving along the bed-form surface.
- The transition zone, where the bed configuration may range from lower flow regime to upper flow regime, depending mainly on antecedent conditions. If the antecedent bed configuration is dunes, flows can increase to values more consistent with the upper flow regime without changing the bed form. Conversely, if the antecedent bed is plane-bed, flows can decrease to values more consistent with the lower flow regime without changing the bed form. Resistance to flow and sediment transport also have the same variability as the bed configuration in the transition zone.
- The upper flow regime, in which resistance to flow is small, and sediment transport is large. Typical bed forms are plane-bed or antidunes. The water surface is in phase with the bed surface, and normally the fluid does not separate from the boundary, except when an antidune breaks.

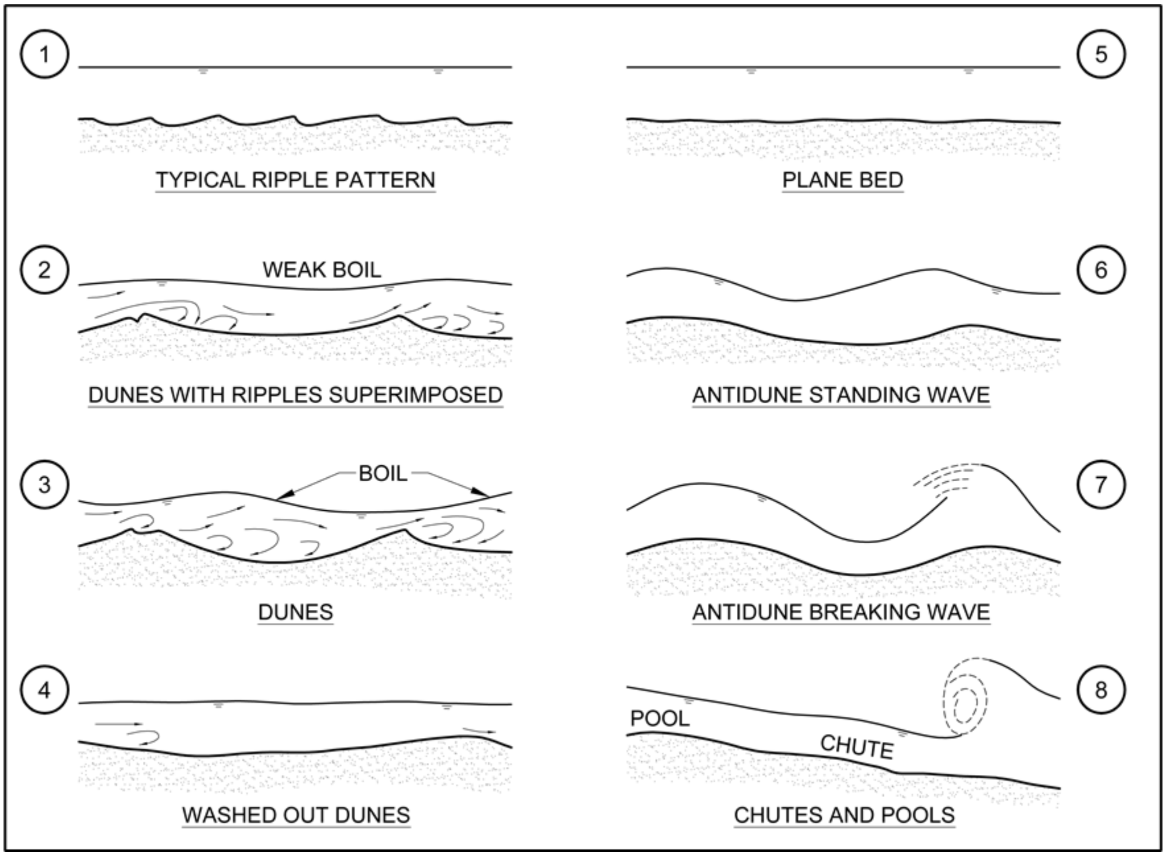

Figure 10.4. Bed forms in sand channels.

Effects of Bed Forms at Stream Crossings. As flows increase during the rising limb of the hydrograph, most sand-bed stream channels shift from a dune-bed to a transition or a plane-bed configuration. The shift in bed form decreases the resistance to flow. The increase in velocity and the corresponding decrease in depth may increase scour depths and the size of riprap required to prevent erosion.

Another effect of bed forms on highway crossings is that dunes on the bed form a fluctuating pattern of scour. Julien and Klaassen (1995) and Karim (1999) present methods for computing bed-form geometry. Karim included laboratory and field data where the crest-to-trough height for dunes ranged from ten to 50 percent of the flow depth. Karim also showed a range of antidune heights between ten and 40 percent of the flow depth. Bennet (USGS 1997) indicated an approximate upper limit as 40 percent of the flow depth. The average dune height equation by Julien and Klaassen is:

$$\frac{\Delta}{y} = 2.5\left(\frac{D_{50}}{y}\right)^{0.3} \tag{10.7}$$

where:

Δ	=	dune crest-to-trough height, ft
D_{50}	=	Median sediment grain size, ft
y	=	Flow depth, ft

Dune lengths can be approximated as 6.5 times the flow depth (Julien and Klaassen 1995). The Manning's n for dune-bed conditions can be more than double that for a plane bed (see Figure 10.5). A change from a dune bed to a plane bed, or the reverse, can have an appreciable effect on depth and velocity. In designing a bridge or a scour countermeasure, it is good engineering practice to assume a dune bed (large n value) when establishing the water surface elevations and a plane bed (low n value) for calculations involving velocity (FHWA 2012b).

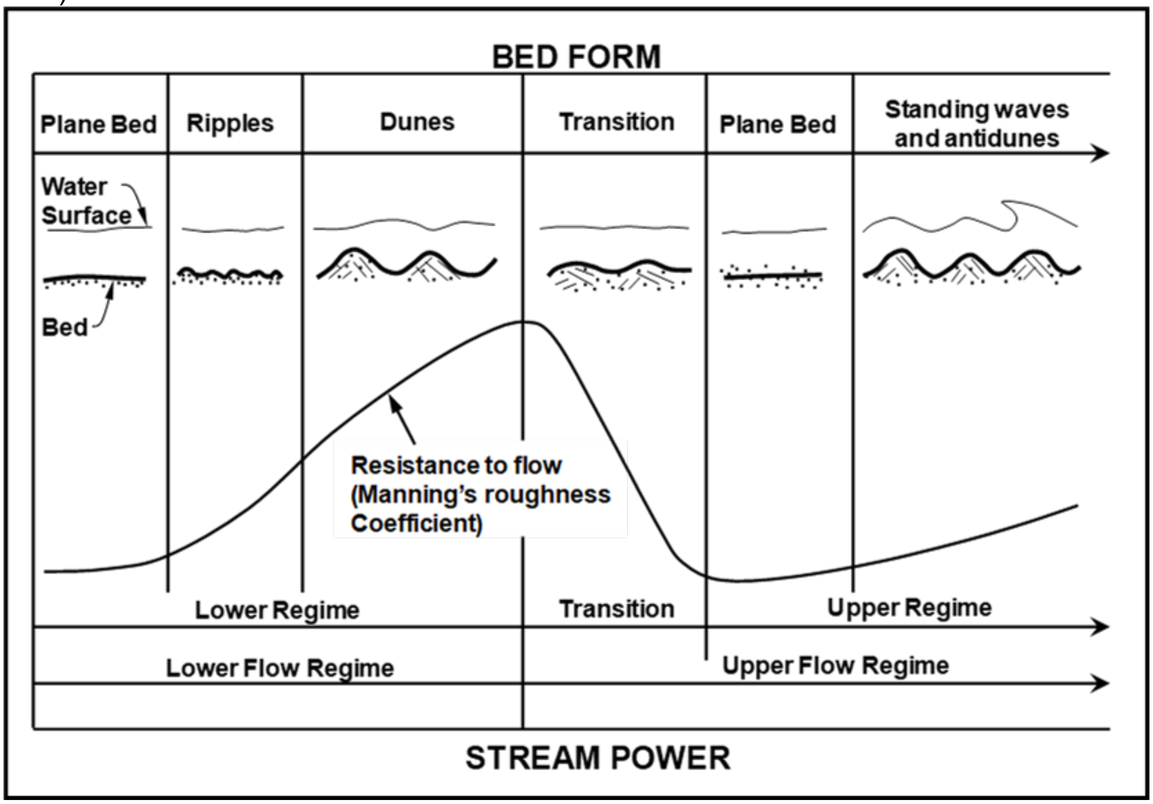

Figure 10.5. Relative resistance to flow in sand-bed channels (after USGS 1989).

10.5 Overview & Selection of Sediment Transport Equations

Researchers through past decades have created multiple bed material transport equations for the various modes of transport. For example, ASCE (2008) discusses sixteen bed load equations. The Meyer-Peter and Müller (1948) equation is a classic bed load equation that engineers still use frequently. The equation has the basic form of:

$$q_b = a\left(\tau_0 - \tau_c\right)^{3/2} \tag{10.8}$$

where:

q_b	=	Bed load discharge per unit width of channel, ft^2/s
a	=	Empirical coefficient

As with the analysis of incipient motion, the bed shear value in the equation typically incorporates only the grain friction portion of the roughness. Many of the equations presented in ASCE (2008) include excess shear stress to the 1.5 power. Because bed shear is proportional to velocity squared (see Equation 10.4), bed-load dominated sediment transport, such as in gravel-bed rivers, is generally proportional to velocity cubed once the bed is mobilized.

The Colby (1964) graphical method for bed material load in sand-bed rivers is another classic method for estimating sediment transport. Suspended sediment transport is dominant for most flow conditions in sand-bed channels. The Colby curves follow a trend of sediment discharge proportional to velocity to the power of 3.5 to 6. These large powers indicate that suspension is highly effective in transporting sediment in sand-bed channels because much of the suspended sediment moves at the average flow velocity.

The Meyer-Peter Müller Equation and the Colby curves illustrate a high sensitivity to the flow velocity. Uncertainty in velocity leads to greater uncertainty in sediment transport calculations. For example, a 10 percent change in velocity can result in a 30 to 80 percent change in sediment transport rate. Therefore, inaccuracies in hydraulic models amplify inaccuracies in sediment transport results.

The bed load increases as flow velocity and shear stress increase, but the suspended load increases more rapidly and can easily dominate the sediment transport process. The bed load is transported in a small fraction of the flow depth (usually taken to be twice the median sediment diameter), and the flow velocity (and bed load velocity) is low near the bed. The flow carries the suspended load through more, and potentially all, of the flow depth (see Figure 10.3). Velocity quickly increases with distance above the bed, so suspended load moves downstream much faster than bed load.

The Colby method provides insight into the sediment transport process. However, Einstein (1950) investigated the suspended load more rigorously. The basis of the Einstein suspended load equation is:

$$q_s = \int_a^{y_0} (v \times c)\,dy \tag{10.9}$$

Where the variables are defined as in Equation 10.6 and:

q_s	=	Suspended load discharge per unit width, ft^2/s
v	=	Velocity at height y above the bed, ft/s

A solution of the integral uses Equation 10.6 for sediment concentration and a vertical velocity profile equation. Figure 10.6 illustrates the concentration and velocity profiles in the water column. The integration depends on a reference concentration determined from the bed load. ASCE (2008) presents nine equations for determining the reference concentration and an easily applied equation (Abad and Garcia 2006) to solve the integration of Equation 10.9. The rate of bed load transport and the concentration profile depend on grain size. Therefore, the method involves performing the integration for the range of grain sizes in the bed material, and the total bed material load is the sum of the proportionate transport rates computed for each size class. Julien (2010) used Equation 10.9 to show that bed load comprises 80 percent or more of the total load when shear velocity divided by fall velocity is less than 0.5, and that suspended load comprises 80 percent or more of the total load when shear velocity divided by fall velocity is greater than 2.0.

ASCE (2008) also presents six empirically based equations for determining total sediment load. These equations have the advantage of being more easily applied. But they are appropriate for use only in conditions similar to the data used in their development or when supported with comparisons to measured data. This concept applies to the use of any sediment transport equation. The HEC-16 document (FHWA 2023a) describes several sediment transport equations and the conditions used in their development. The list is not exhaustive because it focuses on equations in non-proprietary sediment transport models.

The velocity increases from the bed to the water surface following a logarithmic curve. The sediment concentration is highest at the bed and decreases to its lowest value at the water surface.

