IDAHO TRANSPORTATION DEPARTMENT

BRIDGE HYDRAULICS MANUAL

JULY 2021

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

LIST OF ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ACB	Articulating Concrete Blocks
AOP	Aquatic Organism Passage
AS	Abutment Scour
BFE	Base Flood Elevation
CLOMR	Conditional Letter of Map Revision
CN	Curve Number
CS	Contraction Scour
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
HDS	Hydraulic Design Series
HEC	Hydraulic Engineering Center (USACE)
HEC-HMS	Hydraulic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydraulic Engineering Center River Analysis System
HRAM	Hydraulics Risk Assessment Memorandum
IBDM	Idaho Bridge Design Manual
IDF	Intensity-Duration-Frequency
IDWR	Idaho Department of Water Resources
ITD	Idaho Transportation Department
LIDAR	Light Detection and Ranging
LOMR	Letter of Map Revision
LRFD	Load Resistance Factor Design
LTD	Long Term Degradation
NAVD88	North American Vertical Datum 1988
NCHRP	National Cooperative Highway Research Program
NFHL	National Flood Hazard Layer
NFIP	National Flood Insurance Program
NRCS	Natural Resources Conservation Service
NSSDA	National Standard for Spatial Data Accuracy

OHWM	Ordinary Highwater Mark
PS	Pier Scour
RMSE	Root Mean Square Error
S&L	Situation and Layout
SMS	Surface-water Modeling System
SRH-2D	Sedimentation and River Hydraulics – Two-Dimension
TRM	Turf Reinforcement Mat
USACE	U.S. Army Corps of Engineers
USCG	U.S. Coast Guard
USFS	U.S. Forest Service
USGS	U.S. Geological Survey
WMS	Watershed Modeling System

1 BACKGROUND AND GENERAL INFORMATION

1.1 Overview

The purpose of this manual is to guide Idaho Transportation Department (ITD) staff and consultants in the hydraulic aspects of the design of bridges and bridge-sized culverts crossing streams and floodplains. A bridge-sized culvert for the purposes of this manual is one having a hydraulic opening width of 10 feet or more. An appropriate design for such a bridge incorporates the considerations of motorist safety, infrastructure resiliency, and cost effectiveness while preventing unacceptable impacts to the environment and to the flood risk of neighboring properties. This section establishes general expectations as to the hydraulic engineering activities to be performed in the bridge design process along with the associated documentation and deliverables. It describes general criteria and the steps to establishing project-specific criteria. The manual also describes best practices for the various activities of bridge hydraulic design such as hydrologic analysis, hydraulic modeling, regulatory compliance, scour evaluation, and scour countermeasure design.

All structures with an opening less than 10 feet shall use the Roadway Design Manual – Section 600. Bridge deck drainage guidance is provided in the Idaho Bridge Design Manual (IBDM) in Article 2.6.6.

1.2 Hydraulics in Project Scoping

Hydraulic considerations are important in the early phases of scoping a bridge project. Hydraulic concerns should inform early assumptions about the bridge length (or culvert hydraulic opening width) and orientation of the structure. Such concerns include but are not limited to:

- Compliance with local, state, or Federal Emergency Management Agency (FEMA) floodplain and floodway regulations
- Environmental permitting challenges
- The potential for the river channel to migrate
- The space requirements for countermeasures to protect abutments from scour

The engineer responsible for hydraulics on the project should, therefore, begin collecting and examining field and engineering data as early as practicable. The ITD Form 211, *Hydraulic Structures Survey*, provides on the first page a good summary of data that should be compiled early in the project. The information described as content for the Hydraulics Risk Assessment Memorandum (see Section 1.7.1) is all relevant and appropriate for development and consideration during the scoping phase, whenever practicable. Form 211 is included as Appendix A of this manual.

1.3 Risk Considerations

The hydraulic design of a bridge crossing project must include the consideration of risks associated with the project. The types of risks can generally include, but are not limited to:

- Risk to the safety of motorists using the facility
- Risk of damage to the highway infrastructure
- Risk of increasing flood potential to adjacent properties
- Risk of regulatory non-compliance
- Risk of excessive impacts to the environment and habitat values of the stream and floodplain
- Risk of economic losses due to loss of roadway connectivity

This manual presents standards, criteria and guidance that generally apply to bridge projects undertaken by ITD. These standards and criteria represent the minimum for most projects. An assessment of the risks associated with each specific project enables the project team to determine whether the minimum standards are appropriate to address those risks. For example, a decision to increase the freeboard criteria is well justified if the risk assessment reveals a high potential for heavy debris loading during floods.

1.4 New vs. Rehabilitated Structures

The standards, criteria and guidance in this manual apply chiefly to the design of new crossings and replacement bridges and not necessarily to rehabilitation or upgrade projects. In certain rehabilitation cases, though, they may still be relevant. Examples include:

- A superstructure rehabilitation to a bridge with known scour risks where hydraulic and scour analysis are justified to mitigate the scour risk as appropriate to protect the new investment
- A widening of an existing bridge to accommodate more lanes and a higher traffic volume, which increases the potential economic and safety risk

All projects to rehabilitate bridges over streams should be examined to determine whether a situation exists to justify the application of the standards, criteria and guidance in this manual.

1.5 Existing versus Proposed Conditions

The hydraulic analysis of a new crossing or structure replacement requires a comparison of proposed conditions to baseline (usually existing pre-project) conditions. Such a comparison is of critical importance when one of the following conditions exists:

1. The project is located within a floodplain regulated under the National Flood Insurance Program, as described in Section 6.
2. There are existing structures or other infrastructure near the project site whose potential flood risk could be affected by the project.
3. The stability of the channel is found to be at or near a threshold condition such that minor changes in the stream velocity could lead to new channel stability problems that do not currently exist.
4. Sensitive environmental resources or habitat exist in the vicinity of the bridge that could be affected by hydraulic changes induced by the project.

The baseline model is usually a model of existing, pre-project conditions. The proposed conditions model should, to the extent possible, match the baseline model exactly except for physical changes introduced by the project (e.g. changing the bridge abutment locations, span lengths, road profile, alignment, etc.).

The comparison of proposed to baseline conditions serves multiple purposes. Most importantly it will identify and quantify the potential impacts caused by the design and enable the design of mitigation approaches if necessary. The impacts are project specific but could include:

- Impacts to the water surface flood profile for one or more flood recurrence intervals
- Changes in velocity in the vicinity of the bridge leading to channel instability
- Changes in the sediment transport capacity of the channel in the bridge-affected reach

A rehabilitation project may also require a comparison of baseline versus proposed conditions if the dimensions of a pier are to be changed or if any other change is proposed that could affect the bridge hydraulics in low-flow or flood conditions.

1.6 Bridge-Sized Culverts

Culverts that have a total span (hydraulic opening width) of 10 feet or more are considered bridge-sized culverts by ITD. Open-bottom (bottomless) culverts are subject to the same design standards, hydraulic analysis and documentation requirements as bridges. The same design standards, analysis, and documentation requirements also apply to closed-bottom culverts, with exceptions specifically noted in the relevant design sections below. All references to culverts in this manual refer to bridge-sized culverts.

1.7 Bridge Hydraulics Workflow and Documentation Requirements

The workflow chart in Figure 1.1 illustrates the sequence of hydraulics activities and deliverables throughout the design process in a typical ITD bridge design project. The Idaho Bridge Design Manual (IBDM), Article 0.12 provides guidance on the required deliverables and the acceptance process. Hydraulics deliverables are required at three milestones as described in the following sections and as shown in Figure 1.1:

Hydraulics Flow Chart

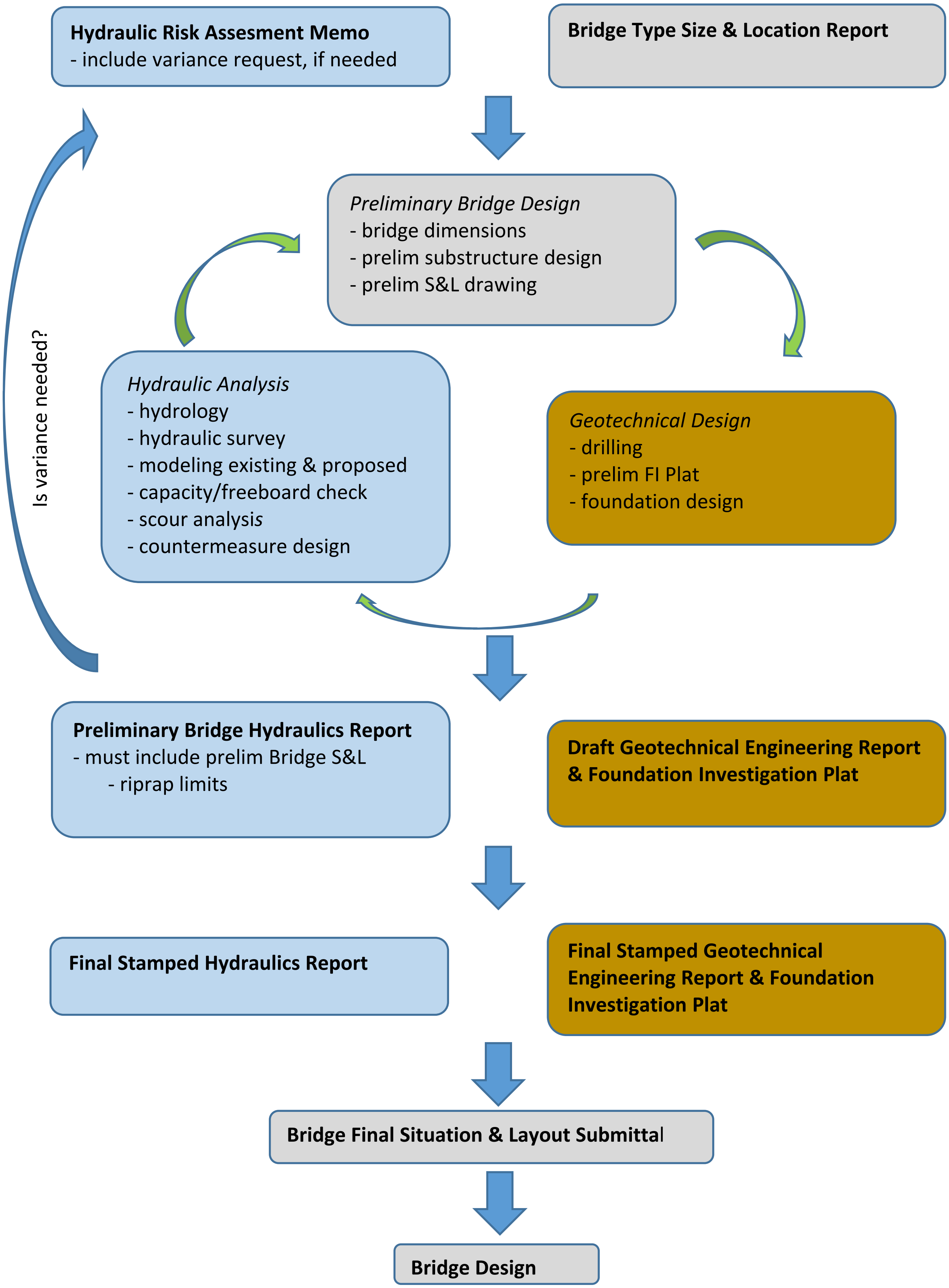

Figure 1.1 Hydraulics activities and deliverables within the bridge design workflow.

1.7.1 Hydraulics Risk Assessment Memorandum

The purpose of the Hydraulics Risk Assessment Memorandum (HRAM) is to document the initial investigation of hydraulics related risks for the project. Its content should follow the detailed outline provided in Appendix B of this manual. The HRAM will summarize the findings of data collection, field and desktop reconnaissance, and a qualitative assessment of the primary risks.

The following data should be collected and examined:

- Any available information about the proposed design (road alignment, bridge length, etc.)
- Notes and photographs from field reconnaissance and geomorphic assessment (a *Field Reconnaissance Checklist* is included in Appendix C as an example)
- Information regarding potential calibration flood events, including date, peak discharge, and photos/locations of high-water marks left by the flood
- Boring logs showing the soil profiles beneath or in the vicinity of the bridge (if available)
- Gradation of the streambed material determined per the guidance in Federal Highway Administration (FHWA) publication HEC-20 *Stream Stability at Highway Structures, Fourth Edition*, (FHWA, 2012a) Section 4.6.3, see Section 7.1.
- Past bridge inspection reports, especially underwater inspections, of the existing bridge (if applicable)
- As-built plans and flood-related maintenance reports of the existing bridge
- Any past hydrology/hydraulic/scour reports for the existing bridge or others nearby crossing the same stream
- Locations of stream gages nearby on the same stream and their periods of peak annual flow records
- FEMA Flood Insurance Rate Maps (FIRMs) and Flood Insurance Study (FIS) Reports relevant to the subject stream reach
- Time history of aerial photographs of the subject stream reach
- Status of sensitive aquatic or riparian habitat in the stream reach

Risk items to be considered and addressed (if applicable) include but are not limited to:

- Flood related risks to motorists (e.g. if roadway overtopping by flooding has historically occurred or if a 100-year or lower flood could potentially overtop the roadway in the proposed condition)
- Risk of impacting the flood profile on residences and other neighboring properties
- Site-specific issues related to ice, debris, hydrologic uncertainties, or navigational clearance that may call for additional freeboard above that required by Section 2 of this manual
- Project located in a FEMA regulatory floodway (requiring the that the project cause no rise in the 100-year flood profile or a Conditional Letter of Map Revision (CLOMR) approved by FEMA prior to starting construction)
- Impact risks to sensitive aquatic or riparian habitat
- Stream stability issues (e.g. widely and rapidly migrating channel or sediment transport)
- Qualitative assessment of scour risk to bridge foundations based on bed material and degree of floodplain constriction
- Whether a variance to the requirements of this manual may be warranted
- Required permits, which could include:

  - *Joint Application for Stream Alteration Permit* administered by Idaho Department of Water Resources (IDWR) and Clean Water Act Section 404 Permit administered by the US Army Corps of Engineers (USACE)
  - Floodplain Development Permit from the floodplain administrator of the local jurisdiction
  - Coast Guard Bridge Permit

The HRAM is required for all projects using this manual and should be submitted to the Bridge Engineer for review and comment. After all comments have been addressed, the HRAM will be accepted by the Bridge Engineer, signaling acceptance.

The HRAM should include a completed page 1 of ITD Form 211 (see Appendix A) as an attachment.

The ITD form 211 is the replacement for ITD Form 210 which in the past covered both culvert and bridge hydraulics. The Form 211 is exclusive to the use of this Bridge Hydraulics Manual and covers all structures 10' and above.

Projects characterized by no overbank flows, being laterally and vertically stable, having low scour risk, or other aspects such that risks within various sections of the HRAM are low or mitigated by examination, are expected to have minimal discussion. However, the applicant of the HRAM is still ultimately responsible for all risks associated with the project and a thorough assessment of each risk.

As illustrated in Figure 1.1, if the need for a variance to the requirements of this manual is determined after the HRAM has been submitted and accepted, the variance request should be prepared, submitted and reviewed as an addendum to this document. The variance will not be granted without an acceptance signature by the Bridge Engineer.

1.7.2 Preliminary Bridge Hydraulics Report and ITD Form 211

The bulk of hydraulics analysis and design work should be completed in coordination with the development of the preliminary Situation and Layout (S&L) bridge plan sheet. This phase of the design is an iterative process conducted by a multi-disciplinary team. The S&L is affected by the hydraulics and the hydraulics are revised to reflect adjustments in the S&L as design is refined. The Preliminary Bridge Hydraulics Report is a required deliverable to document the analysis and support the decisions reflected in the preliminary S&L. The contents of the Preliminary Bridge Hydraulics Report should follow the outline in Appendix D of this manual. The report should refer to the major risk considerations documented in the HRAM and should document the analysis, design and mitigation work that were subsequently performed to address those risks. The report should include:

- Description of the project, its location and the purpose of the study
- Acknowledgement of previous relevant studies (such as FEMA studies)
- Summary description of the watershed and the stream reach
- Reference to and summary of the findings of the HRAM
- The design criteria relevant to the project
  - Flood frequencies for structure freeboard and roadway overtopping
  - Freeboard requirement
  - Flood frequencies for scour design and check flood
- Description of the hydrologic analysis performed and a summary of the results
- Description of the development of the hydraulic model including:

    - Type of model. Two-dimensional (2D) modeling is expected as the default but one-dimensional (1D) modeling in addition to or instead of 2D may be appropriate and should be justified by a variance request.
    - Program(s) used (SRH-2D is expected for 2D modeling and HEC-RAS or HY-8 for 1D modeling).
    - Sources of terrain data (e.g. if Light Detection and Ranging (LiDAR), the date of acquisition, acquired by whom, and the quality level of the data, horizontal and vertical datum of all terrain data).
    - Description of the geometric layout (e.g. cross sections for 1D model or mesh for 2D model).
    - The selection of Manning's n values for different regions of the model domain
    - The different physical conditions modeled (e.g. existing conditions vs. proposed conditions or several proposed alternatives).
    - Model calibration and sensitivity analyses, as applicable.
    - Description of how structures were modeled and any assumptions made.
- Summary of the model results for existing conditions
- Summary of the model results for proposed conditions and the proposed design's performance with respect to the hydraulic design criteria
- Description of the geomorphic concerns and stream stability issues for the bridge site
- Description of the scour evaluation and results (both for the scour design flood and scour check flood) for the proposed design
- Scour profile plots
- Description of the design of scour countermeasures, if any, proposed for the new structure
- Appendix: The preliminary bridge S&L plan sheet. Include the following items below on the S&L plan sheet or additional sheets, as needed. See Article 0.03 of the IBDM for guidance on the bridge S&L:
    - dimensioned riprap and filter system extents
    - total riprap quantity
    - quantity of riprap below Ordinary Highwater Mark (OHWM)
- Appendix: ITD Form 211 fully completed
- Appendix: The Draft Floodplain Development Permit application ready for submission to the community (if required for the project)

Documentation and evidence of the Quality Control process must be submitted in the Preliminary Bridge Hydraulics Report submittal, but as a separate document(s). This documentation should consist of, at a minimum, a completed model review checklist, calculation worksheets initialed by a reviewer, and a comment resolution record showing that review has been performed and the reviewer has been satisfied. Appendix E of this manual provides a model review checklist for 1D and 2D hydraulic modeling. Note that the 2D model checklist has a built-in comment resolution column for convenience and is available as a spreadsheet. All calculations must be reviewed. These may include scour and countermeasure design.

