# Chapter 17
# BRIDGES

MONTANA
Department of Transportation

*HYDRAULICS MANUAL*

*January 2022*

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# Chapter 17
# BRIDGES

## 17.1 INTRODUCTION

### 17.1.1 Overview

Bridges are often required in transportation systems to cross streams and other obstructions. Bridges should be designed as safely as possible while optimizing costs and limiting impacts to property and the environment. Many significant aspects of bridge hydraulic design include regulatory topics, specific approaches for bridge hydraulic modeling, hydraulic model selection, bridge design impacts on scour and stream instability, and sediment transport.

A properly designed bridge is one that balances the cost of the bridge with concerns of safety to the traveling public, impacts to the environment, and regulatory requirements to not cause harm to those that live or work in the floodplain upstream and downstream of the bridge. See HDS 7 (1).

This chapter provides design criteria and guidance for hydraulically designing highway bridges for stream crossings. For a structure designed hydraulically as a culvert, use the design procedures provided in Chapter 11, "Culverts" regardless of its span length, unless a culvert is replacing a bridge; then, also see Section 17.5.7.8.

Chapter 17 provides the following major sections:

- General Considerations, Section 17.2;
- Design Criteria, Section 17.3;
- Bridge Hydraulic Design Process, Section 17.4;
- Hydraulic Modeling Guidelines, Section 17.5;
- Scour Analysis and Design Guidelines, Section 17.6;
- Design Guidelines (Deck Drainage), Section 17.7;
- Detour Guidelines, Section 17.8;
- Documentation, Section 17.9; and
- References, Section 17.10.

### 17.1.2 Bridge Definition

The National Bridge Inspection Standards (23 CFR 650.3) and the AASHTO definition of a bridge is:

> *A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20-feet between under copings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.*

This definition also applies to bridge sized culverts.

## 17.2 GENERAL CONSIDERATIONS

Hydraulic engineers have a wide variety of choices when determining the capacity or location for a new bridge or for an existing bridge that will be replaced. In addition to bridge location considerations, which are discussed in Section 17.2.5, several other factors as discussed in the following sections should also be considered. For further discussion on bridge hydraulic design considerations, see HDS 7 (1).

### 17.2.1 Level of Service

A significant consideration is the level of service that the bridge is expected to provide. If the bridge is remote and carries a low volume of traffic, it can be designed with a lower hydraulic capacity resulting in a smaller and less expensive bridge. This means that the bridge and/or approach roadways may be overtopped more frequently, and the bridge owner can expect that the bridge and approach roadways may require more frequent maintenance and repair. In contrast, if the bridge is on an important route such that significant hardships or economic impacts could be encountered if the bridge is out of service, then it should be designed with a higher hydraulic capacity. This may result in a larger and more expensive bridge and higher approach embankments.

### 17.2.2 Flow Distribution

The proposed facility should not cause any substantial change in the existing flow distribution, unless an adjustment in the flow distribution will improve the stream crossing hydraulics without negatively impacting adjacent properties.

### 17.2.3 Environmental Considerations

The Environmental Services Bureau will coordinate with the Hydraulics Section to specify any environmental constraints such as endangered species and wildlife passage. Environmental will document the environmental constraints in the Wildlife Accommodation Recommendation Memo (WARM) or the Biological Resources Report (BRR).

### 17.2.4 Coordination, Permits, and Approvals

The interests of other governmental agencies must be considered in the evaluation of a proposed stream crossing. Cooperation and coordination with other agencies, especially water resource agencies, is essential; see Chapter 7, “Permits.”

## 17.2.5 Location and Design Considerations

Situating the bridge at the proper location within the floodplain can greatly influence the performance and service life of the crossing. Where practical, the crossing alignment should:

- Cross perpendicular to the river;
- Be located at the narrowest portion of the floodplain;
- Be located on a stable reach of stream;
- Minimize impacts of meander migration; and
- Have appropriately located auxiliary/relief openings, if needed.

## 17.2.6 Hydraulic Considerations for Bridge Layout

The hydraulic engineer should coordinate with the roadway designer and bridge designer to address the bridge layout concerns as discussed in the following sections.

### 17.2.6.1 Roadway Profile

The roadway profile can have a significant effect on bridge crossing hydraulics. Typically, the roadway profile is set to not overtop until the design event is exceeded, and the bridge is designed to provide freeboard above a 100-year flood. When allowing for roadway overtopping, the preferred profile has the low point located away from the bridge. When the overtopping occurs over a long segment of roadway, the associated weir flow is an important component of the overall hydraulic capacity of the crossing. In this case, raising the roadway profile has the potential to increase backwater unless additional capacity is provided in the bridge opening to compensate for the lost roadway overtopping flow capacity. Superelevation must be considered when determining the weir flow.

Where ice buildup is a known issue, consider the historic ice jam elevations when setting the roadway grade and bridge low beam elevations. In addition, additional capacity may be needed at bridges for ice floes. If the roadway grade is set too low, ice and water will push up onto the roadway.

### 17.2.6.2 Abutment Layout

The bridge abutment locations determine the bridge length. In addition to providing the required hydraulic opening, the abutments should be located:

- Outside of the ordinary high water mark; and

- Back from the channel banks where significant problems from ice/debris buildup, scour, or channel stability are anticipated.

The magnitude of local scour at an abutment is a function of the depth and velocity of flow and the volume of flow from the overbank that passes through the bridge opening. It is also a function of where the abutment is located relative to the main channel. Abutment alignment should be designed to minimize flow disruption, debris accumulation, and potential scour. Abutments should be located to fit the site and should not be located in the main channel, if practical. Abutment foundations should be designed to avoid failure due to scour. Scour countermeasures can be used to prevent scour from occurring or to provide additional protection.

Where ice buildup is expected to be a problem, set the toe of spill-through slopes or vertical abutments back from the edge of the channel bank to facilitate the passage of ice.

Where possible, use sloping spill-through abutments. Scour at spill-through abutments is usually less than the scour that occurs at vertical wall abutments for the same hydraulic conditions.

#### 17.2.6.3 Pier/Bent Layout

Where practical, avoid placing piers in the stream channel. When the proposed bridge length requires piers:

- Minimize the number of piers in the active channel.
- Skew the bents to align with the direction of flow for laterally stable streams.
- Use pier shapes that minimize scour and backwater for rivers with changing flow patterns.
- Design pier foundations to avoid failure from scour without the need for scour countermeasures.
- Develop bridge span arrangements and pier types that address long-term performance, change in angle of attack, ice, and debris.
- Avoid placing piers within the abutment riprap.
- Assume that the thalweg could migrate to any of the piers including those located on the floodplain.

On rivers with changing flood patterns, single-column drilled-shaft piers or double-column drilled-shaft piers that are spaced five diameters apart are preferrable for backwater and scour because the impact of the pier is independent of the river's angle of attack. However, when using drilled-shaft piers, the height/vertical depth of the hammerhead should be considered.

### 17.2.7 Maintenance Considerations

Following are some of the maintenance problems that can be encountered and, therefore, should be addressed in project design:

- Scour at piers and abutments caused by the accumulation of debris, or excessive velocities, or both;
- Damage to bridge approach embankments caused by channel encroachment;
- Loss of riprap due to erosion and scour;

- Damage to bridge elements due to debris, ice jams, and excessive velocities;
- Channel aggradation or degradation;
- Clogging of bridge deck drains and scuppers, which may create a hazard to traffic and contribute to deck deterioration;
- Discharges of bridge deck drains that are detrimental to other structural members of the bridge, that spill onto a traveled way below, or that may cause fill and bank erosion;
- Channel migration upstream of bridge openings; and
- Erosion of embankments at bridge ends.

# 17.3 DESIGN CRITERIA

The selection of hydraulic design criteria for determining the waterway opening, roadway profile, minimum low beam elevation, scour potential, riprap design, and other features should focus on the hydraulic performance criteria discussed in this section.

## 17.3.1 MDT Design Criteria

The waterway opening for a bridge should satisfy the site constraints and accommodate the design flood, while satisfying the following criteria.

### 17.3.1.1 Hydraulic Modeling

When sizing a replacement bridge structure, model all existing bridges and their replacement structures in either a 1D HEC-RAS model or a 2D hydraulic modeling program approved by MDT.

### 17.3.1.2 Design Floods

The following flood frequencies are used for bridge hydraulic design to ensure the safety of the travelling public and to ensure no significant backwater damage to adjacent property.

#### 17.3.1.2.1 Design Flood

Section 9.3.2.1 in Chapter 9, “Hydrology” describes the procedure for determining the design flood. Once the design flood has been determined, the crossing must be sized to pass the design flood without overtopping the roadway.

#### 17.3.1.2.2 100-Year Event

The 100-year event or $Q_{100}$ is also referred to as the base flood or the 1% Annual Exceedance Probability event. The following applies:

- The $Q_{100}$ is used to establish the minimum allowable low beam elevation. This is further described in Section 17.3.1.3, "Freeboard and Overtopping."
- The $Q_{100}$ is also used for NFIP mapped floodplain analyses as described in Section 17.3.1.5, "NFIP Identified Areas."
- The $Q_{100}$ backwater produced by the proposed design should not be greater than the existing backwater, where practicable.

#### 17.3.1.2.3 Review Flood

The review flood is the overtopping or the 500-year flood, whichever is smaller. The following applies:

- Include the review flood in the bridge modeling.
- The review flood is used to check for unexpected flood hazards and to evaluate design resiliency. Additional information on resiliency is included in HEC-17, 2nd Edition (2).
- The review flood frequency is not used in designing the size of the structure.

#### 17.3.1.2.4 Scour Flood Design Frequency

The flood design frequency used for scour analysis and countermeasure design is described in Section 17.3.1.4.

### 17.3.1.3 Freeboard and Overtopping

Freeboard is the vertical distance between the low beam of the bridge and the water surface elevation. The freeboard is measured from the water surface in the open water, approximately one bridge length upstream of the bridge, outside of the bridge drawdown area. For 2D models, use the average water surface elevation across the channel. All models for one site should measure backwater at the same location. The following criteria apply to freeboard and overtopping:

- The minimum low beam is typically set 1 ft above the 100-year water surface elevation at the approach section. Additional clearance may be specified to allow for the passage of ice and debris. Local floodplain regulations may require additional freeboard where practicable.
- The minimum low beam elevation should not be lower than the existing low beam elevation, where practicable.

- Where overtopping occurs, it is preferable to set the overtopping location away from the bridge and to set the low beam of the bridge above the overtopping water surface elevation for the design flood frequency to avoid bridge pressure flow.

- The overtopping for the new design should not occur more frequently than the overtopping for existing conditions, where practicable.

## 17.3.1.4 Bridge Scour Flood Selection

Following are descriptions of the floods used for scour analysis and the criteria associated with each event. Scour computations are described in Section 17.4.2.8, "Scour Analysis."

Scour Design Flood. For most bridges, the scour design flood is the $Q_{100}$ event. The scour countermeasures are designed to withstand the $Q_{100}$ event.

Scour Check Flood. The scour check flood for most bridges is the $Q_{500}$ event or the overtopping flood, whichever is less. The scour check flood is used to determine the low scour elevation. The low scour elevation is reported to the Bridge Bureau and Geotechnical Section for the foundation design. If the overtopping flood is less than the $Q_{100}$ event, the overtopping flood will be both the scour design flood and the scour check flood.

High Risk Exception. High risk installations, such as Interstate bridges, four-lane bridges, and bridges with ADT greater than 10,000, should be designed to a higher scour standard. In these instances, the scour design flood is the $Q_{200}$ event and the scour check flood is the $Q_{500}$ event.

Low Risk Exception. If the primary scour criteria cannot be met, a lesser scour standard may be considered for off-system, low-cost bridges that have a $Q_{10}$ or $Q_{25}$ design flood. In these instances, the scour design flood may be lowered to the $Q_{50}$ event, and the scour check flood may be lowered to the $Q_{100}$ event or overtopping event, whichever is less. Discuss the development and usage of a lesser scour standard with the State Hydraulic Engineer before implementation.

### 17.3.1.4.1 Contraction Scour

Compute contraction and local scour for the design flood, the scour design flood, and the scour check flood. The contraction scour for the proposed bridge must be less than 3 ft at the scour design flood. The 3-ft depth has been selected because it is the standard depth of the riprap key. If the depths of contraction scour are too large, it may be necessary to increase the bridge length to reduce scour across the bridge opening.

### 17.3.1.4.2 Abutment Scour

All abutment embankments for bridges over waterways require protection against scour and erosion, regardless of abutment or foundation type. Abutment embankments for bridges over irrigation canals should be evaluated for scour protection on a case-by-case basis.

#### 17.3.1.5 NFIP Identified Areas

In NFIP delineated areas, MDT designs the crossings to meet the FEMA floodplain criteria published in the site-specific Flood Insurance Study (FIS). Typically, these regulations limit the increase in water surface elevations to a 0.5-ft increase for approximate studies and a 0.0-ft increase for detailed studies with defined floodways.

Even in areas with an allowable 0.5-ft rise, MDT typically attempts to provide zero rise in backwater when compared to existing conditions.

Some special cases may require exceeding the published FEMA floodplain criteria or encroaching on the defined floodway (e.g., when it is necessary to place piers in the active channel of a river). These cases will require close coordination with the Floodplain Administrator.

A floodplain permit, obtained from the Floodplain Administrator, is required when a proposed crossing encroaches on a regulatory floodplain. For further information, see Chapter 7, "Permits."

#### 17.3.1.6 Risk Evaluation

Use Section 9.3.4, "Risk Evaluation" to evaluate the risk for each site.

## 17.4 BRIDGE HYDRAULIC DESIGN PROCESS

### 17.4.1 Design Procedure Outline

The following design procedure outline is recommended. Although the project scope and individual site characteristics are unique, this procedure should be applied unless indicated otherwise by MDT:

I. DATA COLLECTION

A. Site Information

1. Topography and Bathymetry;
2. Geology;
3. High-water marks;
4. Anecdotal high water or overtopping observations (from discussions with locals);
5. History of debris accumulation, ice, and scour (generally available from MDT maintenance, local, or county officials);
6. Review of hydraulic performance of existing structures (discuss with maintenance);
7. Review of online bridge inspection and underwater inspection files for information on scour, past hydraulic performance, and scour repairs;
8. Maps and aerial photographs (review of historical photographs to determine lateral stability);
9. Aerial flood photographs;
10. Existing bed or bank instability;
11. Floodplain land use and flow distribution;

12. Proposed roadway alignment, profile, and cross sections;
13. Design data at nearby structures;
14. Rainfall and stream gage records; and
15. Field reconnaissance.

B. Other Studies

1. Federal Flood Insurance Studies;
2. Federal Floodplain Studies by USACE, USGS, and NRCS;
3. Local floodplain studies;
4. USGS and MDT scour studies; and
5. USGS research reports and monitoring studies.

C. Potential Hydraulic Impacts

If needed for the site, collect the following data or review the Location Hydraulic Study Report for:

1. FEMA floodplain designation;
2. Other streams, reservoirs, and water intakes;
3. Structures upstream or downstream;
4. Natural features of stream and floodplain;
5. Channel modifications upstream or downstream;
6. Floodplain encroachments;
7. Sediment types and bed forms (see Chapter 16, "Stream Stability Assessment");
8. Preliminary hydrologic discussions; and
9. Discussions with MDT Maintenance personnel regarding bridge adequacy.

D. Potential Environmental Impacts

Review the Preliminary Field Report (PFR) and the Biological Research Report (BRR), and coordinate with the District Biologist and District Project Development Engineer to identify possible environmental concerns including, but not limited to, the following:

1. Necessary permits (see Chapter 7, "Permits");
2. Endangered species;
3. Fisheries;
4. Wetlands;
5. Environmentally sensitive areas;
6. Navigable waterways (as defined by DNRC); and
7. Recreational impacts (boat launches, parks, watercraft usage).

II. HYDROLOGIC ANALYSIS (See Chapter 9, "Hydrology")

A. Determine watershed characteristics

B. Hydrologic Computations

1. Discharge for historical flood that complements the high-water marks used for calibration; and

2. Determine the discharges for a minimum of the following flood frequencies:

   - 2-year event,
   - Design flood,
   - 100-year flood,
   - Review flood,
   - Scour design flood, and
   - Scour check flood.

III. HYDRAULIC ANALYSIS (In-house Activity 370) (See Section 17.4.2 for a detailed explanation.)

A. Hydraulic performance for existing conditions (calibrated)

B. No Bridge Model

C. Hydraulic performance of proposed designs; include analyses of a sufficient number of bridge opening alternatives (usually at least one opening larger and one opening smaller than the recommended opening) to demonstrate the reasons for selecting the recommended bridge opening.

D. Scour Analysis

E. Preliminary Riprap Layout

F. Recommendation Memo & Hydraulic Report

IV. DESIGN ACTIVITIES (In-house Activity 384)

A. Revise/Update Hydraulic Model (if necessary)

B. Deck Drain Design

C. Finalize Riprap Layout and Proposed Waterway Grading Plan

D. Special Provisions

E. Develop Design Details (if necessary)

F. Complete Hydraulic Data Summary Sheet

V. FINAL DESIGN ACTIVITIES (In-house Activity 390)

A. Complete Design Details

B. Apply for Floodplain Permit (if necessary)

C. Add Floodplain Special Provision (if necessary)

## 17.4.2 Hydraulic Analysis

Figure 17.4-1 presents the MDT process for designing and selecting a hydraulic bridge opening. The process is the same for either 1D or 2D modeling.