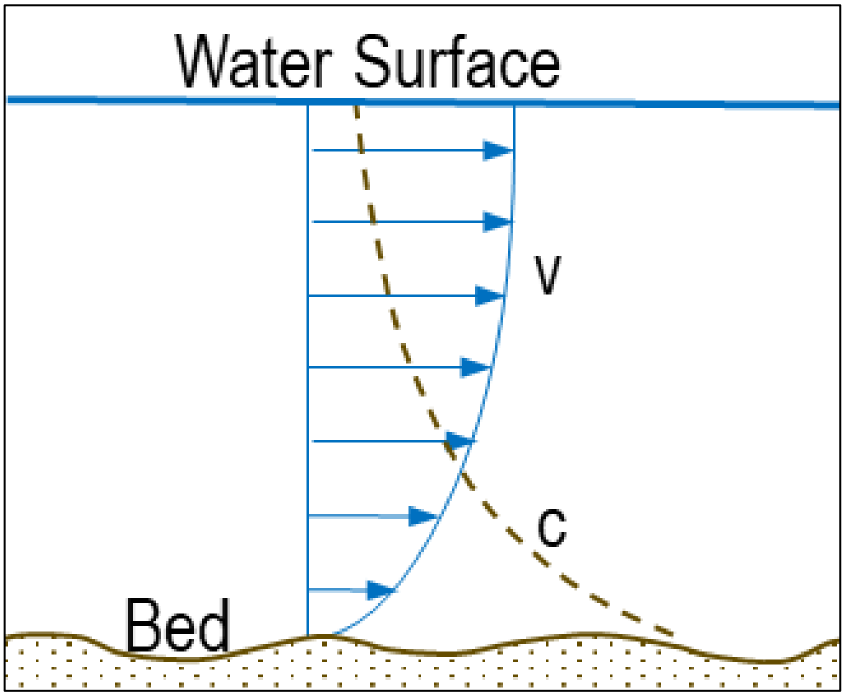

Figure 10.6. Velocity and sediment concentration profiles.

There are several ways of expressing and calculating sediment transport rates. These include volumetric (cfs, m^3/s), mass and weight (tons/day, metric-tons/day), and concentration (ppm, mg/l, sediment volume/total volume, and sediment weight/total weight). HEC-16 provides equations for converting between these expressions (FHWA 2023a).

10.6 Overview of Sediment Transport Modeling Applications for Bridge Projects

As described in Section 10.2, engineers undertake sediment transport analyses and modeling to address specific questions and concerns related to bridge designs. These include contraction scour, long-term degradation and aggradation, effects of structure replacement on channel stability, uninterrupted sediment transport through a bridge, and channel restoration as part of a bridge design project. The model selected for the analysis depends on the situation and model strengths and limitations. The situation also dictates model extents, simulation time period, and types of boundary conditions. Considerations common to all sediment transport simulations are:

- Well-defined hydrologic conditions.
- Accurate representation of hydraulic conditions.
- Appropriate model upstream and downstream extents.
- Identification of sediment inflow and, if applicable, other sediment sources.
- Identification of bed sediment sizes and, if applicable, sediment layers.
- Selection of an applicable sediment transport relationship.

Sediment transport is very sensitive to hydraulic variables, especially velocity. Two-dimensional modeling provides more accurate hydraulic results than 1D modeling. Therefore, 2D sediment transport modeling is more suitable for a wide range of hydraulic conditions, including many bridge hydraulic designs, ranging from relatively straightforward to complex. ASCE (2008) indicates that 1D sediment transport models are appropriate for simulations involving extended river reaches and extended time periods to determine a river's long-term response to natural or man-made changes. This is because of the computational efficiency of 1D models as compared to 2D models. However, as hardware and software continue to advance, 2D models are expected to be more broadly applied.

The following sections explain background and modeling considerations for the range of sediment transport applications listed above. HEC-16 (FHWA 2023a) discusses these considerations in greater detail. Each application requires the engineer to identify bed sediment sizes and sediment layers and select an appropriate sediment transport relationship. The other considerations often vary substantially depending on the application.

10.6.1 Contraction scour

As discussed in Chapter 9, scour is an aspect of hydraulic design that affects nearly all bridges over water. Contraction scour is an erosion and sediment transport process that occurs during flooding. When floodwaters flowing in the river channel and overbanks pass through the constricted bridge opening, the increased velocities lead to higher shear stress and sediment transport capacity. HEC-18 (FHWA 2012b) presents the live-bed and clear-water contraction scour equations. Live-bed contraction scour assumes that the upstream, unconstricted flow is transporting sediment at the hydraulic capacity of the flow. Clear-water contraction scour assumes that there is no transport of sediment at bed-material sizes in the upstream flow.

Sediment transport models can simulate live-bed and clear-water contraction scour. Sediment transport modeling of contraction scour is not a common practice, but it can be helpful when hydraulic conditions are such that the HEC-18 equations would yield unreliable results. Another reason for using sediment transport modeling is that the HEC-18 equations assume the scour reaches its ultimate depth in the design event. By contrast, sediment transport modeling can provide information on the time to reach the ultimate scour. Another reason to consider sediment transport modeling is when bridge backwater is substantially reduced because of the contraction scour. Contraction scour enlarges the bridge opening and reduces the velocity in the bridge. Therefore, the actual backwater caused by a bridge can be less than a fixed bed model predicts. A sediment transport model with a mobile bed can better represent actual flow conditions at the bridge. Figure 10.7 illustrates this concept, showing the water surface profile from a fixed-bed model run of natural (no bridge) and bridge-constricted conditions. The profile also shows the contraction scour and water surface profile predicted by a mobile-bed model incorporating the bridge constriction. The backwater caused by the bridge is less in the mobile-bed model.

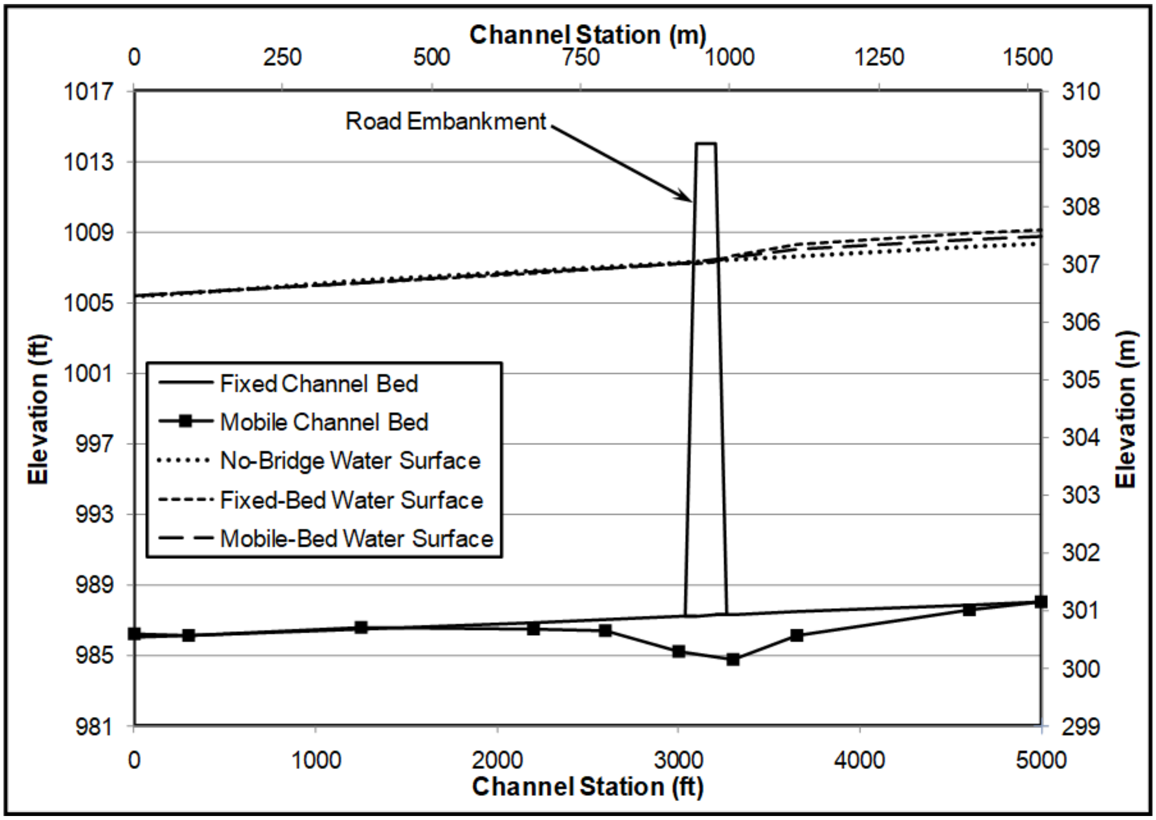

Figure 10.7. Contraction scour and water surface for fixed-bed and mobile-bed models.

The model development considerations for analyzing contraction scour with a sediment transport model include:

- Hydrology: Constant peak discharge or complete hydrograph for the scour design event
- Hydraulics: 2D modeling preferred unless the hydraulic conditions are simple and well represented in a 1D model
- Model Extents: Upstream limit is set to establish the sediment inflow, and downstream limit is set to define the hydraulic boundary condition outside the influence of the contraction scour.
- Sediment Inflow Boundary Condition: Equal to sediment transport capacity.

10.6.2 Long-Term Degradation and Aggradation

The potential for long-term degradation or aggradation is an important bridge design consideration. It can also be very challenging to estimate. Because degradation and aggradation are river system responses to changing basin conditions, the sediment transport modeling often includes large model domains and long time periods. This type of analysis also often includes many sediment sources and non-stationary hydrology. Long-term sediment transport simulations often include:

- Hydrology: The analysis requires daily flow hydrographs extending over many years and often decades.
- Hydraulic Analysis: The analysis usually employs a 1-D model.

- Model Extents: The upstream and downstream extents need to be sufficient to establish the sediment inflow and water surface outside the influence of the reach-scale bed changes. In many cases, the model domain can be tens to hundreds of miles long.
- Sediment Boundary Conditions: These boundary conditions can be the sediment transport capacity or an established sediment rating curve. The analysis often includes tributary sediment inflow sources.

10.6.3 Impacts of Structure Replacement on Future Channel Stability

Existing structures that are undersized can interrupt sediment conveyance along the channel, causing deposition upstream of the structure and erosion downstream of the structure (see Figure 10.8). A hard-bottom culvert can act as a grade control by keeping downstream channel lowering from progressing upstream. These conditions often limit aquatic organism passage into the upstream channel network. When these structures are replaced with a bridge, three-sided culvert, or a culvert with a depressed bottom, the channel can respond both upstream and downstream.

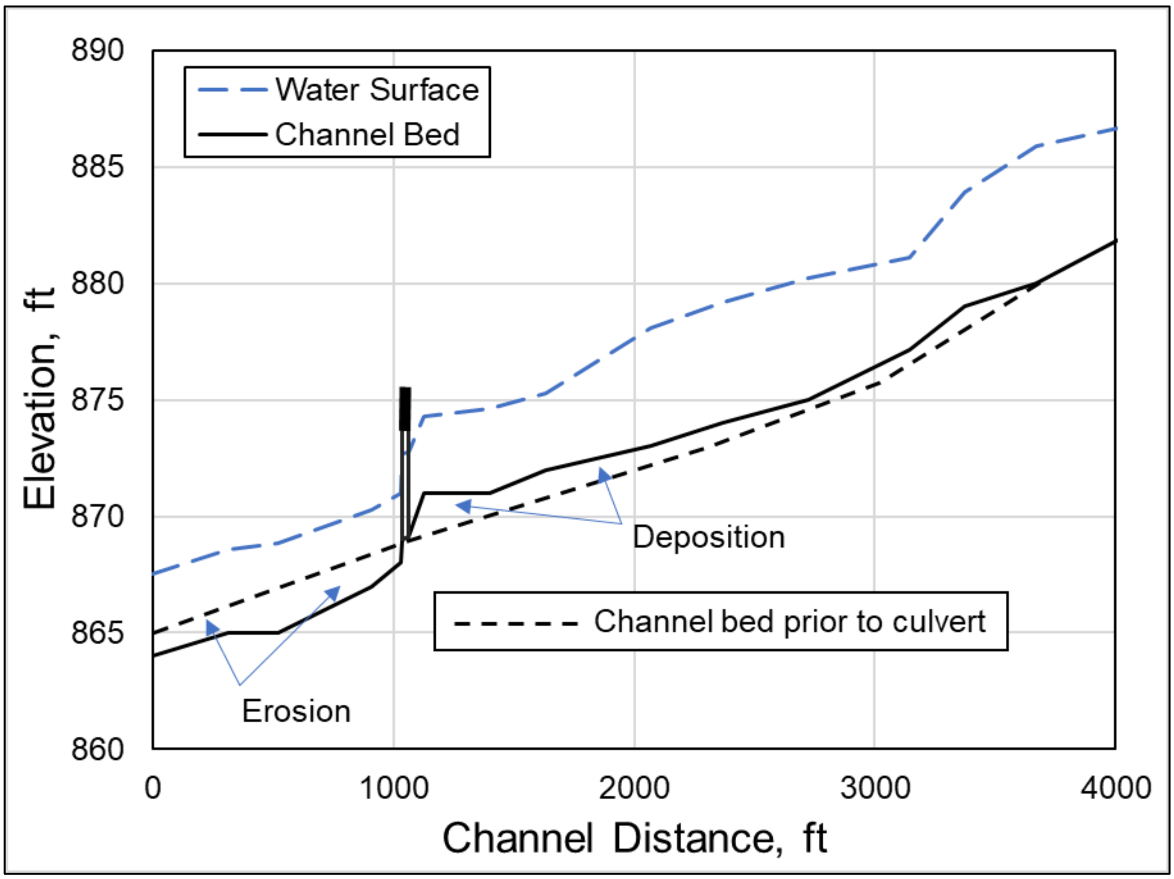

Figure 10.8. Long-stream channel bed and water surface profiles for a culvert with deposition upstream and erosion downstream.