The hydraulic model files must also be provided to ITD with the Preliminary Bridge Hydraulics Report submittal. The model files should include all files necessary to replicate the values provided in the report. The file package should be cleaned of non-essential files and provided as a compressed (e.g.

“zip”) file. The files should include: scatter sets, aerial imagery, and other data sufficient to review model geometry, presented in the project horizontal and vertical datum.

The Preliminary Bridge Hydraulic Report should be submitted to the Bridge Engineer for review and comments for revision. The report should be at a stage where it is ready to be sealed by an Idaho Professional Engineer. The Final Bridge Hydraulics Report will not be reviewed for acceptance until this review is complete and revisions made.

1.7.3 Final Bridge Hydraulics Report and Final Form 211

The Final Bridge Hydraulics Report should be completed before the Final Bridge S&L is completed. This report will have the same content as the Preliminary Bridge Hydraulics Report but will reflect changes to the design that have been incorporated since the Preliminary Situation and Layout and present the results of the final hydraulic and scour analyses. The ITD Form 211 should also be updated and finalized as needed. If a Floodplain Development Permit is required for the project, the approved permit must be included as an appendix to the report. The Final Bridge Hydraulics Report and Final Form 211 must be sealed by a Professional Engineer licensed in the State of Idaho.

The Final Bridge Hydraulics Report submittal must also include a comment resolution record to document that the comments made by ITD during review of the Preliminary Bridge Hydraulics Report submittal have been addressed and responses documented. The submittal must also include a quality assurance cover form, sealed by an Idaho Professional Engineer, showing that the ITD requirements and review have been satisfied. A sample of the quality assurance cover form is included in Appendix E. The quality assurance documents should be provided as separate documents in the submittal and not included as part of the Final Hydraulics Report.

The final hydraulic model files must also be provided to ITD with the Final Bridge Hydraulics Report submittal. The model files should include all files necessary to replicate the values provided in the report. The file package should be cleaned of non-essential files and provided as a compressed (e.g. “zip”) file. The files should include: scatter sets, aerial imagery, and other data sufficient to review model geometry, presented in the project horizontal and vertical datum.

The Bridge Engineer will review the Final Bridge Hydraulics Report submittal to ensure that all review comments have been addressed and that the submittal is complete. The Bridge Engineer will accept the Final Bridge Hydraulics Report and submittal with an acceptance signature on the ITD Form 211.

2 STANDARD DESIGN FREQUENCIES AND CRITERIA

2.1 Hydraulic Capacity Design Standards and Criteria

The expectations for the hydraulic capacity design of ITD bridges and culverts vary depending on the bridge length or the hydraulic opening width of the culvert. The hydraulic opening width of the culvert is the clear span of a single-barrel culvert or the sum of the clear spans in a multiple-barrel culvert, measured perpendicular to the culvert axis. Table 2.1 summarizes the capacity standards and criteria.

Table 2.1 Hydraulic Capacity Standards and Criteria

Bridge Length or Culvert Hydraulic Opening Width	**Design Flood**	**Design Flood Criterion***	**Check Flood**	**Check Flood Criterion**
10 ft and greater, but less than 20 ft	50-year	Minimum 1 ft freeboard to highest point of top of culvert barrel or bridge low chord	100-year	Flow passes through structure without contacting highest point of top of culvert barrel or bridge low chord
All single-span bridges with length 20 ft and greater but less than 50 ft; and all single- or multiple-barrel culverts with hydraulic opening width 20 feet and greater	50-year	Minimum 2 ft freeboard as illustrated in Figure 2.1	100-year	Flow passes through structure without contacting low chord.
All bridges with length 50 feet and greater or with multiple spans	50-year	Minimum 2 ft freeboard at all points	100-year	Flow passes through structure without contacting low chord.
All structures over canals with bridge length or hydraulic opening width 10 ft and greater	Canal Design Flow	Minimum 1 ft freeboard	Canal Maximum Flow	Pass through structure without contacting low chord or top of culvert barrel
Bridges over USCG navigable waterways	Recognized datum per USCG Bridge Permit	Clearance above recognized datum to low chord must equal or exceed requirement of USCG Bridge Permit	N/A	N/A

* These criteria represent the minimum requirements. Considerations such as floating debris or ice potential may call for freeboard or clearance exceeding these criteria.

2.1.1 Freeboard Explained

Freeboard is the vertical clearance between the water surface elevation and the low chord of the bridge. The low chord is the lowest structural member along the length of the superstructure. The water surface elevation value for the freeboard measurement should be taken from a point at a distance equal to the bridge length or the hydraulic opening width upstream of the upstream bridge face but no more than 50 feet. When a bridge 20 feet or longer, but less than 50 feet long, has a bridge low chord with an interior high point (e.g. a crest vertical curve), at least one half of the length of the low chord must provide the specified clearance for the freeboard criterion to be met. This criterion also applies to single- or multiple-barrel culverts with a hydraulic opening width 20 feet or greater. For all bridges 50 feet and longer, and all multiple-span bridges, the specified clearance must be provided along the entire length of the low chord for the freeboard criterion to be met. Figures 2.1 and 2.2 illustrate the concept of freeboard in elevation and profile view, respectively.

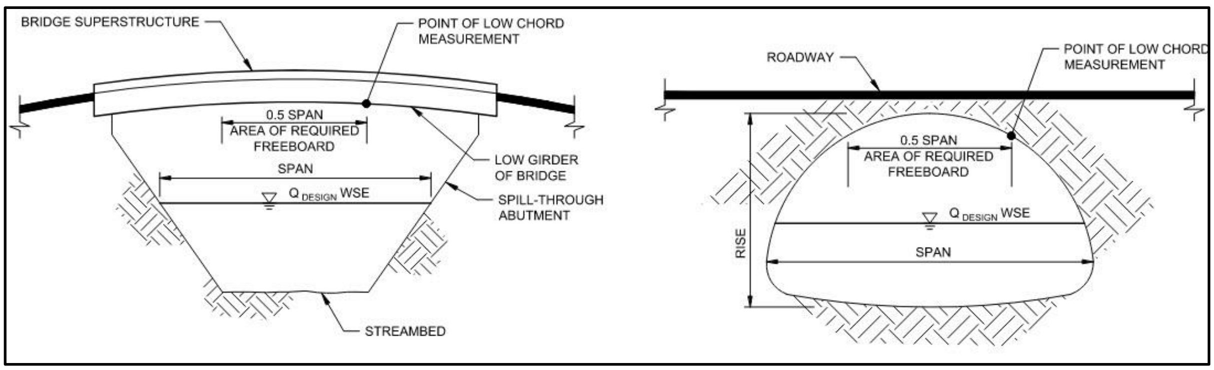

Figure 2.1 Freeboard requirements (bridge elevation view). This applies to single-span bridges 20 ft long and greater but less than 50 feet and single- or multiple-barrel culverts with a hydraulic opening width 20 ft and greater.

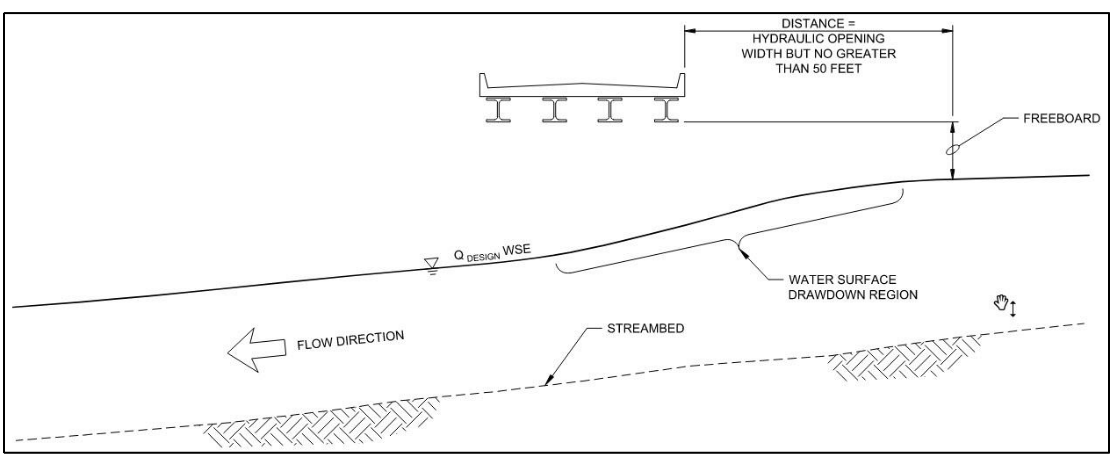

Figure 2.2 Illustration of freeboard requirement (profile view).

2.1.2 Check Flood Impact Limitation

The impact of the project on the 100-year flood profile is an important consideration for bridge or culvert projects crossing floodplains, whether regulated by FEMA or not. In non-regulated floodplains the project should be designed to cause no increase to the 100-year water surface profile over existing conditions to avoid an impact to adjacent structures, critical infrastructure or properties.

FEMA-regulated floodplains could be Zone A, which typically have no Base Flood Elevations (BFEs) and no regulatory floodway, or Zone AE, which typically have both BFEs and a regulatory floodway. **If the project crosses a FEMA Zone AE floodplain, it is imperative to cause no increase in the 100-year flood profile unless a CLOMR is submitted to FEMA and approved prior to the start of construction in the floodway. If the project crosses a FEMA Zone A floodplain, flexibility regarding an increase to the 100-year flood profile may be determined in coordination with the Floodplain Administrator.**

Any project crossing a floodplain, regulated or not, that causes an increase in the 100-year flood profile must be approved by the variance process outlined in Section 2.3. FEMA floodplain and floodway regulations are outlined in Section 6.

2.1.3 Roadway Overtopping

Roadway overtopping is undesirable. It disrupts roadway access and can endanger the traveling public. However, it is not always possible to avoid roadway overtopping because of funding constraints, geometric constraints and other considerations. Historical or potential roadway overtopping should be addressed as a risk in the HRAM. If roadway overtopping is identified as an issue after completing preliminary hydraulic modeling the interdisciplinary team should be notified and alternatives discussed to provide a solution that minimizes impacts. The design of the proposed structure and adjacent roadway should reflect the preferred alternative of the team. Include documentation of the design decisions made related to overtopping, overtopping limits and overtopping depths in the hydraulic report.

2.1.4 Ice and Debris

Neither the relevant FHWA documents nor the guidance provided by the American Association of State Highway and Transportation Officials (AASHTO) provide quantitative guidance on the additional freeboard that should be provided to accommodate ice and debris. The risk for ice and debris should be considered by consulting with ITD maintenance workers and local residents and by examining inspections reports. The risk should be documented in the HRAM. The interdisciplinary design team should be notified, and alternatives discussed to provide a solution to minimize the risk. Recommendations on additional freeboard should be included in the HRAM and documented in the hydraulics report.

2.1.5 Structures over Canals

The Canal Design Flow should be determined in consultation with the canal owner and verified by checking the design and operation manuals, where available, for the operation of the irrigation system. The Canal Maximum Flow should be based on worst-case scenarios of flow entering and staying in the canal upstream of the structure in question. Consideration should be given to the canal's ability to intercept flood flows from adjacent watersheds and other irrigation features or drains and convey them to the bridge. The Canal Maximum Flow should be limited to the maximum carrying capacity of the canal upstream of the bridge, if flows leaving the canal banks would leave the system and not be routed through the bridge. Canal Maximum Flow can often be determined in consultation with the canal owner and verified by checking design and operation manuals, where available for the operation of the irrigation system.

2.2 Scour Design Standards and Criteria

For all bridges, closed-bottom culverts, and open-bottom culverts (e.g. bottomless arches, three-sided stiff-leg structures etc.), the **Scour Design Flood is the 100-year flood (or any lesser flood that is found by hydraulic modeling to produce more severe scour conditions)** and the **Scour Check Flood is the lesser of the 500-year flood or the overtopping flood (or any lesser flood that is found by hydraulic modeling to produce more severe scour conditions than both the 500-year flood and the overtopping flood)**. Scour is to be calculated for both the Scour Design Flood and the Scour Check Flood. The structure must be designed to **withstand the Scour Design Flood at the Service Limit State** and to **withstand the Scour Check Flood at the Extreme Event Limit State** as defined in the AASHTO *LRFD Bridge Design Specifications* (AASHTO, 2020).

For closed-bottom culverts, culvert and appurtenances shall be designed such that the culvert barrel, wingwalls, headwalls, aprons, cutoff walls, and scour countermeasures (if applicable) remain intact and functional in the event of a Scour Design Flood occurrence. The culvert shall be designed such that the culvert barrel is not subject to catastrophic failure in the event of a Scour Check Flood occurrence.

Scour countermeasures shall be designed to protect the structure from scour in a 100-year flood or any lesser flood found by hydraulic modeling to produce more severe scour conditions.

2.3 Variances to these Standards and Criteria

While these standards and criteria are expected to be met on ITD bridge and culvert projects, the circumstances of the project may require a variance to the hydraulic design standards or criteria. Examples include, but are not limited to:

- Raising the existing road profile to meet bridge freeboard criteria would create irreconcilable conflicts such as lost access or failure to meet a fixed grade
- Meeting all standards and criteria would make it impossible to obtain one or more permits (e.g. Floodplain Development Permit, 404 Permit, etc.)
- The project funding vehicle (disaster grant funding, for instance) severely restricts the scope of the project to “replace in kind” without any “betterments’’
- The use of 1D hydraulic modeling instead of, or in addition to the expected 2D modeling is proposed (e.g. for FEMA requirements, or because the waterway is completely one-dimensional, for instance a canal)

Requests for variances to these standards and criteria should be rare and can only be approved by the State Bridge Engineer. If a variance is required, the justification should be explained in the HRAM and should include the following:

- The hydraulic design criteria relevant to the project
- Adverse conditions and/or constraints that are driving the need for a hydraulic design criteria variance
- What it would take to meet all criteria if a variance isn’t granted (significantly greater cost, failure to meet permit requirements, unacceptable schedule for CLOMR approval, etc.)

- Proposed hydraulic design criteria variance
- Discussion of multiple feasible alternatives including the advantages, disadvantages and limitations of each alternative
- Discussion of risks associated with the proposed variance and how the design will mitigate or minimize those risks
- Provide in this section of the HRAM a request and a signature line for the variance to be accepted. The Preliminary Draft Hydraulics Report will not be reviewed until the signed acceptance of the variance.

If the need for a variance to the requirements of this manual is identified after the submittal and acceptance of the HRAM, an addendum to that document must be submitted and accepted by the Bridge Engineer. Acceptance of the variance should be done prior to submission of the Preliminary Bridge Hydraulics Report.

3 HYDRAULIC DESIGN CONSIDERATIONS FOR BRIDGES

3.1 Introduction

The design of a bridge or culvert over water involves a wide variety of decisions across multiple disciplines. Hydraulic issues should be considered early in the design process and will influence many decisions throughout the duration of design and construction. This section provides a brief discussion of each of the major hydraulic design considerations for ITD culverts and bridges over channels and floodplains. Chapter 2 of the FHWA publication HDS 7 *Hydraulic Design of Safe Bridges, First Edition* (FHWA, 2012b) provides a discussion of hydraulic design considerations and regulatory requirements. It can be found at this link.

3.2 Capacity Considerations

3.2.1 Backwater Considerations Affecting Bridge Opening, Culvert Capacity and Road Profile Design Requirements

Normally a roadway crossing of a floodplain, whether featuring a bridge or a culvert, has the potential to cause an increase in the flood profile. This increase is called backwater. Figure 3.1 illustrates the concept of backwater at a bridge. In most cases it is desirable, and in some cases, it is required to design the crossing so that it will cause little or no backwater compared to existing conditions, especially in the check flood. The Floodplain Development Permit from the local floodplain jurisdiction is often the most time-critical permit for a bridge project. It must be obtained before the design can be considered final. When a project involves work in a regulatory floodway, the timeline for a Floodplain Development Permit is typically much shorter if the project can be shown to cause no backwater compared to existing conditions, as documented by a No-Rise Certification. The analysis and documentation requirements for projects crossing floodplains are described in more detail in Section 6.