### 17.4.2.1 Develop and Calibrate Existing Model

Develop a model to accurately represent the existing conditions. Calibrate the existing model to the extent possible. Good practice is to perform a sensitivity analysis on Manning's n values, boundary conditions, etc.

Calibrate the water surface profile model with historical high-water marks and/or gaged streamflow data to ensure that the model accurately represents site conditions. Use the following parameters, in order of preference, for calibrations:

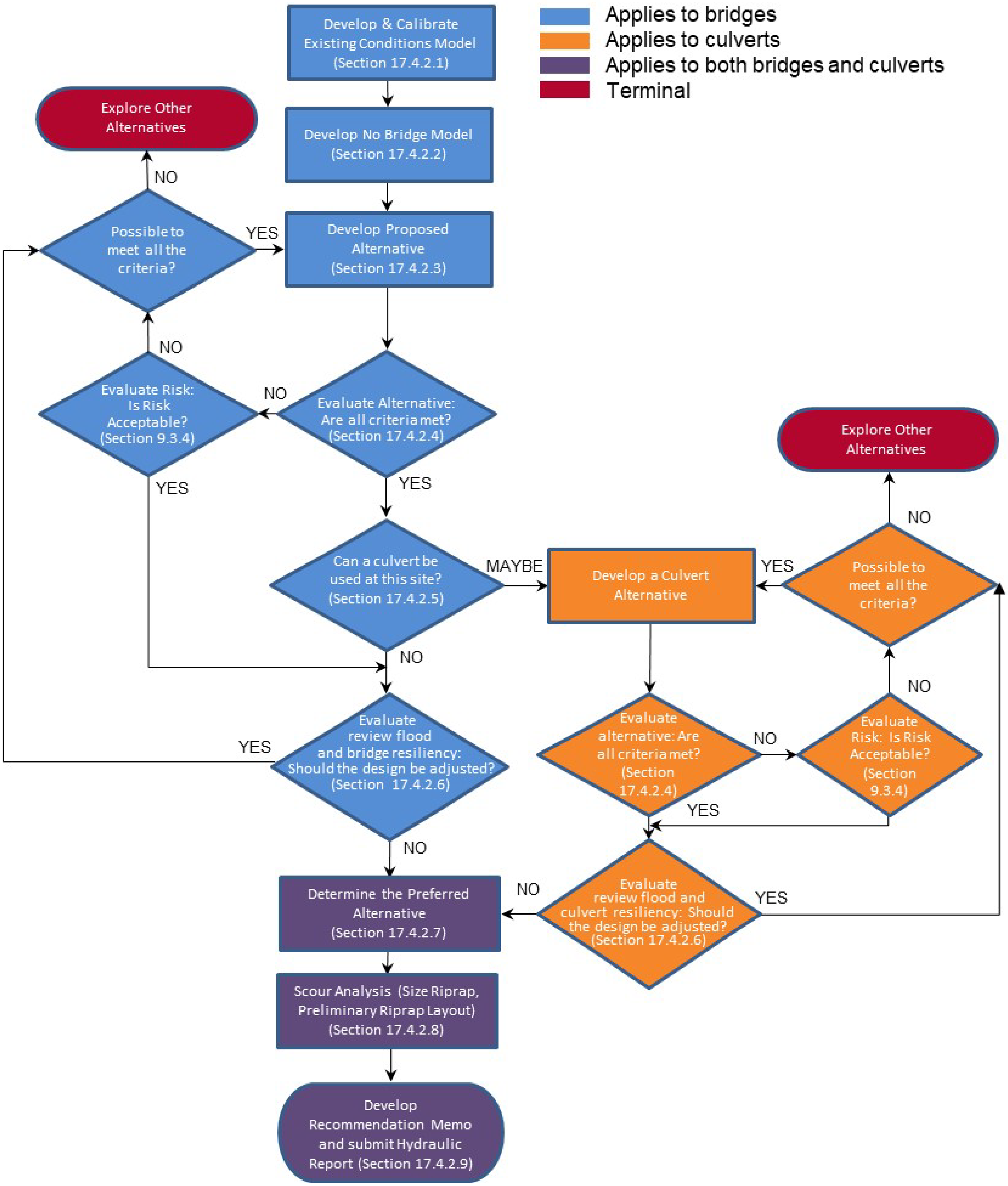
- Manning’s n (1D & 2D);
- Boundary condition(s) (1D & 2D);
- Cross sections (locations and elevations of ineffective flow, levees, blocked obstructions, etc.) (1D); and
- Discharge (review hydrology) (1D & 2D).

The types of information that can be used to calibrate the water surface profile model are discussed in the following paragraphs. Some of these approaches are reasonably accurate, and others only provide an order of magnitude check on the accuracy of the model.

At some locations, a stage-discharge relationship may have already been measured for the bridge channel. These usually exist at gaging stations on streams monitored by USGS. Measured stage-discharge curves will generally yield more accurate estimates of water surface elevation and should take precedence over the analytical methods described below. Consult the USGS NWIS (3) website for measured stage-discharge information.

On streams with a stream gage, it is sometimes possible to relate a known discharge to a known elevation. For example, perhaps the flood of 1978 overtopped the roadway or reached the doorstep of a house. Using the flow data from the stream gage and an approximate elevation of the doorstep, the model can be calibrated to reflect this known condition.

**Figure 17.4-1 — BRIDGE REPLACEMENT HYDRAULIC OPENING DESIGN**



There are also aerial photographs of some flood events available from various sources, including MDT and NRCS. The photographs can be used, supplemented by limited field survey, to determine a water surface elevation on the date of the photograph. With information from a stream gage, this can be used to calibrate the model. On some streams, the time of day of the photograph must be correlated with the time of day at the stream gage. Some streams have significant daily flow variations.

Sometimes, interviewing the local MDT maintenance personnel will indicate that the existing bridge frequently has water near or above the low beam. In this case, the existing bridge model should indicate that the water surface elevation reaches the low beam elevation during a relatively frequent event. Maintenance personnel may also indicate that they have never seen water near the low beam. This lack of information may not help, or it may indicate that the water surface elevation does not reach the low beam except during an infrequent event. The information from interviews can provide only an order of magnitude check on the model.

On streams without a stream gage, the use of the water surface elevations on the date of survey can be used to calibrate the low flow range of the model. With a series of known water surface elevations throughout the model, the model can be adjusted to match the known elevations. This calibration is typically not appropriate for high flows.

For many streams, a flow between the 2-year and the 10-year event is generally contained within the stream banks, and larger events start to inundate the floodplain. A model that indicates that the 100-year flood is contained within the stream banks would generally be suspect.

If a pier is skewed to the flow direction, it is necessary to increase the pier width in the model to reflect the effective width perpendicular to the flow. A similar procedure is necessary to address square bridge abutments on a skewed crossing.

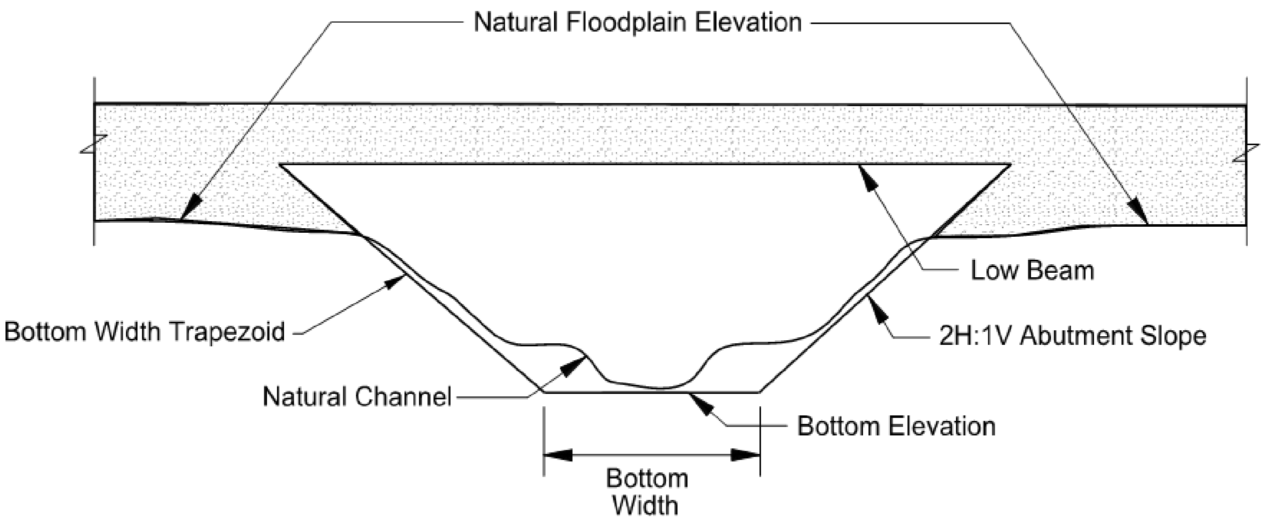
### 17.4.2.2 Develop No Bridge Model

After the existing model is calibrated, remove the bridge and roadway to the natural ground (to the extent practicable) to create a "no bridge" model. This model will be used to compare backwater to a more natural condition without the highway impacts.

### 17.4.2.3 Develop Initial Proposed Alternative

The bridge opening alternatives are defined as a trapezoid section that best represents the natural channel upstream and downstream of the bridge. The trapezoid size is defined by the bottom width, bottom elevation, and abutment side slopes. See Figure 17.4-2. The bridge opening recommendation is based on a trapezoidal section for simplicity, but the hydraulic model should include a natural channel bottom. Tie to floodplain benches upstream and downstream when present or applicable.

**Figure 17.4-2 — INITIAL BRIDGE OPENING ALTERNATIVE TEMPLATE**




Initiate the bridge opening alternatives analysis by developing the minimum bridge opening model using the following assumptions:

- The face of the spill-through abutment should be outside of the natural channel limits.
- Use 2H:1V abutment slopes (other abutment slopes may be used on occasion, but 2H:1V slopes are standard).
- Set the channel bottom for the trapezoid at the lowest channel elevation in the immediate vicinity of the bridge. For bridges with deep local scour, a more reasonable channel bottom elevation may be determined by plotting the average thalweg profile over a long distance and selecting the elevation at the bridge location.
- If a skewed bridge opening is appropriate, determine the skew angle.
- Measure the bottom width perpendicular to the stream.

### 17.4.2.4 Evaluate Alternative

Determine if the alternative meets the following criteria and guidance:

- Site constraints:
  - The alternative fits the site:
    - The abutment slopes align with the upstream and downstream banks.
    - The abutment slopes do not encroach on the natural channel.
    - If appropriate, skew the bridge to align with the channel and floodplain.

- Water surface elevation:
  - Floodplain — The design meets the allowable floodplain criteria as described in Section 17.3.1.5.
  - Measure backwater at a location approximately one bridge length upstream.
  - Backwater compared to “No Bridge” — A guideline for determining an appropriate proposed bridge size is to target a backwater of 0.5 ft and 1.0 ft (or less) at the design event, where practicable.
  - Existing water surface elevation — The proposed 100-year water surface elevation is at or lower than the existing condition (see Section 17.3.1.2.2), or the site risk has been evaluated and an increase in water surface is acceptable (see Section 17.3.1.6).
  - The alternative passes the design flood without overtopping the roadway (see Section 17.3.1.2.1).
  - Freeboard and Overtopping —The freeboard and overtopping meet the site requirements as described in Section 17.3.1.3.
- Contraction scour:
  - The contraction scour is less than 3.0 ft at the scour design event (see Section 17.3.1.4.1).

Adjust the bottom width until all evaluation criteria can be met. In general, good practice is to match the channel width upstream and downstream of the bridge and to tie to floodplain benches upstream and downstream when present. Do not adjust the natural channel width or replace the natural channel with a trapezoid; maintain the natural channel and increase the width of the floodplain benches through the bridge opening to make a larger opening. If roadway embankments impinge on the floodplain or channel, the embankments may need to be removed and a natural channel constructed.

A minimum of three alternatives should be analyzed. Typically, the alternatives include the recommended alternative, one smaller alternative (unless the minimum alternative is selected), and one larger alternative. The differences in the bottom widths among the alternatives should be relative to the bridge length. For example, a bridge with a 100-ft bottom width should have alternatives that vary by approximately 15 ft to 20 ft; a 30-ft bottom width bridge should have alternatives that vary by approximately 5 ft to 10 ft.

If all of the above criteria cannot be met, refer to Section 17.3.1.6 to evaluate the risk.

### 17.4.2.5 Culvert Alternatives

If the above criteria can be met and the proposed bridge length is short, consider if a culvert can be used for the crossing. For example, in many cases, existing 19-ft and 25-ft span timber bridges can be replaced with culverts. Additional considerations include ice, fish passage, wildlife crossings, stock passage, site constraints, fill heights, constructability, and schedule.

When replacing a bridge with a culvert, the allowable headwater used for culvert design is typically the backwater of the existing bridge. Complete a Risk Evaluation per Section 9.3.4 when deciding whether to replace a bridge with a culvert. Where there is no site risk or NFIP floodplain, the backwater may be increased within the limits of Figure 11.3-1 with approval from the State Hydraulic Engineer.

#### 17.4.2.6 Evaluate Review Flood and Bridge or Culvert Resiliency

Evaluate all bridge replacement structures with the review flood to check for unexpected flood hazards:

- Did the water surface increase from the existing condition?
- Are structures being flooded that did not flood under the existing condition?
- Did the overtopping location or the overtopping frequency change?
- Is the structure overtopping? If so, is the structure designed for overtopping?

High-risk or high-value structures warrant additional analysis to determine bridge resiliency to extreme events. Rather than focusing on a single design flow, consider the resilience of the design over a range of possible outcomes that reflect both data and model uncertainty to the extent that these can be quantified. Potential flow ranges include:

- Confidence intervals at the $Q_{100}$ event,
- The range of discharges for the methods used for the hydrologic study, and
- Confidence intervals at the $Q_{500}$ event.

Based on the results of the review flood and/or resiliency analyses, consider adjusting the design if warranted.

For further information on resiliency, see HEC 17, 2nd Edition (2).

#### 17.4.2.7 Determine the Preferred Alternative

Use the list below to evaluate a preferred bridge alternative:

- Backwater,
- Velocity through the structure,
- Contraction scour,
- Freeboard,
- Site constraints,
- Future maintenance,
- Structure configuration (pier locations), and
- Environmental considerations.

Backwater, contraction scour, or site constraints typically control the recommended hydraulic opening, but the recommended bridge opening should consider the entire list.

### 17.4.2.8 Scour Analysis

Use the methods in HEC-18 to complete a total scour analysis for the preferred alternative:

- Long term degradation of the riverbed,
- Contraction scour at the bridge, and
- Local scour at the piers and abutments.

Size the riprap for the abutments using HEC 18 (4). MDT uses a minimum of Class II riprap for bridge abutments.

Complete a preliminary riprap layout that shows the proposed horizonal and vertical riprap extents and an estimation of the proposed finished-grade contours showing how the riprap ties into the upstream and downstream banks.

### 17.4.2.9 Develop a Recommendation Memo and Report

Follow the templates provided in Appendix 17A, and prepare a recommendation memo and hydraulic report. The recommendation memo is used by the Road Design Section and Bridge Bureau to lay out the recommended bridge alternative. The hydraulic report should document the assumptions and decisions made through the bridge hydraulic design process.

# 17.5 HYDRAULIC MODELING GUIDELINES

The analysis approach for bridge hydraulics is either one-dimensional (1D) or two-dimensional (2D) modeling. Section 17.5.1 includes information on selecting the most appropriate approach. This section provides background on developing input data and other considerations that are common to all bridge hydraulic analyses regardless of the specific approach.

## 17.5.1 Hydraulic Model Selection

*Hydraulic Design of Safe Bridges*, Hydraulic Design Series No. 7 (HDS 7) (1) provides a detailed discussion of both 1D and 2D hydraulic modeling techniques. In addition, HDS 7 compares and contrasts 1D and 2D modeling. MDT uses both 1D and 2D models and, prior to requesting the survey, the hydraulic engineer decides whether to use a 1D or 2D model to analyze the bridge crossing.

Generally, simple bridge crossings with uniform channels and no overtopping may be completed with a 1D model. In addition, most FEMA floodplain analyses will require a 1D model. However, complex bridge crossings are more accurately represented with a 2D model, which can then be used to calibrate or inform the 1D model.

MDT encourages the use of 2D hydraulic models in appropriate situations consistent with staff expertise and project resources. 2D models represent waterways and their interactions with bridges, or other transportation infrastructure that encroaches on a waterway, in a more comprehensive application than 1D models. 2D models allow a more realistic variation of key variables, including velocity and water surface elevation, across a river or other water body. In addition, the 2D model provides better longitudinal variation in these same variables. This improved representation results in better hydraulic designs that improve the stewardship of project resources, including time and budget. Figure 17.5-1 compares appropriate 1D and 2D modeling applications.