It is often possible to determine the impacts of the existing structure by evaluating the longitudinal stream bed profile, especially if the pre-construction profile is available for comparison. It may be necessary to evaluate the potential upstream and downstream channel response to a structure replacement project. This is another application for sediment transport analysis. This type of analysis requires:

- Hydrology: The analysis requires daily flow hydrographs extending over several years.
- Hydraulics: The analysis often employs 1D models, but 2D models are more suitable when the extents of the impact are not excessive.
- Model Extents: The upstream and downstream extents need to be sufficient to establish the sediment inflow and water surface outside the influence of the reach-scale bed changes and to encompass an area greater than the anticipated response reach.
- Sediment Boundary Conditions: These boundary conditions can be the sediment transport capacity or an established sediment rating curve. The analysis often includes tributary sediment inflow sources.

10.6.4 Maintaining Sediment Conveyance Through a Structure

Avoiding the impacts illustrated in Figure 10.8 is a consideration for many bridge projects. Even though the channel bed through a bridge can respond to changing watershed conditions, a structure could still interrupt long-stream sediment conveyance resulting in upstream deposition and downstream bed erosion.

HEC-16 (FHWA 2023a) explains a method for comparing long-term sediment supply to sediment transport capacity in a channel design reach. The method draws from the Capacity Supply Ratio (CSR) concept from Soar and Thorne (2001). Bledsoe et al. (2017) developed a spreadsheet tool to apply the method. The tool compares the long-term sediment transport capacity from a supply reach to that of a downstream design reach. It uses simple hydraulic calculations combined with sediment transport calculations over a flow record or flow duration curve. The tool determines a range of channel conditions (slope, width, depth, and sinuosity) for CSR values of 1.0, which indicates that the capacity of the design reach has the same long-term sediment transport capacity as the supply reach. Underpinning this approach as a design tool is the assumption that the design channel is free to adjust width, depth, and planform in the future in a state of dynamic stability. The inputs for this method include:

- Hydrology: The tool requires direct input of a flow duration curve or flow records to develop a flow duration curve representing long-term hydrology over the complete flow record.
- Hydraulics: Normal-depth hydraulics are computed for the supply and design reaches.
- Model Extents: The tool uses representative cross sections and slopes for the supply and design reaches.
- Sediment Boundary Conditions: The tool computes sediment transport capacity for the supply reach based on the bed-material grain size distribution.

10.6.5 Channel Restoration

Some bridge replacement projects include channel restoration activities. One goal of channel restoration is to design a channel that is in long-term balance with the upstream supply. The CSR tool described in the previous section was developed for this express purpose and the considerations described above apply as well. Because the channel restoration reach is likely outside the road right-of-way, there may also be greater freedom to develop a design that achieves long-term sediment transport balance and freedom for the channel to adjust. Sediment balance can also be achieved through the bridge reach and transportation corridor right of way but additional constraints, such as bridge width constraining channel widths, slopes, sinuosity, may limit the design alternatives.

10.7 Alluvial Fans

Alluvial fans are very dynamic sedimentary landforms that can create significant hazards to highways because of floods, debris flows, deposition, channel incision, and avulsion (Schumm and Lagasse 1998). The National Research Council Committee on Alluvial Fan Flooding (NRC 1996) defines an alluvial fan as a sedimentary deposit that is convex in cross-profile and located at a topographic break, such as the base a mountain, escarpment, or valley side. An alluvial fan is formed by stream-flow and/or debris-flow sediments and has the shape of a fan either fully or partially extended. Figure 10.9 shows an alluvial fan in Death Valley, California. As the bed material and water reaches the flatter section of the stream and exits the canyon, the coarser bed materials can no longer be transported because of the sudden reduction in slope, velocity, and channel confinement. Consequently, the deposited material builds a cone or fan out into the unconfined valley. The sediment loads are typically first deposited in the existing channels during storm events potentially blocking or restricting flow which in turn can lead to lateral migration or channel avulsion as flows are able to occupy adjacent areas of lower elevation. Alluvial fans often feature unstable channel geometries and rapid lateral movement resulting from the decline in slope and valley confinement. The NRC committee determined that alluvial fan hazards can include (1) flow path uncertainty below the fan apex, (2) abrupt deposition and ensuing erosion of sediment as a stream or debris flow loses competence to carry material eroded from the steeper, upstream source area, and (3) hazardous conditions and risks that elevation or fill will not reliably mitigate.

Figure 10.9. An alluvial fan formed where Wineglass Canyon enters Death Valley, California, used by permission from W. B. Miller (1998).

Potential avulsions, deposition, channel blockages, and channel incision present challenges for highway design at alluvial fans. A reconnaissance of the fan and its drainage is needed to identify the potential changes, allowing the highway design team to develop countermeasures to minimize these impacts. FEMA's *Guidelines for Determining Flood Hazards on Alluvial Fans* contains helpful information on recognizing alluvial fan landforms and methods for defining active and inactive areas (FEMA 2000). Any study of alluvial fans should include a geomorphic map delineating active and inactive portions of the fan and the identification of problem sites within the active portions of the fan. For example, local aggradation in a channel can lead to avulsion because avulsion is likely to occur in places where deposition has raised the floor of the channel to a level that is nearly as high as the surrounding fan surface. This condition can be identified in the field by observation or by the surveying of cross-fan profiles (Schumm and Lagasse 1998).

French (1987) cautions that alluvial fan hydraulics are highly unsteady and 2D in nature. Analysis of hydraulic and sediment transport conditions on alluvial fans should include in-depth geomorphic evaluation. Historically, FEMA used probabilistic methods of hazard delineation (Dawdy, 1979) for the National Flood Insurance Program (NFIP). ASCE (2008) indicates that 2D models are available for modeling flow and sediment transport on alluvial fans, specifically mentioning FLO-2D (Obrien 2009). Debris flow modeling was also recently added as a 2D modeling capability within HEC-RAS 6.0.

10.7.1 Hazard Mitigation Measures for Alluvial Fans

The only way to eliminate the geomorphic and flood-related risk to infrastructure adjacent to or on an alluvial fan is to avoid the fan entirely. However, this may be impractical or impossible for some projects. In these instances, design teams responsible for transportation projects impacted by alluvial fans have an assortment of potential approaches to mitigating and reducing the potential hazards. The mitigation techniques generally include 1) roadway alignment, 2) sediment control and conveyance, and 3) monitoring, operations, and maintenance.

Roadway Alignment: The road alignment may depend on the highly diverse habitat found on a specific alluvial fan. However, it is vital to understand the dynamics of each possible alignment. An apex alignment is located as close as possible to the upstream apex of the fan. Here, the channel is still confined by the topography, reducing the ability of the stream to avulse or migrate laterally. This alignment would entail a single-span bridge or culvert but would also maximize the magnitude of the flows, hydraulic forces, and sediment loads. A crossing at this location would likely require a relatively large structure with erosion countermeasures. However, this alignment benefits from greater certainty regarding the location of the flow path and the flow magnitude.

A mid-fan alignment follows a route midway between the apex and the toe of the fan and crosses multiple channels or flow paths. Dividing the primary channel into several branches means a reduction in the magnitude of flows and sediment loads at each individual crossing, potentially resulting in smaller structure sizes. However, the lack of channel confinement at this location subjects the road and crossings to risks associated with soil instabilities, lateral migration, and avulsion. Additionally, over the service life of the crossings, it is conceivable that any one of the channels on the fan may incise enough to capture the entire flow, potentially overwhelming a single crossing. Generally, mid-fan alignments are not preferred because uncertainties, design challenges, and construction costs are higher than for an apex alignment.

A toe alignment runs along the toe, or distal edge of the alluvial fan. Here, the uncertainty in the positions and magnitudes of flow crossing locations is highest. However, flow magnitude at any given location is, in theory, minimized due to the increased potential for flow dispersion and infiltration over the fan. Furthermore, due to the distance from the fan apex, most of the coarse sediment, wood, and debris is expected to be deposited on the upper portions of the fan before reaching the road, reducing the risk of bulking flows, blockages, and avulsion. Like the mid-fan alignment, this alignment will have a series of smaller crossings. Again, these smaller crossings are at risk of being overwhelmed if incision occurs and allows for a rapid increase in conveyance potential.

Sediment Control & Conveyance: Alluvial fans are an area of instability and transition between mountain canyons, which are high-energy, sediment source regions, and larger valley bottoms or plains regions. This transition from high velocity and transport capacity to low velocity and transport capacity leads to significant amounts of deposition which forms these features. Source control, managing the sediment supply, can reduce the risk of filling existing, usually more stable, channel, which forces the stream to respond by migrating lateral or avulsing. Source control, at its simplest, means limiting land-use disturbances within the watershed and maintaining well-vegetated riparian corridors. However, transportation departments usually do not have authority over watershed land use. Natural and human activities, like wildfire, insect damage, over-grazing, mass wasting events, unsustainable forest management, and land development, can drastically increase sediment yields in the watershed.

In some circumstances, the risk to a highway or community is sufficient to warrant the construction of structures to trap and retain source sediment, wood, and flow. This usually involves building a dam, drop structure, sediment trap, or debris basin (Zech et al., 2014) upstream of an alluvial fan. The traps and basins are emptied periodically or after significant flood events. Long-term costs and environmental impacts of operation, maintenance, and disposal of material collected by storage structures are significant considerations for road project sustainability (Zech et al., 2014).

Conveyance structures for controlled passage of flood and debris flows on alluvial fans, such as armored channels, flumes, or guide banks, have been used in numerous locations. Figure 10.10 shows a community near Palm Desert, CA that has constructed guide banks on an alluvial fan to consolidate the irregular flow path and an armored channel with check structures to convey the flow through the development and control the sediment deposition. The variable density and viscosity properties associated with mud and debris flows create significant challenges and uncertainties when designing these facilities. Management and removal of the deposited materials at the downstream ends of these conveyance structures is also a key component to the success of conveyance structures.

Long-term Monitoring Operations and Maintenance: Alluvial fans are dynamic river landforms that constantly evolve in response to variable inputs of water, sediment, wood, and debris supplied from the contributing watershed upstream. Given the unique and time-variant characteristics of alluvial fans, locating transportation infrastructure on or adjacent to an alluvial fan involves a long-term commitment to inspection, monitoring, and maintenance to make the highway as safe, resilient, and reliable as possible. Depending on site-specific circumstances, effective long-term programs may include:

- Assessment of hydraulic conveyance capacity of all crossing structures such as bridges and culverts. Excessive degradation or aggradation are a typical signs of conveyance issues and should be noted adjacent to all structures.
- Assessment of the hydraulic conveyance and location of flow paths on the fan as they relate to the crossing structures.
- Inspection, cleaning, and repair sediment control and conveyance structures protecting the roadway corridor.

Figure 10.10. An aerial view of a development on the southern edge of Palm Desert, CA that has mitigated the risk associated with alluvial fans by constructing guide banks that lead to an armored channel with check structures.

CHAPTER 11
OTHER BRIDGE HYDRAULICS TOPICS AND CONSIDERATIONS

11.1 Hydraulic Forces on Bridge Elements

Bridge design engineers analyze the bridge's stability under various loading conditions. Rivers, streams, and coastal water bodies exert significant forces on bridge structures during floods, storm surges, or wave attack. The hydraulic forces potentially acting on a bridge include hydrostatic, buoyancy, drag, and wave forces. Impact by vessels and forces exerted by debris or ice are also closely tied to hydraulics. Bridge designers require information from the hydraulic analysis results to evaluate the hydraulic forces on bridge elements.

Bridge designers typically follow the *AASHTO LRFD Bridge Design Specification*s 9th Edition (herein referred to as the *LRFD Specifications*) (AASHTO 2020), with some State-specific modifications, in evaluating forces and loads on bridges. This section briefly summarizes the guidelines of *LRFD Specifications*, along with information and insights from other references.

11.1.1 Hydrostatic Force

The weight of water exerts hydrostatic pressure in all directions. The pressure value is the product of the height of water above the point of consideration and the unit weight of water. Thus, the pressure is greatest at the lowest point of a submerged element and is zero at the water surface elevation.

The hydrostatic force acting on a bridge element in a particular direction is the summation, or integral, of the product of the pressure and the surface area of the bridge element projected in the plane perpendicular to the direction of the force. Hydrostatic forces on one side of a bridge are partly or entirely balanced by opposing hydrostatic forces acting on the other side. Any imbalance in the hydrostatic force is due to variation in the water surface elevation. Informing bridge designers of the water surface elevation upstream and downstream of the bridge for the design flood enables them to evaluate the hydrostatic forces.

11.1.2 Buoyancy Force

Buoyancy is an uplift force equivalent to the weight of water displaced by the submerged portion of any bridge component. It can pose a threat to a submerged bridge superstructure if the superstructure design incorporates large, enclosed voids as with a box-girder or if air pockets develop between girders beneath the deck. As discussed later in this chapter, buoyancy is also a factor in evaluating wave-related forces on bridge decks. If a pier includes a large empty void, the buoyant uplift force acting on the pier may be significant. Informing bridge designers of the water surface elevation upstream and downstream of the bridge for the design flood enables them to evaluate the hydrostatic forces.