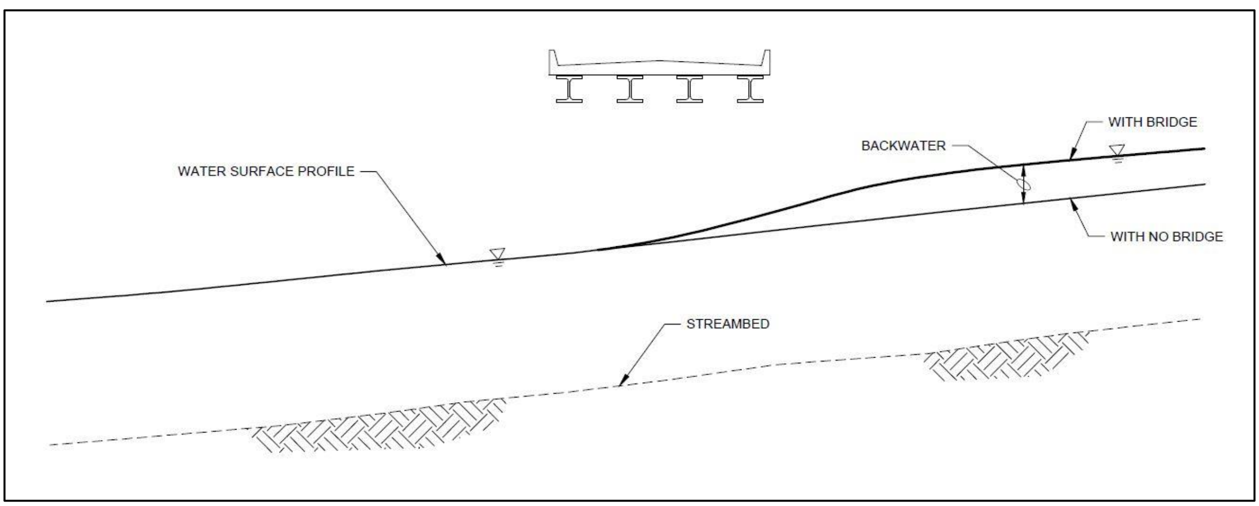

Figure 3.1 Illustration of backwater at a bridge.

Backwater Effects of Bridge Length: Many factors influence the potential backwater, but bridge length or culvert span (which is the hydraulic opening width) is a dominant factor. In general, given a particular design discharge at a given crossing location, a shorter bridge will have less hydraulic capacity and cause more backwater than a longer bridge. For this reason, hydraulic analysis should be incorporated early in the project design to establish the minimum bridge length needed to avoid causing an unacceptable backwater height.

Backwater Effects of Bridge Superstructure: The requirements in Section 2 call for the bridge superstructure to be high enough that the low chord is some distance (freeboard) above the design flood water surface profile and at least as high as the check flood water surface profile. One important reason for this is that backwater can be significantly increased by the water surface submerging the low chord and causing a pressure flow condition. Freeboard provides a measure of protection against floating debris build up that can cause a pressure flow effect. Figure 3.2 illustrates a pressure-flow condition that would be expected to increase backwater at a bridge.

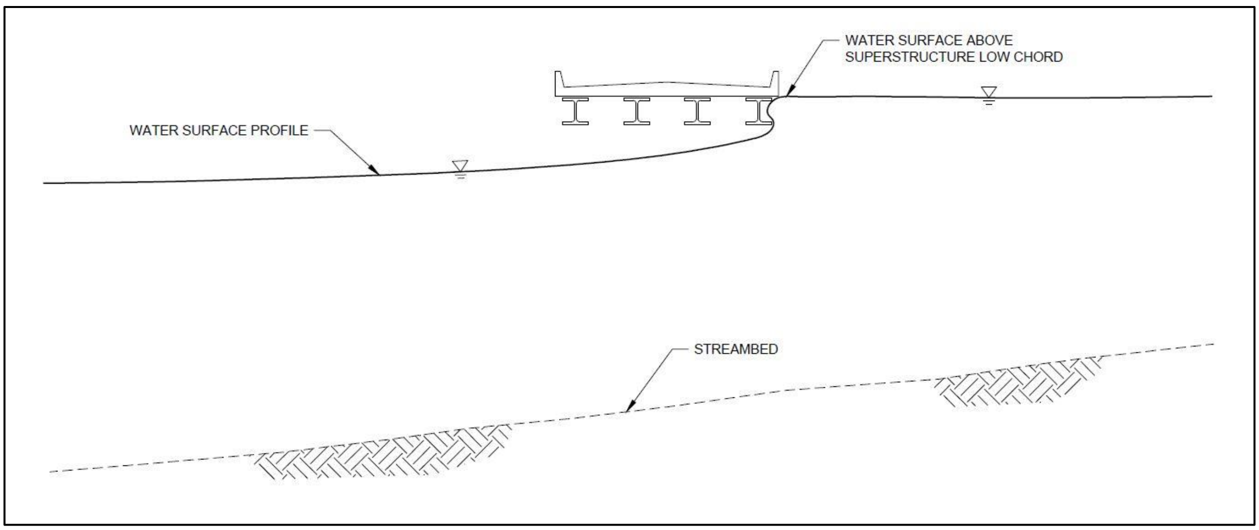

Figure 3.2 Illustration of pressure flow.

Effects of Bridge Piers and Abutments: Bridge piers have a small influence on backwater compared to the bridge length and provision of freeboard. However, in cases where the piers are not aligned with the flow direction, they can form significant obstructions to flow that can cause undesirable backwater and scour. It is desirable, whenever possible, to align the pier axis to the anticipated flow direction. For piers that are aligned with the flow direction, the plan view shape of the pier has an effect on the drag (and thus the backwater potential) exerted by the pier. HDS 7 provides recommended values of drag coefficients for piers of different shapes. Piers with squared off ends have the highest drag coefficient. Oval shaped piers with semi-circular ends have a lower drag coefficient. Piers with a long elliptical shape that are aligned with the flow direction have a much lower drag coefficient than either.

Beyond the obvious relationship between abutment location and bridge length, the configuration and alignments of abutments can influence the backwater potential. Spill-through abutments are generally expected to lead to less backwater than vertical abutments. If vertical abutments are to be used, the abruptness of the hydraulic transition can be somewhat mitigated by using angled wingwalls at the abutment corners. Whether vertical or spill-through abutments are used, abutments aligned with the flow are likely to cause less backwater than misaligned abutments. Figure 3.3 illustrates a pier and two abutments that are misaligned.

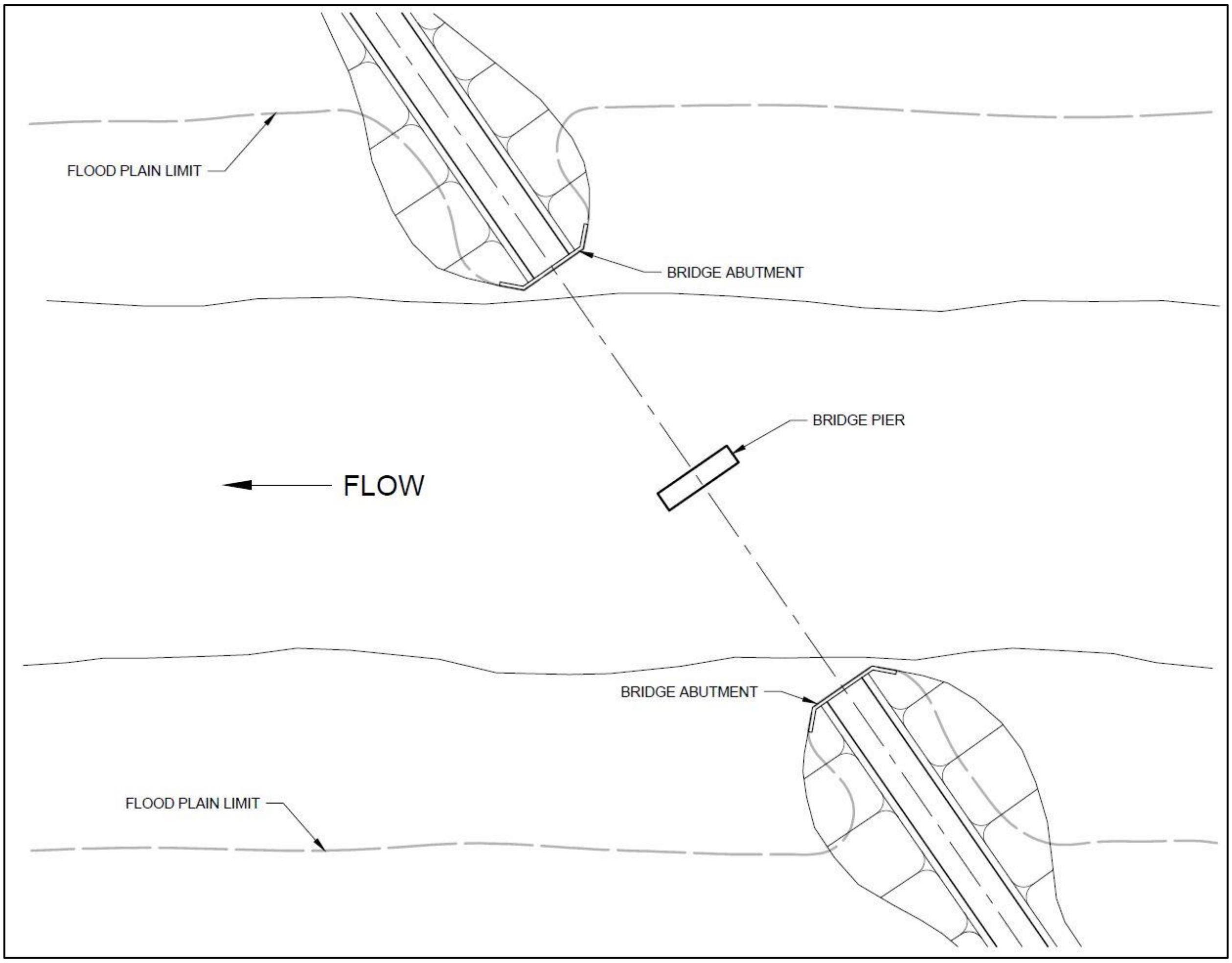

Figure 3.3 Illustration of a pier and abutments that are misaligned to the flow direction.

Backwater at Culverts: Backwater at a culvert occurs when the headwater elevation (the water surface elevation upstream of the culvert) is increased over existing conditions by the project. The relationship between discharge and headwater at a culvert crossing depends on the type of flow control at the culvert. If a culvert is operating in inlet control, the headwater is a function of the discharge, the size and shape of the culvert barrel, and the entrance condition. If a culvert is operating in outlet control, the headwater is dependent on all of those factors plus the length and roughness of the culvert barrel and the tailwater elevation downstream of the culvert. Inlet control at a culvert occurs when flow inside the barrel is supercritical and the flow passes through critical depth at the culvert inlet. Outlet control occurs when flow inside the barrel is subcritical or when the barrel is flowing full. The capacity design process for a culvert, once the design discharge is determined, involves finding the most economical culvert barrel size, shape and inlet configuration that will pass the design flow with an acceptable headwater, and without excessive backwater compared to existing conditions. Hydraulic analysis and design conditions for culverts are discussed thoroughly in FHWA publication HDS 5 *Hydraulic Design of Highway Culverts, Third Edition* (FHWA, 2012c). It is available at this link.

Backwater Effects of Road Profile Design: Highways crossing floodplains usually include encroachment into the floodplain by road embankments in addition to the bridge or culvert structure. If a flood overtops the roadway, increases in backwater are limited because a portion of the flow can pass over the road rather than being forced through the bridge or culvert. For bridge replacement projects, the existing crossing often has a relatively low road profile and therefore a significant amount of overtopping in the design flood or check flood. As discussed in Section 2, the road profile should be raised high enough to avoid overtopping. This approach allows ITD to meet its objectives for motorist safety and service reliability. Raising the road profile, however, has the trade-off of forcing more flow to go through the bridge or culvert. This will lead to increased backwater unless the structure capacity is increased to compensate for the loss of overtopping flow.

3.2.2 Bridge Freeboard

Whatever the allowable backwater may be, an additional capacity consideration is the freeboard. The definition of freeboard and the criteria for providing freeboard in the design are presented in Section 2. Freeboard allows for some amount of floating debris to pass under the bridge superstructure in the design flood. It also accommodates the uncertainty inherent in the estimate of the discharge and uncertainty in the hydraulic analysis and aids in preventing pressure flow, with its undesirable effects on both backwater and scour. If the potential exists for exceptionally large quantities or dimensions of floating debris, the designers should consider providing freeboard exceeding the criteria in Section 2. Such a potential could be indicated by a history of debris problems at the existing bridge to be replaced or at other nearby bridges over the same river. Additional freeboard can be provided to the extent that the project constraints can reasonably accommodate it.

3.2.3 Culvert Headwater Limitations

When designing a culvert, the allowable headwater for the design flood and check flood should be determined after consideration of several potential limiting factors. Section 2 presents the criteria for culverts of different sizes. Beyond those requirements, other considerations include the presence of certain features that may be inundated by the potential headwater in a design flood or check flood. Examples could include:

- Residences or other buildings
- Other roads or railroads
- Ditch berms or other high points which, if submerged, would allow flows to pass into another cross-drainage basin

3.3 Scour and Stream Stability Considerations

3.3.1 Scour at Bridge Substructure Foundations

Floods eroding bed material from the foundations of piers and abutments are the most common cause of bridge failures. Bridge scour during flood events is a significant design consideration for bridges over water. Section 2 presents the appropriate flood frequencies to be used for the scour design flood and scour check flood. Section 7 provides a more detailed description of scour processes and the evaluation of scour potential.

The following types of scour can threaten bridge foundations:

General scour: This is caused by long-term channel instability (lateral and/or vertical). This type of scour is the result of ongoing channel bed movement along a reach extending a significant distance upstream and/or downstream of the bridge. The streambed might be gradually decreasing (degrading) or increasing (aggrading) over time, or the channel bed and banks could be shifting laterally (left or right), narrowing, or widening. Any highly unstable channel reach is undesirable for a crossing location. If the crossing must be located within such a reach, however, measures should be taken in the bridge design to accommodate the potential range of channel movement, both laterally and vertically. HEC-20 describes methods for qualitative and quantitative evaluation of long-term channel instability potential. It is available at this link.

Contraction scour: The reduction in flow width (and pressure flow when the bridge low chord is inundated) at a bridge crossing causes contraction scour. The highway embankments and bridge abutments encroach into the floodplain from the sides and the bridge piers also reduce the flow width, though to a much lesser degree. The reduction in flow width leads to a reduction in flow area and consequently an increase in flow velocity. The increased velocity leads to contraction scour, which can occur in both the main channel and in overbank areas within the bridge waterway opening. Contraction scour potential is increased when the roadway approach embankments and abutment fill slopes encroach significantly into the floodplain. It is further increased when the low chord is inundated. Contraction scour potential is reduced when embankment and abutment fills do not encroach significantly into the floodplain and the bridge low chord is not inundated.

Pier Scour: The flow obstruction caused by a pier and its shape leads to local accelerations and vortices that generate local scour around the base of the pier. The potential for pier scour is higher for wider piers, or long piers that are not aligned with the flow direction.

Abutment scour: Scour occurs at an abutment when the floodplain flow is obstructed by the abutment itself and/or the approaching road embankment. The flow passing through the bridge from directly upstream of the abutment abruptly converges with the redirected floodplain flow in the vicinity of the upstream corner of the abutment. This condition leads to a locally high velocity and strong vortices adjacent to the abutment, causing localized scour. It also produces a wake eddy downstream of the abutment, which can erode the downstream face of the roadway. Even though it is a local scour phenomenon, abutment scour is integrally related to contraction scour in the bridge opening. Thus, the FHWA-adopted method of computing abutment scour is essentially a modified contraction scour equation. The most important influence on the potential for abutment scour is the amount of encroachment by the road embankment into the floodplain.

The methods of evaluating contraction, pier and abutment scour are described in detail in the FHWA publication HEC-18 *Evaluating Scour at Bridges, Fifth Edition* (FHWA, 2012d) which is available at this link.

3.3.2 Incorporating Scour Considerations in Bridge Design

The potential for scour of all types described above must be quantified prior to the bridge foundation design process. The foundation of each pier must be designed to withstand the estimated total scour at that pier (general scour plus contraction scour plus pier scour), at the limit states indicated in Section 2, without the aid of scour countermeasures such as riprap at the base of the pier. Meeting this requirement usually requires the use of driven piles, drilled shafts, caissons, or other types of deep foundation. When foundation costs are high and sensitive to the estimated scour, the design process should be iterative. Increasing the bridge length may significantly reduce the scour potential and decrease the foundation cost sufficiently to justify the longer bridge.

An abutment must be designed to withstand the computed abutment scour, but that design is allowed to incorporate a properly designed scour countermeasure to prevent the formation of abutment scour. Detailed guidance for the design of different types of bridge scour countermeasures is provided in FHWA publication HEC-23 *Bridge Scour and Stream Instability Countermeasures, Third Edition* (FHWA, 2009) which is available at this link. Additional considerations for the design of bridge abutments and abutment countermeasures in consideration of scour potential are described in FHWA Tech Brief HIF 19-007 *Hydraulic Considerations for Shallow Abutment Foundations* (FHWA, 2018), which can be found at the same link provided above for HEC-23.

Shallow foundations in abutment design should only be used after careful consideration of the geotechnical analysis deliverables and calculated scour potential. ITD has generally found that for perennial stream applications, providing shallow foundations and its required scour countermeasures to the depths required is cost prohibitive when compared to deep foundations. Additionally, the level of effort to provide both the foundation and countermeasure to that depth often disturbs the existing soils and creates an additional risk for scour and has undesirable environmental impacts. ITD has seen good performance of deep foundations with respect to minimizing scour risk and in general prefers deep foundations for crossings of perennial streams.

3.3.3 Scour Considerations at Culverts with Closed Bottoms

The design of closed bottom culverts must include consideration of the potential for scour at the outlet. Scour occurs at culvert outlets because of the concentration of flow from a floodplain into a much smaller opening, which results in flow exiting the culvert at a higher velocity than would occur under natural (unencroached) conditions. The outlet scour potential is influenced most dominantly by the culvert outlet velocity, but also by the bed materials in the channel downstream of the culvert.

Inlet control culverts, which have steep barrel slopes and supercritical flow in the barrel, are expected to have more outlet scour potential than outlet control culverts. Outlet scour potential must be evaluated, however for both control types. Methods for computing outlet scour potential are detailed in FHWA publication HEC-14 *Hydraulic Design of Energy Dissipators for Culverts and Channels, Third Edition* (FHWA, 2006) which is available at this link. When significant outlet scour (greater than 3 feet) has been calculated for the scour design flood or scour check flood, a scour mitigation measure should be incorporated into the design to prevent the formation of a scour hole. HEC-14 provides guidance on the design of scour mitigation measures downstream of culverts. For relatively low outlet velocities, a simple riprap apron may be appropriate.