**Figure 17.5-1 — BRIDGE HYDRAULIC MODELING SELECTION**

| Bridge Hydraulic Condition | Hydraulic Analysis Method | |
|---|---|---|
| | 1-D | 2-D |
| Small streams | ● | ◗ |
| In-channel flows | ● | ◗ |
| Narrow to moderate-width floodplains | ● | ◗ |
| Wide floodplains | ◗ | ● |
| Minor floodplain constriction | ● | ◗ |
| Highly variable floodplain roughness | ◗ | ● |
| Highly sinuous channels | ◗ | ● |
| Multiple embankment openings | ◗/○ | ● |
| Unmatched multiple openings in series | ◗/○ | ● |
| Low skew roadway alignment (< 20°) | ● | ◗ |
| Moderately skewed roadway alignment (> 20° and < 30°) | ◗ | ● |
| Highly skewed roadway alignment (> 30° ) | ○ | ● |
| Detailed analysis of bends, confluences, and angle of attack | ○ | ● |
| Multiple channels | ◗ | ● |
| Detailed flow distribution at bridges | ◗ | ● |
| Significant roadway overtopping | ◗ | ● |
| Upstream controls | ○ | ● |
| Countermeasure design | ◗ | ● |

● Well suited or primary use
◗ Possible application or secondary use
○ Unsuitable or rarely used
◗/○ Possibly unsuitable depending on application

*Source*: HDS 7 (1)

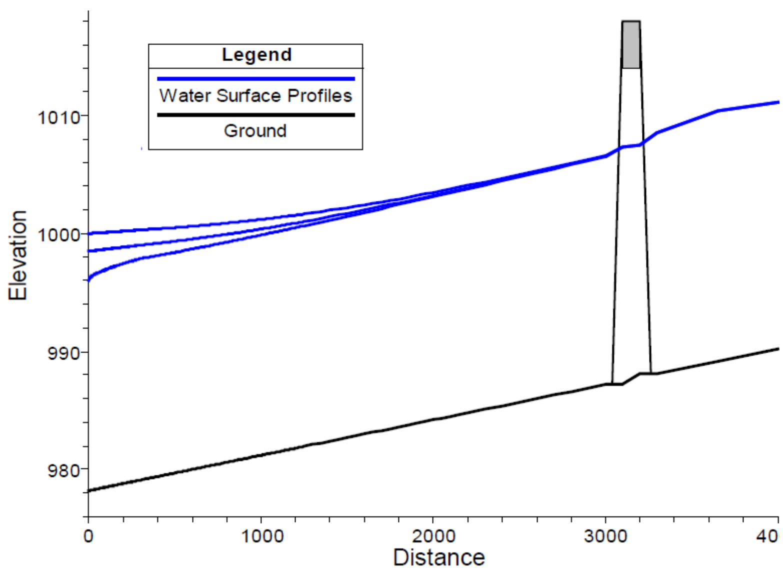
All numerical hydraulic models (1D and 2D) incorporate simplifying assumptions, require certain types of input data, and operate under specific implementation limitations. The goal of all hydraulic model studies is to simulate anticipated flow conditions as accurately as possible within project constraints, without violating the assumptions or ignoring the limitations of the model. Thus, the hydraulic engineer must be aware of and understand the underlying assumptions of the model selected because they form the limitations of that approach.

Select the modeling approach based primarily on its advantages and limitations, considering also: the importance of the structure, potential interactions with the waterway, and potential project impacts, cost, and schedule.

## 17.5.2 Selecting Upstream and Downstream Model Extent

If the downstream starting water surface elevation is not known with confidence, then extending the model further downstream will provide more distance for the model to reach equilibrium. This is illustrated in Figure 17.5-2, which shows water surface profiles for a simple bridge model. The three profiles are for the same discharge, the only difference being the downstream boundary condition. Each profile represents a valid solution to the equations of fluid motion. The downstream boundary is located far enough downstream so that the profiles converge, and the 4.0 ft of initial difference is eliminated before reaching the bridge. Convergence is reached if a range of starting water surface slopes all converge to essentially the same elevation downstream from the structure.

**Figure 17.5-2 — FLOW PROFILES WITH DOWNSTREAM BOUNDARY**




*Source*: HDS 7 (1)

Generally, the modeling extent for a two-dimensional model should extend at least two floodplain widths upstream and downstream of the bridge. It is also desirable to select locations to begin and end the model where flow is reasonably one-dimensional, especially at the downstream boundary or at a location with a hydraulic control. This is because the downstream boundary is usually specified as a constant water surface elevation along the boundary.

Modeling extents for 1D models should extend approximately 1500 ft downstream of the bridge and approximately 500 ft upstream of the bridge. Flatter slopes should start further downstream and far enough upstream that the existing, no bridge, and proposed water surface profiles converge away from the bridge location.

With both 1D and 2D models, the further the boundary is located away from the crossing being assessed, the less influence the boundary condition will have on the results. Where there are other structures or hydraulic controls either upstream or downstream that will influence or can be impacted by the project, extend the modeling to include these structures.

## 17.5.3 Identifying and Selecting Model Boundary Conditions

An important part of the hydraulic engineer's responsibility is to select representative boundary conditions for the hydraulic analysis. Peak discharge is used for steady flow analyses and flood hydrographs are used for unsteady flow analyses.

For one dimensional models, the downstream water surface must be specified or computed for subcritical flow computations. For supercritical flow, the upstream condition is specified and, for mixed flow conditions, the downstream and upstream conditions are specified.

The model extent (Section 17.5.2) and boundary conditions should be selected based on identifiable hydraulic controls or on other reliable information. There are several types of hydraulic controls that can establish the boundary condition for a model. These include slope breaks where critical depth occurs (from flat to steep in the downstream direction), diversion dams, bridges, roads, natural channel constrictions, and other structures.

The discussion in the following sections relates to a downstream boundary condition for both 1D and 2D hydraulic models. This guidance also applies to upstream boundary conditions for supercritical or mixed flow 1D models. Boundary conditions are discussed further in Section 17.5.7.4.

### 17.5.3.1 Water Surface

A known water surface is very commonly used in hydraulic modeling as a starting downstream condition. Sources of water surface elevation data may include:

- FEMA Flood Insurance Study (FIS),
- Gage data,
- Surveyed water surface elevation, and
- Observed high water marks.

### 17.5.3.2 Normal Depth and Energy Slope

Normal depth occurs when the bed profile, water surface, and energy grade line are all parallel and the flow depth and velocity do not change along the channel flow path. This occurs relatively infrequently in natural rivers, although it can be a reasonable approximation for establishing boundary conditions in many situations.

### 17.5.3.3 Critical Depth

Critical depth is a relatively well-defined boundary condition that occurs when a control structure produces a sudden drop in the channel. Critical depth in natural channels is unusual except in steep, bedrock, or boulder-bed channels. In HEC-RAS (5), critical depth is defined as the minimum total energy. In a natural channel, total energy includes the energy correction coefficient ($\alpha$); therefore, roughness and flow distribution impact the determination of critical depth. Critical depth should be confirmed as reasonable before its use as a boundary condition in natural channels.

### 17.5.4 Tailwater Relationship (Confluence or Large-Water Body)

Where the bridge is located within the hydraulic influence of a downstream confluence or large water body (e.g., river or lake), the hydraulic engineer can perform the following to determine the frequency of the corresponding tailwater:

- Use conservative engineering judgement; for example, use the same flood recurrence intervals for both the design location and the downstream water body; or

- Apply the procedure in NCHRP Web-Only Document 199, *Estimating Joint Probabilities of Design Coincident Flows at Stream Confluences*, Appendix H, Step-by-Step Application Guide, which is available online.

### 17.5.5 Manning's n Value Selection

Manning's n selection in natural channels depends heavily on engineering experience, because of the many factors involved and their variation. Photographs of channels and floodplains for which the discharge has been measured and Manning's n has been calculated are very useful. The following may be used as guides for Manning's n selection:

- USGS, Water Supply Paper (WSP) 2339 (6);
- USGS, WSP 1849 (7);
- USGS WRI 85-4004 "Determination of Roughness Coefficients for Streams in Colorado" (8);
- "Roughness Characteristics of New Zealand Rivers" (9);
- Flood Insurance Studies for the area;
- Chapter 10, "Channel Design"; or
- Open Channel Hydraulics, by Ven Te Chow.

For situations outside the hydraulic engineer's experience, a more regimented approach is presented in WSP 2339 (6). Once the Manning's n values have been selected, verify or calibrate the values with historical high-water marks and/or gaged streamflow data. When calibration data does not exist, perform a sensitivity analysis and adjust the Manning's n values within acceptable ranges to determine how much impact they have on the water surface elevation. Calibration is discussed further in Section 17.4.2.1.

The hydraulic engineer should select values that are within acceptable ranges, and the justification should be documented.

### 17.5.6 1D Modeling Guidelines

In addition to the information below, refer to HDS 7 (1) for a comprehensive discussion on 1D modeling.

#### 17.5.6.1 Cross Sections

The geometry of stream channels and their adjacent floodplains is defined by cross sectional coordinates of lateral distance and ground elevation that locate individual ground points. The cross section is taken normal to the flow direction along a single, straight line where possible but, in wide floodplains or bends, it may be necessary to use a section along intersecting straight lines (e.g., a dog-leg section). A plot of each cross section is essential to reveal any inconsistencies or errors. See HDS-7 for additional information.

Locate cross sections to be representative of the sub-reaches between them. Cross sections should be:

- Located at all major breaks in bed profile;
- Located at the toes of the embankment slopes both upstream and downstream of the bridge;
- Located one bridge length upstream for backwater analysis;
- Located at areas of interest such as houses;
- Placed at points of minimum and maximum cross-sectional areas;
- Placed at shorter intervals in expanding reaches and in bends;
- Placed at shorter intervals in reaches where the conveyance changes greatly as a result of changes in width, depth, or roughness;
- Located at points where roughness changes abruptly; for example, where the floodplain is heavily vegetated or forested in one sub-reach but has been cleared and cultivated at the adjacent sub-reach;
- Placed between sections that change radically in shape, even if the two areas and the two conveyances are nearly the same;
- Placed at shorter intervals in reaches where the lateral distribution of conveyance in a cross-section changes radically from one end of the reach to the other, even though the total area, total conveyance, and cross-sectional shape do not change substantially;
- Located at locations of estimated flow splits;
- Located at and near control sections, including locations such as irrigation diversion structures; and
- Bend where necessary so that the cross section is always perpendicular to the flow direction.

Include cross sections far enough upstream to model areas of significant risk, especially floor elevations and the ground around buildings.

The effects of almost all undesirable features of nonuniform, natural stream channels can be lessened by taking more cross sections. However, consideration must also be given to the time, cost, and effort to locate and survey additional cross sections. The criteria for cross section locations serve, therefore, to call attention to the considerations behind the need for cross sections and to assist the hydraulic engineer in understanding the anomalies in computed profiles if cross sections are omitted.

The following stream locations require cross sections at shorter intervals to better model the change in conveyance:

- Major breaks in bed profile;
- Abrupt changes in roughness or shape;
- Control sections (e.g., free overfalls, bends, contractions); or
- Other abrupt changes in channel slope or conveyance.

Compute the conveyances of each subsection separately to determine the flow distribution, and then add them together to determine the total flow conveyance. Choose the subsection divisions carefully so that the distribution of flow or conveyance is nearly uniform in each subsection (see USGS TWRI (10)).

### 17.5.6.2 Bridge Data Input

For contraction/expansion areas, ensure that the:

- Coefficient losses are entered correctly (usually 0.3 contraction and 0.5 expansion, and extend through the contracting and expanding zones for the bridge or at other natural contractions/expansions);

- Ineffective areas are located correctly (usually, 1:1 contraction upstream and 1:2 expansion downstream); and

- Cross sections are located at fully expanded sections upstream and downstream.

If the bridge is in pressure flow (i.e., the water is above the bridge low chord), use the following to select the high-flow bridge modeling approach. Use the energy method to model bridges if the ratio of the (EGL to the existing ground) to (low chord to existing ground) is less than or equal to 1.2. If the ratio is greater than 1.2, use the pressure/weir method. When the ratio is less than 1.2, pressure flow has not fully developed.

If overtopping of the roadway is occurring, the pressure/weir method may need to be used to calculate the weir flow over the roadway, even if the bridge is not in pressure flow.

If a significant amount of water is overtopping the roadway, the overtopping area may need to be modeled as a conveyance area using a multiple opening analysis.

### 17.5.6.3 Output Analysis

After a model has been established, the following should be considered:

- Compare top width, cross sectional area, and velocity between sections. These parameters should not vary significantly between adjacent cross sections.

- Plot thalweg, computed water surface elevations, and critical depth elevations to detect irregularities that may indicate problems with the model.

- Critical depth error messages should be carefully reviewed. Critical depth generally should not occur, except possibly at the bridge section. Critical depth at a cross section makes that section a control section, and the downstream channel has no influence on the water surface elevations upstream from the control section. Output with numerous critical depth messages should be highly suspect.

- For detailed Flood Insurance Studies, it is generally necessary to match the published FIS water surface elevations within 0.1 ft for the existing condition. This can sometimes be accomplished by obtaining the original FIS model from FEMA. When it is not possible to match the FIS, the Hydraulics Section should be consulted for additional direction.

- Where modeling difficulties occur, adding additional cross sections may help. If necessary, additional cross sections can be added from the available survey, by interpolation, or by copying the nearest cross section and adjusting for slope.

- Check for continuity between cross sections, especially when overtopping occurs. The following should be consistent:

- Flow widths;

- Flow in right overbank, channel, left overbank;

- Ineffective flow area locations; and

- Manning's n-values.

- Ineffective elevations should be modified in all cross sections that have ineffective flow areas that are caused by the roadway, both upstream and downstream of the roadway crossing. For example, if the roadway overtops at the 50-year flood, all water surface elevations for the 50-year flood and greater should be effective and all water surface elevations below the 50-year flood should be ineffective.

- When overtopping occurs, the flows in the overbanks upstream should be consistent with the flows in the overbanks downstream. Similarly, the flow in the bridge section upstream should be consistent with the flow in the bridge section downstream. The overtopping flow should be included in the comparisons.

### 17.5.7 2D Modeling Guidelines

In addition to the information below, refer to HDS 7 (1) and the FHWA- HIF-19-061, "2D Hydraulic Modeling for Highways in the River Environment - Reference Document" (11) for a comprehensive guide to the use of 2D modeling.

### 17.5.7.1 Metadata

Most 2D modeling platforms include a metadata section or function, which is a location where important information relating to the model can be stored. The Hydraulics Section requires that, at a minimum, the following information be included in the metadata for each model:

- Project information — name, number, control number;
- Modeler information — name, title, company;
- Hydrology information — brief summary on hydrologic method used;
- Terrain data (may include multiple sources/surveys) — sources, types, dates (bathymetry collection date versus topography collection date), vertical datums, and coordinate systems;
- Boundary condition source — brief summary on where boundary conditions (mostly downstream) came from and confidence in accuracy;
- Roughness parameters — source (e.g., imagery), values, assumptions;
- Hydraulic structures — source of data, coordinate system, vertical datum;
- Background, GIS, other — source, coordinate system, vertical datum;
- Monitoring line correlation (e.g., LN1 is used for the line that is farthest upstream); and
- Simulation naming conventions — include a brief summary of simulation naming conventions unless it is obvious in the name of the simulations.

### 17.5.7.2 Floodplain and Channel Topography Data

The topographic data used to develop a 2D model is typically segregated into two areas — the bathymetry data and the remaining channel and floodplain above the water surface. Recommended practice is to gather enough topography data to develop a terrain that spans beyond the anticipated limits of the model in all directions.

Bathymetry data, the elevation of a channel below the water surface, is most often collected in one of two ways — through ground survey collected by personnel or sonar collected from a boat. Typically, when ground survey is utilized, it is through the collection of several channel cross sections that span from top of bank to top of bank of the channel. These cross sections are then used to develop the channel topography. This channel topography is established by "stamping" the channel into the floodplain topography or by developing a "channel-only" topography set that can be merged with other data.

The collection of the terrain data necessary for the development of the floodplain topography can be accomplished in several methods; the following are the most common: Photogrammetry survey conducted by MDT survey forces in conjunction with limited ground survey work necessary to capture any void areas. Another method is to collect the data using LIDAR. Due to discrepancies in vertical data, nationally available DEM data is not recommended for use in MDT hydraulic models.

Topographic data should be analyzed for continuity between collection methods. Ensure that floodplain topography and the bathymetry topography transition well. If individual cross sections are used to create a low-flow channel, ensure that triangles do not span the channel causing obstructions within the topography. Use breaklines for accurate triangulation.

When it is known that a 2D model will be used, presurvey meetings should be held with the entity collecting the survey to discuss what is needed and to determine the best method(s) to gather the needed survey data. The collection dates of the topographic data and the bathymetry data should be coordinated to prevent significant gaps between collection dates.

Note that an MDT survey is typically completed in state plane coordinates, with US survey feet in the vertical plane and International survey feet in the horizontal plane. A combination scale factor (CSF) is also typically developed and can be used to adjust the model projection.