11.1.3 Stream Pressure on Piers

The *LRFD Specifications* uses the term "stream pressure" for the pressure associated with the drag exerted on the structure by flowing water. To compute stream pressure on a pier by the *LRFD Specifications,* one multiplies the square of the flow velocity by a drag coefficient. The *LRFD Specifications* gives the same expression for both longitudinal and lateral stream pressure on a pier. It provides a set of drag coefficients, dependent on the shape of the upstream pier nose and whether debris is lodged against the pier, for longitudinal pressure. It provides a second set of drag coefficients, dependent on the skew angle of the flow, for lateral pressure.

The hydraulic engineer reports the magnitude and direction of the local impinging flow velocity at each pier for the design event, as well as the flow depth and debris accumulation potential so that the bridge designer can compute the stream pressure on piers.

11.1.4 Stream Pressure and Lift on Submerged Bridge Superstructures

Bridge designs usually place the superstructure above the design flood elevation. However, some projects may require analyzing forces for flood scenarios that submerge all or a portion of the bridge superstructure. As described in the document *Hydrodynamic Forces on Bridge Decks* (FHWA 2009c) the FHWA applied physical and CFD modeling to investigate the hydrodynamic forces on inundated bridge decks, specifically:

- The drag force acting parallel to the flow direction and tending to push the superstructure off the piers and the abutments.
- The lift force acting vertically and tending to lift the superstructure.
- The overturning moment resulting from unevenly distributed forces and tending to rotate the superstructure about its center of gravity.

The physical modeling and CFD modeling focused on three different superstructure design types: one with six flanged girders, one with three larger rectangular girders, and a third with a highly streamlined cross-sectional shape. Figure 11.1 shows a CFD results plot for a six-girder bridge model.

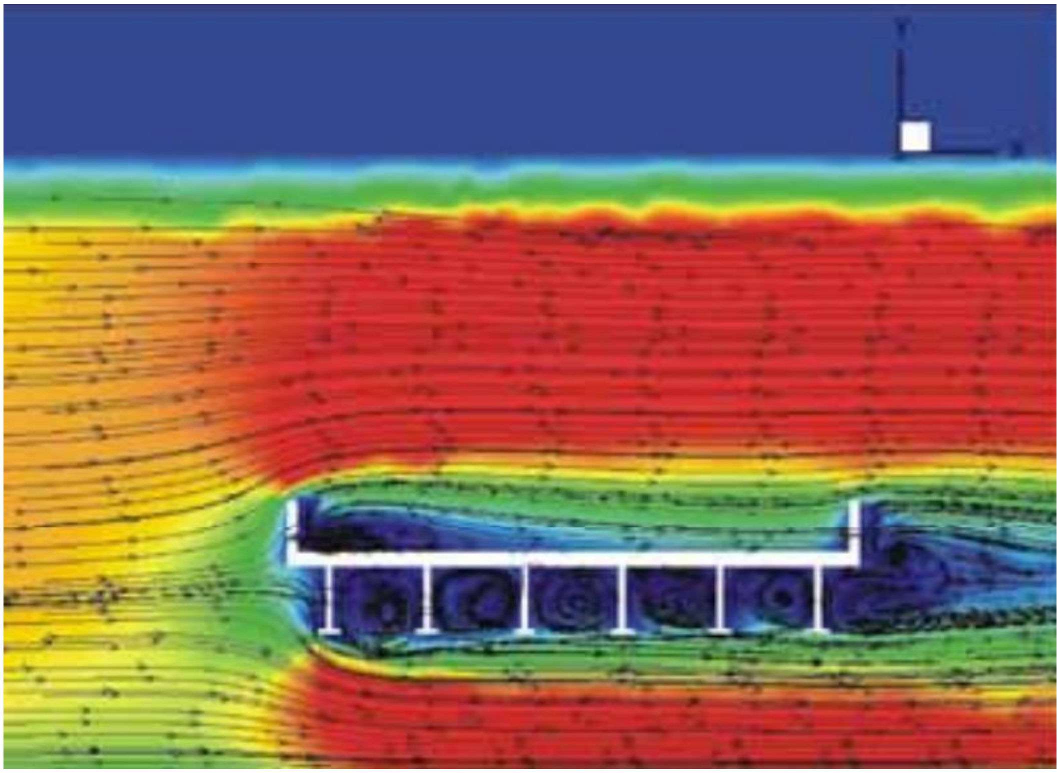

Figure 11.1. CFD results plot showing velocity direction and magnitude from a model of a six-girder bridge (from FHWA 2009c).

The *Hydrodynamic Forces on Bridge Decks* (FHWA 2009c) document provides equations for determining the drag coefficient, lift coefficient, and moment coefficient as functions of the inundation ratio for each superstructure type. Bridge designers can use the equations in this report to evaluate the bridge's stability for large flood events that would inundate the superstructure.

A bridge designer requires certain information from the hydraulic analysis to determine the forces, including the water surface elevation, depth, and velocity upstream of the bridge. One can adjust the drag coefficient to accommodate a bridge superstructure type other than a six-girder or three-girder bridge design.

11.1.5 Wave Forces

Chapter 8 briefly discusses considerations for bridges in coastal settings and briefly discusses waves and their potential impacts on bridges. Waves striking a bridge superstructure impart forces acting both horizontally and vertically. The magnitudes of the forces depend on several factors, including the tide level, storm surge, and properties of the anticipated waves. The *Guide Specifications for Bridges Vulnerable to Coastal Storms* (AASHTO 2008) provides guidelines for designing bridges subject to wave forces in coastal settings. HEC-25 (FHWA 2020) presents an alternative method for estimating wave loads on bridge decks. A bridge designer estimating wave loads requires certain information from a coastal engineer about the tidal hydraulics and the wave setting:

- The maximum probable wave height for the design event.
- The wave length.
- The wave period.
- The upwind fetch over which wave can be generated.
- The water surface elevation at the bridge for the design storm tide, including local wind setup where appropriate.
- The stream bed elevation at the bridge.
- The current velocity from hydrodynamic modeling for the design event.

Hydrodynamic models provide the storm tide height, current velocity, and wave properties for each design event considered. Absent numerical wave modeling, the *Coastal Engineering Manual* (2002) provides empirical methods for calculating wave heights. The wave properties depend on the wind speed, duration and direction, the upwind fetch length, the water depth at the bridge, and the water depth over the fetch.

11.1.6 Effects of Debris

Debris accumulations on bridges can dramatically increase the hydraulic forces exerted on bridge piers and superstructures. The *LRFD Specifications* (AASHTO 2020) provides guidance on incorporating debris potential into the stream pressure calculations by assigning a drag coefficient and estimating the cross-sectional area of the debris blockage.

A research project by the NCHRP used physical modeling to examine debris forces on bridges. The report, titled *Design Specifications for Debris Forces on Highway Bridges* (NCHRP 2000), recommends separate evaluations of the drag force and hydrostatic force from debris accumulations. For evaluating the drag force, the report provides envelope curves and tables to aid in assigning the drag coefficient for debris on piers and superstructures. The drag coefficient is a function of the blockage caused by the debris and the Froude number in the contracted section (the bridge waterway). The report also provides valuable guidance on selecting the reference velocity for use in the drag force or stream pressure calculations. The difference in water surface elevation from the upstream side of the debris accumulation to the downstream side of the bridge is an essential factor in the hydrostatic force calculation.

Another research project by the NCHRP used field observations, a photographic database, and extensive physical modeling to investigate the effects of debris on bridge pier scour. The resulting report, titled *Effects of Debris on Bridge Pier Scour* (NCHRP 2010c), provides refined guidance on estimating the potential dimensions of debris flow blockage, on incorporating debris into one- and 2D models, and on computing an effective pier width for pier scour calculations based on the estimated debris dimensions. When the potential for debris accumulation on the bridge is significant, the hydraulic engineer provides the bridge designer with the estimated dimensions and reference elevation of the potential debris blockage. The hydraulic engineer should also recommend an appropriate drag coefficient for the debris, based on the NCHRP Report.

11.1.7 Effects of Ice

When ice accumulates at a bridge and forms an ice jam, significant problems can develop. Some of the negative consequences include bridge scour and bank erosion, even during times of low streamflow. Ice jams also impart significant lateral forces on the bridge. Like debris blockages, ice jams magnify the stream pressure forces by increasing the surface area of pressure application. An inordinate amount of backwater often accompanies ice jams, which increases the hydrostatic force. The elevation of ice accumulation significantly influences the bridge stability calculations. The LRFD Specifications provides an extensive discussion on the evaluation of ice forces.

The design team should perform site-specific research to assess whether ice jamming is relevant. If it is a concern, the hydraulic engineer develops hydrologic and hydraulic information to assist the bridge designer in evaluating ice forces. It may be beneficial, for instance, to determine the months of the year when ice jamming is most likely to occur. Streamflow records would then be studied to assess the potential for flooding during the most likely ice jamming months and to identify a reasonable yet conservative flow rate for assessing the potential elevation of an ice jam on the bridge. Field reconnaissance may reveal evidence of the elevation range of ice jam formation. The Transportation Association of Canada has published the *Guide to Bridge Hydraulics* (TAC 2004), which includes information on estimating the stage and thickness of ice jams. The USGS and South Dakota Department of Transportation conducted field research to evaluate factors affecting ice forces at selected bridges in South Dakota (SDDOT 2002), focusing on the maximum ice thickness and ice crushing strength. The research report provided equations to estimate ice thickness for South Dakota bridge designs.

Ice can exert other forces on a bridge beside the increase in stream pressure and hydrostatic force mentioned above. Large ice floes striking bridge piers can generate significant impact forces. Large sheets of ice can experience thermal expansion, generating lateral pressure on the bridge. Ice adhering to the bridge structure during water level increases can impart uplift forces. The hydraulic engineer can assist the bridge designer in assessing the potential range of water levels associated with these forces.

11.1.8 Vessel Collision

The potential for impact forces from vessel collisions is a design consideration for bridges crossing navigable waterways. Bridge designs, wherever practicable, minimize the probability of a vessel impact. Advisable practices include providing appropriate vertical clearance above the water surface, keeping piers as far away from navigation channels as practicable, and avoiding the placement of piers near a bend in a navigation channel. Navigating large ships and barges can be very difficult at bends, especially in high-velocity waterways. Locating one or more bridge piers near a bend in a high-velocity waterway with barge or ship traffic dramatically increases the risk of a vessel impact.

After taking appropriate precautions in locating the bridge and bridge piers, it is still necessary to allow for some probability of vessel collision. The type of vessel to be considered depends on the waterway and the typical boat traffic. The AASHTO *LRFD Bridge Design Specifications* (AASHTO 2020) and the AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges (AASHTO 2010) describe methods for selecting an appropriate design vessel and assessing the probability of a vessel collision. The bridge designer typically evaluates more than one vessel collision scenario.

One potential scenario spelled out by the AASHTO *LRFD Specifications* is a case of an empty barge breaking loose from its mooring and hitting a bridge pier under peak 100-year flood conditions. The flood conditions include the presence of half of the long-term scour (long-term degradation) and half of the flood-specific scour (contraction scour and pier scour) coincident with the vessel striking the pier. The hydraulic engineer informs the bridge designer of the peak 100-year flood velocity, flow direction, depth, water surface elevation, long-term degradation, and total scour to enable evaluation of the impact force. The required velocity is the local velocity impinging on the pier in question. It is usually appropriate to report the same velocity, flow direction, and depth used in the scour calculations when providing information for vessel impact forces under flood conditions.

Another commonly considered case is a fully loaded vessel transiting along the navigation channel and errantly striking the bridge during typical waterway conditions. The AASHTO *LRFD Specifications* states that the appropriate velocity and water surface for such a scenario are those associated with yearly mean conditions, combined with half of the estimated long-term scour depth. If streamflow records are available for the stream reach, one can use the annual mean of the daily mean flow rates to represent yearly mean conditions. In a tidal waterway, it is more appropriate to select one or more specific tidal levels, such as mean high water, to represent typical waterway conditions.

Bridge piers located in the vicinity of seaports or major shipping channels are potentially exposed to enormous vessel impact forces that are not readily accommodated in the bridge structure design. In such cases, it is common to incorporate separate structural dolphins, with or without fender racks, to prevent a bridge impact. Dolphin installations can potentially aggravate the scour potential at the bridge piers they are protecting, which justifies careful hydraulic analysis at the design stage.

11.2 BRIDGE DECK DRAINAGE DESIGN

11.2.1 Objectives of bridge deck drainage design

Appropriate bridge deck drainage design protects public safety, supports efficient traffic flow, and prevents or minimizes water-related damage to the bridge. Relevant design measures include the use of appropriate cross slopes and longitudinal slopes on the bridge deck, along with hardware such as inlets, scuppers, and drainage pipes. While the concerns and design approaches are comparable to roadway pavement drainage design, significant differences exist because of the physical and geometric constraints of installing a drainage system on a bridge.

FHWA document HEC-21 *Design of Bridge Deck Drainage* (FHWA 1993) provides extensive information to aid in designing deck drainage systems. This section briefly summarizes the design considerations for bridge deck drainage, drawing heavily from HEC-21.