3.4 Permits

A bridge project requires numerous permits. The typical permit requirements that are related to rivers and streams are the Joint USACE/State of Idaho Permits; Coast Guard Bridge Permit; and Floodplain Development Permit. Floodplain Development Permits are discussed in Section 6.

3.4.1 Joint USACE/State of Idaho Permits

The Idaho Stream Channel Protection Act states that the Idaho Department of Water Resources (IDWR) must approve in advance any work being done within the beds and banks of a continuously flowing stream. Any project that would alter a stream channel requires a Stream Channel Alteration Permit from the IDWR and a Section 404 permit from the USACE. These permits are requested through a single application, the *Joint Application for Permits*, which can be found at this link. The permit application requires information about the site and project including, but not limited to:

- The reason or purpose for the project
- A detailed description of each activity that will take place within waters of the United States, including wetlands, dimensions, equipment, construction methods, sediment and turbidity controls, hydrological changes, winter/summer stream flows, borrow sources and disposal locations
- A description of alternatives considered to avoid impacts or measures taken to minimize and/or compensate for impacts to water of the United States, including wetlands
- A proposed mitigation plan or a justification as to why a mitigation plan is not needed
- The type and quantity (cubic yards) of materials to be discharged (placed) below the OHWM or within wetlands
- Exhibits that show the OHWM in cross section view and plan view
- The type and quantity (in acres and cubic yards) of impacts to waters of the United States, including wetlands. The impacts of interest include but may not be limited to filling, backfilling and bedding, land clearing, dredging, flooding, excavation and draining
- A listing of impacts, by activity, to stream banks and shorelines of lakes or reservoirs quantified in linear feet
- A listing of impacts, by activity, to wetlands
- A description of construction phasing and temporary facilities
- Contact information for all property owners adjacent to impacts

Early consultation with ITD's Environmental Planners (at headquarters and/or the district office) is essential for successful attainment of the IDWR Stream Channel Alteration Permit and the Section 404 Permit. To assist in documentation for the permit, the Preliminary and Final Bridge Hydraulics Reports should include the quantity, size and footprint of riprap below the OHWM and within delineated wetland areas.

OHWM can be estimated using hydraulic modeling results for the flood recurrence intervals identified during resource agency coordination (typically 1.5 to 2-year flood). If necessary, the OHWM can be verified and refined by environmental/wetland delineation specialists.

3.4.2 Coast Guard Bridge Permit

Certain waters in Idaho are considered "Navigable Waterways of the United States," and are subject to the regulation of the United States Coast Guard (USCG). Any new or replacement bridge crossing a designated Navigable Waterway of the United States will require a Section 9 USCG Bridge Permit. Early consultation is required to determine the need for a permit. If a permit is required, the USCG will determine the navigational clearance requirements (vertical and horizontal) for the proposed bridge. Additional discussion on Navigable Waters can be found in the Idaho Bridge Design Manual (IBDM) in Article 2.3.3.1. A list of these waters in Idaho can found at this link.

3.5 Riparian Habitat

In addition to wetlands and the waters of the United States, other riparian habitat can be impacted by bridge and culvert projects. The placement of riprap on channel banks to laterally stabilize the channel upstream and downstream of a bridge is an example of a potential impact to riparian habitat. If the geomorphic assessment indicates that stream bank protection will be required a significant distance upstream and downstream of the bridge, impacts to riparian habitat may be an important design consideration. The ITD Environmental Planners at headquarters and/or the district office should be consulted.

The impact of stream stabilization can be mitigated by using void-filled riprap, riprap buried beneath planted topsoil, willow-planted riprap, and other environmentally sensitive bank protection concepts. Making provision for riparian habitat need not automatically lead to a diminished bank protection function, so long as design is carried out properly. Design guidance can be found in, but is not limited to, FHWA HEC-23, National Cooperative Highway Research Program (NCHRP) Report 544 *Environmentally Sensitive Channel and Bank Protection Measures*. (NCHRP, 2005) and the Natural Resources Conservation Service (NRCS) publication *National Engineering Handbook, Part 654* (NRCS, 2008), which can be found at this link.

3.6 Aquatic Organism Passage (AOP)

According to Idaho Law, bridge crossings over naturally fish-bearing streams should not result in a design that acts as an impassible barrier for fish and other aquatic organisms. Accordingly, the passage of fish and other aquatic organisms should be considered in the design of bridges and culverts. Problems with AOP are much more common at culverts with closed bottoms than at bridges. Barriers can exist in the form of drops at the culvert outlet that exceed reasonable organism jump heights; excessive water velocity in the culvert barrel; inadequate flow depth in the barrel; drop inlet configurations; debris accumulations; and excessive turbulence.

The regulatory AOP requirements vary depending on the stream being crossed and the resource agencies having jurisdiction. Early coordination with the ITD Environmental Planners at headquarters and/or the district office is necessary to determine the AOP requirements for the project. AOP issues should be examined before project scoping and discussed in the HRAM.

Methods of evaluating and designing culverts for AOP are provided in FHWA publications HDS 5 and HEC-26 *Culvert Design for Aquatic Organism Passage, First Edition* (FHWA, 2010), both of which can be found at this link. Many other helpful resources exist for AOP, including the U.S. Forest Service's AOP Program resources. An important Forest Service document on AOP is *Stream Simulation: An Ecological Approach to Providing Passage for Aquatic Organisms at Road-Stream Crossings* (USFS, 2008) which can be found at this link. The method and guidance for AOP should be coordinated with ITD Environmental Planners and Resource Agencies having jurisdiction.

3.7 Considerations for Temporary Conditions During Construction

Bridge and culvert projects over water often require temporary features to make construction feasible. Examples of such features include but are not limited to:

- Temporary detour bridges (usually short and/or low profile) spanning the stream for maintenance of traffic flow
- Temporary detour roads across the stream for maintenance of traffic flow, consisting of temporary fill and one or more culvert pipes

- Temporary work bridges or causeways providing construction equipment access
- Temporary fill placed in a portion of the river to divert flow around a work platform for a pier, a pier scour countermeasure, or riprap at an abutment or along a streambank
- Cofferdams or berms to keep river flows out of the work area
- Temporary diversion of river flow through or around the work area by diverting all the river flow into a pipe
- Berms or turbidity barriers to keep disturbed sediment in the construction work areas out of the river flow

If ITD is responsible for the design of a temporary detour crossing carrying traffic, it should be designed such that the probability of overtopping or erosion damage to the detour is less than 10%. Thus, if the detour duration is one year, the detour should be designed for a 10-year flood. The risk of exceedance of a flood of a certain magnitude over a certain duration can be computed using the equation

$$R = 1 - (1 - AEP)^n \qquad \textit{Eqn 3.1}$$

Where:

R= the risk of flood exceedance during the duration

AEP = the annual exceedance probability of a flood of a given magnitude (the inverse of recurrence interval)

n = duration of the detour in years

Temporary detour crossings should be protected from erosion using suitable countermeasures so that temporary fills are not damaged by the design flood for the detour. For motorist safety, the contractor must be prepared to implement a closure of the detour if flow conditions occur, or are expected to occur, that exceed the capacity of the temporary detour crossing. The hydraulic design requirements for temporary work bridges, causeways, diversions or cofferdams, should be determined on a case-by-case basis.

Most temporary features for construction, if they were in place during the occurrence of a 100-year flood, would cause an increase in the flood profile. For projects located in a FEMA floodplain, the contractor must schedule construction activities such that temporary construction features will not be in the river or floodway during the normal high flow season. Additionally, the contractor must be prepared to promptly remove temporary construction features from the river if any significant flood is expected to occur (e.g. in case of flash flood warnings for the area or rapidly rising river flows).

In completing the *Joint Application for Permits*, both permanent and temporary structures within jurisdictional waters must be outlined. No mitigation is required for temporary features as long as they are removed and conditions restored after the project is constructed.

4 HYDROLOGIC ANALYSIS

4.1 Introduction

While hydrology has a much broader definition for general usage, this section focuses on determining the flow for a range of annual exceedance probabilities crossing a highway at a given bridge or culvert location. Specifically, the hydraulic analysis requires as input the peak discharge rate, or a discharge hydrograph, associated with a range of flood exceedance probabilities. The exceedance probability is commonly expressed as the flood frequency or return period. For instance, floods with annual exceedance probabilities of 0.01 and 0.02 are commonly referred to as the 100-year flood and 50-year flood, respectively.

For ITD designs of bridges and bridge-sized culverts (except those crossing canals) the capacity design flood and capacity check flood are the 50-year and 100-year events. The scour design flood and scour check flood are the 100-year and 500-year events. Lower flood recurrence intervals (such as the 1.5-year, 2-year, or 10-year floods) may be of interest for estimating channel forming discharges or OHWM, or for the design of temporary structures for construction.

This section describes acceptable methods of hydrologic analysis for ITD bridge and culvert projects. It explains the data required for each method and the suitability of each method for different sites and situations. The FHWA publication HDS 2 *Highway Hydrology, Second Edition* (FHWA, 2002) provides extensive guidance on hydrologic analysis for transportation projects. It can be found at this link.

4.2 Factors Affecting Flood Hydrology

For a particular recurrence interval, the peak flood discharge at a particular crossing location is influenced by several factors in two categories: meteorological characteristics and watershed characteristics.

The meteorological characteristics that influence flood magnitude include:

- The average annual precipitation, which varies in Idaho from 10 inches to more than 60 inches
- The rainfall depth/frequency relationship for the watershed area
- The intensity and duration of rainfall for the watershed area
- The potential for rain on snow events
- Snowpack and snowmelt factors in the spring

The watershed characteristics that influence flood magnitude include:

- The size (area) of the watershed
- The slopes of the land surfaces in the watershed
- The abstractions and infiltration losses of rainfall on the land surface (affected by soil type, slope and vegetative ground cover)
- Potential aggravating watershed conditions such as hydrophobic soils and reduced ground cover after wildfires
- The percentage of impervious area in the watershed
- The time required for runoff to reach the crossing location (affected by land surface and stream slopes within the watershed.
- Future land changes due to items such as development, tree harvesting, dredging, wildfires, etc.

- Regulated rivers with impoundments for reservoirs or removal of the impoundments.

4.3 Peak-Flow Hydrologic Analysis Methods for ITD Projects

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

4.3.1 Flood Frequency Analysis Using Gaged Data

The United States Geological Survey (USGS) maintains streamflow gages throughout Idaho that provide a record of annual peak discharge rates. These annual peaks can be analyzed statistically, using the Log-Pearson Type III (LP-III) distribution, to develop an estimated relationship between recurrence interval and peak discharge. The nationally recognized standard of practice for flood frequency analysis of gaged data is set forth in *Guidelines for Determining Flood Flow Frequency, Bulletin 17C* (USGS, 2019) which can be found at this link. The Bulletin 17C methods are incorporated into the latest version of the USGS software *PeakFQ* which can be downloaded at this link. Figure 4.1 is an example output plot from *PeakFQ*.

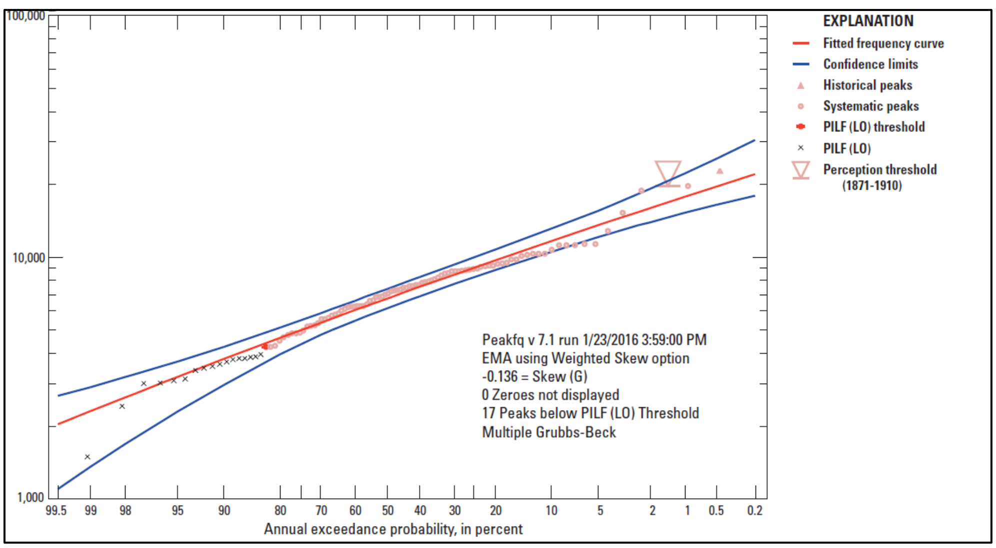

Figure 4.1 Example output plot from PeakFQ (taken from USGS 2017)

For ITD projects, if a gage exists at or near the project site with a record of 20 years or more, a flood frequency analysis is the preferred hydrology approach. Alternatively, a flood frequency analysis developed for that gage by the USGS can be used. Flood frequency analysis for Idaho rivers is further explained in the USGS publication *Estimating Peak-Flow Frequency Statistics for Selected Gaged and Ungaged Sites in Naturally Flowing Streams and Rivers in Idaho* (USGS, 2017), which is available at this link. USGS gage locations can be found using the online resource *StreamStats*, described below.

If the project crosses a stream with a USGS gage but not at the gage location, the gage analysis may be transferred to the project site by using a drainage-area ratio adjustment (raised to an exponent). The drainage area of the ungaged site should be between 0.5 and 1.5 times the drainage area at the gage if gage data transfer is to be used. The USGS publication referenced in the paragraph above (USGS, 2017) provides the equation for the drainage-area ratio adjustment and a table listing the exponent values for different regions and annual exceedance probabilities.

4.3.2 Published FEMA Data

If the floodplain has an associated FEMA Flood Insurance Study (FIS), the FIS report will include the peak discharge rates for several recurrence intervals at various locations along the floodplain. These discharge rates will generally be used for bridge design analysis unless other reliable methods provide evidence that the FEMA discharge rates are inaccurate. Whether or not they are used for design purposes, the FEMA discharge rates from the effective FIS are to be used for all modeling and analysis aimed at floodplain regulation compliance for the purpose of obtaining a Floodplain Development Permit (see Section 6). Figure 4.2 shows an example Summary of Discharges from the FIS report for Boise County, Idaho.

Location	Drainage Area (square miles)	Peak Discharges 10-Year	50-Year	(cfs) 100-Year	500-Year
Payette River At Horseshoe Bend	2,230	21,000	26,000	28,000	38,300
South Fork Payette River					
Below Middle Fork Payette River	1,196	12,800	17,000	18,800	22,900
Above Middle Fork Payette River	779	9,450	12,500	13,800	16,800
South Fork Payette River At Lowman	456	6,300	7,760	8,330	9,570

Figure 4.2 Example Summary of Discharges from FIS report.

4.3.3 Published Regional Regression Equations

When the project crosses a stream that does not have a gage nearby with at least 20 years of record, the preferred approach is to use published regional regression equations that are derived from the flood-frequency relationships at selected gages within a region. The USGS, in cooperation with ITD, developed updated regional regression equations for Idaho and published them in the document *Estimating Peak-Flow Frequency Statistics for Selected Gaged and Ungaged Sites in Naturally Flowing Streams and Rivers in Idaho,* referenced above. The equations are provided for each of 6 regions (Regions 1_2, 3, 4, 5, 6_8, 7). A portion of the state was excluded from the analyzed regions and is referred to in the USGS report as Region 0. Region 0 was excluded because of the extent of flow regulation, significant interaction between groundwater and surface water, and high infiltration rates. Figure 4.3, taken from the USGS report, shows the region boundaries for the regional regression equations. The report includes a table of the ranges of basin characteristics, including watershed area, that were used in developing the equations. Caution should be used in applying these equations for watersheds that do not fall within the indicated ranges. The online *StreamStats* application described below is a tool that greatly facilitates the use of the regional regression equations.

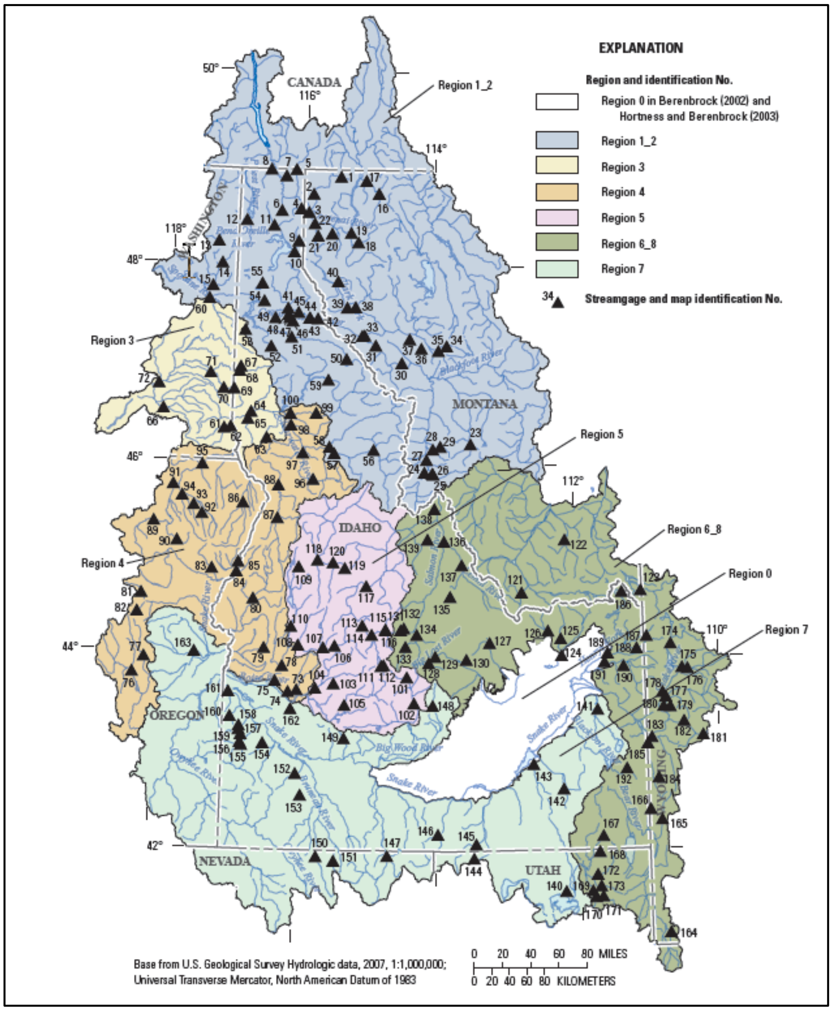

Figure 4.3 Regions for the USGS Regional Regression Equations (USGS, 2017).