#### 17.5.7.3 Mesh

The first step to establish the limits of the mesh is to determine the best locations for the upstream and downstream boundary conditions. The inlet boundary condition should be located where the flow would appear to be confined to one channel, is uniform, and is hydraulically stable. Exit boundary conditions should be positioned at a straight section that is as narrow as possible given the site conditions.

When developing the initial extents of the 2D modeling mesh, ensure that the extents are captured by the available topography data. Typically, the mesh should contain less than 100,000 elements, even for larger rivers. The size of the mesh elements and the total elements are dependent on the site being modeled. It is recommended to use larger elements in the broad floodplain areas that see little change in flow direction, and then utilize smaller elements in areas where more hydraulic definition is desired (e.g., along a flood control dike). In short, the element size should be congruent with the change in slope of the area in question. The density of the points used to develop the model terrain should also be considered. If ground shots are spaced far apart, the mesh element size should be larger.

Two standard types of mesh element can be created in SMS/SRH-2D (i.e., the MDT preferred 2D model) — triangular elements and quadrilateral elements. Other 2D modeling platforms allow for elements with more sides. A mixed use of mesh element types is best for model accuracy and efficiency.

When developing a mesh, it is important that all features that will drive the hydraulics of the site are properly defined. Features such as the primary channel, flow training dikes, roadways, or buildings of concern should be delineated with arcs to ensure that they are properly shown in the mesh contours. Triangular elements are adequate for most of the floodplain area outside the main channel.

Quadrilateral elements are preferred for use for the following features:

- Primary channels and, at times, well-defined side channel or braided sections;
- Roadways, especially if overtopping is expected; and
- Flow training features (e.g., dikes).

When delineating channels within a mesh, it is preferable to use quadrilateral elements placed as parallel to the flow as possible. Use several vertices across the channel; elements should not be smaller than the anticipated flow depth. The orthogonal length of the elements will typically be longer than the width, unless in-stream structures are present.

When a bridge does not experience pressure flow and contains piers, it is acceptable to utilize triangular elements through the bridge opening to achieve an acceptable mesh quality and better define the flow patterns around the piers. The more refined the mesh is under the bridge, the more accurate the velocity distribution will be.

Embankments such as roadways and flow training dikes should use a minimum of two quadrilateral mesh elements across the top of the feature and a minimum of two mesh elements along the side slopes.

To observe in-depth water surface elevations at desired structures/obstructions, the mesh elements should be ½ to ⅓ the size of the obstruction itself.

The quality of the mesh should be reviewed, and errors fixed where possible, especially in the area of concern (e.g., upstream and downstream of the bridge opening).

### 17.5.7.4 Boundary Conditions

The two boundary conditions that must be included in a 2D model are the inlet boundary condition arc and the outlet boundary condition arc. These arcs should be as simple as possible (minimal vertices) and be drawn as perpendicular to channel flows as possible. The inlet boundary condition is typically in the form of a known flow rate for steady state models and hydrographs for unsteady state models.

The outlet boundary condition is typically in the form of a known or estimated water surface elevation for steady state models or a stage-elevation curve for unsteady models.

Properly size the inlet and outlet boundary arcs for the flow being assessed. The inlet arc should span the anticipated area that would be carrying water at the given peak flow rate. Likewise, the outlet arc should span the area of the floodplain carrying water at the end of the model. The arc lengths may need some adjustment after initial model runs.

Within 2D models several hydraulic features are modeled as boundary conditions. These features include bridges in pressure flow, culverts, and obstructions. Bridge boundary arcs are discussed in Section 17.5.7.7, and culvert boundary arcs are discussed in Section 17.5.7.8. Also, see Section 17.5.3 for further discussion on boundary conditions.

### 17.5.7.5 Materials Roughness Coverages (Manning's n values)

An initial materials coverage should be developed based on field visits, photos, and aerial photos of the site. See Section 17.5.5 for additional information on selecting appropriate Manning's n values. The materials coverage should extend to the same limits as the mesh or further to ensure that the mesh has a materials coverage throughout.

1D models use the Manning roughness coefficient to account for multiple losses in addition to roughness, and the Manning's roughness coefficient in a 2D model only represents the roughness loss. Therefore, 2D models may have lower roughness values than a 1D model to reach the same calibration elevation.

If calibration data is available, the materials roughness coefficients will be the first variables adjusted to achieve model calibration.

A materials roughness sensitivity analysis should be completed if calibration data is not available. Adjust the roughness coefficients within acceptable ranges to determine their effect on the final solution of the model.

### 17.5.7.6 Model Controls

SMS/SRH-2D model controls are discussed below:

<u>Model Name</u>. The simulation description and the case name are used to describe the model simulation in the outputs. These fields should provide adequate information to describe the simulations (e.g., existing condition, 100-year).

<u>Time Control</u>. The model time is controlled by setting the time step and by selecting the start and end time.

> <u>Time Step</u>. The calculation time step should be set between 1 and 10 seconds, depending on the smallest element size in the mesh. Complex models or models involving multiple bridges or culverts may require a time step less than 1 second to achieve a stable result. It is recommended to start with a larger time step and reduce it as necessary to help minimize model run times.
>
> <u>Start Time and End Time</u>. The total run time is set by entering the start time and end time and is used to direct the model the number of calculations to make with the time step that is specified. For steady state models, the modeler can read the outputs from the monitoring lines and observe at what time step the model converged. Then, the modeler can shorten the end time to the minimum time needed to reach continuity. Shortening the end time will help to achieve a quicker calculation time for the model.

<u>Initial Condition</u>. The initial condition of the model will typically be dry; however, the use of a model restart file or a starting water surface elevation can be used to speed up model run times after initial test runs.

<u>Data Output</u>. The data output rate should be in the range of 5 to 30 minutes, depending on total model run time. Data output is how often the model reports results.

### 17.5.7.7 Bridges

Bridge openings can be modeled differently in SMS/SRH-2D, depending on the opening configuration and whether the bridge is in pressure flow for any of the assessed flows.

Bridges not in pressure flow are modeled as an opening in the terrain, and the bridge superstructure is not included in the model.

If a bridge will be in pressure flow for any of the assessed flows, then boundary arcs must be placed at the upstream and downstream faces of the bridge. These arcs should span far enough to reach a topographic elevation at or above the top of the bridge deck. Failure to extend these arcs far enough will produce faulty results. Additional information about modeling bridges in pressure flow may be found in FHWA-HIF-19-061, "Two-Dimensional Hydraulic Modeling for Highways in the River Environment – Reference Document" (11).

The quadrilateral elements through the bridge opening need to be:

- Perpendicular to the bridge opening,
- Extend at least the width of the boundary arcs, and
- Extend upstream and downstream of the boundary arcs.

Triangular elements cannot be used between pressure flow arcs.

When a bridge will be modeled for pressure flow and has piers:

- Quadrilateral elements must still be utilized through the opening,
- The maximum recommended mesh element size is the width of the pier,
- The minimum mesh element size is the pier width divided by 3,
- A buffer row of mesh elements must be placed between multiple column piers, and
- An arc must be included between the piers but not touching the piers to achieve better mesh quality.

To correctly model bridge deck overtopping, check the box indicating overtopping in the pressure flow boundary conditions setup window. Selecting this option will also produce an overtopping flow output file ("case name"_OT1.dat).

Bridges with piers can be modeled in two ways within SMS/SRH-2D. The Hydraulics Section's preferred method is to model the piers as holes in the mesh. This method allows the model to more accurately analyze the interaction with channel flows and the pier configurations.

Less preferred is to model the piers as obstructions by placing boundary arcs upstream and downstream of the bridge. This method computes the head loss through the bridge opening due to the pier obstruction and does not show how velocities and angles of attack are affected by the piers.

#### 17.5.7.8 Culverts

Culverts in SMS/SRH-2D models are modeled in a 1D regime using HY8. The HY8 file is linked to the 2D model for calculations within the model run. The following guidance should be followed when modeling a culvert in 2D:

- Place boundary arcs upstream and downstream of the culvert.
- The boundary arc location and mesh elevation must correspond to the culvert invert elevations listed in HY8.

- Use quadrilateral elements upstream and downstream of the culvert boundary arcs and align the elements with the culvert ends and flow direction. Check that the boundary arcs are snapped to the appropriate mesh elements. Also, verify that the mesh elevations are not blocking the culvert entrance or exit.

- Link the HY8 file to the culvert boundary arcs; remember that the last culvert viewed in the HY8 file will be the one connected to the 2D model. It is recommended to create a separate HY8 file for each culvert in a 2D model.

Place monitoring lines upstream and downstream of the culvert boundary arcs. These monitoring lines can be used to check continuity through the culvert and also used to check against the culvert output file ("case name" _HY1.DAT in SMS/SRH-2D).

If a culvert is a box shape and does not flow full for the range of flows being assessed, then it may alternatively be modeled as a square opening in the mesh in SMS/SRH-2D.

### 17.5.7.9 Model Monitoring

All 2D models should include monitoring points and monitoring lines to check the stability and the continuity of a model. Monitoring <u>points</u> should be placed within the mesh at the both the downstream and upstream ends of the model. Monitoring <u>lines</u> are recommended at both the upstream and downstream model boundaries and upstream and downstream of hydraulic features like bridges and culverts.

Monitoring points and lines are placed in their own monitor coverage. Within SMS/SRH-2D, monitoring point output files are labeled "Case Name_PT1.DAT." Monitoring lines will be labeled with the case name and LN1.DAT. The monitoring lines are numbered in the order in which they are drawn.

### 17.5.7.10 Model Review

Review the model to assess the model validity and check for possible errors. Perform the following:

- Review mesh elevations (compare to terrain data).

- Check continuity (monitor line output).

- Check stability.

- Check solution convergence (steady state).

- Review velocity ranges.

- Review hydraulics at boundary conditions.

- Plot water surface profile along the thalweg and through the bridge and review.

- Check the Froude number coverage to ensure that the model does not experience prolonged sections that are over 1.0.

After a model has been determined to be valid, proposed alternative should be evaluated per the discussion in Section 17.4.2.4.

## 17.6 SCOUR ANALYSIS AND DESIGN GUIDELINES

### 17.6.1 Overview

After the bridge waterway opening has been established, the hydraulic engineer should evaluate the estimated scour that will occur at each of the bridge elements. HEC 18 (4) recommends that bridge scour be evaluated as interrelated components.

All scour analyses should include:

- Channel instability/long-term degradation (see Section 17.6.3), and
- Contraction scour (see Section 17.6.4):
  - Local scour (see Section 17.6.5),
  - Pier scour (see Section 17.6.5.1), and
  - Abutment scour (see Section 17.6.5.2).

In addition to the components above, some scour analyses may also include:

- Pressure flow scour (see Section 17.6.9.3.3),
- Debris scour (see Section 17.6.6), and
- Scour resistant materials (see Section 17.6.7).

Refer to HEC 18 (4) for a thorough discussion on scour and scour prediction methodology. Refer to Chapter 16, "Stream Stability Assessment" and HEC 20 (12) for discussion on lateral and vertical stream stability. Refer to HEC 23 (13) for a discussion on designs for scour countermeasures.

The scour analysis should also consider the scour history of the existing bridge. The scour history can be determined by reviewing bridge inspection reports, from discussions with maintenance personnel, by reviewing the flood history, and through site visits. In some instances when further information is needed, a geotechnical evaluation may be requested to determine the difference between local materials and the materials that were deposited in the scour hole after a flood event or help determine the elevation and erodibility of bedrock.

### 17.6.2 Scour Flood Magnitudes

MDT practice is to evaluate scour at the design flood, the scour design flood, and either the scour check flood or the overtopping flood, whichever is smaller. At some locations, the greatest pier scour will occur at smaller flows, due to changes in the angle of attack. When roadway overtopping occurs, the maximum scour may occur just before the overtopping event when the maximum hydraulic pressure is exerted on the bridge.

The scour design flood is used to design scour countermeasures, and the scour check flood is used to determine the low scour elevation for the foundation design.

Scour design and check floods are defined in Section 17.3.1.4.

### 17.6.3 Channel Instability

Within the context of safe bridge design, channel instability includes any channel change that can threaten a bridge foundation. The change may be natural or result from a variety of human activities. Channel instability can create changes in channel geometry that expose foundations and increase scour during floods. Chapter 16, "Stream Stability Assessment," which is based on HEC 20 (12), provides guidance on evaluating channel instability at bridges. Even though these changes may be gradual or episodic, they are usually cumulative and are considered long term because they alter the channel over the life of the bridge. Therefore, the potential for vertical and horizontal change must be considered in safe bridge design.

Channel instability not only considers the existing conditions but also potential future conditions. Factors that may need to be considered 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 floodplain materials,
- Vegetation and land-use and potential land use changes, and
- Sediment sources and supply.

Vertical and lateral instability are often identified during bridge inspections, through channel reconnaissance during bridge design, or through comparison of recent and historic aerial photography. Hydraulic modeling and sediment transport analysis can also be used to evaluate channel instability. As discussed in HDS 7 (1) and Chapter 16, "Stream Stability Assessment," sediment transport analysis can be used to evaluate channel aggradation and degradation trends over the life of a bridge. Even when sediment transport modeling is not performed, hydraulic models, especially two-dimensional models, can provide insight into vertical and lateral channel instability potential. Locations where channel flow velocity is much higher than upstream or downstream may be prone to bed or bank erosion.

Models can be used to predict a future condition, but they can also be used to evaluate potential future conditions by configuring the model for expected channel changes. Model results should never be interpreted without considering the river characteristics. Geologic controls, sediment characteristics, vegetation characteristics, and manmade features may counteract erosion that may be expected from reviewing model results. It is important that the channel reconnaissance be performed and that the hydraulic engineer develops an understanding of a wide range of fluvial geomorphic processes and potential channel response as discussed in HEC 20 (12).

### 17.6.3.1 Vertical Instability

Vertical instability is usually referred to in terms of aggradation and degradation. Aggradation and degradation are long-term streambed profile changes due to natural or man-induced causes. Aggradation is the deposition of bed load due to a decrease in the energy gradient. A braided channel is frequently an indication of aggradation. Degradation is the scouring of bed material due to increased stream sediment transport capacity that results from an increase in the energy gradient. A head cut is frequently an indication of degradation.

Vertical change results from a long-term excess or deficit in sediment supply and from degradation caused by headcutting and a loss of a downstream control. Long-term trends in discharge also impact channel geometry because channels that convey larger flows tend to be wider and deeper. If a channel consistently conveys more water than it did historically, the channel will enlarge. This can occur due to increased runoff from urbanization, from climate change, and due to many other causes. Bridge inspection reports that include repeat cross section measurements are useful in identifying aggradation and degradation problems and trends. The sediment transport chapter of HDS 7 (1) includes the discussion of sediment continuity and how sediment transport concepts can be used to analyze aggradation and degradation when there is an imbalance of sediment supply and transport capacity.

Headcuts occur when channel degradation progresses up the channel and are caused when the downstream base level of a channel is lowered. Figure 17.6-1 from HDS 7 (1) shows a headcut that will migrate upstream and through the bridge crossing during future runoff events. Features of a headcut that can threaten a bridge include long-term degradation that persists after the headcut has migrated upstream of the bridge, the plunge pool when headcut is under the bridge, and channel widening that occurs because of bed lowering, which can destabilize channel banks.

**Figure 17.6-1 — HEADCUT DOWNSTREAM OF A BRIDGE**

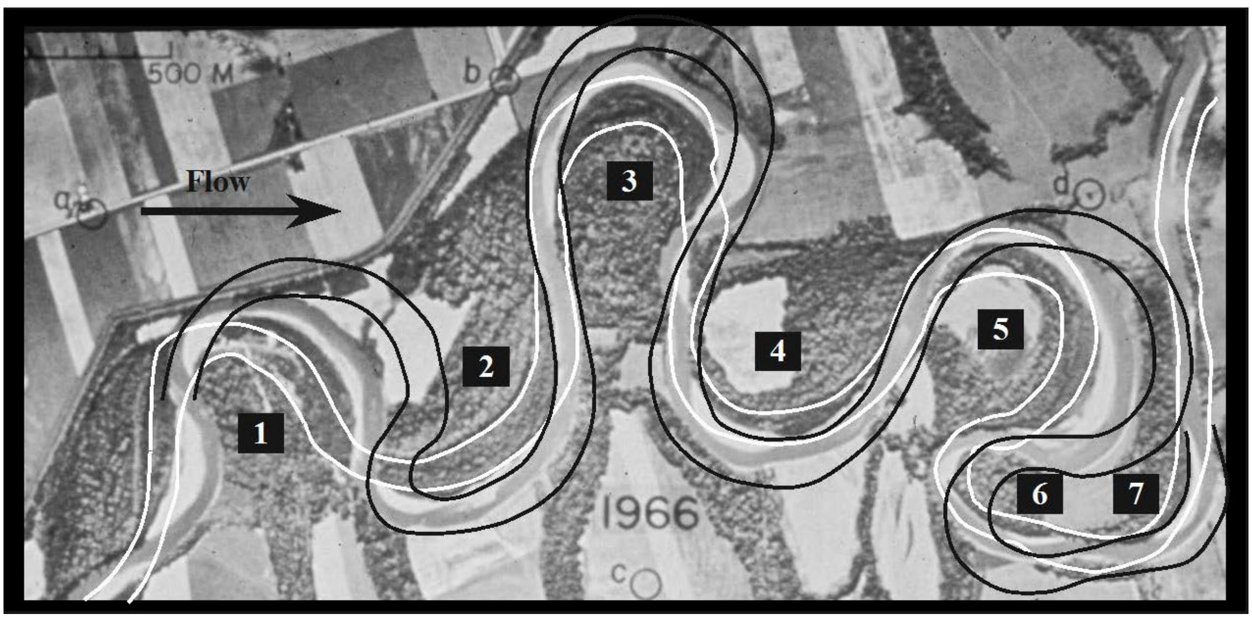
*Source*: HDS 7 (1)

### 17.6.3.2 Lateral Instability

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 historic aerial photography is not only useful for identifying the potential for lateral instability problems at a bridge but also can be used to make predictions of channel location during the life of the bridge. These photo-comparison techniques are presented in HEC 20 (12). A single flood can also cause extreme channel migration and widening, which for some regions can present significant challenges for bridge design.