11.2.2 Bridge Deck Drainage Considerations

Minimizing Spread Width and Flow Depth: Runoff flow spreading into traffic lanes on a bridge deck causes safety risks and reduces traffic service levels. The flow encroaching into traffic lanes, if deep enough, can cause hydroplaning, an extremely hazardous condition in which a film of water separates vehicle tires from the road surface. The spread width and depth of flow are both functions of the runoff discharge, the shoulder width, the cross slope of the deck, and the longitudinal grade of the bridge. Figure 11.2 is a cross-section sketch illustrating the concept of spread width.

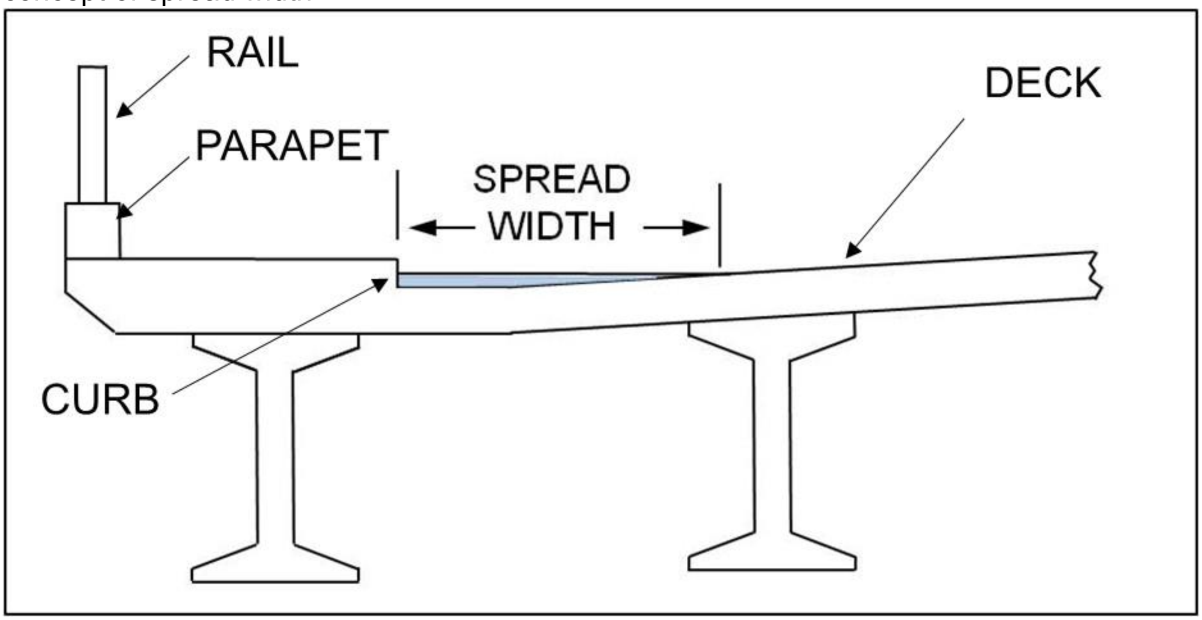

Figure 11.2. Sketch illustrating spread width of bridge deck drainage.

The design can mitigate excessive spread width by removing all or a portion of the runoff from the deck surface. Various types of inlets or scuppers can remove runoff. Handling the flow removed from the deck surface comes with its own challenges and considerations, as described later in this section. Keeping the spread width and depth below objectionable amounts may require multiple scuppers or inlets. Shorter bridges or those on a steeper slope may not require inlets anywhere along the deck to achieve acceptable spread width. HEC-21 (FHWA 1993) describes a methodology and provides equations for determining the inlet spacing requirements for a bridge.

Most transportation agencies have established design criteria regarding the acceptable spread width associated with the design rainfall event. The route classification, traffic flow, and design speed are factors in setting the spread width criteria. A typical requirement for high-speed, high-volume routes is that the spread width caused by the design rainfall event may not encroach beyond the shoulder into a traffic lane.

Superelevation Transitions: Superelevation transitions on a bridge deck can be problematic for drainage design. Potential issues include:

- Flow leaving the gutter on one side of the road and crossing to the other side.
- A sag in the gutter profile, causing water to pond.
- A locally flattened cross slope allowing excessive flow spread.

If the deck design includes a superelevation transition, the engineer may need to mitigate potential problems by placing one or more inlets or scuppers just upslope of the beginning of the transition.

Protecting Road Embankments at Bridge Ends: Erosion damage commonly occurs on the road embankment slopes adjacent to bridge ends because of inadequate control of bridge deck drainage. An appropriate design minimizes the problem by delivering the flow safely to the bottom of the embankment without erosion. HEC-21 advises intercepting gutter flow with roadway drainage inlets on the approaches at both ends of the bridge. The design conveys the intercepted flow by pipes to the bottom of the embankment or to an existing storm drain system. The design can allow some flow to bypass the specified roadway inlet if there is curbing of sufficient height and length to convey the bypassed flow to the next drainage inlet or erosion-protected outfall location, thus protecting the embankment from erosion damage.

Minimizing Drainage-Related Damage at Bridge Joints: Water seeping from the deck through bridge joints can corrode the girders, bearings, and substructure. For that reason, some transportation agencies require inlets on the bridge deck to capture flow before it runs across the joint at the downslope end of the bridge, even if there are no other inlets on the deck. The interception capacity of a single bridge deck inlet is quite limited. Therefore, the beneficial function of bridge deck inlets placed at the downslope end of a bridge deck is primarily to intercept nuisance flows such as runoff from minor rainfall events and snowmelt, rather than to keep joints dry during high-intensity rainfall events.

Design Rainfall Intensity for Bridge Deck Drainage: The runoff flow rate accommodated in bridge deck drainage design is directly related to the short-duration rainfall intensity, which is the expected temporal rainfall rate over a brief time period (usually 10 minutes or less). Transportation agencies typically link the criteria for acceptable spread width and protection of the embankments at the bridge ends to a standard design recurrence interval (AEP) for rainfall intensity. The 10 percent rainfall AEP appears commonly as the design standard for moderate- to high-volume roads. HEC-22 (FHWA 2009b) provides information to aid in selecting the design rainfall frequency for deck drainage.

11.2.3 Practical Considerations in Design of Bridge Deck Inlets and Drainage Systems

Dimensional Limitations of Bridge Deck Inlets: The inlets used in roadway pavement drainage applications are unsuitable for bridge deck applications because the structural dimensions of a bridge deck do not accommodate them. Roadway pavement drainage typically drops through a long curb opening or gutter grate into a large concrete catch basin, which discharges it through a pipe into an outfall or a storm drain system.

Bridge deck inlets, by necessity, usually have a smaller footprint on the bridge deck surface. Large openings may cause extensive complications in the design and construction of deck reinforcement. Bridge deck inlets are typically rectangular or round cast-iron grates that allow runoff to drop into shallow inlet chambers constructed of formed concrete, ductile iron, or welded steel. HEC-21 provides illustrations of several standard inlet configurations and explains the factors that affect the interception capacity of bridge deck inlets. Grates with bars parallel to the traffic direction are the most hydraulically efficient. However, many new bridges and bridge widenings accommodate bicycle traffic. Such bridges require bicycle-safe grates, which have bars perpendicular to the traffic direction. Vane grates are tilted or curved with the top edges inclined in the upstream direction and can make perpendicular-bar grates more efficient.

Handling Intercepted Runoff: The drainage removed from the deck either falls through a vertical scupper or enters an underdeck drainage system. A vertical scupper may discharge drainage directly into the air under the bridge or extend down to the ground in a downspout installed on a pier. Many situations prohibit discharging the runoff directly into receiving waters beneath the bridge due to stormwater quality concerns or regulations. In such cases, the engineer may design an underdeck drainage system to carry the intercepted flow to a storm drain system or an appropriate stormwater quality feature. Roads, railroads, residential, commercial, or industrial development beneath the bridge may also make direct discharge from the bridge deck unacceptable.

Underdeck drainage systems are problematic to bridges' construction, maintenance, and aesthetics. They can be difficult to clean, and the pipes are subject to breakage. It is broadly accepted practice to avoid them unless the setting or regulations require them. When avoidance is not feasible, underdeck drainage systems should be as short as possible. Underdeck drainage pipe is usually ductile iron, PVC or fiberglass, and is smaller than the conduit used in underground storm drains. Figure 11.3 is a photograph of an installed underdeck drainage system constructed of fiberglass pipe.

Figure 11.3. Underdeck bridge drainage system.

Water Quality Impacts on Receiving Waters: When a bridge crosses a broad waterway, the design may need to avoid negatively impacting the water quality. When environmentally sensitive waters are present, the State's water quality regulations likely prohibit the direct discharge of the bridge deck runoff into the stream. In such cases, the design provides for runoff capture and conveyance to an acceptable stormwater quality mitigation feature (a stormwater best-management practice, or BMP). The NCHRP report titled *Assessing the Impacts of Bridge Deck Runoff Contaminants in Receiving Waters* is a resource to help identify, assess, and manage the water quality aspects of bridge deck runoff (NCHRP 2002). A more recent NCHRP report, *Bridge Stormwater Runoff Analysis and Treatment Options* (NCHRP 2014), presents an analysis process for identifying stormwater management strategies. Among the treatment strategies described in the report is to mitigate the problem by treating an equivalent segment of roadway that drains to the same receiving water. As of December 2022, NCHRP had a study underway entitled *Development of On-Bridge Stormwater Treatment Practices*, NCHRP 25-61. The study aims to develop practical on-bridge stormwater treatment applications and provide information about for their selection, design, placement, and maintenance.

Maintenance Considerations: Bridge deck inlets become clogged with debris, even under the best conditions. The design ideally keeps debris at or above the bridge deck surface and in areas that are easy to reach and safe for maintenance crews to service. This practice minimizes the required maintenance effort and promotes the efficiency of the bridge deck drainage system. It is appropriate to place inlets at the shoulder's outer edge and make the shoulder as wide as feasible. Inlets located within traffic lanes are inappropriate unless current and projected traffic volumes are very low.

Upslope Interception: The paragraphs above describe many challenges inherent in managing large runoff flows on the bridge deck. An ideal practice, wherever feasible, is to intercept all or nearly all of the roadway drainage with inlets upslope of the bridge. This practice allows managing only the runoff generated on the deck itself.

11.3 Considerations for Temporary Conditions during Construction

Bridge projects often require temporary features to make construction feasible. Examples include, but are not limited to:

- Detour bridges spanning the stream to maintain traffic flow.
- Detour roads across the stream to maintain traffic flow, consisting of temporary fill and one or more culvert pipes.
- Work bridges or causeways to provide construction equipment access.
- Fill placed in a portion of the river to divert flow around a work platform for a pier, pier scour countermeasure, riprap at an abutment or along a streambank.
- Cofferdams or berms to keep river flows out of the work area.
- Diversion of river flow through or around the work area by diverting all the flow into a pipe.
- Berms or turbidity barriers to keep disturbed sediment in the construction work areas out of the river flow.

Many transportation agencies specify a minimum design event for temporary detours. A common requirement is to use at least the 50 percent AEP (2-year recurrence interval) event for detour design, provided the detour is in place for less than a year. It is prudent to examine the risk of overtopping at a temporary detour for safety considerations. The equation below provides the exceedance probability of a specific AEP flood for a given duration:

$$P = 1 - (1 - AEP)^{n} \tag{11.1}$$

where:

P	=	probability of flood exceedance during the duration
n	=	duration of the construction and detour period, in years
AEP	=	annual exceedance probability of the flood

For example, one could use Equation 11.1 to determine that a 20 percent AEP (5-year) flood has a 36 percent exceedance probability for a 2-year construction period. Other considerations might include providing freeboard from the design water surface to the detour surface and establishing a response plan (by the owner or the contractor) to close the detour if a flood begins to rise in the stream. Designing the temporary detour for a given flood event ideally includes protecting it from erosion damage.

Temporary work bridges, causeways, diversions, or cofferdams, while not involving risk to the traveling public, also warrant some consideration of a design flood event. A flood during construction could destroy work in progress or derail the project schedule. Ideally, the project contract documents make clear which party owns the risk for flood damage to the work, contractor equipment, schedule, etc. Equation 11.1 can help understand the degree of risk and mitigate it.

Some rivers have a definite flood season, outside of which floods of any significant size are very unlikely. Many temporary detour crossings, causeways, and other facilities are in place for only part of a year. A partial-year construction period would ideally occur outside the typical flood season for the project location. For example, suppose flooding on the stream is known to always occur in May, June or July, and the construction duration is October through January. In that case, the probability of a 50 percent AEP flood occurring during construction is considerably less than 50 percent. The risk of such a scenario can be assessed with high confidence if a nearby streamflow gage exists with daily flow records over a substantial period of record. Apart from seasonal considerations, the engineer can use daily gage data to determine the probability of the discharge remaining below a threshold value for a certain number of days consecutively (see example in Figure 11.4).