4.3.4 USGS *StreamStats*

The USGS has developed an interactive online tool for hydrologic analysis. The *StreamStats* application for Idaho uses the regional regression equations from the 2017 USGS report referenced in the previous section (Section 4.3.3). The tool automatically delineates the watershed draining to the point of interest requested by the user and then evaluates the basin characteristics and calculates the peak discharge for several recurrence intervals using the regional regression equations. Also, within *StreamStats* the user can click on a USGS gage location, and the tool will provide information about the gage along with the calculated flood-frequency relationship. *StreamStats* is accessed at this link.

4.4 Other Available Methods

Any method besides those above will require a variance. If none of the methods above are practicable because of lack of available data, or if they are deemed unsuitable for a particular site, other methods are available and are described in HDS 2. Some other methods that are occasionally used include the following.

4.4.1 NRCS TR-55 Method

The NRCS TR-55 peak discharge method is consistent with FHWA HDS 2 and described in detail in the NRCS publication *Urban Hydrology for Small Watersheds*, (NRCS, 1986). This method applies to watershed areas less than 25 square miles. This method uses rainfall data; land use and ground cover, as represented by the user's assignment of a Curve Number (CN); and the parameter Time of Concentration (T_c) which reflects how rapidly runoff can reach the point of interest from the most remote part of the watershed. The 24-hour rainfall volume for the flood recurrence interval of interest is a required input for this method. This value can be obtained from the National Oceanic and Atmospheric Administration Atlas 2 *Precipitation-Frequency Atlas of the Western United States: Volume 5-Idaho* (NOAA, 1973) at this link.

4.4.2 Rational Method

The Rational Method is a commonly used hydrology approach for storm drains and some culverts. It may be used for watersheds up to 200 acres in size. This method uses the Rational Formula:

$$Q = CiA \qquad \textit{Eqn 4.1}$$

Where

Q = the peak discharge in cfs
C = the runoff coefficient, a function of the ground cover and imperviousness
i = the rainfall intensity in inches per hour for the given recurrence interval in inches/hour
A = the watershed area in acres

The value of the rainfall intensity (i) is taken from an Intensity-Duration-Frequency (IDF) curve for the locality. IDF curves have already been developed for some communities (for instance the City of Boise *Stormwater Management Design Manual* (City of Boise, 2018) contains an IDF curve for use in the city. The engineer enters the curve with a known value of the Time of Concentration (which is the minimum time required for all parts of the basin to be contributing runoff), reads up to the selected recurrence interval or return frequency, and reads across to the rainfall intensity in inches per hour, which is the value of i in the Rational Formula. HDS 2 provides guidance on determining an appropriate value for C.

4.4.3 Rainfall-Runoff Hydrograph Methods

It is generally expected that one of the peak flow methods described above will be suitable for an ITD bridge or culvert project. Some situations, however, may call for a hydrologic analysis using rainfall-runoff methods. Examples of situations calling for rainfall-runoff methods include:

- Large watersheds having no streamflow gage, where more accuracy is needed than can be provided by the available regional regression equations
- When a hydrograph is required rather than just the peak flow, for instance when evaluating downstream hydrologic effects of a project

- When the duration of flow above a certain threshold is needed (e.g. duration of road overtopping)
- When stream routing effects or storage in the watershed are expected to significantly attenuate peak flows at the project site (e.g. wide, low gradient floodplains over a long reach)
- When flood duration is expected to be a limiting factor in calculating scour depth
- When the watershed is complex and includes several subbasins of widely varying characteristics (e.g. rural to urban, mountainous to flat, partly burned watersheds, etc.)
- When it is important to estimate the effects of future changes such as land use in the watershed or climate-related changes in rainfall

Many rainfall-runoff methods are available. The methods differ in several ways, but for the most part they have the following input requirements in common:

- The area of the watershed (often the watershed is subdivided into subbasins)
- Ground cover and soil characteristics of the watershed (often in the form of Curve Numbers) to determine how much of the total rainfall becomes runoff versus infiltrating into the ground
- The total design rainfall depth and the time distribution of the depth (a rainfall hyetograph)
- Travel time and/or time of concentration for runoff to reach the downstream end of the watershed or subbasins
- A unit hydrograph for the watershed or each subbasin (often the standard NRCS unit hydrograph)
- A method of transforming the hydrograph from one collection point (the downstream end of a subbasin) downstream to the point of interest (referred to as hydrograph routing)

The program *HEC-HMS*, developed and maintained by the USACE Hydrologic Engineering Center, is a public-domain tool that can be used to perform rainfall-runoff analysis using a variety of methods. It can be downloaded from this link. The program *WMS*, developed, maintained and marketed by Aquaveo, is another versatile and widely used tool for rainfall-runoff analysis. Many other software programs exist for conducting rainfall-runoff hydrology.

4.5 Determining Design and Check Flood Discharge Rates for Canals

When a project involves a crossing of an irrigation canal, it is necessary to determine the design or operational flow for the canal and the maximum or bank-full flow. Design or operational flow refers to the discharge that the canal owner (irrigation district, irrigation company, ditch company, etc.) intends or expects to be able to convey through the canal to perform its intended system function. If the crossing project were to obstruct flow or cause backwater under design flow conditions, the system operations would be impacted negatively. The design or operational flow (which may be referred to by a different name within the canal jurisdiction) should be provided by the canal jurisdiction.

The canal maximum or bank-full flow refers to the maximum discharge that the canal can convey to the crossing site without losing flow over the top of one or both banks. This value should also be provided by the canal jurisdiction, but if necessary, it can be determined through hydraulic modeling of the canal upstream of the crossing to determine the discharge at which the canal’s capacity is exceeded.

4.6 Documenting the Hydrologic Analysis

Whatever method is chosen for the hydrologic analysis on an ITD bridge or culvert project, the analysis decisions, inputs and results must be thoroughly documented in the Preliminary Bridge Hydraulics Report. The documentation must include but not be limited to:

- Description and graphic depiction of the watershed
- Summary of the pertinent watershed characteristics

- Description of the hydrologic method used and the reason for selecting that method
- If flood frequency analysis was performed using gage data, provide a table of the flood peaks used and how the skew coefficients were assigned
- How the variables were chosen for the regional regression equations, if relevant
- How the rainfall depths and intensities were determined, if relevant
- How the Time of Concentration was determined, if relevant
- How the Curve Number was assigned, if relevant
- How the Runoff Coefficient was determined, if relevant
- A graphical depiction of the IDF curve, if the Rational Method was used
- Documentation from the canal company for the canal design flow and canal maximum flow

5 HYDRAULIC ANALYSIS

5.1 Hydraulic Modeling Approaches

Multiple hydraulic analysis tools are available for bridge projects. The methods and software selected will influence the value of the analysis results. The current state of practice in transportation hydraulics is to use 1D or 2D hydraulic modeling. HDS-7 and HDS-5 provide guidance on the selection and use of methods for the analysis of bridges and culverts. The FHWA reference document *Two-Dimensional Hydraulic Modeling for Highways in the River Environment* (FHWA, 2019) discusses at length the assumptions and limitations of 1D and 2D hydraulic models and is available at this link. Table 5.1 summarizes the comparison of 1D and 2D models.

Table 5.1 Summary of 2D Hydraulic Model Benefits Compared to 1D Models (taken from FHWA *Two-Dimensional Hydraulic Modeling for Highways in the River Environment*)

Hydraulic Variables	1D Modeling	2D Modeling
Flow direction	Assumed by user	Computed
Flow paths	Assumed by user	Computed
Channel roughness	Assumed constant between cross sections	Roughness values at individual elements used in computations.
Ineffective flow areas	Assumed by user	Computed
Flow contraction and expansion through bridges	Assumed by user	Computed
Flow velocity	Averaged at each cross section	Computed at each element
Flow distribution	Approximated based on conveyance	Computed based on continuity and momentum
Water Surface Elevation	Assumed constant across entire cross section	Computed at each element

The default expectation for ITD bridge projects is that hydraulic analysis will be conducted with the 2D model *SRH-2D* within the graphical user interface *SMS* by Aquaveo. The use of 1D modeling must be approved by ITD. If approval for 1D modeling is sought, the justification must be provided in the Hydraulic Risk Assessment Memorandum as a variance request. The use of other 2D models or 1D models must be approved by ITD. Possible justifications for 1D modeling include:

- FEMA compliance modeling when the effective models are 1D and the local floodplain administrator insists on 1D modeling
- The stream being crossed is completely one-dimensional (e.g. a canal)
- The proposed structure is a bridge-sized culvert with straight forward approach and exit flow conditions and no anticipated roadway overtopping

When 1D modeling is approved, it will be performed with the USACE program *HEC-RAS*. In the case of culverts, FHWA culvert program *HY-8* is acceptable if tailwater conditions are straightforward.

Requests for use of HEC-RAS or HY-8 must be done using the variance process and will require justification for it use as outlined in Section 2.3 of this manual.

5.2 Hydraulic Modeling Workflow for Bridge Design

To ensure desirable outcomes, any hydraulic analysis for bridge design should include:

- Analysis of existing conditions
- Evaluation of alternatives
- Analysis of the proposed conditions
- A thorough quality control process

There is a logical workflow that proceeds from one component to the next, but in many cases this process is iterative by necessity. An understanding of the purpose of each component should guide the engineer through the following workflow. This workflow is independent of the steps required to adhere with FEMA Floodplain and Floodway regulations, described in Section 6.

5.2.1 Existing Conditions Analysis

Most bridge projects strive to maintain or improve the hydraulic performance of the structure. This performance can be measured by flow capacity, scour potential, and flood hazard impacts to upstream or downstream properties. A thorough understanding of existing conditions is necessary to assess and compare the hydraulic performance of any proposed design. Therefore, an existing conditions model should be developed and analyzed for all flood recurrence intervals of interest. This analysis is the baseline analysis discussed in Section 1.5. If a suitable previously created hydraulic model already exists, the engineer may choose to review and update it for use as the existing conditions model.

The existing conditions model should accurately represent the hydraulic performance of the existing structure and the river and floodplain as they currently exist. This model should incorporate the best available terrain, land use, and hydrologic data available.

The purpose of the existing conditions model is to serve as a baseline for comparison to the proposed alternatives and eventually to the proposed design. To serve as an accurate basis for comparison, the existing conditions model should match the proposed conditions model(s) in all aspects except for those to be altered by the project. These aspects should include (but are not limited to):

- Spatial extents
- Spatial resolution (cross-section or mesh spacing)
- Terrain data
- Boundary and initial conditions (flow rates and water surface elevations)
- Numerical model parameters

5.2.2 Evaluation of Alternatives

After the existing hydraulic conditions have been established, a series of simplified proposed alternatives should be modeled. A variety of design alternatives such as span lengths, deck thicknesses, pier configurations, and abutment locations may be evaluated. The alternative models need not be highly precise, as the purpose is to compare options and find the best alternatives. The results of these alternatives should be evaluated against desirable project outcomes and constraints. An interdisciplinary team should work together to evaluate the performance and feasibility of alternatives and to select a preferred design alternative(s) for further development.

5.2.3 Development of Selected Design

Once proposed alternative(s) have been selected, the process of refining and analyzing the proposed condition model can begin. The purpose of this model is to gain an understanding of the hydraulic performance of the proposed bridge or culvert crossing, verify and demonstrate that it meets the hydraulic design requirements (standards, criteria, impact limitations) and provide detailed hydraulic data needed for the design of the bridge. These data are needed to evaluate the flow capacity, scour potential, structural loading, countermeasure design, and flood risks associated with the proposed design. As mentioned earlier, the final model of proposed conditions ideally should differ from the existing conditions model only in those aspects to be altered by the project.

5.2.4 Quality Control Process

The hydraulic engineer should employ best practices in model development to ensure an accurate understanding of the hydraulic performance of the project and to provide reliable data for design. This is true of both the existing conditions and proposed conditions model. A good description of best practices in 1D hydraulic modeling for bridge projects can be found in HDS 7. The best practices in 2D modeling for bridge projects are explained in detail in FHWA's *Two-Dimensional Hydraulic Modeling for Highways in the River Environment*.

In addition to following best practices, the modeling should be subjected to a thorough and well-documented quality control process. The Bridge Hydraulics Report must include documentation and evidence of the quality control process (a completed model checklist and comment resolution form showing that review has been performed and the reviewer has been satisfied). *Two-Dimensional Hydraulic Modeling for Highways in the River Environment* provides a review checklist for 2D hydraulic models. Appendix A of Chapter 0 of the IBDM provides a checklist for 1D hydraulic models. The timing of the quality control review should follow the workflow described in Section 1.7 of this manual.

The quality control process will ensure proper function of the hydraulic model and validate the underlying model input data. Any errors or inadequacies in the model must be documented, evaluated, and corrected. Such issues to be considered include, but are not limited to:

- Model resolution
- Geometric representation of terrain, structures and other hydraulic controls
- Hydrologic flow rates and model boundary conditions
- Model parameters
- Model stability and convergence issues

5.3 Data Requirements

Regardless of the type of analysis performed, a significant amount of input data is required. To the extent possible, input data should be acquired before model development begins.

5.3.1 Model Extents

The hydraulic engineer must first estimate upstream and downstream extents of the model domain. *Two-Dimensional Hydraulic Modeling for Highways in the River Environment* describes the factors to consider when determining the model extents. The main concerns with model extents are to:

- Place the downstream end far enough downstream of the project area so that the results in the project area are not sensitive to error that may be introduced at the downstream boundary condition (usually a known or assumed water surface elevation)
- Place the upstream end far enough upstream of the project so that the results in the project area are not sensitive to error that may be introduced at the upstream boundary condition (most significantly the flow distribution at the inflow boundary)
- Place the upstream end far enough upstream of the project so that the water surface impacts of the project will be fully captured within the model

A flow constriction or grade break that would cause critical depth, or a location where the water surface elevation for the discharge being modeled is a known value is an ideal location for the downstream end of the model. In the absence of clear advantageous locations for the upstream and downstream model limits, *Two-Dimensional Hydraulic Modeling for Highways in the River Environment* Suggests initially locating the model limits at least two floodplain widths away from the bridge crossing. However, each scenario is unique. The hydraulic engineer should evaluate boundary condition location for each project given that project's unique considerations and location.

A sensitivity analysis should be conducted once the model has been developed and simulations can be run, to determine if the results in the project area are sensitive to reasonable changes in the downstream boundary condition value and/or small changes in the flow distribution at the inflow boundary. The sensitivity analysis may indicate that the model extents need to be increased in the upstream and/or downstream direction or that the range of boundary conditions uncertainty be reduced with refined estimates.

The lateral extents of the model must be sufficient to cover the full extent of inundation for the highest discharge being modeled.

5.3.2 Terrain Data

Accurate terrain data should be collected for the full model extents. If the model extents haven't yet been determined at the time of terrain data acquisition, a general recommended guideline is to acquire data at least three flood plain widths upstream and downstream of the area of interest, as illustrated in Figure 5.1. This general guidance helps ensure that the terrain data will be sufficient to cover the full model domain extents. Note that the recommendation of three floodplain widths exceeds the suggested initial model extents presented in the previous section. The recommendation here for a longer extent of terrain data allows for the possibility that the required model extents will exceed the initial required model extent estimate.

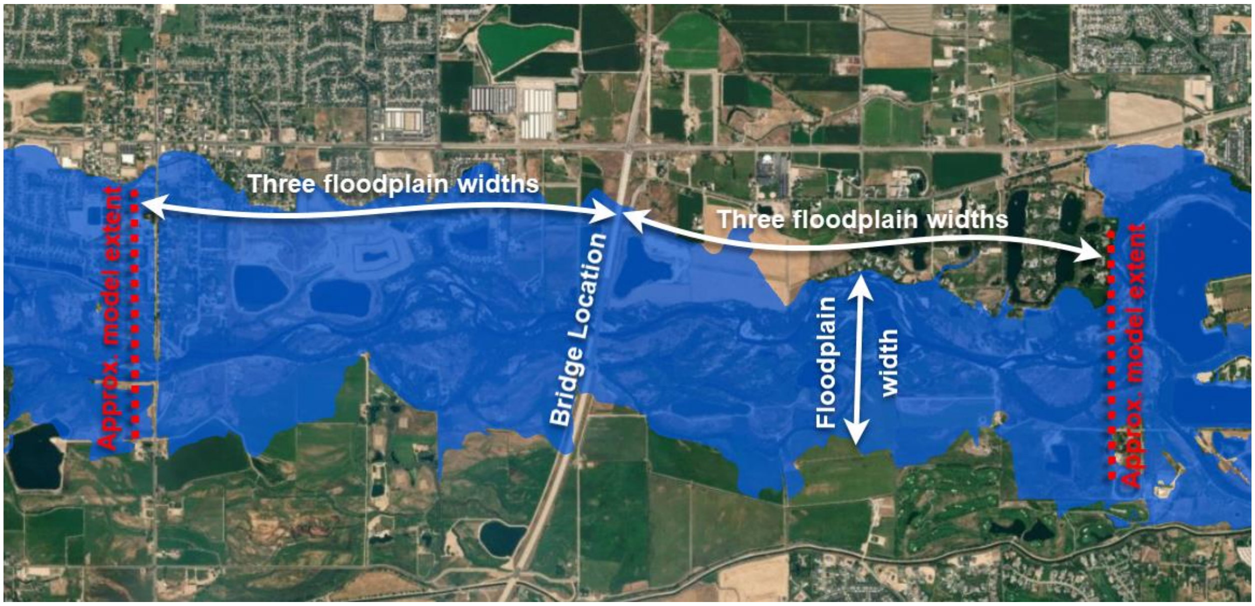

Figure 5.1 Guidance for terrain data acquisition extents if model extents are not yet known.