Figure 17.6-2 from HEC 20 (12) shows the banklines observed in 1937 and estimated for 1998, overlain on the 1966 aerial photo. Inspection of the estimated banklines reveals that Bend 1 would encroach into the levee to the north by 1998, while the growth of Bend 5 would likely cutoff Bends 6 and 7. Registration points for the photographs are noted with the letters a, b, c, and d.

**Figure 17.6-2 — AERIAL PHOTO OF THE WHITE RIVER IN INDIANA IN 1966 SHOWING THE ACTUAL 1937 BANKLINES (WHITE) AND THE PREDICTED 1998 BANKLINE POSITIONS (BLACK)**




*Source*: HEC 20 (12)

## 17.6.4 Contraction Scour

Contraction scour is a lowering of the streambed across the stream or waterway bed at the bridge, resulting from a contraction (or constriction) of the flow. At bridges, contraction scour results from a constriction of the flow area caused by approach fills in the floodplain or, to a lesser extent, by bridge piers in the waterway. Contraction scour should be computed using the equations described in HEC 18 (4).

Highways, bridges, and natural channel contractions are the most commonly encountered causes of contraction scour, also termed general scour.

Contraction scour is 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. 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. HEC 18 includes equations for determining the critical velocity when bed material movement is initiated based on flow depth and particle size.

Clear-water conditions occur for fine sediment sizes (sands and fine gravel) only when flow velocity is low and for coarse sediment sizes (coarse gravel and cobbles) even for a relatively high velocity. 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 often not found in channel beds in significant amounts and does not generally impact either clear-water or live-bed contraction scour. The water may be turbid due to suspended transport of silt and clay but is still considered as clear-water from the perspective of bed material transport.

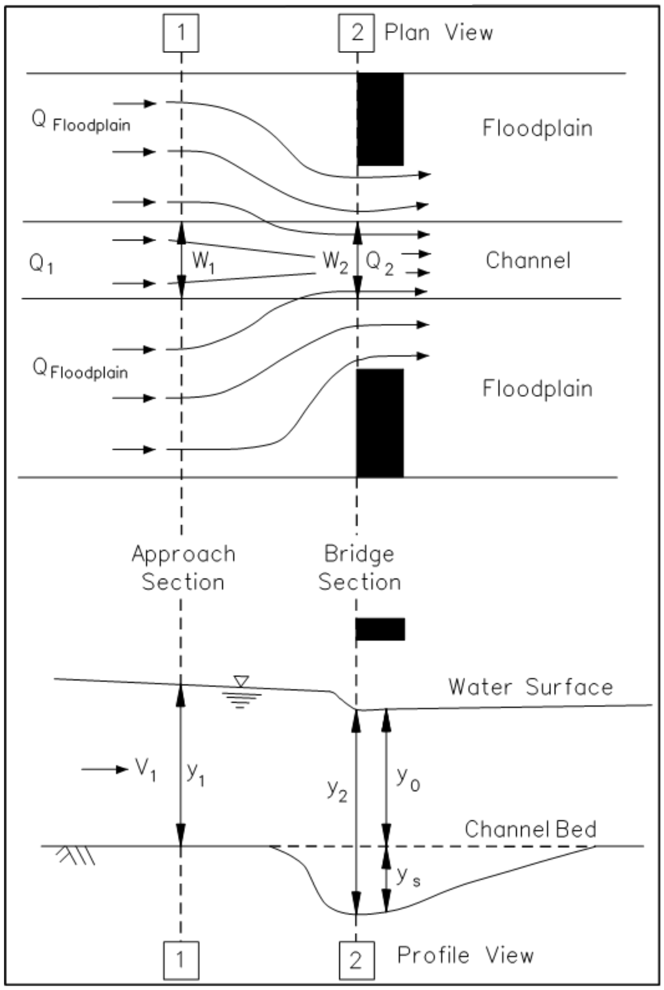
For clear-water contraction scour, the flow velocity in the bridge opening is sufficient to move bed material even though the upstream flow velocity is too low for bed material movement. For live-bed contraction scour, the higher flow velocity in the bridge opening has a greater capacity for transporting sediment than the upstream flow velocity. In either case, there is an imbalance between sediment supply and sediment transport capacity, and contraction scour occurs. 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. The erosion process takes time; therefore, depending on the duration of the flood, the ultimate scour depth may not be achieved during a single flood event.

Accurate contraction scour calculations depend on having accurate estimates of flow distribution at the approach and bridge cross sections. Flow is divided into channel, left floodplain, and right floodplain in the fully expanded flow upstream of the bridge, and divided into channel, left setback (floodplain), and right setback areas under the bridge. Bridges that do not have floodplains under the bridge (set back abutments) use the upstream channel width and discharge and the total bridge water surface width and discharge. The discharges and widths control the contraction scour process; see Figures 17.6-3.

## 17.6.5 Local Scour

Local scour occurs where the flow field is disrupted by an obstruction. The term “local” is used because scour is in the vicinity of the obstruction, not across the entire channel or bridge section. The flow is redirected and accelerates, vortexes form, and turbulence increases. The two most common types of local scour at bridges are pier scour and abutment scour. Ice and debris can also impact local scour.

**Figure 17.6-3 — CONTRACTION SCOUR VARIABLES**




*Source*: HDS 7 (1)

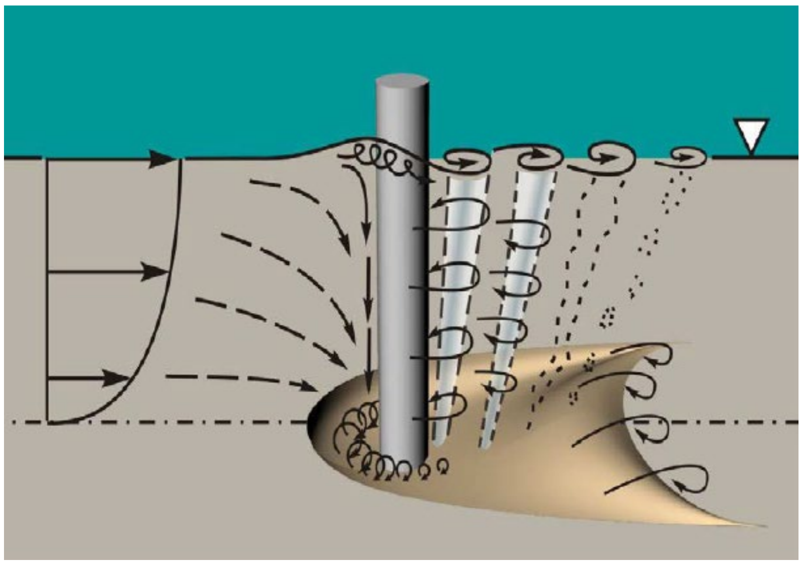
Factors that affect the magnitude of local scour depth at piers and abutments are:

- Velocity of the approach flow,
- Depth of flow,
- Width of the pier,
- Discharge intercepted by the abutment and returned to the main channel at the abutment,
- Length of the pier if skewed to flow,
- Size and gradation of bed material,
- Angle of attack of the approach flow to a pier or abutment,
- Shape of a pier or abutment,
- Bed gradation or composition, and
- Ice formation or jams and debris.

### 17.6.5.1 Pier Scour

Pier scour is illustrated in Figure 17.6-4 (from HDS 7). The velocity upstream of the pier accelerates around the pier, and flow is directed downward along the front face of 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 vortexes on the sides of the pier. Sediment deposition occurs in the wake area downstream of the pier.

**Figure 17.6-4 — THE MAIN FLOW FEATURES FORMING THE FLOW FIELD AT A CYLINDRICAL PIER**



*Source*: HDS 7 (1)

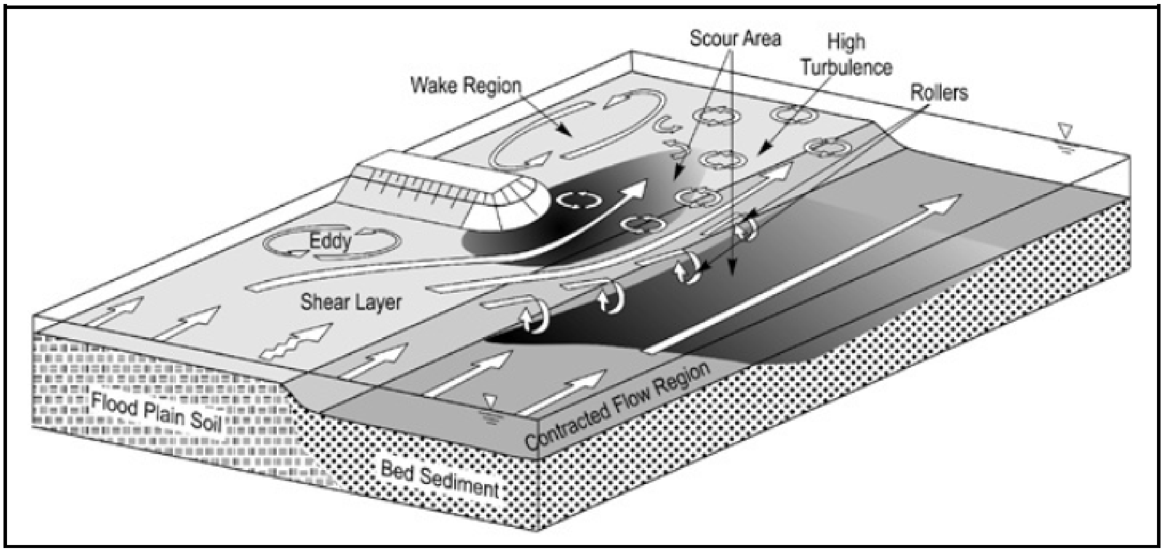
### 17.6.5.2 Abutment Scour

Scour occurs at abutments when the roadway embankment and abutment obstruct the flow. The flow obstructed by the abutment accelerates through the bridge and causes vortices along the abutment. The magnitude of local scour at an abutment is a function of depth and velocity of flow, the skew of the embankment to the floodplain, and the volume of flow from the overbank that passes through the bridge opening. It is also a function of where the abutment is located relative to the main channel.

Abutment scour can result in geotechnical failures of the embankment or channel bank materials. Once the geotechnical failure depth is reached, scour will not increase in depth but will progress laterally, potentially creating a free-standing abutment foundation that would act more as a pier from the standpoint of scour.

Figure 17.6-5 (from HDS 7) illustrates abutment scour processes. Where abutments are set well back from the channel, abutment scour is located entirely on the floodplain. Where abutments are set in or close to the channel, abutment scour can occur entirely in the channel or in the floodplain and channel. When the abutment is set close to the channel, channel sediment and floodplain soil characteristics (e.g., grain size and cohesion) factor into the proportion of scour that will occur in the floodplain versus in the channel.

**Figure 17.6-5 — FLOW STRUCTURE IN FLOODPLAIN AND MAIN CHANNEL AT A BRIDGE OPENING**




*Source*: HDS 7 (1)

### 17.6.6 Debris Scour

Debris is a common problem at bridges, especially during floods. Debris loading and impact forces can damage piers, decks, and girders, and debris can reduce the waterway opening thereby increasing upstream flooding. All types of scour can be increased due to debris collection. Increased pier scour is the most common type of debris scour problem. In addition, contraction scour is increased when debris blocks a portion of the bridge opening, and pressure scour (Section 17.6.9.3.3) is increased when debris collects on the bridge deck and girders. See HEC 18 for further guidance on debris scour.

### 17.6.7 Scour-Resistant Materials

Use caution when determining the scour resistance of bed materials and the underlying strata. With sand size material, the passage of a single flood may result in the predicted scour depths. Conversely, in scour-resistant material including bedrock, the maximum predicted depth of scour may not be realized during the passage of a specific flood; however, some scour-resistant material may be lost. Commonly, this material is replaced with more easily scoured material. A later flood may reach the predicted scour depth.

Serious scour has been observed to occur in materials commonly perceived to be scour resistant (e.g., consolidated soils and glacial till, so-called bedrock streams and streams with gravel and boulder beds). Even though a bridge has survived a flood of some magnitude, this does not ensure that the bridge will survive the same size flood in the future.

The hydraulic engineer in consultation with the geotechnical engineer must assess if the bed material will scour during the life of the structure or will be scour-resistant.

### 17.6.8 Abutment Riprap for Proposed Bridges

MDT practice is to design and place rock riprap at bridge abutments. Abutment slopes at hydraulic crossings are typically 2H:1V, unless otherwise specified by the MDT Geotechnical Section. Use of riprap on the abutment, with the bottom of the key at or below the level of contraction scour, is generally considered to be an adequate countermeasure. The abutment riprap generally wraps around the abutment and remains within the right-of-way. Site conditions may require that the riprap be extended upstream beyond the right-of-way, or a guide bank may need to be constructed. When a pier is close to the abutment, it may be prudent to extend the abutment riprap beyond the pier. HEC 23 (13) should be used for hydraulic design, sizing, and placement of rock riprap at bridge abutments.

Use the following procedure to design abutment riprap.

#### 17.6.8.1 Riprap Size

Consider the following guidelines when sizing abutment riprap:

- Size the riprap per the abutment riprap equations in HEC 23.
- Calculate the size using the variables from the scour design flood.
- Check the size using the variable from the scour check flood.
- Use a minimum of Class II riprap to protect bridge abutments.
- Select a riprap size from the MDT standard riprap sizes (e.g., Class II or III).

Consider upsizing and/or thickening the riprap if:

- The size of riprap based on the scour design flood riprap is close to Class III.
- There is a history of ice jams or debris at the bridge.
- There is a history of scour at the bridge site, upstream or downstream.
- The scour check flood does not overtop and flows through the bridge.
- There is a history of riprap failures at the bridge.

#### 17.6.8.2 Riprap Key Thickness and Elevation

When determining the riprap key thickness and elevation, consider the following:

- Typically, the top of the riprap key is set at the channel bottom elevation described in Section 17.4.2.3, "Initial Proposed Alternative."

- Riprap placed at abutments should have a minimum thickness and key dimensions per MDT Detailed Drawing 613-16.

- In some instances, the key elevation may be lowered and the thickness may be increased to provide additional protection from contraction scour, long-term degradation, and/or channel migration.

### 17.6.8.3 Riprap Extents

Carefully consider the riprap extents and layout. Consider the following guidelines:

- The horizontal limits of the abutment riprap should extend past the abutment wall twice the hydraulic depth of the scour design flood or 25 ft, whichever is larger. In some instances, the riprap will wrap around the embankment fill or, in sites with deep channels, the riprap may extend along the channel bank.
- The horizontal riprap extents should be measured from the face of the abutment back along the embankment if the riprap is wrapping around the embankment fill or from the outside corner of the abutment if the riprap is extending along the channel bank.
- The horizontal riprap should be extended on all four corners of the bridge.
- Abutment slopes should be protected with rock riprap to a minimum elevation of 1 ft above the 100-year flood.

### 17.6.8.4 Riprap Filter

To prevent a transport of fines through the riprap, which can ultimately cause a riprap failure, a filter is required between the native soils and all riprap. MDT uses geotextile for the filter under bridges and adjacent to critical infrastructure:

- The geotextile is installed per MDT Detailed Drawing 613-16.
- Consult the Geotechnical Section to determine the geotextile class.
- Occasionally, a granular filter layer may be used in sites away from critical infrastructure if the site is low risk and the riprap and granular filter can be installed in a dry condition.

### 17.6.8.5 Preliminary Riprap Layout

Develop a preliminary layout to illustrate the riprap concept and include with the bridge hydraulic recommendation memo:

- Show the riprap footprint.
- Show the contours for the proposed ground for the bridge opening from the channel to the top of the riprap.
- The proposed ground contours should connect to the existing contours upstream and downstream of the opening and align with the channel.
- Show the centerline of the bridge bottom-width trapezoid and the skew.

### 17.6.8.6 Final Riprap Layout

Develop a final riprap layout and contour grading plan after the Plan-in-Hand review:

- Once the roadway grade and bridge layout are set, work with the Road Design Section and Bridge Bureau to develop a final contour grading plan detail.
- The detail should include:
  - A finished surface contour grading plan;
  - Sufficient information to place the riprap and construct the abutment slopes, channel, and floodplain benches as necessary;
  - A coordinate table for the riprap elevations and locations and any other pertinent features;
  - Plan and profile views; and
  - Cross sections as necessary.