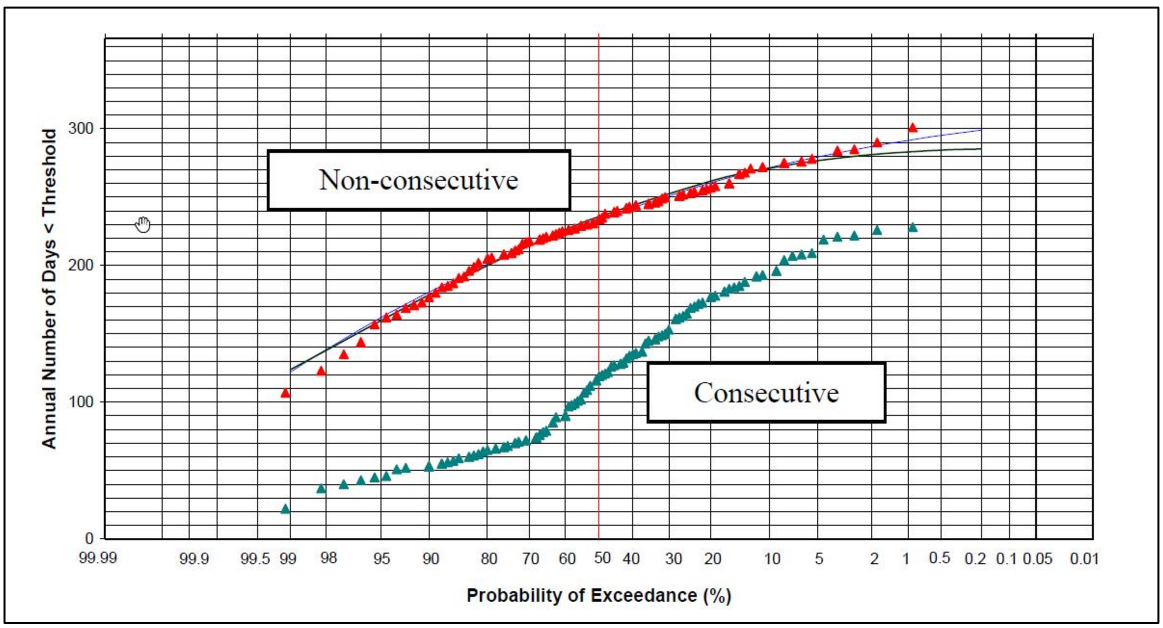

Figure 11.4. Example of probability analysis for consecutive days of flow below a threshold value of a specific discharge (from FHWA National Hydraulics Team).

Some floodplain regulatory jurisdictions limit the allowable impacts of temporary features. Many temporary features for construction, if they were in place during the occurrence of a 100-year flood, would cause an increase in the flood profile. The floodplain authority may require scheduling construction activities such that temporary features will not be in the river or floodway during the flood season. The authority may require the contractor to demonstrate readiness and ability to promptly remove temporary construction features from the river if a flood above a specific threshold occurs.

CHAPTER 12
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Appendix A – Glossary

ABUTMENT (BRIDGE): A substructure at the end of a bridge that supports the bridge superstructure and laterally supports the roadway embankment as it approaches the bridge. The slope fill interior to the bridge opening (spill-through abutment) is often considered part of the abutment.

ABUTMENT EMBANKMENT PROTECTION: A countermeasure that protects the abutment fill and adjacent embankment fill up to the hydraulic design flood or road overtopping flood, whichever is greater. The abutment foundation is designed to be stable for total scour for all events up to and including the Load Resistance Factor Design (LRFD) extreme event limit state, i.e., the countermeasure protects the abutment fill and embankment fill up to the point of road overtopping and the stability of the bridge is not dependent on the countermeasure.

ABUTMENT SCOUR COUNTERMEASURE: A countermeasure that protects the abutment foundation against scour for all events up to and including the LRFD extreme event limit state. The stability of the abutment foundation depends on the countermeasure as the abutment is not designed to be stable for the scour condition that includes local abutment scour.

ACCELERATION: Time rate of change in magnitude or direction of the velocity vector.

ADVERSE SLOPE: The hydraulic condition where the bed slope in the direction of flow is negative and normal depth is undefined.

AGGRADATION: The general and progressive buildup of the longitudinal profile of a channel bed due to sediment deposition.

ALLUVIAL CHANNEL: A channel wholly in alluvium; no bedrock is exposed in channel at low flow or likely to be exposed by erosion.

ALLUVIAL FAN: A fan shaped deposit of material at the place where a stream issues from a narrow valley of high slope onto a plain or broad valley of low slope. An alluvial cone is made up of the finer materials suspended in flow while a debris cone is a mixture of all sizes and kinds of materials.

ALLUVIAL STREAM: A stream that has formed its channel in cohesive or non-cohesive materials that have been and can be transported by the stream.

ALLUVIUM: Unconsolidated material deposited by a stream in a channel, floodplain, alluvial fan, or delta.

APPROACH SECTION: The cross section upstream of a bridge where flow is fully expanded in the floodplain.

ASTRONOMICAL TIDE: The tidal levels and character which would result from gravitational effects, e.g., of the earth, sun, and moon, without any atmospheric influences.

ATTENUATION: 1) The reduction in peak flow between inflow and outflow hydrographs for a reservoir, detention pond, or river reach. 2) A lessening of the height or amplitude of a wave with distance.

AVERAGE VELOCITY: The velocity at a given cross section or location determined by dividing discharge by the cross-sectional area of the flow.

AVULSION: Sudden change in the channel course that usually occurs when a stream breaks through its banks; often associated with a flood or a catastrophic event.

BACKWATER: An increase in water surface elevation relative to the elevation occurring under natural channel and floodplain conditions or compared to a condition that excludes the bridge and approach embankments but the includes other structures. It is induced by a bridge or other structure that obstructs or constricts the free flow of water in a channel.

BANK: Sides of a channel between which the flow is normally confined.

BANK, LEFT OR RIGHT: One side of a channel as viewed in a downstream direction.

BANKFULL DISCHARGE: Discharge that, on average, fills a channel to the point of overflowing.

BANK PROTECTION: Engineering works for the purpose of protecting streambanks from erosion.

BASE FLOODPLAIN: The area subject to flooding by the flood or tide event with a one percent change of being exceeded in any given year.

BATHYMETRY: The below-water ground elevation of a channel, lake, ocean, or other body of water.

BAY: 1) a body of water almost completely surrounded by land but open to some tidal flow communications with the sea. 2) A recess in the shore or an inlet of a sea between two capes or headlands, not so large as a gulf but larger than a cove.

BED: The bottom of a channel or other water body bounded by banks. Also refers to the material placed within and embedded culvert.

BEDFORM: A recognizable relief feature of the bed of a channel, such as a ripple, dune, plane bed, antidune, or bar. Bedforms are the consequence of the interaction between hydraulic forces and the bed sediment.

BED LAYER: A flow layer, multiple grain diameters thick (usually two) immediately above the bed and associated with bed load sediment transport.

BED LOAD DISCHARGE (OR BED LOAD): The quantity of bed load passing a cross section of a stream in a unit of time.

BED LOAD: Sediment that is transported in a stream by rolling, sliding, or skipping along the bed or very close to it; considered to be within the bed layer (contact load).

BED MATERIAL: Material found in and on the bed of a stream or other water body (may be transported as bed load or in suspension).

BEDROCK: Solid rock exposed at the surface of the earth or overlain by soils and other unconsolidated material.

BED SHEAR: Force per unit area exerted by a fluid flowing past the channel bed, bank, or other boundary.

BED SLOPE: Inclination of the channel bottom.

BENT: A frame structure. Sometimes used to mean bridge pier. See pile bent.

BOULDER: A rock fragment or particle whose intermediate diameter is greater than 250 mm.

BOUNDARY CONDITION: Any condition that encloses and feeds information into the model domain. These are often environmental conditions, e.g., water levels, waves, currents/flows, drifts, etc. used as boundary input to numerical models. Some boundary conditions may not be obvious and may be inherent within models, such as bottom boundary conditions (i.e., the condition that forces the water to stay above ground).

BRIDGE OPENING: The cross-sectional area beneath a bridge that is available for conveyance of water.

BRIDGE OWNER: Any Federal, State, Local agency, or other entity responsible for a structure defined as a highway bridge by the National Bridge Inspection Standards (NBIS). [23 CFR § 650.305]

BRIDGE PIER: See pier.

BRIDGE SECTION: The cross section at a bridge. For scour calculations in water surface profile models (e.g., HEC-RAS (see Acronyms)), typically the upstream adjacent section to the internal bridge sections.

BRIDGE WATERWAY: Area of a bridge opening available for flow, as measured below a specified stage and normal to the principal direction of flow.

CAPACITY SUPPLY RATIO: The ratio of the total bed material load transported by the historic sequence of flows in the design reach compared to that in the sediment supply reach immediately upstream.

CAUSEWAY: Rock or earth embankment carrying a roadway across water.

CELERITY: Propagation speed of a wave.

CHANNEL: The bed and banks that confine surface flow of a stream.

CHECK DAM: A low dam or weir across a channel used to control stage or degradation.

CHORD: A line along the bottom of girders (low chord) or top of the bridge obstruction including deck, curbs, and walls (high chord) typically along the bridge faces.

CLAY (MINERAL): A particle with diameter between 0.00024 and 0.004 mm.

CLEAR-WATER SCOUR: Scour at a pier or abutment (or contraction scour) when there is no movement of the bed material upstream of the bridge crossing at the flow causing bridge scour.

CLIMATE CHANGE: Any significant change in the measures of climate lasting for an extended period. Climate change includes major variations in temperature, precipitation, or wind patterns, among other environmental conditions, that occur over several decades or longer. Changes in climate may manifest as a rise in sea level, as well as increases in the frequencies and magnitudes of extreme weather events now and in the future. (FHWA Order 5520)

COBBLE: A rock fragment or particle with intermediate diameter between 64 and 250 mm.

COINCIDENT FLOW: The combination of peak flows or flow hydrographs at a confluence.

COMPUTATIONAL FLUID DYNAMICS (CFD): The use of applied mathematics and physics for the purpose of describing or visualizing fluid behavior or movement.

CONFLUENCE: The junction of two or more streams.

CONSTRICTION: A natural or artificial section, such as a bridge crossing, channel reach or dam, with less flow capacity than the upstream river channel and floodplains.

CONTACT LOAD: Sediment particles that roll or slide along in almost continuous contact with the streambed (bed load).

CONTOUR: A line on a map or chart representing points of equal elevation, depth, velocity, or other property.

CONTRACTED SECTION: An area in the vicinity of the bridge or culvert with suitably consistent hydraulic (velocity and depth) conditions to allow calculation of contraction scour. Examples include (A) the bank toe to bank toe width for a main channel area, (B) top of bank to top of bank width for a main channel area, (C) top of bank to toe of abutment slope for an area where the abutment is set back from the channel, and (D) toe of abutment to toe of abutment where a relief bridge does not include a channel.

CONTRACTION: The effect of channel or bridge constriction on flow streamlines.

CONTRACTION SCOUR: In a natural channel or at a bridge crossing, the removal of material from the bed and banks across all or most of the channel width. This component of scour results from a contraction of the flow area at the bridge which causes an increase in velocity and shear stress on the bed at the bridge.

CONTROL SECTION: A location where the water surface is uniquely defined for a given discharge.

CONVEYANCE: The capacity of the channel to accommodate flow.

CORIOLIS: A force representing the acceleration due to the Earth's rotation, capable of generating currents. It causes moving bodies to be deflected to the right in the Northern Hemisphere and to the left in the Southern Hemisphere. The "force" is proportional to the speed and latitude of the moving object. It is zero at the equator and maximum at the poles.

COUNTERMEASURE: A measure intended to prevent, delay or reduce the severity of hydraulic erosion or scour.

CRITICAL DEPTH: In hydraulic analysis, the depth when flow has a Froude number of 1.0. The depth associated with the minimum total energy to pass a given flow through a given cross section.

CRITICAL SHEAR STRESS: The minimum amount of boundary shear stress capable of initiating sediment or soil particle motion (i.e., the point of incipient motion).

CRITICAL SLOPE: The hydraulic condition where the bed slope produces a normal depth equal to critical depth.

CRITICAL VELOCITY: In sediment transport analysis, the velocity when a bed material particle size is at incipient motion.

CROSS SECTION: A transect normal to the flow direction of a channel or floodplain.

CROSSING: The relatively short and shallow reach of a stream between bends; also, crossover or riffle.

CURRENT: 1) The flowing of water. 2) That portion of a stream of water which is moving with a velocity much greater than the average or in which the progress of the water is principally concentrated. 3) Ocean currents can be classified in several different ways. Some important types include the following: A) Periodic – a result of the effect of the tides; such Currents may be rotating rather than having a simple back and forth motion. The currents accompanying tides are known as tidal currents; B) Temporary - due to seasonal winds; C) Permanent or ocean - constitute a part of the general ocean circulation. The term drift current is often applied to a slow broad movement of the oceanic water; D) Nearshore - caused principally by waves breaking along a shore.

CUTOFF: (a) A direct channel, either natural or artificial, connecting two points on a stream, thereby shortening the original length of the channel and increasing its slope; (b) A natural or artificial channel which develops across the neck of a meander loop (neck cutoff) or across a point bar (chute cutoff).

DATUM: Any permanent line, plane, or surface used as a reference to which elevations are referred.