Terrain data for a typical bridge hydraulic analysis will often come from at least two types of sources: topographic mapping of the floodplain overbank areas and bathymetric survey of the lower (underwater) portion of the main river channel. Modern aerial mapping is often in the format of dense LiDAR point clouds and bathymetric data is usually in the form of boat-based fathometer readings or cross sections obtained by surveying across the channel. Bathymetric data usually has far fewer data points than the above-water aerial mapping. Bathymetric data is merged with aerial data to create a continuous terrain surface model.

The terrain data must be acquired in, or reprojected to, the standard horizontal coordinate system and projection for the project. That will ensure that the hydraulic model inputs and results will be in the same coordinate system as the project. The vertical datum of the terrain data must be the North American Vertical Datum 1988 (NAVD 88) unless otherwise specified by the ITD project team.

Two-Dimensional Hydraulic Modeling for Highways in the River Environment provides useful guidance on acquiring, evaluating, and merging terrain data. It includes a detailed discussion of accuracy standards for terrain data. For ITD bridge projects the accuracy requirements are as follows:

- Within all areas of key interest (along and adjacent to the crossing alignment and at significant hydraulic controls throughout the model domain) the terrain data should meet the National Standard for Spatial Data Accuracy (NSSDA) standard of RMSE 0.3 feet
- Throughout the remainder of the model domain the terrain data should meet the NSSDA standard of RMSE 0.6 feet

The terrain data requirements and standards are the same for 1D and 2D modeling. Both methods of modeling require high quality terrain data. Given the same terrain dataset, 2D modeling will yield more accurate and insightful hydraulic results than 1D modeling.

5.3.3 Structure Data

A combination of survey, field verified as-built plans, and proposed design information is needed to accurately represent the existing and proposed structures. For bridges this information should include:

- Low chord elevations and dimensions
- Bridge deck details and thickness
- Railing elevations
- Abutment locations, shape, and dimensions
- Wingwall locations and geometry
- Pier locations and geometry
- The profile and typical cross section(s) of the road embankments encroaching into the floodplain

5.3.4 Ground Cover and Land Use Data

The Manning's n values used in the hydraulic model should be based upon the engineer's observation of ground cover and land use. Aerial imagery and photographs should be recent and detailed. A site visit is necessary to establish the type, nature, and condition of the vegetation and land use depicted in the aerial imagery and field photographs. All roughness values should be documented, justified, and reported.

Manning's n values should be calibrated if detailed water surface elevation and flow data from a relatively recent high flow event is available. Calibration involves running a model with discharge values and boundary conditions matching known values from a past high flow event and comparing the computed water surface elevation to observed high-water marks from the same high flow event located at various locations throughout the model domain. Manning's n values are then refined, if necessary, to produce a water surface profile that better matches the high-water marks. The appropriate tolerance varies depending on the qualities of the high water marks, in general +/- 0.5ft is a desirable maximum tolerance.

Nearby stream gage data that would have captured the peak discharge of the high flow evnet greatly aids the calibration process. Absent gage data, the USGS often performs post-high flow event investigations to arrive at an estimate of the peak high flow discharge. High-water marks are ideally identified and surveyed shortly after the high flow event, but some can be found years later, on structure walls and trees, for instance. Road maintenance crews can also be useful sources of high-water information from a recent past high flow event, as they are often called upon to assist with emergency maintenance operations during high flow events.

5.3.5 Boundary conditions

In general, two types of boundary conditions are required for a hydraulic model. The first is the discharge rate entering the model from upstream. Section 4 provides guidance on hydrologic analysis to determine discharge rates.

The downstream water surface elevation is the second type of boundary condition. The downstream water surface elevation boundary condition should be located a sufficient distance from the area of interest and be based upon known information ideally (e.g. a water surface elevation from a FEMA Flood Insurance Study Profile or an established relationship between stage and discharge). If such known information is not available to establish the downstream boundary condition, a normal depth water surface elevation may be acceptable. HDS 7 and *Two-Dimensional Hydraulic Modeling for Highways in the River Environment* provide extensive guidance on establishing and evaluating boundary conditions.

6 FEMA FLOODPLAIN AND FLOODWAY REGULATIONS

6.1 The FEMA National Flood Insurance Program (NFIP)

The NFIP was established by the National Flood Insurance Act of 1968. The program aims to reduce the impact of flooding on private and public structures by providing affordable insurance to property owners and by encouraging communities to adopt and enforce floodplain management regulations. Communities (city and county governments) that choose to participate in the NFIP, thereby making flood insurance available to members of their communities, are bound by the federal regulations under 44 Code of Federal Regulations (CFR) Sections 59 through 80. The participating communities must adopt and enforce ordinances, codes and regulations that uphold the federal regulations to avoid sanctions under the NFIP.

FEMA oversees the creation, maintenance, and updating of maps, called Flood Insurance Rate Maps (FIRMs) of the flood hazard areas associated with the major streams and rivers in each community. When an ITD project involves work within or affecting a floodplain depicted on a current FIRM, a Floodplain Development Permit must be obtained from the community having jurisdiction before the work in the floodplain may proceed. If the project affects regulated floodplains in more than one jurisdiction (for instance when the center of the river being crossed is the boundary between two jurisdictions) all affected jurisdictions must issue Floodplain Development Permits.

6.2 Determining Project Specific Requirements

Early in project scoping, the hydraulic engineer should determine whether the stream being crossed is within an NFIP mapped floodplain. FEMA's online Map Service Center provides access to the FIRM maps and Flood Insurance Studies (FIS) for all participating communities at this link. The relevant FIRM panels and FISs, once accessed, can be downloaded in whole or in part. FIRM maps are the authoritative depictions of NFIP floodplain limits. Another helpful online resource is FEMA's *National Flood Hazard Layer (NFHL) Viewer*, found at this link. Figure 6.1 is a screen capture from the NFHL Viewer.

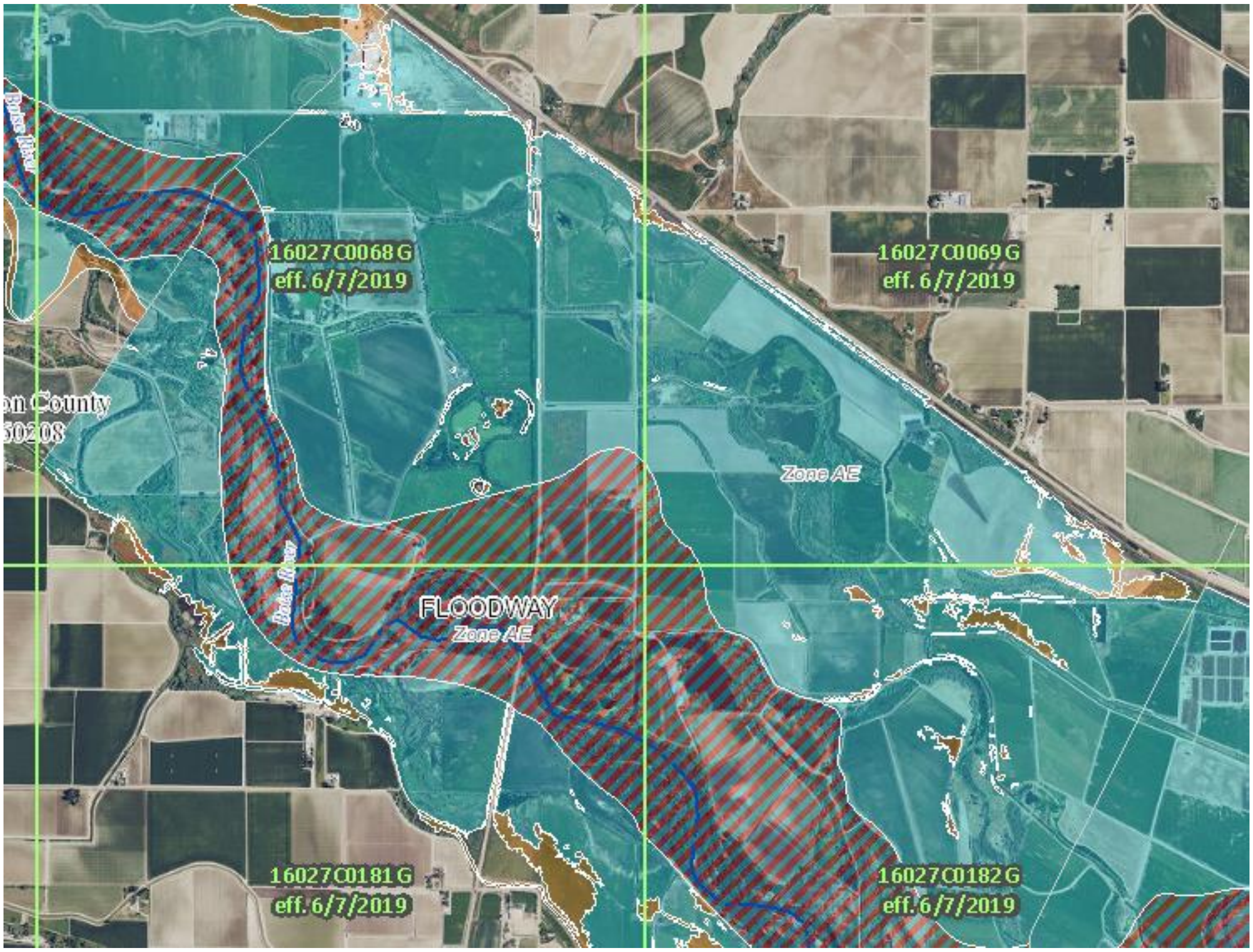

Figure 6.1 Screen capture from the National Flood Hazard Layer Viewer showing a segment of the Zone AE floodplain of the Boise River. Note the flood fringe (blue shading) and the regulatory floodway (hatched area).

Examination of the relevant FIRM panel(s) will reveal whether the project is within an NFIP regulated floodplain. It will further indicate whether the floodplain being crossed contains a regulatory floodway. The floodway is a zone of special restriction and is further described below, in Section 6.3. The FIRM panel will also identify the communities having jurisdiction over the floodplain in the potentially affected area. If the project is expected to involve work within an NFIP floodplain and/or floodway, the hydraulic engineer should begin communication with the floodplain administrator of each potentially affected community as early as possible to determine the specific requirements of that community for a Floodplain Development Permit. The findings of this early investigation and communication should be documented in the HRAM.

6.3 Floodplain Zone Designations

The two basic types of NFIP floodplain zones are approximate floodplains, known as Zone A, and detailed-study floodplains, known as Zone AE. A floodplain with a Zone A designation has been mapped by approximate methods and does not have an officially recognized longitudinal flood profile. Because there is no associated flood profile, there are also no Base Flood Elevations (BFEs). A Zone A floodplain typically has had only an approximate hydrologic or hydraulic analysis or none at all.

A Zone AE floodplain has been the subject of a detailed hydrologic and hydraulic analysis approved by FEMA. It also has an associated flood profile, and BFEs established along the length of the floodplain reach. Zone AE floodplains typically consist of two zones, the floodway and flood fringe. The analysis and documentation requirements for a project will depend partly upon whether the affected floodplain is Zone A or Zone AE and whether work will take place within the floodway.

6.4 Analysis and Documentation Requirements

The analysis and documentation required to obtain a Floodplain Development Permit may vary for different jurisdictions in Idaho, and therefore the floodplain administrator of the community having jurisdiction should be consulted as early as possible. The following information describes the minimum requirements that should be expected based on the federal regulations.

6.4.1 Zone AE Floodplains

If the project will include work within the flood fringe of a Zone AE floodplain but no work will occur in the regulatory floodway, the information needed to obtain a Floodplain Development Permit may be minimal. To obtain a Floodplain Development Permit, it is typically only necessary to show on the design plans that the project work will encroach into the flood fringe but not into the floodway.

If the project will include work in the floodway (e.g. placing fill or constructing piers or abutments), it must be demonstrated that the project will not cause any increase to the base (100-year) flood profile. A No-Rise Certification must be submitted to and approved by the local jurisdiction's floodplain administrator before the Floodplain Development Permit can be issued. The No-Rise Certification is a standard form, stamped by a registered Professional Engineer, stating that the project will not cause a rise in the 100-year flood elevations, floodway elevations or floodway widths at published cross sections in the flood insurance study nor at unpublished cross sections in the vicinity of the proposed development. The form must be accompanied by technical documentation showing that the statement is supported by hydraulic modeling. IDWR has prepared a standard No-Rise Certification form, which is included as Appendix F of this manual.

The floodplain administrator will require, at a minimum, a comparison of the proposed project conditions versus the existing conditions. The input to these two models should be the same in every respect except for modifications associated with the proposed project. With 2D modeling of proposed and existing conditions, a reasonable approach is to demonstrate the following:

- the discharge-weighted mean BFE along evaluation lines will not increase
- no existing insurable structure will be subject to a calculated water surface elevation increase within its building footprint based on 2D model results
- the variability of the proposed condition BFE vs. the discharge-weighted mean BFE along any evaluation line is within +/- 0.5'.

Evaluation lines for 2D no-rise and floodway analysis may be set either in the same locations as effective FEMA 1D cross-section lines (for reaches with effective 1D hydraulic modeling only) or at 2D model existing-condition BFE contour lines.

This must be demonstrated for proposed versus existing conditions models having no floodway encroachment, and also for proposed versus existing conditions models in which the model domain's lateral edges follow the lines of the FEMA floodway encroachments (or alternatively having element edges align with the floodway lines and using unassigned material types for all elements outside the floodway lines). Modeling to demonstrate no rise must use the FEMA 100-year flood discharge for the reach of interest.

If the project involves work in the floodway and the hydraulic modeling shows that the project will cause an increase in the 100-year flood profile or floodway profile, the federal regulations require that a CLOMR be submitted to and approved by FEMA prior to the issuance of a Floodplain Development Permit. The timeline for approval of a CLOMR is open-ended and could run from six months to well over a year. A CLOMR, once approved by FEMA, is a statement of how the floodplain mapping will change after the project featured in the CLOMR application is completed. All affected property owners are notified of any change in the flood risk to their properties that would result from the proposed changes.

A CLOMR does not change the effective regulatory floodplain mapping, because it is describing a proposed future development. When an ITD project requires a CLOMR to obtain the Floodplain Development Permit, it will be necessary after construction to prepare and submit a Letter of Map Revision (LOMR), using a model representing as-built conditions. The LOMR, once approved by FEMA, will change the effective regulatory mapping. The approval process for both CLOMR and LOMR processes involves extensive FEMA review, potentially with many resubmittal cycles before approval is met.

6.4.2 Zone A Floodplains

Zone A floodplains do not have regulatory floodways established. Some communities have stringent requirements governing work in Zone A floodplains. At a minimum per federal regulations, it must be demonstrated that the proposed project, along with reasonably foreseeable future projects that could occur, will not cause more than 1 foot of increase in the flood profile. Since Zone A floodplains typically do not have associated hydrologic and hydraulic analyses, it will be necessary to analyze the hydrology and develop a hydraulic model of both existing and proposed conditions. Coordination with the local floodplain administrator is needed to reach agreement on the method of hydrologic analysis and on what type and level of documentation must be submitted for the Floodplain Development Permit when a Zone A floodplain is involved.

6.4.3 Requirements for CLOMRs and LOMRs

FEMA has established standards and guidance for reviewing and incorporating 2D modeling in floodplain studies. It is anticipated that many future floodplain studies will be conducted using 2D models. At the present time, however, the majority of Zone AE floodplains in the NFIP were developed using 1D hydraulic modeling. From a regulatory standpoint, the most straightforward approach to hydraulic analysis for the purpose of preparing a CLOMR or LOMR request is to use the same approach that was used for the floodplain study that established the mapping.

A specific modeling sequence is expected in CLOMR and LOMR request submittals. The preparer is required to develop:

1. A Duplicate Effective Model in which the engineer runs the relevant portion of the effective model (obtained from FEMA or the local floodplain jurisdiction) through the current version of the same modeling software (e.g. HEC-RAS 1D) to ensure that the results match the effective model. The extents of this portion of the model should be sufficient that the floodplain impacts of the project are fully contained within the modeled reach.

2. A Corrected Effective Model in which the engineer corrects any errors that are found in the Duplicate Effective Model (for instance the existing bridge might not be represented in the model). Also, in the Corrected Effective Model, the engineer adds cross sections or model resolution in all locations that will be needed in later models in the series to accurately depict the addition of the project. For instance, adding cross sections in the locations that will be needed to represent the bridge in the proposed conditions model. The Corrected Effective model may also incorporate more detailed topographic information than was used in the Duplicate Effective Model, but it must not reflect any man-made physical changes since the date of the effective model.
3. An Existing or Pre-Project Conditions Model. This model is a revised version of the Corrected Effective Model that reflects any manmade physical changes that have occurred since the date of the effective model. If no such changes have occurred, then the Existing Conditions Model will be identical to the Corrected Effective Model.
4. The Proposed, or Post-Project Conditions Model. This model is the same as the Existing Conditions Model except for modifications necessary to represent the effects of the project.