## 17.6.9 Cumulative Scour Analysis

The following sections describe the procedure for a scour analysis.

### 17.6.9.1 Step 1: Determine Scour Analysis Variables

1. Based on Section 17.3.1.4, determine the magnitude of the discharges for the selected flood for the scour analysis.
2. Where the bridge is just upstream of a river confluence, it may be necessary to look at coincidental flows to determine the maximum scour potential. Coincidental tailwater events are discussed in Section 17.5.4. To determine the greatest scour potential, select the lowest reasonable downstream water surface elevation and use the scour check flood discharge through the structure.
3. Use a hydraulic model to calculate the results for the discharges defined above. For routine situations, use a 1D model and, for complex flow situations, use a 2D hydraulic model. From computer analysis and from other hydraulic studies, determine input variables such as the discharge, velocity, and depth needed for the scour calculations. Further guidance on variable selection is given in Section 17.6.9.3 and Section 17.6.9.4.
4. Collect and summarize the following information as appropriate and use the information to make a qualitative evaluation of the site with an estimate of the potential for stream movement and its effect on the bridge:
   - Boring logs to define geologic substrata at the bridge site;

- Bed material size, gradation, and distribution in the bridge reach (bed material sample collection is requested in the LHSR and is discussed in the LHSR memo template);
- Scour history of the existing bridge;
- History of flooding;
- Location of bridge site with respect to other bridges in the area, confluence with tributaries close to the site, bedrock controls, man-made controls (e.g., dams, old check structures, river training works), and confluence with another stream downstream;
- Character of the stream (e.g., perennial, flashy, intermittent, gradual peaks);
- Geomorphology of the site (e.g., floodplain stream; crossing of youthful, mature, or old age stream; crossing of an alluvial fan; meandering, straight, or braided stream);
- Erosion history of the stream;
- Development history (consider present and future conditions) of the stream and watershed;
- Photos of past scour including ground photographs and aerial photographs;
- Scour history from bridge inspection reports and Maintenance staff;
- History of sand and gravel mining from the streambed or floodplain upstream and downstream from the site; and
- Other unanticipated factors not included in the above discussion that could affect the bridge.

### 17.6.9.2 Step 2: Determine the Magnitude of Long-Term Degradation or Aggradation

Using the information collected in Step 1, determine the magnitude of long-term degradation at the bridge. Use historic records, observational data, or other empirical methods to determine the potential for long-term degradation and then, factor that value into the total scour depth. If the analysis concludes that there will be long-term aggradation, it should be noted in the records but should not be included in the total scour depth outlined in Step 6.

### 17.6.9.3 Step 3: Compute the Magnitude of Contraction Scour

Using the information collected in Step 1, compute the magnitude of the contraction scour using the equations and procedures in HEC 18 (4). Use careful consideration when selecting the variables for contraction scour, and refer to Figure 17.6-3.

### 17.6.9.3.1 Scour Approach Section

First, consider the location of the approach section. The approach section for calculating scour is likely in a different location than the approach section for measuring backwater. The scour approach section will be used to collect the variables for both the contraction scour and the abutment scour calculations. The scour approach section should be located outside the influence of the bridge. When selecting the location for the scour approach cross section, consider the following:

- In 1D models, look at a table that shows the average velocity in the channel by cross section. Working upstream from the bridge, look for the average velocity in the channel to decrease and then become relatively consistent. The cross section where the velocity has stabilized is the approximate cross section to use for the scour approach section.

- In 2D models, the flow vectors in the floodplain should be parallel to the channel (not turning towards the bridge).

### 17.6.9.3.2 Selecting a Width

Determine whether the flow at the scour approach section is transporting bed material by following the procedure in HEC-18. If the stream is transporting bed material, the scour is "live bed" and if the stream is not transporting sediment, the scour is “clear water.”

Note: Although HEC-18 indicates a bottom width, the top width is acceptable instead of the bottom width if the hydraulic engineer is consistent when selecting widths within the same analysis (top width upstream and top width through bridge opening).

If the scour is live bed, use the width that is actively transporting bed material:

- In 1D models, determine this width from the flow distribution table by looking for the width associated with velocities greater than or equal to the critical velocity for the site.

- In 2D models, observe this width by displaying the velocities that are greater than or equal to the critical velocity.

If the scour is clear water, use the width of the upstream channel:

- In 1D models, it is important that the bank points are set at the top of the bank-full channel if selecting this value as the channel width from a summary table.

- In 2D models, set this width at the edge of the vegetation as seen in an aerial or at the top of the channel.

### 17.6.9.3.3 Pressure Flow Scour (Vertical Contraction Scour)

Pressure flow scour or vertical contraction scour occurs when the water surface elevation at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure. Pressure flow under the bridge results from a buildup of water on the upstream bridge face and a plunging of the flow downward and under the bridge. At higher approach flow depths, the bridge can be entirely submerged with the resulting flow being a complex combination of the plunging flow under the bridge and the flow over the bridge. Refer to HEC 18 (4) for more information pertaining to pressure flow scour.

### 17.6.9.4 Step 4: Compute the Magnitude of Local Scour at Piers

Using the information collected in Step 1, compute the magnitude of local pier scour using the equations and procedures in HEC 18 (4). Apply the following for pier scour computations:

- There are numerous pier scour methodologies including those for standard piers, wide piers, complex piers, coarse bed streams, cohesive materials, erodible rock, and debris accumulation. Refer to HEC 18 for the appropriate pier scour equations and methodologies.

- Because the thalweg of a stream can move, use the maximum depth and velocity just upstream of the bridge face, and apply the calculated scour depth to all piers.

- When computing total scour, the amount of pier scour is added to the amount of contraction scour to determine the total scour at the pier.

Other considerations when computing pier scour include:

- The skew angle between the pier and the flow direction may change at different water surface elevations. In some cases, more severe pier scour occurs at lower flows, because the flows are not lined up well with the piers. Review of flood photographs and/or 2D models can be helpful in determining the appropriate angle of attack.

- Where debris is a consideration and could be caught on the pier, the scour may increase because the effective width of the pier increases. Follow the HEC 18 guidelines when considering debris impacts on pier scour.

- Evaluate the hazards of ice and debris buildup when considering the use of multiple pile bents in stream channels. Where ice or debris buildup is a problem, consider the bent a solid pier to estimate scour. Consider the use of other pier types where clogging of the waterway area could be a major problem.

- Scour analyses of piers near abutments need to consider the potential of higher local velocities and greater angles of attack from the flow coming around the abutment. When the distance between the abutment toe and the near-pier (pier adjacent to the abutment) is less than or equal to 10 times the flow depth approaching the abutment, the pier scour should be checked against the abutment scour, and the larger of the two estimates should be used to design the near-pier foundation.

- Local scour holes at piers and abutments may overlap one another in some instances. If local scour holes do overlap, the scour is indeterminate and may possibly be deeper than independent estimates at one or the other. The top width of a local scour hole on each side of the pier ranges from 1.0 to 2.8 times the depth of local scour, depending upon soil characteristics. A top width value of 2.0 times the depth of local scour on each side of a pier is suggested for practical applications of plotting the scour cone.

There are also several considerations in selecting the location for the piers, including:

- The spacing of the piers should be wider than the expected debris length.
- Where ice or debris are considerations, avoid placing piers near the bank on the outside of a bend.

Where the channel has a thalweg that is well defined and appears to be unlikely to migrate substantially, the piers should be kept out of this area. One way to determine the long-term stability of the thalweg near existing structures is to compare the recent survey to the cross section shown on the general layout for the existing bridge.

### 17.6.9.5 Step 5: Compute the Magnitude of Local Scour at Abutments

Using the information collected in Step 1, compute the magnitude of abutment scour using the equations and procedures in HEC 18 (4). When completing abutment scour calculations:

- Refer to HEC 18 for the appropriate abutment scour equations and methodologies.
- Select the scour variables using the same scour approach section that was defined during the contraction scour calculation.

All abutment embankments for bridges over waterways require protection against scour and erosion, regardless of abutment or foundation type or calculated scour depth. Abutment embankments for bridges over irrigation canals should be evaluated for scour potential and may or may not require abutment protection.

### 17.6.9.6 Step 6: Plot the Total Scour Depths and Evaluate the Design

#### 17.6.9.6.1 Step 6A — Plot the Total Scour Depths

On the cross section of the stream channel or other general floodplain at the bridge crossing, plot the estimate of long-term bed elevation change, contraction scour, and local scour at the piers and abutments using the following guidance. Also, make note of any lateral stream movement that may occur during the life of the bridge:

1. Long-term elevation changes may be either aggradation or degradation. However, only degradation is considered in the total scour assessment.
2. Contraction scour is plotted from and below the long-term degradation line.
3. Pier scour is plotted from and below the contraction scour and long-term degradation lines.
4. Plot the calculated NCHRP abutment scour elevation below the long-term degradation line.

5. Plot the depth of scour and scour hole width at each pier and/or abutment. Use 2.0 times the depth of local scour to estimate the scour hole width on each side of the pier and/or abutment.

### 17.6.9.6.2 Step 6B — Evaluate the Total Scour Depths

1. Evaluate whether the computed scour depths are reasonable and consistent with the interdisciplinary team's previous experience and engineering judgment. If not, carefully review the calculations and design assumptions to modify the depths. These possible modifications must reflect sound engineering judgment.

2. Evaluate whether the local scour holes from the piers or abutments overlap between spans. If so, local scour depths can be larger because the scour holes overlap. For new or replacement bridges, the length of the bridge opening should be reevaluated and the opening increased or the number of piers decreased as necessary to avoid overlapping scour holes.

3. Evaluate the impact from factors such as lateral movement of the stream, stream flow hydrograph, velocity and discharge distribution, movement of the thalweg, shifting of the flow direction, channel changes, type of stream, or other items.

4. Evaluate whether the calculated scour depths appear reasonable for the conditions in the field relative to the laboratory conditions under which the equations were developed. The first objective in evaluating the reasonableness of the scour results is to confirm that the results of the hydrologic and hydraulic analysis are reasonable and accurate. All of the methods used to compute scour rely on accurate input to the procedures.

   If the calculated scour depths appear too deep, consider an iterative approach for computing scour by recalculating the hydraulic variables after long-term degradation and/or contraction scour are accounted for. This may provide a more realistic scour evaluation and decrease the total scour depth.

5. Evaluate cost, safety, etc. Also, account for additional opening requirements and factors that may complicate the scour computations due to ice and/or debris effects.

6. In the design of bridge foundations, the bottom foundation elevation(s) should be located such that the foundation is stable at elevations below the total scour elevation(s); i.e., the foundation design cannot rely on material above the total scour line to provide load capacity.

### 17.6.9.6.3 Step 6C — Re-evaluate the Bridge Design

Reevaluate the bridge design based on the scour computations and evaluation. Revise the design as necessary. This evaluation should consider the following questions:

1. Is the waterway area sufficiently large (e.g., are contraction, pier, and abutment scour amounts too large)?

2. Are the piers too close to each other or to the abutments (e.g., do the scour holes overlap)? Estimate the top width of a scour hole on each side of a pier at 2.0 times the depth of scour. If scour holes overlap, local scour can be deeper.

3. Is there a need for relief structures? If so, how large should they be and where should they be located?

4. Are bridge piers and abutments properly aligned with the flow and located properly relative to the stream channel and floodplain?

5. Is the bridge crossing of the stream and floodplain in a desirable location? If the location presents problems:

   a. Can it be changed?

   b. Can any of the following be used to provide for an acceptable flow pattern at the bridge — guide banks, setback abutment(s) from the channel, or add relief bridge(s)?

6. Is the hydraulic study adequate to provide the necessary information for foundation design?

   a. Are flow patterns complex and should a two-dimensional, water surface profile model be used for analysis?

   b. Is the foundation design safe and cost effective?

#### 17.6.9.6.4 Step 6D — Calculate the Low Scour Elevation for the Bridge Plans

The low scour elevation is listed on the bridge recommendation memo. This elevation is used to ensure that the bridge foundation elements are set low enough. To calculate the low scour elevation, subtract the contraction scour and pier scour for the scour check flood event from the low channel elevation. Show the variables used in Figure 17.6-6:

**Figure 17.6-6 — LOW SCOUR ELEVATION FOR SCOUR CHECK FLOOD $Q_{XXX}$**

| Channel Bottom Elevation (ft) | subtract | Contraction Scour (ft) | subtract | Pier Scour (ft) | equals | Low Scour Elevation (ft) |
|---|---|---|---|---|---|---|
| XXXX.XX | – | X.XX | – | X.XX | = | XXXX.XX |

### 17.6.9.7 Step 7: Design Abutment Countermeasures for Proposed Bridges

Proposed piers will be designed for pier scour; therefore, proposed bridges will not have pier scour countermeasures.

Proposed abutments will have scour countermeasures. Follow the guidance in Section 17.6.8 to:

1. Size the riprap and select a filter.
2. Determine the extents of the riprap.
3. Develop a preliminary riprap layout with the recommendation memo.
4. Develop a final riprap layout after the Plan-in-Hand review.

### 17.6.10 Considerations for Evaluating Scour at Existing Bridges

Scour evaluations for existing bridges should be completed per the guidance in HEC 18 and Sections 17.6.1 through 17.6.7 above, with the following exceptions:

- For existing bridges, aggradation and degradation at the site may be considered when evaluating scour.
- Streambed armoring may also be considered when evaluating scour.
- When the top of the pier footing is above the contraction scour elevation, the width of the footing needs to be considered in the scour analysis.
- HEC 18 has specific guidance for evaluating scour at existing bridges.
- Mitigation actions must be taken for scour critical bridges. See Section 17.6.10.1.
- Scour countermeasures are allowed for existing bridge piers, but not for proposed bridge piers. See Section 17.6.10.2.

#### 17.6.10.1 Scour Critical Bridges

A scour critical bridge is a bridge with a foundation element that has been determined to be unstable for the observed or evaluated scour condition. Hydraulic, geotechnical, and structural engineers individually or as a team use an engineering analysis to determine if a bridge is scour critical.

If a bridge is determined to be scour critical, the Hydraulics Section will prepare a Plan of Action (POA) that documents the concerns with the bridge and the actions to be taken by maintenance before, during, and after a flood event to protect the traveling public.

In addition, the Hydraulics Section will request that the Bridge Bureau change the National Bridge Inventory code for Item 113 — Scour Critical Bridges to a 3 or lower until the bridge scour can be mitigated. Codes for the NBI Item 113 are shown in Figure 17.6-7. See Figure 17.6-8 for the three scour depth types (i.e., Examples A, B, and C) that apply to the NBI Item 113 codes.