DEBRIS: Floating or submerged material, such as logs, other vegetation, and trash, transported by a stream.

DEGRADATION (BED): A general and progressive (long-term) lowering of the channel bed due to erosion over a relatively long channel length.

DEPOSITION: The geological process of adding sediments, soil, rocks, and silts to landform or landmass. Deposition in rivers is typically found when the sediment transport capacity decreases and the suspended material can no longer be carried by the river.

DESIGN FLOW (DESIGN FLOOD): The peak discharge, volume (if appropriate), stage or wave crest elevation of the flood associated with the annual exceedance probability selected for the design of a highway asset.

DIKE (GROIN, SPUR, JETTY): A structure extending from a bank into a channel that is designed to: (A) reduce the stream velocity as the current passes through the dike, thus encouraging sediment deposition along the bank (permeable dike); or (B) deflect erosive current away from the streambank (impermeable dike).

DISCHARGE: Flow rate. The volume of water passing through a channel or structure in a given period.

DIURNAL: Having a period or cycle of approximately one tidal day.

DIURNAL TIDE: A tide with one high water and one low water in a tidal day.

DRAG FORCE: The force acting between the flow and an obstruction.

DRAINAGE BASIN: The land area contributing runoff to a location along the channel.

DRIFT: Alternate term for vegetative debris.

DRILLED SHAFT: A deep foundation constructed with fresh concrete and reinforcing steel placed into a cylindrical drilled hole. Also called a drilled pier or caisson.

DYNAMIC EQUILIBRIUM: A state of balance between continuing processes. A channel is in dynamic equilibrium when it adjusts to varying flow, sediment, and biological inputs without a trend toward a substantially different condition.

EDDY: A vortex type motion of a fluid flowing contrary to the main current, such as the circular water movement that occurs when the main flow becomes separated from the bank.

ENCROACHMENT: Human activity, occupation, or construction within the river or its floodplain including highway fill, new construction, substantial improvements, and other transportation development.

ENERGY CORRECTION COEFFICIENT (α): A correction factor that is applied when the average velocity is used to compute kinetic energy because the total energy of a velocity distribution is greater than the energy computed from the average velocity.

ENERGY GRADE LINE (EGL): In one-dimensional hydraulic modeling, the profile line representing total energy, which is the sum of the water surface elevation and the kinetic energy head.

ENERGY SLOPE: The slope of the energy grade line at a cross section for 1D hydraulic models and at a point in 2D hydraulic models. Also called friction slope, although this document uses "friction slope" to refer to the averaged energy slope between 1D hydraulic model cross sections.

EPHEMERAL STREAM: A stream or reach of stream that does not flow continuously throughout the year.

EROSION: Displacement of soil particles due to water, ice, or wind action.

ESTUARY: 1) The region near a river mouth in which the fresh water of the river mixes with the salt water of the sea and which receives both fluvial and littoral sediment influx. 2) The part of a river that is affected by tides.

EXIT SECTION: The cross section downstream of a bridge where flow is fully expanded in the floodplain.

FALL VELOCITY: Velocity at which a sediment particle falls through a column of still water.

FETCH: The distance or area in which wind blows across the water forming waves. Sometimes used synonymously with fetch length and generating area.

FETCH LENGTH: The horizontal distance (in the direction of the wind) over which wind generates waves and wind setup.

FILL SLOPE: The side or end slope of an earth fill embankment. Where a fill slope forms the streamward face of a spill-through abutment, it is regarded as part of the abutment.

FLASHY STREAM: A stream characterized by rapidly rising and falling hydrograph stages. Typically associated with mountain streams, highly urbanized catchments, and arid environments.

FLOOD-FREQUENCY CURVE: A graph indicating the probability that the annual peak discharge will exceed a given magnitude, or the recurrence interval corresponding to a given magnitude.

FLOODPLAIN: A nearly flat, alluvial lowland bordering a stream or coastal water body that is subject to inundation by floods.

FLOODWAY: See Regulatory Floodway.

FLOW RESISTANCE: The boundary impediment to flowing water depending on several factors, including boundary roughness, vegetation, irregularities, etc.

FLUVIAL GEOMORPHOLOGY: The science dealing with morphology (form) and dynamics of streams and rivers.

FLUVIAL SYSTEM: The natural river system consisting of: (A) the drainage basin, watershed, or sediment source area. (B) tributary and mainstem river channels or sediment transfer zone. (C) alluvial fans, valley fills and deltas, or the sediment deposition zone.

FREEBOARD: The vertical distance of the lowest structural member of the bridge superstructure above the water surface elevation of the overtopping flood. Also, the vertical distance above a design stage that is allowed for waves, surges, drift, and other contingencies.

FRICTION (HYDRAULIC): See flow resistance.

FRICTION SLOPE: In this document, the averaged value of energy slope between 1D hydraulic model cross sections. See energy slope.

FROUDE NUMBER: A dimensionless number that represents the ratio of inertial to gravitational forces in open channel flow.

GEOMORPHOLOGY/ MORPHOLOGY: That science that deals with the form of the Earth, the general configuration of its surface, and the changes that take place due to erosion and deposition.

GLOBAL MEAN SEA LEVEL RISE: The sea level rise averaged across the world's oceans. This is the average change in sea level due to a change in the volume of the world's ocean basins and the total amount of ocean water. Vertical land movement is not included.

GRADE-CONTROL STRUCTURE (SILL, CHECK DAM): A structure placed bank to bank across a stream channel (usually with its central axis perpendicular to flow) for the purpose of controlling bed slope and preventing scour or headcutting.

GRADUALLY VARIED FLOW: A flow condition where streamlines are essentially parallel and vertical accelerations are negligible.

GRAVEL: Rock fragment or particle with intermediate diameter between 2 to 64 mm.

GUIDE BANK: A body of igneous rocks extending upstream from the approach embankment at either or both sides of the bridge opening to direct the flow through the opening. Some guide banks extend downstream from the bridge (also spur dike).

HEADCUTTING: Channel degradation associated with abrupt changes in the bed elevation (headcut) that generally migrates in an upstream direction.

HIGH TIDE: The maximum elevation reached by each rising tide.

HIGH WATER: The maximum height reached by a rising tide. The height may be solely due to the periodic tidal forces, or it may have superimposed upon it the effects of prevailing meteorological conditions. Nontechnically, also called the high tide.

HIGHER HIGH WATER: The higher of the two high waters of any tidal day. The single high water occurring daily during periods when the tide is diurnal is considered to be a higher high water.

HORIZONTAL SLOPE: The hydraulic condition where the bed slope is zero and normal depth is infinite.

HYDRAULIC CONTROL: 1) A control section where water surface is uniquely defined for the discharge. 2) A natural or constructed feature that locally diverts or impedes flow, such as high ground in a floodplain, natural or constructed levees, spurs, etc.

HYDRAULIC GRADE LINE (HGL): In open-channel flow, the profile line that is the water surface elevation.

HYDRAULIC MODEL: A numerical or small-scale representation of a flow situation.

HYDRAULIC RADIUS: The cross-sectional area of a stream divided by its wetted perimeter.

HYDRAULIC STRUCTURES: The facilities used to impound, accommodate, convey or control the flow of water, such as dams, weirs, intakes, culverts, channels, and bridges.

HYDRAULICS: Applied science of the behavior and flow of liquids, especially in pipes, channels, structures, and the ground.

HYDROGRAPH: A graph of the time distribution of discharge at a point on a stream.

HYDROLOGY: The science concerned with the occurrence, distribution, and circulation of water on the earth.

HYDROSTATIC PRESSURE: The pressure as it varies with depth in still water. Also, the pressure of flowing water that is not affected by vertical accelerations other than gravity.

INCIPIENT OVERTOPPING: The point at which road overtopping is beginning to occur.

INEFFECTIVE FLOW: An area of flow where water is not being conveyed in a downstream direction (e.g., eddies and ponded areas upstream or downstream of an embankment).

INITIATION OF MOTION: The hydraulic condition when bed material, often the median grain size, begins to move and sediment transport of bed material occurs.

INLET: 1) A short, narrow waterway connecting a bay, lagoon, or similar body of water with a large parent body of water. 2) An arm of the sea (or other body of water) that is long compared to its width and may extend a considerable distance inland.

INTERMEDIATE DIAMETER: The length measured along the axis perpendicular to the longest and shortest axes of a rock particle (i.e., sand, gravel, cobbles, and boulders).

INVERT: The lowest point in the channel cross section or at flow control devices such as weirs, culverts, or dams.

JOINT PROBABILITY: The probability of occurrence of two events. The events may be independent or may be correlated.

KINETIC ENERGY: The energy due to the velocity of water or other fluid equal to one-half mass times velocity squared. In hydraulics, kinetic energy often excludes mass and includes gravity as velocity squared divided by 2.0 times gravity to be measured as a height.

KNICKPOINT: A headcut in non-cohesive alluvial material.

LAGOON: A shallow body of water, like a pond or sound, partly or completely separated from the sea by a barrier island or reef. Sometimes connected to the sea via an inlet.

LATERAL EROSION: Erosion in which the removal of material is extended horizontally as contrasted with degradation and scour in a vertical direction.

LEVEE: An embankment, generally landward of the top of bank, that confines flow during high-water periods, thus preventing overflow into lowlands. Levees can be constructed or natural (see natural levee).

LIDAR: An aerial survey method that illuminates the target with laser light and measures the reflected pulses. Formerly LiDAR. Stands for Light Detection And Ranging.

LIVE-BED SCOUR: Bridge scour when there is bed material transport in the channel upstream of the bridge.

LOAD (OR SEDIMENT LOAD): The amount of sediment being moved by a stream.

LOCAL SCOUR: The removal of material from around piers, abutments, spurs, and embankments caused by an acceleration of flow and resulting vortices induced by obstructions to the flow.

LONGITUDINAL PROFILE: The profile of a stream or channel drawn along the length of its centerline. In drawing the profile, elevations of the water surface or the thalweg are plotted against distance as measured from the mouth or from an arbitrary initial point.

LOW TIDE: The minimum elevation reached by each falling tide.

LOW WATER: The minimum height reached by a falling tide. Nontechnically, also called the low tide.

LOWER LOW WATER: The lower of the two low waters of any tidal day. The single low water occurring daily during periods when the tide is diurnal is considered to be a lower low water.

MEAN HIGH WATER: The average height of the high waters over a 19-year tidal epoch. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All high water heights are included in the average where the type of tide is either semidiurnal or mixed. Only the higher high water heights are included in the average where the type of tide is diurnal. So determined, mean high water in the latter case is the same as mean higher high water.

MEAN HIGHER HIGH WATER: The average height of the higher high waters over a 19-year tidal epoch. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.

MEAN LOW WATER: The average height of the low waters over a 19-year tidal epoch. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All low water heights are included in the average where the type of tide is either semidiurnal or mixed. Only lower low water heights are included in the average where the type of tide is diurnal. So determined, mean low water in the latter case is the same as mean lower low water.

MEAN LOWER LOW WATER: The average height of the lower low waters over a 19-year tidal epoch. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. Frequently abbreviated to lower low water.

MEAN SEA LEVEL: The average height of the surface of the sea for all stages of the tide over a 19-year tidal epoch, usually determined from hourly height readings.

MEANDER OR FULL MEANDER: A meander in a river consists of two consecutive loops, one flowing left followed by one flowing right, or vice-versa.

MEANDERING STREAM: A stream having a sinuosity greater than some arbitrary value. The term also implies a moderate degree of pattern symmetry, imparted by regularity of size and repetition of meander loops. The channel generally exhibits a characteristic process of bank erosion and point bar deposition associated with systematically shifting meanders.

MEDIAN DIAMETER: The particle diameter of the 50th percentile point on a size distribution curve such that half of the particles (by weight, number, or volume) are larger, and half are smaller (D_{50}).

MIGRATION: The change in position of a channel by lateral erosion of one bank and accretion of the opposite bank.

MILD SLOPE: The hydraulic condition where the bed slope is less than critical slope and normal depth is greater than critical depth.

MIXED TIDE: A type of tide in which the presence of a diurnal wave is conspicuous by a large inequality in either the high or low water heights, with two high waters and two low waters usually occurring each tidal day. In strictness, all tides are mixed, but the name is usually applied without definite limits to the tide intermediate to those predominantly semidiurnal and those predominantly diurnal.

MODEL EXTENT: The limits of a model domain including boundary conditions and fixed boundaries.

MOMENTUM CORRECTION COEFFICIENT (β): A correction factor that is applied when the average velocity is used to compute momentum because the total momentum of a velocity distribution is greater than the momentum computed from the average velocity.

MUD: A soft, saturated mixture mainly of silt and clay.

MULTIPLE OPENINGS: Road embankments that have two or more bridges (and/or culverts) located along the embankment.

NATURAL LEVEE: A low ridge that slopes gently away from the channel banks that is formed along streambanks during floods by deposition.

NONSTATIONARITY: A characteristic of time series data such that the data are heterogeneous. Trends over time prevent historical data from being used to estimate future conditions.

NON-UNIFORM FLOW: A flow condition in which the velocity changes in magnitude or direction or both with distance. The convective acceleration components are different from zero. Examples are flow around a bend or flow in expansions or contractions.