In addition to the hydraulic modeling described above, a CLOMR or LOMR request ordinarily includes:

- A FEMA MT-2 form providing specific technical details about the request
- A narrative report explaining the project and the reason for the request
- Design plans (for a CLOMR) or as-built plans (for a LOMR) depicting the project work
- Topographic work maps, in a scale acceptable to FEMA, depicting the effective floodplain and floodway boundaries and the revised floodplain and floodway boundaries
- An annotated copy of the FIRM panel showing the changes to the floodplain inundation
- Revision information that will update the effective FIS Report (e.g. changes to flood profile plots and floodway data tables)
- Payment of the FEMA CLOMR or LOMR review fee

At project close-out, ITD will prepare and submit all data required as conditions of the Floodplain Development Permit.

7 STREAM STABILITY, SCOUR, AND COUNTERMEASURES

7.1 Introduction

Understanding how the stream channel and floodplain are likely to change over the service life of a crossing is critical to the design and protection of bridge piers and abutments. FHWA document HEC-18 provides extensive guidance for conducting scour analysis at bridges. FHWA documents HEC-20 and HEC-23 provide stream stability analysis and hydraulic countermeasure design information. ITD adopts these informational documents as guidance for bridge design. These documents are updated on a regular schedule and are supplemented between editions with TechBrief documents found on the FHWA Hydraulics website at this link, including HIF-19-007: Hydraulic Considerations for Shallow Abutment Foundations.

7.2 Stream Stability Assessment

A stream stability assessment will help inform the appropriate placement, size, and configuration of the piers and abutments, evaluation of scour potential, and hydraulic countermeasure design. A stream stability assessment is required for all bridge and bridge-sized culvert evaluations and designs.

7.2.1 Data Required

The following information is necessary to complete a stream stability assessment:

- Historic channel profile elevation information
- Historic channel cross-section geometry and location information
- Knowledge of upstream and downstream hydrologic and sediment continuity controls
- Spatial and stratigraphic soil / geologic information
- Land use and management information

7.2.2 Desktop Reconnaissance and Data Mining

Prior to field review a desktop reconnaissance should be performed to obtain and/or produce the following data:

- Inspection records and time-series cross-sections plotted on a common datum (to assess past vertical and lateral stability trends at the bridge site)
- Specific gage analysis and evaluation of water surface elevations at nearby gage sites (to assess past vertical stability trends)
- Topographic and, if available, geologic maps (to assess landforms, channel and floodplain characteristics and soil types)
- Time-series aerial imagery with bank lines outlined and annotated (to examine historical channel alignment changes; past land use changes; required model extents; floodplain vegetation types and changes over time; and to aid in establishing sediment sampling locations)
- As-built plans
- Geotechnical investigation reports
- Planned locations for bed sediment gradation sampling:
    - main channel approach section (surface and subsurface if significantly different in type or apparent gradation)
    - overbanks if significantly different from subsurface material at approach channel section

- at bridge channel and overbanks if significantly different from approach section materials

7.2.3 Field Reconnaissance

A field reconnaissance site visit should be performed by an experienced and knowledgeable practitioner. An example set of field reconnaissance sheets (adapted from HEC-20) is included in Appendix C of this manual.

Bed sediment sampling: collect a sub-armor-layer Bulk sample collection and sieve analysis of channel bed material at the approximate location of the approach section unless bed is coarse enough that bulk sampling is not practicable. In that case, a Wolman Count procedure should be used to capture a grain size distribution , converted to a gradation by weight using the methods presented in HEC-20 at this link.

7.2.4 Lateral Stability Assessment

Most natural channels are prone to lateral movement over time. The nature and speed of these processes can have a significant impact on the design and protection of piers and abutments. HEC-20 provides guidance on how to identify and assess lateral stream migration.

7.2.5 Vertical Stability Assessment

Aggradation (deposition) or raising of the stream bed over time can reduce channel capacity, leading to more frequent out-of-bank flood events, and result in decreased lateral stability. Degradation (erosion), or lowering of the stream bed, can result in significant risk to pier and abutment foundations. Both aggradation and degradation can result from upstream or downstream changes in hydrology, hydraulics, and sediment continuity. HEC-20 provides extensive guidance on how to identify and assess vertical stream stability.

7.2.6 General Stability Assessment

Lateral and vertical instability analyses should calculate whether each pier or abutment will be subject to degradation and/or main-channel hydraulic conditions (due to lateral channel movement) over the service life of the structure. The results of both the lateral and vertical stability evaluations should be documented, justified, and applied to each bridge substructure element.

7.3 Bridge Scour Evaluation

Bridge scour evaluation methods apply to bridges, bridge-sized bottomless arches and three-sided stiff-leg structures. A properly performed scour analysis will provide a calculated estimate of the scour potential at each pier or abutment of a bridge. Scour estimates should be used to establish a minimum design scour elevation for each pier or abutment for the scour design flood and the scour check flood. These design scour elevations can then be used to inform the design of bridge foundations, and to set the appropriate scour countermeasure design embedment. The results of the stream stability analysis must be considered as part of the scour evaluation.

If a given pier or abutment is potentially subject to main-channel hydraulic conditions during the design life of the bridge, then it should be evaluated both for its current hydraulic conditions and for main-channel hydraulic conditions. The pier or abutment should then be designed for the minimum (lowest) calculated design post-scour elevation.

Bridge scour evaluation includes estimation of general scour, contraction scour and local scour. For abutments, the recommended scour calculation method combines contraction scour and local scour calculations. ITD requires the use of the methods described in HEC-18 for scour evaluation.

7.3.1 General Scour (Vertical and Lateral Instability)

The designer must account for the combined vertical effect of long-term degradation potential and lateral long-term instability of the main channel. For instance, if the main channel is expected to degrade by a certain amount and the main channel is expected to migrate a certain distance over the service life of the structure, then any pier that is within the current main channel limits or within the calculated migration zone would be evaluated using the thalweg as the starting elevation and subtracting the long-term degradation before decreasing bed elevation by the contraction and local scour potential. This process is illustrated in the righthand portion of Figure 7.1.

7.3.2 Contraction Scour

Contraction scour is a general lowering of the bed elevation beneath a structure due to contraction of the flow width or depth. It occurs simultaneously with local scour (pier scour and abutment scour) during a flood event. The HEC-18 methods used to evaluate contraction scour are based on the fundamental concepts of continuity and sediment transport.

Contraction scour is computed with different methods depending on whether the reach upstream of the bridge can supply and transport significant quantities of bed material sediments. The gradation of the approach-section sediment sample obtained during field reconnaissance should be used to evaluate sediment transport conditions at the approach section.

A properly selected approach cross-section:

- Represents hydraulic flow characteristics that are not within the bridge’s immediate zone of hydraulic influence, meaning it is upstream of the zone of flow contraction
- Accurately represents the sediment transport characteristics of the upstream channel segment
- Only considers the approach cross-section width that is transporting bed material sediment (if operating under “live-bed” conditions)

HDS 7 presents general information on how to locate an appropriate approach section in a 1D model. *Two-Dimensional Hydraulic Modeling for Highways in the River Environment* provides information to help locate the approach cross section in a 2D model. Failure to appropriately locate the approach cross-section will result in unreliable contraction scour calculations.

If the upstream reach does not transport significant quantities of bed material sediments, contraction scour is calculated for that flow using “clear-water” contraction scour equations, and particular consideration should be given to the channel and overbank sediment characteristics at the bridge opening, as the results are highly sensitive to the gradation and type of sediment present at the bridge.

7.3.3 Local Scour

Local Scour is caused by the local flow accelerations, flow separations, vortices and turbulence that form as water is forced around an obstruction. Two types of local scour are of primary interest to the bridge designer: pier scour and abutment scour.

Pier Scour

The flow obstruction caused by a pier leads to local accelerations and vortices that generate local scour around the base of the pier. The potential for pier scour is higher for wider piers, or long piers that are not aligned with the flow direction. Pier scour depths shall be calculated using the HEC-18 pier scour equations. Practitioners should carefully evaluate the angle at which flow is attacking the pier and the location from which hydraulic variables (impinging flow velocity and depth) are extracted. This location should be from 3 to 7 pier widths upstream, outside the zone of immediate hydraulic influence, and represent the hydraulics to which the pier is being subjected. If the channel stability analysis or 2D model results indicate more than minor variability in the flow patterns and channel alignment over the design life of the bridge, a representative maximum impinging velocity and corresponding depth and angle of attack should be evaluated for each pier.

Abutment Scour

Scour occurs at an abutment when the floodplain flow is obstructed by the abutment itself and/or the approaching road embankment. The flow passing through the bridge from directly upstream of the abutment abruptly converges with the redirected floodplain flow in the vicinity of the upstream corner of the abutment. This condition leads to a locally high velocity and strong vortices adjacent to the abutment, causing localized scour. It also produces a wake eddy downstream of the abutment, which can erode the downstream face of the roadway. Even though it is a local scour phenomenon, abutment scour is integrally related to contraction scour in the bridge opening. The NCHRP 24-20 abutment scour-calculation methods presented in HEC-18 shall be used to estimate abutment scour. The NCHRP 24-20 methods combine contraction scour and local scour effects and provide a single combined local and contraction scour depth value.

7.3.4 Debris Accumulation Effects on Scour

Debris accumulation during flooding events is a problem common to bridges. As expounded upon in "Effects of Debris on Bridge Pier Scour," NCHRP Report 653, debris accumulation increases all scour potential types and can damage piers, decks, and girders. Debris clusters are highly variable between bridge locations and between flooding events of the same location. HEC-20 (FHWA 2012a) provides guidance on upstream debris production potential and debris collection potential depending on the pier location within the channel. HEC-18 (FHWA 2012d) provides debris cluster scour relationships on piers. When the debris cluster size and the flow depth cause these two scour holes to coincide, the maximum scour depth at a pier is likely to occur. 1D hydraulic models can hydraulically simulate debris blockage at individual piers by reducing flow area in the bridge opening. HDS 7 (FHWA 2012b) explains approaches to representing piers in a 2D model to allow for debris collection. The low chord of the bridge can be adjusted in both 1D and 2D models to account for debris collected at the bridge deck.

7.3.5 Total Combined Scour

Once the components of scour have been evaluated, the practitioner must calculate and plot the total scour and minimum scour design elevation for each pier and abutment. Total scour is the combined result of lateral and vertical instability, contraction scour, and local scour. Total scour should be evaluated as following:

- Total scour at a pier is the sum of general scour, contraction scour, and local pier scour
- Total scour at an abutment is the sum of general scour and abutment scour (which incorporates contraction scour in the NCHRP 24-20 method)

- The combined total scour at each pier and abutment should be reported as a post-scour design elevation, referencing the project vertical datum

When plotting total scour please note:

- If a pier or abutment is potentially subject to main channel hydraulics during its service life based on the lateral and vertical stability assessment, the combined effect of these instabilities must be accounted for in design. Shift the reference (starting) ground elevation for contraction and local scour depth down to the channel thalweg elevation and subtract any calculated degradation potential. The scour depth should be plotted downwards from the existing thalweg elevation to assess the minimum design elevation for that pier or abutment.
- If a pier or abutment will not be subject to main channel hydraulics during its service life, the reference elevation for scour depth is the existing terrain elevation at that location.

Figure 7.1 presents an example of how total scour should be plotted. The left half of the figure assumes that the stream is laterally stable, while the right half of the figure assumes that the stream is laterally unstable.

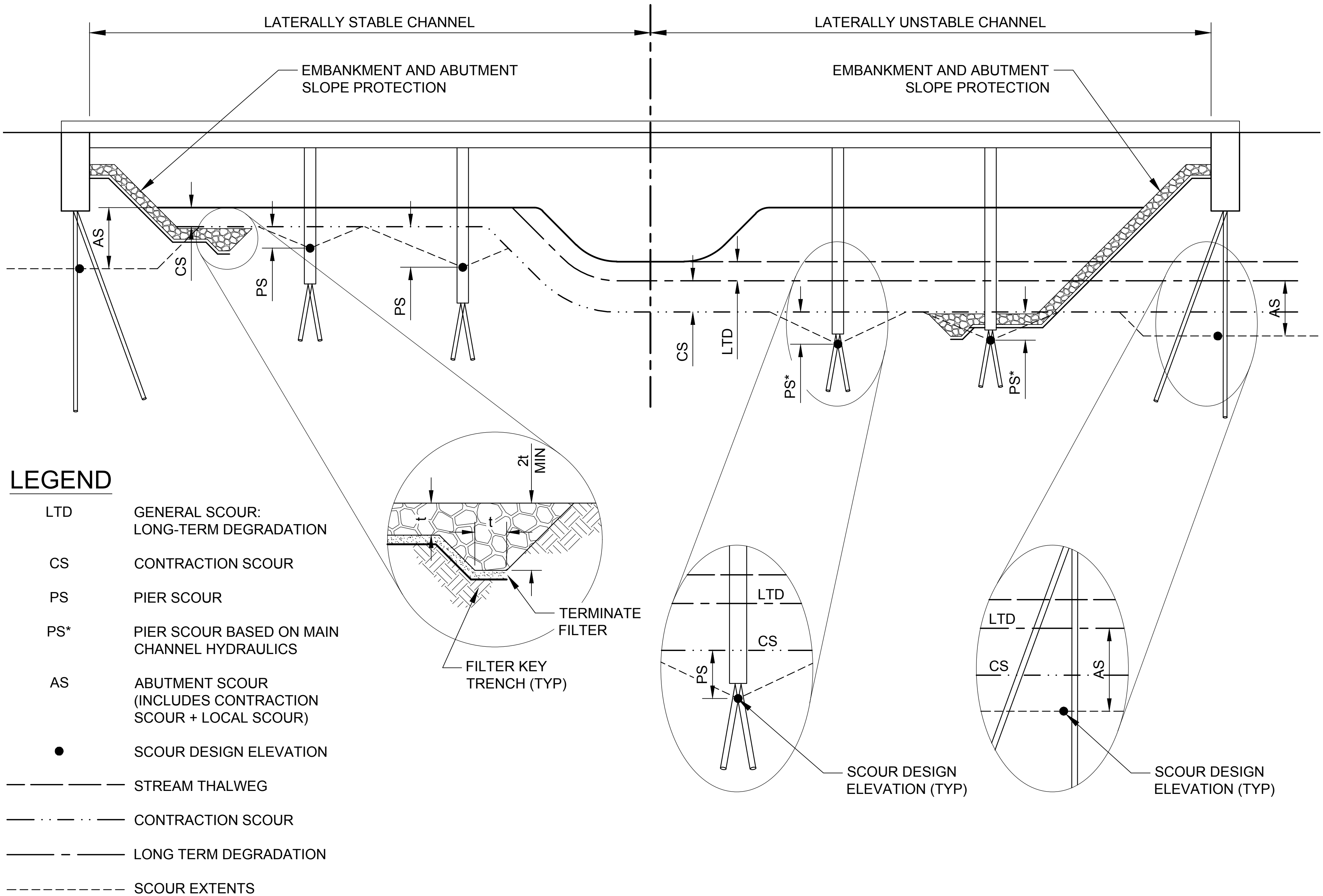

Figure 7.1 Total scour cross-section plot example.

7.4 Scour Analysis at Closed-Bottom Culverts

Closed-bottom culverts (pipes, pipe-arches and four-sided box culverts) are not subject to scour in the barrel, but the potential exists for scour problems at the outlet and inlet. The culvert barrel, headwalls, wingwalls and other appurtenances should be designed to remain functional and structurally stable per Section 2. General scour in the form of long-term degradation could eventually cause the downstream channel bed to lower, causing a drop from the culvert outlet to the streambed. With or without general scour, a local scour hole could form downstream of the culvert due to concentrated velocities exiting the culvert.

Outlet scour should be computed using the methods outlined in HEC-14. For closed-bottom culverts, the design scour elevations should include any calculated general scour in addition to the computed outlet scour hole depth and should be referenced to the pre-scour downstream channel bed elevation. In the case of buried-bottom culverts, which are set below the alluvial channel invert by design, scour should be evaluated starting at the culvert bottom elevation.

7.5 Stream Stability and Scour Countermeasures at Bridges and Bridge-Sized Culverts

Scour countermeasures shall be designed to protect the structure and abutment fill from scour for Scour Design Flood conditions (See Section 2.2.) The most common armoring systems used in Idaho incorporate angular rock riprap. The armoring should be underlain by a suitable granular or geosynthetic filter fabric system. See HEC-23 for recommended filter design techniques, abutment rock riprap design procedures, and Tech Brief HIF-19-007, Figures 6-10 for typical placement sections. If riprap is used, the median stone size (D_{50}) shall not be less than 12 inches.

Note that a suitable filter layer and adequate embedment are required elements of any scour countermeasure design. ITD standard practice is to use a geosynthetic filter fabric (geotextile) in lieu of a granular filter system. ITD's Standard Construction Specifications lists two Riprap/Erosion Control Geotextile (Section 718.06) as options. It is recommended to specify the high survivability geotextile in most applications. Recommendations for geotextile to be provided are given in the Geotechnical Engineering Report (see Figure 1.1 – bridge design workflow). Part of the hydraulic design process is to verify that the recommended geotextile meets the design guidance provided in HEC-23.

The HEC-23 design guide for abutment rock riprap design procedures and Tech Brief Figures 6-10 for typical placement sections provide some details on the placement of geotextile in relation to the riprap. ITD's standard practice includes the following: For riprap against abutments, piers and wingwalls, provide geotextile up against the substructure one half the riprap depth. At the toe of riprap or along the horizontal riprap apron, terminate the geotextile at the bottom of the riprap. Do not extend the geotextile up along the face of the riprap nor key in the geotextile down vertically below the riprap by burying. Along the leading (upstream), lateral (channelward) and trailing (downstream) edges of riprap placements, provide a termination (key-in) trench. See Figure 7.1 for a typical section. ITD applies HEC-23 methods, supplemented with Tech Brief HIF 19-007, to inform proper roadway embankment and abutment protection dimensions and elevations for bridge design.