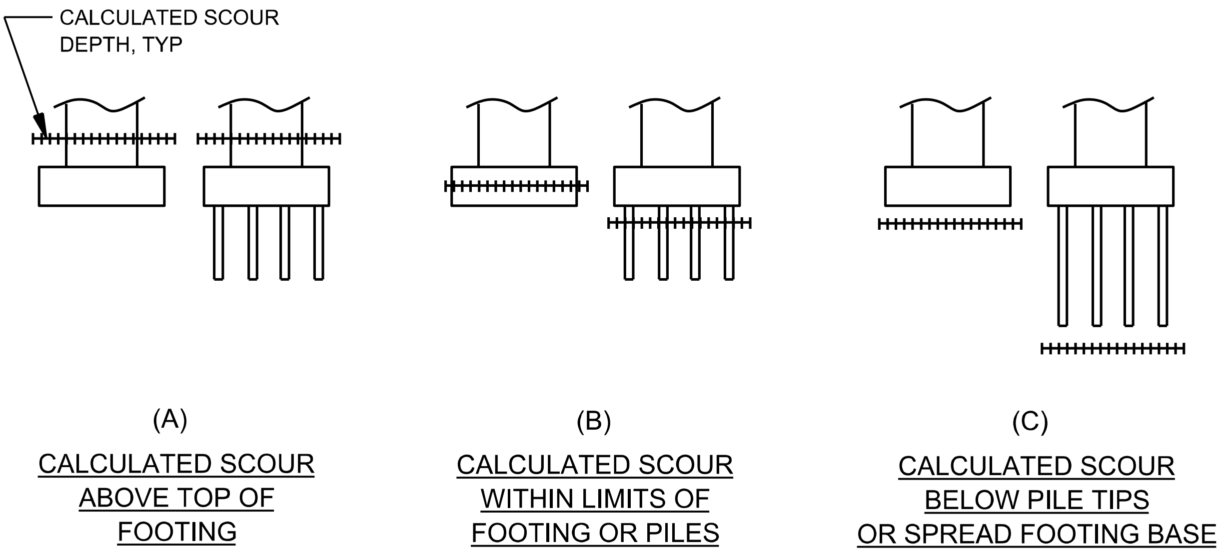
## Figure 17.6-7 — NBI ITEM 113 CODES

| Code | Description<br>(See FHWA Recording and Coding Guide[5,6]) |
|---|---|
| N | Bridge not over waterway. |
| U | Bridge with "unknown" foundation that has not been evaluated for scour. Until risk can be determined, a plan of action should be developed and implemented to reduce the risk to users from a bridge failure during and immediately after a flood event[3]. |
| T | Bridge over "tidal" waters that has not been evaluated for scour but considered low risk[2]. Bridge will be monitored with regular inspection cycle and with appropriate underwater inspections until an evaluation is performed. ("Unknown" foundations in "tidal" waters should be coded U.) |
| 9 | Bridge foundations (including piles) on dry land well above flood water elevations. |
| 8 | Bridge foundations determined to be stable for the assessed or calculated scour conditions[2]. Scour is determined to be above top of footing (Example A) by assessment (i.e., bridge foundations are on rock formations that have been determined to resist scour within the service life of the bridge[4]), by calculation or by installation of properly designed countermeasures[3]. |
| 7 | Countermeasures have been installed to mitigate an existing problem with scour and to reduce the risk of bridge failure during a flood event. Instructions contained in a plan of action have been implemented to reduce the risk to users from a bridge failure during or immediately after a flood event. |
| 6 | Scour calculation/evaluation has not been made. (Use only to describe case where bridge has not yet been evaluated for scour potential.) |
| 5 | Bridge foundations determined to be stable for assessed or calculated scour condition. Scour is determined to be within the limits of footing or piles (Example B) by assessment (i.e., bridge foundations are on rock formations that have been determined to resist scour within the service life of the bridge[4]), by calculations or by installation of properly designed countermeasures[3]. |
| 4 | Bridge foundations determined to be stable for assessed or calculated scour conditions; field review indicates action is required to protect exposed foundations[3]. |
| 3 | Bridge is scour critical; bridge foundations determined to be unstable for assessed or calculated scour conditions:<br>• Scour is within limits of footing or piles (Example B).<br>• Scour is below spread footing base or pile tips (Example C). |
| 2 | Bridge is scour critical; field review indicates that extensive scour has occurred at bridge foundations, which are determined to be unstable by:<br>• A comparison of calculated scour and observed scour during the bridge inspection, or<br>• An engineering evaluation of the observed scour condition reported by the bridge inspector in Item 60. |
| 1 | Bridge is scour critical; field review indicates that failure of bents/abutments is imminent. Bridge is closed to traffic. Failure is imminent based on:<br>• A comparison of calculated and observed scour during the bridge inspection, or<br>• An engineering evaluation of the observed scour condition reported by the bridge inspector in Item 60. |
| 0 | Bridge is scour critical. Bridge has failed and is closed to traffic. |

References:

1. FHWA Technical Advisory T 5140.23, "Evaluating Scour at Bridges," October 28, 1991, (14)
2. HEC 18, *Evaluating Scour at Bridges*, Fourth Edition (4)
3. HEC 23, *Bridge Scour and Stream Instability Countermeasures*, Second Edition (13)
4. FHWA Memorandum, "Scourability of Rock Formations," July 19, 1991 which is available online and has been replaced with HEC 18, Chapter 4, Section 4.6.
5. FHWA-PD-96-001, *Recording & Coding Guide for the Structure Inventory & Appraisal of the Nation's Bridges*, 1995
6. FHWA Memorandum, "Revision of Coding Guide, Item 113 – Scour Critical Bridges," April 27, 2001, available online

**Figure 17.6-8 — SCOUR DEPTH (NBI Item 113)**




## 17.6.10.2 Existing Bridge Scour Mitigation

Scour critical bridges may be addressed by:

- Installing scour countermeasures; see Section 17.6.10.2.1;
- Structural measures; see Section 17.6.10.2.2; and
- Scour monitoring; see Section 17.6.10.2.3.

### 17.6.10.2.1 Existing Bridge Scour Countermeasures

As necessary, MDT uses the following scour countermeasures on existing bridges:

- Rock riprap,
- Partially grouted riprap,
- Grout-filled bags, and
- Guide banks.

Once an engineered scour countermeasure is installed, the NBI Item 113 code should be changed to a 7. Post installation, the countermeasures will be monitored through the bridge inspection program.

Figure 17.6-9 provides the HEC 23 (13) assessment of the suitability of these methods for various river environments. See HEC 23 for design guidelines.

**Figure 17.6-9 — SUITABLE RIVER ENVIRONMENTS FOR HYDRAULIC SCOUR COUNTERMEASURES**

| Scour Counter-measure | Suitable River Environment | | | | | | |
|---|---|---|---|---|---|---|---|
| | **River Type** | **Stream Size** | **Bend Radius** | **Bed Material** | **Debris/ Ice Load** | **Bank Slope** | **Floodplain** |
| | B = braided<br>M = meandering<br>S = straight | W = wide<br>M = moderate<br>S = small | L = long<br>M = moderate<br>S = short | C = coarse<br>S = sand<br>F = fine | H = high<br>M = moderate<br>L = low | V = vertical<br>S = steep<br>M = mild | W = wide<br>M = moderate<br>N = narrow |
| Grout-Filled Bags | ✓ | ✓ | ✓ | ✓ | M, L | M | ✓ |
| Rock Riprap | ✓ | ✓ | ✓ | ✓ | ✓ | S, M | ✓ |
| Partially Grouted Riprap | ✓ | ✓ | ✓ | ✓ | ✓ | S, M | ✓ |
| Guide Banks | ✓ | W, M | ✓ | ✓ | ✓ | ✓ | W, M |

✓ *Suitable for the full range of characteristics*

*Source:* HEC-23 (13) (Modified by MDT)

## 17.6.10.2.2 Structural Measures

In some cases, a structural measure such as a pier retrofit may be preferred to a scour countermeasure. In these cases, the hydraulic engineer will work with the structural and geotechnical engineers to determine the best solution.

## 17.6.10.2.3 Scour Monitoring

For existing scour-critical bridges, monitoring and closing a bridge during high flows and subsequent inspections after the flood may be an effective countermeasure to reduce the risk to the traveling public. However, this does not reduce the risk of collapse of the bridge due to scour, and the NBI Item 113 remains a 3. The monitoring approach should be carefully considered based on traffic volumes, emergency vehicle routes, availability of monitoring personnel, and available alternative routes. If monitoring is selected as the countermeasure option, develop a location-specific Plan of Action (POA) to ensure that the appropriate actions are taken when the target flood elevations are reached (see FHWA TA 5140.23 (14)). If scour monitoring instrumentation is proposed, consult HEC 23 (13).

# 17.7 DESIGN GUIDELINES (DECK DRAINAGE)

This Section provides guidelines and procedures for designing bridge deck drainage systems. Criteria are provided for determining the hydraulic design of the system (e.g., design flood frequency, allowable water spread). MDT design practices for the system components are discussed. For additional guidance, see HEC 21 (28) and HDS 7 (1).

## 17.7.1 Importance of Bridge Deck Drainage

The bridge deck drainage system includes the bridge deck, sidewalks, railings, curbs, and inlets (or scuppers). Bridge end drainage is discussed in Section 17.7.2.2. The primary objective of the drainage system is to remove runoff from the bridge deck before it collects in the gutter to a point that exceeds the allowable design spread. See Section 14.3.3 for MDT design criteria on storm drainage design flood frequency and spread on roadways, which also applies to bridge decks.

Proper bridge deck drainage provides many other benefits, including:

- Efficiently removing water from the bridge deck enhances public safety by decreasing the risk of hydroplaning.
- Long-term maintenance of the bridge is enhanced.
- The structural integrity of the bridge is preserved.
- Aesthetics are enhanced (e.g., the avoidance of staining substructure and superstructure members).
- Erosion on bridge end slopes is reduced.
- Stormwater quality requirements are satisfied.

## 17.7.2 Deck Drainage Coordination

The following Sections outline the responsibilities of the hydraulic engineer with respect to bridge deck drainage.

### 17.7.2.1 Design of Deck Drainage System

The hydraulic engineer:

- Calculates the spread width on the deck based on the design frequency and determines the need for deck drains,
- Coordinates with the bridge engineer to select the type of deck drain, and

- Determines the hydraulic inlet spacing on the bridge deck to intercept the calculated flow to meet the allowable water spread criteria.

The bridge engineer incorporates the drainage design information into the structural design of the bridge plans.

#### 17.7.2.2 Bridge End Drainage Coordination

The hydraulic engineer is responsible for recommending the drainage design for any runoff approaching or leaving the bridge deck. These recommendations are sent to roadway designer. Bridge end drainage is discussed further in Section 17.7.3.9.

### 17.7.3 Bridge Deck Drainage Design Considerations

#### 17.7.3.1 Type of Drainage System

MDT generally favors an open drainage system for its bridge decks. An underdeck drainage system is required where downspouts are prohibited and the spread width criteria are exceeded without any deck drains. See Section 17.7.3.4 for the determination of a maximum deck length without deck inlets.

#### 17.7.3.2 Deck Slope

To provide proper bridge deck drainage, the absolute minimum longitudinal gradient is 0.2%; preferably, the longitudinal gradient will not be less than 0.4%. The hydraulic engineer should verify that these minimum slopes are met at the Alignment and Grade review.

The transverse drainage of the bridge deck must be accommodated by providing a suitable roadway cross slope, typically 2%. If superelevation is involved, additional analysis may be required to ensure that the spread width criteria can be met.

#### 17.7.3.3 Sag Vertical Curves

If practicable, no portion of a bridge should be located on a sag vertical curve. If the bridge must be located on a sag vertical curve, the low point of the curve should not be located on the bridge or the approach slab. The low point of the sag vertical curve should be located a distance sufficiently far from the bridge so that a longitudinal grade of at least 0.4% is maintained on the bridge.

#### 17.7.3.4 Maximum Length of Bridge without Inlets

On a continuous grade, the maximum bridge length that requires no bridge deck drainage inlets can be determined. In other words, the drainage basin area (e.g., the bridge deck) will not generate a sufficient runoff to produce a gutter flow that, at any point, exceeds the allowable water spread on the bridge deck.

The hydraulic engineer should use the following equation and the known site conditions to determine if drainage inlets are needed or if the bridge length is short enough to design the bridge without drainage inlets:

$$L = \frac{24{,}393.6\left(S_x^{1.67}\right)\left(S^{0.5}\right)\left(T^{2.67}\right)}{CniW} \qquad \text{Equation 17.7-1}$$

Where:

| | | |
|---|---|---|
| L | = | maximum allowable bridge length without drainage inlets, ft |
| S | = | longitudinal slope, ft/ft |
| $S_x$ | = | cross slope, ft/ft |
| W | = | width of drained deck*, ft |
| C | = | runoff coefficient** |
| i | = | rainfall intensity, in/h |
| n | = | Manning's n** |
| T | = | maximum allowable spread, ft |

* *For normal crown cross sections, this distance is typically measured from the centerline of bridge to the outside edge of deck or barrier, whichever controls. For a fully superelevated cross section, this distance is measured between the outside edges of deck or barrier.*

** *For typical decks, C = 0.9 and n = 0.016.*

### 17.7.3.5 Location of Inlets

The hydraulic engineer is responsible for performing the hydraulic analysis to determine the hydraulic inlet spacing on the bridge deck. Use the Rational Method, as discussed in Section 9.7 to estimate the runoff. Then, use the following to determine the location of inlets:

- Use Figure 14.3-1 "Minimum Design Flood Frequency and Spread."
- Use the FHWA Hydraulic Toolbox or other applicable software (see Chapter 8, "Hydraulics Software") with the gutter geometrics to determine the roadway spread.

Consider the following factors in selecting the final location of inlets:

- Where possible, locate deck drains where they will discharge onto abutment riprap.
- It is desirable to collect 100% of the runoff up gradient from expansion joints, especially where the approaching roadway is a curb-and-gutter section.
- Generally, embankment protectors are used upstream and downstream of bridge ends to prevent water from flowing onto the bridge and to capture water flowing off the bridge (see Section 17.7.3.9).

These additional considerations will impact the overall inlet layout and must be considered in addition to the inlet spacing based on the spread width calculated by the hydraulic analysis.

### 17.7.3.6 Downspouts

Although the deck drain locations are typically specified by the hydraulic engineer, the downspouts are designed by the bridge engineer. Downspouts, where used, should be of a rigid, corrosion-resistant material not less than a 4-in. inside diameter pipe. The bridge engineer should consider the following from HEC-21 (15) when locating downspouts:

1. Location with Respect to Structural Elements. Downspouts typically extend below structural elements. Do not locate downspouts within 5 ft of the end of any substructure units or where water could easily blow over and run down a substructure element. Do not locate downspouts where a 45° cone of splash beneath the downspout will touch any structural component. Ensure that downspouts do not encroach upon the required vertical or horizontal clearances.

2. Location with Respect to Ground. A free fall exceeding 25 ft will sufficiently disperse the falling water so that minimal erosion damage will occur beneath the bridge HEC 21 (15). Where less than 25 ft of free fall is available, erosion protection on natural ground beneath the outlet may be needed.

3. Location with Respect to a Waterway. Prior to locating downspouts over a waterway, coordination may be required with the Environmental Services Bureau.

4. Railroads. Do not provide downspouts over railroad right-of-way unless otherwise agreed to by the Railroad.

5. Other Exclusions. Avoid locating downspouts over the traveled way portion of an underpassing highway, sidewalk, or unpaved embankment.

### 17.7.3.7 Structural Considerations

The primary structural considerations in drainage system design are as follows:

1. Coordination with Reinforcement. Inlet sizing and placement must be compatible with the structural reinforcement and other components of a bridge.

2. Corrosion and Erosion. Design the drainage system to deter runoff and the associated corrosives from contacting vulnerable structural members and to minimize the potential for eroding embankments. To avoid corrosion and erosion, the design must include the proper placement of outfalls. In addition, water should be prevented from running down the joint between the approach roadway and bridge and thereby undermining an abutment or wing wall.

### 17.7.3.8 Maintenance Considerations

The drainage system will not function properly if it becomes clogged with debris, ice, or snow. Therefore, the bridge engineer must consider maintenance requirements in the design. The bridge engineer should avoid drainage designs that provide inadequate room for maintenance personnel on the bridge deck or access beneath the bridge or that provide unsafe working areas for maintenance personnel.

### 17.7.3.9 Bridge End Drainage

To address erosion issues at bridge ends, embankment protectors will be placed at all four corners of bridges unless their elimination can be justified (e.g., corners are on the high side of a superelevated section). Refer to Section 11.6.1 in the MDT *Road Design Manual* and Detailed Drawing 603-28 for detailed guidance on installation.

The installation of embankment protectors should be included in all projects that involve major grading. The use of drain chutes may be considered where the bridge abutment riprap layout may make the installation of embankment protectors impractical.

## 17.8 DETOUR GUIDELINES

When MDT designs a temporary detour to be included in a contract plan set:

- Determine the detour flood frequency based on Appendix 9A, Design Frequency for Temporary Facilities;

- If a culvert is selected to convey the waterway, determine the minimum-sized pipe(s) that will pass the design flow with HW/D less than or equal to 1.0; and

- If a bridge is selected to convey the waterway, provide a minimum waterway opening of sufficient size to accommodate the 2-year flood event, spanning the active channels with a 1-ft minimum freeboard. See Section 206 of the MDT *Standard Specifications*.

The following are additional design considerations when sizing a hydraulic opening for a temporary detour:

- A detour located upstream of the roadway may potentially wash out and hit the construction site.
- A detour located downstream of the construction site may cause backwater at the work area.
- The detour may need to be upsized for debris or ice conditions.
- Perennial streams may have additional construction challenges than intermittent streams.
- Temporary hydraulic facilities may need to address environmental concerns.
- If MDT designs the detour structure, floodplain modeling/permitting may be required.
- If the detour overtops, consider that:
  - Riprap may be needed at the sag, and
  - The use of fabrics to protect overflow section may be warranted.

## 17.9 DOCUMENTATION

### 17.9.1 Bridge Recommendation Memo and Report

Use the template provided in Appendix 17A to document design assumptions and decisions.

## 17.10 REFERENCES

1. **FHWA.** *Hydraulic Design of Safe Bridges, Hydraulic Design Series Number 7.* Washington, DC : Federal Highway Administration, 2012. FHWA-HIF-12-018.

2. —. *Highways in the River Environment - Floodplains, Extreme Events, Risk, and Resilience, Hydraulic Engineering Circular No. 17, 2nd Edition.* Washington, DC : Federal Highway Administration, 2016. FHWA-HIF-16-018.

3. **USGS.** *National Water Information System* . Washington, DC : US Geological Survey. Available at http://waterdata.usgs.gov/nwis.

4. **FHWA.** *Evaluating Scour at Bridges, Fifth Edition. Hydraulic Engineering Circular No. 18.* Washington, DC : Federal Highway Administration, US Department of Transportation, 2012. FHWA-HIF-12-003.

5. **USACE.** *HEC-RAS, River Analysis System Hydraulic Reference Manual.* s.l. : US Army Corps of Engineers, 2016. Report No. CPD-69.

6. **USGS.** *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains.* Washington, DC : US Geological Survey, 1984. Water Supply Paper 2339 (also published as FHWA-TS-84-204).

7. —. *Roughness Characteristics of Natural Channels.* Washington, DC : US Geological Survey, 1978. Water Supply Paper 1849.