NOR'EASTER: A common storm type in the North Atlantic Ocean which produces northeast winds along the United States Atlantic seaboard.

NORMAL DEPTH: A condition when the water surface slope and energy grade slope are parallel and equal to the bed slope. Also, a boundary condition where the water surface is computed from a preset energy grade slope.

NUMERICAL MODEL: A computer representation of a physical process.

ONE-DIMENSIONAL MODEL: A numerical model that calculates variables (velocity, depth, etc.) changing predominantly in one defined direction, x, along the channel.

ORIFICE FLOW: Flow through a bridge where the upstream low-chord is submerged but the downstream low-chord is not.

OVERBANK FLOW: Water movement that overtops the bank either due to stream stage or to overland surface water runoff.

OVERTOPPING FLOW: The bridge hydraulic condition when flood waters are flowing over the approach embankment and/or bridge.

PARALLEL BRIDGES: Bridges located in series along a channel where flow does not fully expand between the bridges.

PERENNIAL STREAM: A stream or reach of a stream that flows continuously for all or most of the year.

PHYSICAL MODEL: A laboratory scaled model of a hydraulic or coastal process.

PIER (BRIDGE): An intermediate substructure unit located between the ends, or abutments, of a bridge.

PILE: An elongated member, usually made of timber, concrete, or steel, that is driven into the earth and serves as a structural foundation component of a river-training structure or bridge.

PILE BENT: A frame structure consisting of piling and pile cap acting as a bridge pier.

POTENTIAL ENERGY: The gravitational energy due to the elevation and depth of water or other fluid equal to mass times the acceleration of gravity times height. In hydraulics, potential energy often excludes mass and gravity, and is measured as an elevation.

PRESSURE FLOW: The flow condition when the bridge low chord is submerged by high flows.

RAPIDLY VARIED FLOW: Flow where streamlines are not parallel or have significant curvature, pressure is not hydrostatic, and vertical accelerations are substantial.

REACH: For purposes of this document, a segment of stream length that is arbitrarily bounded or characterized by a consistent attribute.

RECESSION: The decreasing discharge after the peak flood flow.

RECURRENCE INTERVAL: The reciprocal of the annual exceedance probability (AEP) of a hydrologic event (also return period).

REGIME: The stability condition of a stream or its channel. A stream is in regime if its channel has reached an equilibrium form consistent with its flow characteristics. Also, the general pattern of variation around a mean condition, as in flow regime, tidal regime, channel regime, sediment regime, etc. Also used to mean a set of physical characteristics of a river.

REGULATORY FLOODWAY: 23 CFR § 650.105(m) defines this term: *"Regulatory floodway shall mean the flood-plain area that is reserved in an open manner by Federal, State or local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount (not to exceed 1 foot as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program)."*

RELATIVE SEA LEVEL RISE: Sea level change at a coastal location relative to the land. This includes both the eustatic sea level rise component and the vertical land movement component. This is the sea level change measured by long-term tide gages.

RELIEF BRIDGE: An opening in a road embankment on a floodplain to permit passage of overbank flow.

RESILIENCE: The ability to anticipate, prepare for, and adapt to changing conditions and withstand, respond to, and recover rapidly from disruptions. [23 U.S.C. § 101(a)(24)].

REVETMENT: Rigid or flexible armor placed to inhibit scour and lateral erosion.

RIPARIAN: Pertaining to anything connected with or adjacent to the banks of a stream (corridor, vegetation, zone, etc.).

RIPRAP: Layer or facing of rock, meeting common specifications, placed to armor a structure or embankment from erosion. Riprap has also been applied as wire-enclosed riprap, matrix riprap, and vegetated riprap. Common usage of the term often applies to the rock suitable for such applications.

RISK: *"… the consequences associated with hazards (including climatic) considering the probabilities of those hazards. It shall include the potential for property loss and hazard to life during the service life of the highway"* (23 CFR § 650.105(o)).

ROCK: Geomaterial (material of geologic origin) that is sufficiently large or hard that excavation involves relatively great effort (i.e., drilling, wedging, blasting, or other methods).

ROUGHNESS COEFFICIENT: Numerical measure of the frictional resistance to flow in a channel, as in the Manning or Chezy formulas.

ROUTING: A technique used to predict changes in the shape of a flow hydrograph at the flood wave moves down a river or through a reservoir.

RUNOFF: The portion of a precipitation discharged from a watershed into the stream network during and following the event of either perennial or intermittent form.

SAND: Rock fragments or particles with intermediate diameter between 0.062 and 2.0 mm.

SCOUR: Erosion of streambed or bank material due to flowing water; often considered as being localized around piers and abutments of bridges (see degradation, contraction scour, local scour, total scour).

SCOUR APPRAISAL: A risk-based and data-driven determination of a bridge's vulnerability to scour, resulting from the least stable result of scour that is either observed, or estimated through a scour evaluation or a scour assessment.

SCOUR ASSESSMENT: The determination of an existing bridge's vulnerability to scour which considers stream stability and scour potential.

SCOUR EVALUATION: The application of hydraulic analysis to estimate scour depths and determine bridge and substructure stability considering potential scour.

SEDIMENT CONCENTRATION: Weight or volume of sediment relative to the quantity of transporting (or suspending) fluid.

SEDIMENT CONTINUITY: An analysis that accounts for sediment inflow, sediment outflow, and erosion or storage of sediment along a river reach.

SEDIMENT DISCHARGE: The quantity of sediment that is carried past any cross-section of a stream in a unit of time. Discharge may be limited to certain sizes of sediment or to a specific part of the cross-section.

SEDIMENT LOAD: The amount of sediment being moved by a stream.

SEDIMENT OR FLUVIAL SEDIMENT: Fragmental material transported, suspended, or deposited by water.

SEDIMENT YIELD: The total sediment outflow from a watershed or a drainage area at a point of reference and in a specified time period. This outflow is equal to the sediment discharge from the drainage area.

SEEPAGE: The slow movement of water through small cracks and pores of the bank material.

SEMIDIURNAL: Having a period or cycle of approximately one-half of a tidal day (12.4 hours). The predominant type of tide throughout the world is semidiurnal, with two high waters and two low waters each tidal day. The tidal current is said to be semidiurnal when there are two flood and two ebb periods each day.

SEMIDIURNAL TIDE: A tide with two high waters and two low waters in a tidal day.

SHEAR STRESS: Force or drag per unit area developed at the channel bed by flowing water.

SILT: A particle whose diameter is in the range of 0.004 to 0.062 mm.

SIMILITUDE: A relationship between full-scale flow and a laboratory flow involving smaller, but geometrically similar boundaries.

SINUOSITY: The ratio between the thalweg length and the valley length of a stream.

SKEW: The condition when a bridge opening is not perpendicular to flow or when a pier is not aligned with the flow.

SLOPE (OF CHANNEL OR STREAM): The fall per unit length along the channel centerline or thalweg.

SOIL: Any unconsolidated geomaterial composed of discrete particles with interstitial spaces in between.

SPILL-THROUGH ABUTMENT: A bridge abutment having a fill slope on the streamward side. The term originally referred to the "spill-through" of fill at an open abutment but is now applied to any abutment having such a slope.

SPUR: A permeable or impermeable linear structure that projects into a channel from the bank to alter flow direction, induce deposition, or reduce flow velocity along the bank.

STABILITY: A condition of a channel when, though it may change at different times of the year as the result of varying conditions of flow and sediment charge, there is no appreciable change from year to year; that is, accretion balances erosion over the years. See dynamic equilibrium.

STABLE CHANNEL: A condition that exists when a stream has a bed slope and cross section which allows its channel to transport the water and sediment delivered from the upstream watershed without aggradation, degradation, or bank erosion.

STAGE: The water surface elevation of a stream with respect to a reference elevation.

STEADY FLOW: Flow that remains constant with respect to time.

STEEP SLOPE: The hydraulic condition where the bed slope is greater than critical slope and normal depth is less than critical depth.

STILLWATER LEVEL: Commonly abbreviated as SWL. The surface of the water if wave and wind action were to cease.

STREAM: A body of water that may range in size from a small rill to a large river. The term is sometimes applied to a natural channel or drainage course formed by flowing water whether it is occupied by water or not.

STREAMBANK EROSION: The removal of soil particles or a mass of particles from a bank surface due primarily to water action. Other factors such as weathering, ice and debris abrasion, chemical reactions, and land use changes may also directly or indirectly lead to bank erosion.

STREAMBANK FAILURE: Collapse of a bank due to an unstable condition such as removal of material at the toe of the bank by scour. Streambank failures can occur as sloughing, slumping, caving, and mass failures.

STREAMBANK PROTECTION: Any technique used to prevent erosion or failure of a streambank.

STREAMLINE: An imaginary line within the flow that is tangent everywhere to the velocity vector.

STREAMTUBE: An element of fluid bounded by a pair of streamlines.

STORM SURGE: A rise (typically over several minutes) in average water level above the normal astronomical tide level due to the action of a storm. Storm surge results from wind stress, atmospheric pressure reduction, and wave setup.

STORM SURGE HYDROGRAPH: A graph of the variation in the rise in SWL with time due to a storm.

SUBCRITICAL FLOW: Open channel flow characterized by low velocities, large depths, mild slopes, and a Froude Number less than 1.0.

SUBMERGED ORIFICE FLOW: Flow through a bridge where water submerges the upstream and downstream low-chords.

SUPERCRITICAL FLOW: Open channel flow characterized by high velocities, shallow depths, steep slopes, and a Froude Number greater than 1.0.

SUPPLY REACH: An alluvial and generally stable reach of river used to estimate the sediment supply of the upstream area.

SUSPENDED SEDIMENT DISCHARGE: The quantity of suspended sediment (that is sediment supported by turbulence in the flow) per unit time that passes through a stream cross-section.

TAILWATER: The waters, and specifically the water surface elevation, located immediately downstream of a bridge, culvert, or other hydraulic structure.

THALWEG: The line following the lowest elevation of the riverbed.

THREE-DIMENSIONAL MODEL: A numerical hydraulic model that computes three components of velocity.

TIDAL EPOCH: An 18.6-year time period used to calculate sea levels.

TIDE: The periodic rising and falling of the water that results from gravitational attraction of the Moon and Sun and other astronomical bodies acting upon the rotating Earth. Although the accompanying horizontal movement of the water resulting from the same cause is also sometimes called the tide, it is preferable to designate the latter as tidal current, reserving the name tide for the vertical movement.

TOE OF BANK: That portion of a stream cross section where the lower bank terminates and the channel bottom or the opposite lower bank begins.

TOPOGRAPHY: The arrangement of the natural and artificial physical features of an area.

TOTAL SCOUR: The sum of long-term degradation, contraction scour, and local scour.

TOTAL SEDIMENT LOAD: The sum of suspended load and bed load or the sum of bed material load and wash load of a stream (total load).

TSUNAMI: A long-period wave caused by an underwater disturbance such as a volcanic eruption or earthquake. Commonly miscalled "tidal wave."

TURBULENCE: The motion of fluids in which local velocities and pressures fluctuate irregularly in a random manner as opposed to laminar flow where all particles of the fluid move in distinct and separate lines. Turbulent flows are predominant in nature.

TWO-DIMENSIONAL (2D) MODEL: A numerical hydraulic model that computes two components of velocity. Depth-averaged models derived from the shallow water equations are often referred to as 2D models.

ULTIMATE SCOUR: The maximum depth of scour attained for a given flow condition. May require multiple flow events and in cemented or cohesive soils may be achieved over a long time period.

UNIFORM FLOW: Flow of constant cross section and velocity through a reach of channel at a given time. Both the energy slope and the water slope are equal to the bed slope under conditions of uniform flow.

UNIT DISCHARGE: Discharge per unit width (may be average over a cross section, or local at a point). Determined for an area by dividing discharge by width and for a point by multiplying velocity and depth.

UNSTEADY FLOW: Flow of variable discharge and velocity through a cross section with respect to time.

VELOCITY: The speed and direction of flow expressed as distance per unit time. The average flow velocity is the velocity at a given cross-section determined by dividing discharge by cross-sectional area.

VELOCITY HEAD: The representation of flow kinetic energy as an equivalent height equal to velocity squared divided by 2.0 times gravity.

VERTICAL ABUTMENT: A tall abutment, usually with wingwalls, that has no fill slope on its streamward side.

VORTEX: A turbulent eddy in the flow generally caused by an obstruction such as a bridge pier, abutment, or sump wall (e.g., horseshoe vortex).

WASH LOAD: Suspended material of very small size (generally clays and silts but can include sands) originating primarily from erosion on the land slopes of the drainage area and not found in appreciable quantities in the bed.

WATERSHED: The upstream area of land that drains to a specific location - i.e., the land upstream of a bridge specifically draining to the bridge.

Appendix B Units

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	l
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 l shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	shorttons(2000lb)	0.907	megagrams (or "metric ton")	Mg (or"t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-lamberts	3.426	candela/m^2	cd/m^2
		FORCEandPRESSUREorSTRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
ml	mittitlters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or"t")	megagrams (or "metric ton")	1.103	shorttons(2000lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	tux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-lamberts	fl
		FORCEandPRESSUREorSTRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is thesymbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)