8. —. *Determination of Roughness Coefficients for Streams in Colorado.* Lakewood, CO : USGS, 1985. Water-Resources Investigations Report 85-4004.

9. **National Institute of Water and Atmospheric Research Limited, 2nd edition.** *Roughness Characteristics of New Zealand Rivers.* Riccarton, Christchurch, New Zealand : Water Resource Publications, 1998.

10. **USGS.** *Computation of Water Surface Profiles in Open Channels.* Washington, DC : US Geological Survey, 1984. Techniques of Water Resources Investigation, Book 3, Chapter A15.

11. **FHWA.** *Two-Dimensional Hydraulic Modeling for Highways in the River Environment - Reference Document.* Austin, TX : Federal Highway Adiministration, 2019. FHWA-HIF-19-061.

12. —. *Stream Stability at Highway Structures. Hydraulic Engineering Circular No. 20, Fourth Edition.* Washington, DC : Federal Highway Administration, US Department of Transportation, 2012. FHWA-NIF-12-004.

13. —. *Bridge Scour and Stream Instability Countermeasures: Experience, Selection and Design Guidance - 3rd Edition, Hydraulic Engineering Circular No. 23.* Washington, DC : Federal Highway Administration, 2009. FHWA-NHI-09-011 (Volume 1), FHWA-NHI-09-012 (Volume 2 "Design Guidelines").

14. —. *Evaluating Scour at Bridges. Technical Advisory (TA 5140.23).* Washington, DC : Federal Highway Administration, US Department of Transportation, October 28, 1991.

15. —. *Design of Bridge Deck Drainage, Hydraulic Engineering Circular 21.* Washington, DC : Federal Highway Administration, 1993. FHWA-SA-92-010.

# Appendix 17A — BRIDGE OPENING RECOMMENDATION MEMO AND REPORT TEMPLATES

Montana Department of Transportation
PO Box 201001
Helena, MT 59620-1001

*(Letterhead provided as an example and may be updated to current letterhead or consultant letterhead.)*

## *Memorandum*

To: "Click here and type name" , P.E. *(Design Project Manager)*
"Click here and type title"

From: Name, P.E. *(Sign Electronically)*
"Click here and type title"

Thru: Name, P.E. *(Sign Electronically)*
Hydraulics Engineer

Date: DRAFT until signed

Subject: [Project Number]
[Project Name]
UPN [UPN]
**Hydraulics Bridge Opening Recommendation Memo**

*Provide a brief discussion of the purpose of the recommendation memo including the names of the water bodies crossed, i.e., "The recommendations contained in this memo are for the proposed bridge opening on No Name Creek near………"*

Additional site-specific hydraulic background information concerning drainage features in on the project is available in the Hydraulic Bridge Opening Report, file: XXXXXXXHYCSP00X.DOCX found on PCMS.

### RECOMMENDATIONS

| | |
|---|---|
| Drainage Area | XXX.X $mi^2$ |
| Centerline of Bottom Width | Station XX+XX.XX (if stationing not available use coordinates) |
| Channel Bottom Width | XX ft |
| Channel Bottom Elevation | XXXX.XX ft |
| Channel Slope | 0.XXXX-ft/ft |
| XX-yr Flood Event *(Hydraulic Design Event)* | XXXX $ft^3/s$ |
| XX-yr Flood Stage Elevation | XXXX.XX ft |
| XX-yr Flood Velocity | X.XX ft/s |
| 100-yr Flood Discharge | XXXX $ft^3/s$ |
| 100-yr Flood Stage Elevation | XXXX.XX ft |

| | |
|---|---|
| 100-yr Flood Velocity | X.XX ft/s |
| 500-yr Flood Discharge | XXXXX ft/s |
| 500-yr Flood Stage Elevation | XXXX.XX ft |
| 500-yr Flood Velocity | X.XX ft/s |
| 2-yr Flood Discharge | XXXX ft/s |
| 2-yr Elevation | XXXX.XX-ft |
| Skew | None/XX° LT/RT |
| Bank Protection | Class X Riprap |
| Abutment/Riprap Slope | XH: 1V *(2H:1V typical)* |
| Minimum Low Beam Elevation | XXXX.XX ft |
| Low Scour Elevation *Check Flood* | XXXX.X ft |
| Ice *(Select One)* | *Light (0-0.5 ft),*<br>*Moderate (0.5 ft – 1 ft),*<br>*Severe (Greater than 1 ft)* |
| Debris *(Select One)* | *None (very light or no debris)*<br>*Light (small limbs or sticks)*<br>*Medium (limbs or large sticks)*<br>*Heavy (logs or trees)* |

Notes:

- Water surface elevations are XXX ft upstream from the bridge (*If using HEC-RAS list Station number*) and include backwater.
- Velocities shown are the highest average channel velocity through the bridge.
- Overtopping occurs at *list event or state an event greater than the 500-year* at *list location/station*.
- *List bridge modeling assumptions including number of spans, size of piers modeled, height of hammerhead, etc.*

*The following additional recommendations discuss additional important information and details that may be pertinent for the road and bridge designers to develop an alignment and grade.*

Bridge Skew

*If there is a skew to the bridge, provide a brief discussion for the reasoning and discuss existing versus proposed skews. If there is no skew, the section is not necessary.*

Bridge Grade & Drainage

*Discuss the desired deck drainage for the proposed structure; i.e., provide a minimum of 0.4% grade, etc. Deck drains and embankment protectors will be located during the 384 Activity after the bridge grade and length are determined.*

Low-Beam Elevation

*Explain whether one or two feet of freeboard are required for the minimum low beam elevation and why. (Manual* Section 17.3.1.3*)*

Scour

*Discuss the calculated scour. Include tables to show the calculated scour depths and how the low scour elevation was calculated.*

**Scour Depths**

| Flow ($ft^3/s$) | Contraction Scour (ft) | Pier Scour (ft) | Abutment Scour Elev (ft) |
|---|---|---|---|
| $Q_{Design}$ = XXXX | X.XX | X.XX | XXXX.XX |
| $Q_{100}$ = XXXX | X.XX | X.XX | XXXX.XX |
| $Q_{500\ or\ OT}$ = XXXX | X.XX | X.XX | XXXX.XX |

**Low Scour Elevation for Scour Check Flood $Q_{XXX}$**

| Channel Bottom Elevation (ft) | minus | Contraction Scour (ft) | minus | Pier Scour (ft) | equals | Low Scour Elevation |
|---|---|---|---|---|---|---|
| XXXX.XX | - | X.XX | - | X.XX | = | XXXX.XX |

Riprap Layout and Site Grading

*Include a preliminary layout and brief discussion of the planned riprap for this bridge. Also include a discussion of any anticipated channel modifications. The final riprap layout and grading plan will be completed during the 384 Activity.*

Floodplain Permitting

*Provide a brief discussion about floodplain permitting requirements for the bridge. Highlight any items that will affect the alignment and grade of the roadway.*

This bridge is not located in a regulated floodplain and will not require a floodplain permit.

*Or*

This bridge is located in a *Zone A/detailed floodplain* on Panel XXXX and will require a floodplain permit from *the local floodplain community; i.e., Rosebud County*.

Improvements Over Existing Bridge

*Describe any improvements that are being made from the existing bridge to the proposed bridge especially regarding natural resources. (Reducing number of spans, decreasing backwater, etc.)*

Site Risk

*Briefly describe any site risk. (Manual Section 9.3.4)*

If you have any questions regarding this memo or the recommendations contained in this memo, please contact *(your name with phone number and e-mail address).*

This memo (filename) and the Preliminary Hydraulics report (filename) are also available on PCMS.

Attachments: Preliminary Riprap Layout, Hydraulic Report

e-copies: *List may be modified as needed.*

Bridge Engineer
District Bridge Engineer
Highways Engineer
Hydraulics Engineer
Hydraulic Operations Engineer
District Hydraulic Engineer
Road Design Engineer (*headquarters design*)
Road Design Designer (*if known*)
District Design Supervisor (*District Design*)
District Engineering Services Supervisor
District Projects Engineer
District Geotechnical Engineer
District Biologist
District Project Development Engineer

*Montana Department of Transportation*
*PO Box 201001*
*Helena, MT 59620-1001*

**Bridge Hydraulics Report**
**Name of Crossing**
**Project Name:**
**Project Number:**
**UPN:**
**By:**
**Date:**

## INTRODUCTION

This report describes…

## GENERAL PROJECT INFORMATION

*(Can copy and paste this from the Hydrology Report.)*

Provide a brief description of the project location, route, project limits, and scope of work.

Include a general description of the project area and drainage basin characteristics (terrain, land use, etc.).

Include Caveat: This hydraulic study is valid for the MDT bridge replacement project at this location and should not be used as a floodplain study or for any other purpose.

## SITE INFORMATION

Provide a description of the Existing Bridge and site. (Include photos)

Year Constructed
Type of structure (truss, timber, concrete, steel, etc.)
Low Beam Elevation
Number of Spans
Type of piers
Abutment slopes
Skew

Describe the site history including scour, repairs, flood history, etc. (Include)

Describe the Site Risk (nearby structures, etc.)

Describe whether the project site is within a FEMA delineated floodplain. If it is in a floodplain list the community and the map number.

Discuss the streambed size and sample method.

Describe the design constraints including freeboard, ice, debris, horizontal alignment and vertical profile limitations, etc.

## HYDROLOGY SUMMARY

A complete description of the hydrologic analyses, flooding history, design event determination, and existing pipe capacities for this project is documented in the bridge hydrology report XXXXXXXHYCSP001.pdf that was completed X/XX/XXXX.

Below is a summary of the calculated discharges at the bridge. The selected hydrology method is xxx, and the Design Event for this project is the XX-year.

**Hydrology Summary Table**

| Selected Method | Drainage Area (mi$^2$) | $Q_2$ (ft$^3$/s) | $Q_{10}$ (ft$^3$/s) | $Q_{25}$ (ft$^3$/s) | $Q_{50}$ (ft$^3$/s) | $Q_{100}$ (ft$^3$/s) | $Q_{500}$ (ft$^3$/s)* |
|---|---|---|---|---|---|---|---|
| | | | | | | | |

**Note: $Q_{500}$ may change to $Q_{200}$ if the bridge is being replaced with a culvert.*

## HYDRAULIC MODELING

List the hydraulic model and version number used to complete the analysis.

Describe the source of the site survey and topographic data used in the hydraulic model. Include the vertical datum and horizontal coordinate system.

**Existing Bridge Model**

- List the Modeling Assumptions/Methods
  - Manning's n (1D & 2D)
  - Boundary Conditions (1D & 2D)
  - Mesh Development and Quality (2D)
    - Include Mesh Quality Plot
  - Skew (1D & 2D)
    - If the bridge is skewed to the flow, how is this addressed in the modeling.
  - Pressure Flow Modeling Approach (if applicable) (1D & 2D)
- Describe the overtopping location and frequency.
- Describe how the model was calibrated.
- Describe upstream and downstream modeling extents, and the model's sensitivity to the boundary conditions.

**No Bridge Model**

Example text: A "No Bridge" model was developed to show the river conditions without the bridge at the project site or roadway abutments in place. The results for the No Bridge model are shown in the summary table, and additional output is included in Appendix X.

**Proposed Alternatives**

Describe proposed alternatives analyzed include for each alternative:

Bottom width & elevations
Assumed bridge configuration (number of spans, types of piers, etc.)
Abutment configuration (slope, relation to the active channel, etc.)
Minimum low beam
Freeboard
Contraction scour

**Results**

Develop a table that includes, at a minimum, the information shown in the sample table below. Highlight the recommended alternative.

| Conditions | Flow ($ft^3/s$) | WSE (ft) at Sta 11820 | Backwater Depth (ft) From No Bridge | Backwater Change from Existing (ft) | Velocity Inside Structure (ft/s) | Contraction Scour (ft) |
|---|---|---|---|---|---|---|
| Natural Channel (No Bridge) | $Q_2$ = 208 | 4365.45 | - | -0.46 | 2.17* | - |
| | $Q_{50}$ = 1515 | 4367.01 | - | -1.64 | 3.38* | - |
| | $Q_{100}$ = 2225 | 4367.31 | - | -2.39 | 3.54* | - |
| | $Q_{500}$ = 4200 | 4367.94 | - | -4.07 | 3.90* | - |
| Existing Conditions (Approximately 55′ Bottom Width) | $Q_2$ = 208 | 4365.91 | 0.46 | - | 2.44 | - |
| | $Q_{50}$ = 1515 | 4368.65 | 1.64 | - | 7.29 | - |
| | $Q_{100}$ = 2225 | 4369.70 | 2.39 | - | 9.55 | - |
| | $Q_{500}$ = 4200 | 4372.01 | 4.07 | - | 12.07 | - |
| 75′ Bottom Width Proposed Bridge | $Q_2$ = 208 | 4365.75 | 0.30 | -0.16 | 2.19 | - |
| | $Q_{50}$ = 1515 | 4368.36 | 1.35 | -0.29 | 5.97 | 1.7 |
| | $Q_{100}$ = 2225 | 4369.27 | 1.96 | -0.43 | 7.39 | 2.5 |
| | $Q_{500}$ = 4200 | 4371.26 | 3.32 | -0.75 | 11.18 | 5.0 |
| 95′ Bottom Width Proposed Bridge | $Q_2$ = 208 | 4365.76 | 0.31 | -0.15 | 2.18 | - |
| | $Q_{50}$ = 1515 | 4367.86 | 0.85 | -0.47 | 5.50 | 1.3 |
| | $Q_{100}$ = 2225 | 4368.98 | 1.67 | -0.72 | 6.74 | 1.9 |
| | $Q_{500}$ = 4200 | 4371.18 | 3.24 | -0.83 | 9.45 | 4.3 |
| 135′Bottom Width Proposed Bridge | $Q_2$ = 208 | 4365.77 | 0.32 | -0.14 | 2.19 | - |
| | $Q_{50}$ = 1515 | 4367.80 | 0.79 | -0.71 | 4.41 | 0.8 |
| | $Q_{100}$ = 2225 | 4368.62 | 1.31 | -1.08 | 5.32 | 1.2 |
| | $Q_{500}$ = 4200 | 4370.15 | 2.21 | -1.86 | 7.32 | 2.1 |

**Velocity averaged from 11820 & 11480 upstream and downstream of bridge.*

Discuss the results and why the recommended alternative was selected. Include discussion on the overtopping frequency and location of the selected alternative.

Describe any improvements that are being made from the existing bridge to the proposed bridge especially regarding natural resources. (Reducing number of spans, decreasing backwater, etc.)

**Scour**

Describe the calculated scour for the recommended alternative and include the tables below.

**Scour Depths**

| Flow ($ft^3/s$) | Contraction Scour (ft) | Pier Scour (ft) | Abutment Scour (ft) | |
|---|---|---|---|---|
| | | | Left | Right |
| $Q_{Design}$ = XXXX | X.XX | X.XX | X.XX | X.XX |
| $Q_{100}$ = XXXX | X.XX | X.XX | X.XX | X.XX |
| $Q_{500 \text{ or } OT}$ = XXXX | X.XX | X.XX | X.XX | X.XX |

**Low Scour Elevation for Scour Check Flood $Q_{XXX}$**

| Channel Bottom Elevation (ft) | minus | Contraction Scour (ft) | minus | Pier Scour (ft) | equals | Low Scour Elevation |
|---|---|---|---|---|---|---|
| XXXX.XX | - | X.XX | - | X.XX | = | XXXX.XX |

**Riprap**

Describe the calculated riprap class. Include the calculations in Appendix D.

Also describe the proposed riprap layout and extents. Include a figure in Appendix D.

**No Rise/Encroachment Analysis**

*If a No Rise/Encroachment analysis is required, follow the procedure described in Section 7.2.2.3 and incorporate the documentation described in Section 7.2.2.5.*

**Appendices (Minimum)**

**<u>Appendix A. Hydrology Report</u>**

**<u>Appendix B. Model Output</u>**

For the Existing Conditions and Selected Proposed Alternative in a 1D/HEC-RAS models, include the following:

Cross-section locations with cross-sections labeled, preferable on an aerial
Profile plot with critical depth turned on
Plot of cross sections with water surfaces turned on
Table 1
Table 2
Six Section Bridge Table

For the Existing Conditions and Selected Proposed Alternative 2D hydraulic models, include the following For the Design Event, $Q_{100}$, and $Q_{500}$/OT Events:

Water surface elevation with contours

Water depth
Difference in water surface elevation between the existing and proposed conditions
Velocity with vectors
Bed shear stress
Plan view distribution of the Froude Number
Plan view of the materials coverage and Manning's n values
Profile plot of the water surface and ground profiles
Oblique view of the ground elevations with the 100-yr water surface extents
Oblique view of the mesh geometry including bridge

**Appendix C. Scour Calculations**

All Proposed Alternatives:
- Scour Design Flood Contraction Scour

Selected Proposed Alternative
- Full Scour Analyses for both Scour Design and Scour Check Floods
  - Long Term Degradation
  - Contraction Scour
  - Pier Scour
  - Abutment Scour
  - Scour Sketch

**Appendix D. Riprap Calculations and Schematic**

**Appendix E. Photos**