There are a wide variety of alternative armor technologies. If suitable size, durability, and gradation of angular rock riprap cannot be economically sourced or if construction of a rock riprap placement is not practicable given site conditions, then the designer should consider alternative treatments, including

Articulating Concrete Blocks (ACBs), Matrix Riprap (partially grouted riprap), or gabions (in sand-bed channels only).

In wide floodplains with laterally unstable channels and/or significant abutment scour potential, guide banks designed in accordance with HEC-23 guidance should be considered where practicable. Guide banks reduce scour potential at the bridge, reduce the probability of scour damage to the abutment foundation, and can limit lateral instability potential in the immediate vicinity of the bridge crossing.

FHWA does not currently recommend vegetative treatments for protection of critical infrastructure elements, such as bridge or culvert crossings. However, ITD recognizes that it is good practice to incorporate biotechnical treatments in consultation with resource agency input as an adjunct to HEC-23 compliant scour countermeasures, and all countermeasures should be set back as practicable, buried/covered with suitable growth media or alluvium as appropriate, and planted per FHWA Tech Brief 19-007 recommendations. The NRCS publication *Guidance for Stream Restoration* (NRCS, 2014) provides information on the use of biotechnical bank stabilization techniques.

Designers shall not adjust calculated total scour depths and elevations to account for proposed scour countermeasures. Roadway embankment and abutment fill slope protection shall be designed using HEC-23 abutment scour countermeasure design guideline procedures as amended by HIF-19-007 (Tech Brief) updates.

7.5.1 Bridges, Bottomless Arches and Three-Sided Stiff-Leg Structures

ITD does not permit designs of bridges, bridge-sized bottomless arches, or bridge-sized stiff-leg structures that are dependent on hydraulic countermeasures for structural stability for either the scour design flood (service limit state) or the scour check flood (extreme limit state) conditions (see Section 2.2). ITD also requires protection at the abutments of these structures to protect roadway embankment fills from calculated scour design flood conditions. All hydraulic countermeasure designs shall be developed and constructed in accordance with HEC-23 standards as supplemented by Tech Brief 19-007 considerations. ITD adopts Figures 6-10 of this Tech Brief as general updates to HEC-23 typical sections for abutment scour countermeasure design.

7.5.2 Bridge-Sized Closed-Bottom Culverts

Bridge-sized closed-bottom culverts shall be designed with a downstream cutoff wall designed to retain soil to the downstream channel thalweg (invert) elevation minus long-term degradation potential, or three feet below the invert elevation, whichever is deeper. The cutoff wall may be designed to retain soil for the scour design flood conditions. However, if the downstream cutoff wall is not designed to retain soil for the scour design flood conditions, then an outlet scour countermeasure designed to remain in place and be functional for the scour design flow conditions shall be provided to prevent outlet scour from undermining the culvert outlet and cutoff wall. HEC-14 provides design guidance for culvert scour depth calculation, design of energy dissipators and scour countermeasures for culvert outlets. For relatively low outlet velocities, a simple riprap apron may be appropriate.

ITD standard practice is to provide countermeasures at the upstream end of the culvert mirroring what is designed and provided at the downstream end. This includes cutoff walls and simple riprap aprons, as appropriate.

7.5.3 Approach Roadway Overtopping

If ITD approves a variance to allow an approach roadway embankment design that is designed to provide hydraulic relief to the crossing (i.e. overtop) for events less than the hydraulic design (capacity) event, then overtopping embankment protection on the embankment side slopes shall be designed and installed to protect the roadway embankment from erosion damage. This countermeasure shall be designed to withstand the maximum calculated overtopping flows produced by the scour design flood or the hydraulic design (capacity) flood, whichever is greater.

If suitable vegetation (dense grasses or dense riparian vegetation such as mixed whip willow species) can be established on the embankment slope, then a soil-filled and planted Turf Reinforcement Mat (TRM) should be provided for the lateral extents of overtopping. The embankment slope and TRM should be designed to remain stable for the calculated maximum shear stress in the overtopping region, keyed under the roadway edge of pavement, and keyed in at the toe of slope to one foot below the calculated apron scour depth. If suitable vegetation cannot practicably be established on the embankment side slopes due to site conditions, then an alternate overtopping protection and filter system shall be designed using HEC-23 methods.

8 REFERENCES

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APPENDIX A

ITD FORM 211

LINK

Your Safety • Your Mobility
Your Economic Opportunity

Bridge Structures Hydraulics Survey

ITD 0211 (Rev. 05-21)
itd.idaho.gov

General Data

Project Key Number	Bridge Key Number (proposed)	Project Number	Date
Project Title		Local Structure Name	
Station/Milepost		Latitude/Longitude	Vertical Datum (all elevations in this form)
Location		County	
Roadway Identification		Crossing Type ☐ Creek ☐ River ☐ Canal ☐ Other	
Feature Crossed		A Tributary Of	

Hydrologic Data

Hydrology Methods Used to Determine Design Flows ☐ USGS Stream Gage Statistics (Bulletin 17C) ☐ Flood Insurance Study ☐ USGS Regression Equations/StreamStats ☐ Canal Flows Data ☐ Other (Describe)
Description of Watershed or Canal
Drainage Basin Area ☐ mi^2 ☐ acres

Stream/Crossing Data

☐ Natural Stream ☐ Other	List Months in the Dry, If Any	Streambed Material Size, D_{50} inches
Character of Streambed ☐ Stable ☐ Aggrading ☐ Degrading ☐ Headcutting		
Describe Streambed		
Flow Controlled ☐ Upstream ☐ Downstream	If Controlled, Explain	
Stream Carries an Appreciable Amount of Ice ☐ No ☐ Yes	Ice Thickness in	Stream Carries an Appreciable Amount of Debris ☐ No ☐ Yes (describe)
Located in Regulatory Floodplain (or adjacent to) ☐ Yes ☐ No	NFIP Community Name	
Flood Insurance Rate Map (FIRM) Panel Number & Date	Regulatory Floodway ☐ Yes ☐ No	If Yes, Floodway Map Panel Number

Existing Structure

☐ Bridge ☐ Culvert ☐ None (Describe the Bridge, Culvert or existing conditions)		
General Condition		Year Constructed
Describe Any Existing Adverse Conditions		
Structure Dimensions, Diameter, Etc.	Type of Bridge Piers ☐ Spread Footings ☐ Piles	Number of Piers

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Bridge Design – Signed & Sealed to Project File

No additional copies required

Bridge Structures Hydraulics Survey

Total Structure Clear Span Normal to Channel ft	Bridge Clearance Above Q_{Design} WSE ft	Velocity(max) Through Structure at Q_{Design} fps
Streambed Elevation at Upstream Face ft	Streambed Slope through Structure ft/ft	
Existing Structure Carried Flow Adequately (per current standards) ☐ Yes ☐ No	If No, Explain	

Design Flows & Proposed Condition Values

Flood	Discharge	Water Surface Elevation	Velocity***
Ordinary High Water [Q]*	cfs	ft	fps
Design [Q]**	cfs	ft	fps
Base [Q_{100}] (Scour Design)	cfs	ft	fps
Scour Check [Q]	cfs	ft	fps
Canal Design Flow	cfs	ft	fps
Canal Max Flow (scour check)	cfs	ft	fps

* OHW as determined by resource agency request for environmental considerations (generally $Q_{1.5}$ or $Q_{2.0}$)
** Use Q_{50} for bridges and bridge-sized culverts 10' or greater.
*** Maximum velocity through the structure.

Proposed Bridge (10' or greater)

Structure Type	Number and Length of Spans	
Skew Angle of Bridge °	Streambed Elevation at Upstream Face ft	Streambed Slope through Structure ft/ft
Total Bridge Clear Span Normal to Channel ft	Distance from Upstream Structure Face to Point Where Freeboard is Measured ft	Bottom of Superstructure Elevation (low chord) ft
Flow Angle to Pier(s) °	Q_{50} Freeboard ft	Q_{100} Freeboard ft

Scour Calculations

Maximum Scour at Abutment				
Flood	General Scour	Abutment Scour (NCHRP 24-20 method includes contraction scour in abutment scour calcs)	Total Scour	Scour Elevation
Scour Design [Q_{100}]	ft	ft	ft	ft
Scour Check [Lessor of: Q_{500} or $Q_{overtopping}$]	ft	ft	ft	ft
Canal Max Flow	ft	ft	ft	ft

Maximum Scour at Pier					
Flood	General Scour	Pier Scour	Contraction Scour	Total Scour	Scour Elevation
Scour Design [Q_{100}]	ft	ft	ft	ft	ft
Scour Check [Lessor of: Q_{500} or $Q_{overtopping}$]	ft	ft	ft	ft	ft
Canal Max Flow	ft	ft	ft	ft	ft

Scour Countermeasures

Countermeasure Design Flow [minimum Q_{100}] cfs	Calculated Riprap Size, D_{50} ft	Design Riprap Size, D_{50} ft
Countermeasure Filter Type		Riprap Thickness ft

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Bridge Structures Hydraulics Survey

In addition to the above information, submit and check each of the following that apply.

- ☐ Hydraulic Report* (see Bridge Hydraulics Manual for format) including the following as applicable:
- ☐ Hydraulic Risk Assessment Memorandum
- ☐ A vicinity map, such as a county map, with the location of the structure clearly indicated.
- ☐ A contour map of the structure site showing 1 foot contours.
- ☐ A roadway centerline profile to the same scale as the contour map.
- ☐ A streambed profile of the entire hydraulic reach analyzed for the structure including the location of the structure.
- ☐ A typical proposed roadway section at the structure.
- ☐ Elevation view of the upstream face of proposed structure with dimensions.
- ☐ Documentation supporting hydrology methods used to develop design flows.
- ☐ Photographs of the existing structure and channel upstream and downstream from the site.
- ☐ Channel change or canal lining details (typical section, plan and profile, and limits).
- ☐ Computations for scour based on Scour Design Flood flow and Scour Check Flood flow or Maximum Canal flow.
- ☐ Letter of approval from canal company or irrigation district.
- ☐ Floodplain data including copy of map panel at the structure location (FIRMette).
- ☐ Floodplain Development Permit from the Local Floodplain Administrator if the structure is located in the 100-year floodplain.
- ☐ Preliminary Bridge Situation & Layout Plan Sheet
- ☐ Riprap details (typical section, dimensions, limits, size, toe embedment, etc.) for proposed locations.

A hydraulic report should accompany bridges and bridge-sized culverts with hydraulic openings greater than 10’.

Prepared By	Title	Engineer’s Signature and Seal

Accepted by LHTAC Administrator, Bridge Engineer, or District Engineer	Signature/Date

Distribution: Consultant – Signed & Sealed to Bridge Design/LHTAC Project Manager
Bridge Design – Signed & Sealed to Project File

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APPENDIX B

OUTLINE FOR
HYDRAULICS RISK ASSESSMENT
MEMORANDUM

HYDRAULICS RISK ASSESSMENT MEMORANDUM (OUTLINE)

1 PROJECT BACKGROUND

- Location: Route; stream crossed; county (*Figure: Location map*)
- Existing crossing (if there is one): Hydraulic opening of structure; number of spans; approach roadway profile (Figures: Field photographs of existing structure and crossing)
- Proposed Work: New crossing or replacement; will road be realigned; approximate hydraulic opening of proposed structure; number of spans (*Figure or attachment: plan view drawing of proposed project, showing existing alignment and structure*)

2 WATERSHED

- Name of river or stream crossing
- Watershed characteristics: Size; annual precipitation; terrain and slope; ground cover and land use (*Figure: Drainage basin map showing basin delineation, topography, highways, tributaries and stream gage locations*)
- Note locations of stream gages nearby on the same stream and their periods of peak annual flow records
- Discuss expected method to determine hydrology and design flows (best method based on watershed, project requirements and available information and resources)
- Known ecological sensitivities: endangered/threatened species; fish passage concerns in the river; aquatic or riparian habitat

3 PAST HYDRAULIC OR HYDROLOGY STUDIES

- Past ITD studies
- Other state or local agencies
- Federal agencies

4 FEDERAL EMERGENCY MANAGEMENT AGENCY (FEMA) NATIONAL FLOOD INSURANCE PROGRAM (NFIP) STATUS

If the project is not located in a FEMA regulated floodplain, this section will simply state that in a sentence. If it is located in a FEMA floodplain, the information below will be provided in this section.

- Flood zone designation: Zone AE with a regulatory floodway; Zone AE with no regulatory floodway; Zone A (approximate)
- Floodplain administration jurisdiction: Name of city or county and NFIP community number
- Other jurisdictions that could be affected

- FIRM panel(s) depicting the project location and possible impact areas (*Figure or Attachment: Excerpt of FIRM panel for project area*)

4.1 Floodplain Regulatory Constraints Arising from NFIP Status and Jurisdiction

- Floodplain Development Permit required, potentially from more than one jurisdiction
- No-Rise Certification: per jurisdiction's requirements if the project involves encroachment in a regulatory floodway
- Regulations, codes or ordinances of the jurisdiction that may be more stringent than the NFIP

4.2 FEMA Technical Data

- Date of effective study
- Hydraulic model used for effective study
- Discharge rates listed in the effective study (*Table: discharge rates for every recurrence interval listed in the Flood Insurance Study*)
- Base flood elevation just upstream of proposed bridge location (*Figure or Attachment: Flood profiles from Flood Insurance Study*)

5 OTHER PERMIT REQUIREMENTS AND APPROVALS

- Joint Application for USACE 404/Idaho Stream Alteration
- Coast Guard Bridge Permit
- Irrigation District License Agreement

6 FLOOD RISK ASSESSMENT

Note: If a hydraulic analysis of existing crossing is available (e.g. from a FEMA study or other earlier study) use that study to develop content for this subsection. Otherwise assess qualitatively

6.1 Risk to the Crossing Facility

- What recurrence interval flood would be expected to overtop the road or bridge deck of the existing crossing?
- Can the proposed design reduce overtopping frequency, and would it be beneficial to motorist safety to do so?
- Does the existing crossing provide sufficient freeboard per Table 2.1 of the Bridge Hydraulics Manual (BHM)?
- Can the proposed design increase the freeboard, and would it be beneficial to do so?
- Is it possible or likely that a variance to the standards and criteria in BHM Section 2 will be necessary? If so explain (the most common reason for this is limitations to how high

the approach road profile can be raised because of nearby intersections, railroad crossings, private accesses, etc.)

6.2 Risk of Offsite Impacts

- Is this a new crossing on a new alignment? If so the risk of offsite flood impacts (e.g. increasing the flood profile upstream) is likely high
- Is this a replacement of a crossing with extensive overtopping of the roadway? If so the risk of offsite impacts is high unless:
  - The road profile is kept low in the proposed design
  - The bridge opening capacity is increased (usually by a longer bridge) to offset loss of road overtopping
- Is this a replacement of a crossing with minimal roadway overtopping and with existing deficiencies in the hydraulic opening (e.g. narrow opening, pressure flow, badly skewed piers) that can readily be corrected in the proposed design? If so, the risk of offsite impacts is low
- Is a no-rise design required?
  - If the project encroaches on a NFIP regulatory floodway: Yes
  - If adjacent structures would be impacted by an increase in the flood profile: Yes
  - If no-rise dictated by more stringent requirements of the local jurisdiction: Possibly
- What mitigation strategies can be incorporated in the proposed design to provide no rise or minimize the offsite impact?

6.3 Hydraulic Modeling Approach

- One-dimensional (1D) or 2-dimensional (2D) modeling recommended for project (default is 2D)
- Are both 1D and 2D required (e.g. 2D for analysis and design, 1D for FEMA compliance and no-rise)
- If 1D recommended, give justification

7 GEOMORPHIC ASSESSMENT

7.1 Channel Characteristics

- Make use of Figure 2.6 in FHWA HEC-20 4th Edition (*Figure: annotated version of HEC-20 Figure 2.6 with notations specific to site*)
- Channel size (small, medium or wide)
- Flow habit (e.g. ephemeral, perennial, canal)
- Bed material (from streambed material sampling and testing)
- Valley setting
- Floodplains
- Apparent incision
- Channel boundaries
- Tree cover on banks
- Sinuosity
- Braided or anabranched?

- Width variability and development of bars

7.2 Lateral Stability

- Examining historic aerial imagery (<u>Figure: historic channel alignment overlay</u>)
- Examining bridge inspection reports
- Findings of site reconnaissance (observed vertical or cut banks, undercut tree roots, recent bar development, etc.) (*<u>Figures: field photographs</u>*)
- Summary of lateral channel instability risks at bridge site: Low, moderate or high potential for channel migration or bank erosion

7.3 Vertical Stability

- Any known degradational or aggradational problems in this reach of the stream (e.g. problems with existing bridge foundations or underground utility crossings becoming exposed; frequent channel dredging needed)?
- Examining channel cross section plots from time series of bridge inspection reports (*<u>Figure: Plots of channel cross sections from bridge inspection reports covering as long a duration as possible, common vertical reference point</u>*)
- Examining nearby stream gages for rating curve shifts
- Findings of site reconnaissance (observed terraces in floodplain, apparent incision, exposed bridge piles, exposed utility crossings, culvert acting as grade control against downstream degradation) (*<u>Figures: field photographs</u>*)
- Summary of vertical instability risks: Low, moderate, or high potential of long-term degradation or aggradation

8 QUALITATIVE SCOUR RISK ASSESSMENT

8.1 Scour Conditions at Existing Structure

- Item 113 code from inspection report
- Visible scour issues at existing structure (*<u>Figures: field photographs</u>*)
- Presence and condition of existing scour countermeasures (*<u>Figures: field photographs</u>*)

8.2 Potential for Scour Under Proposed Design

- Contraction scour potential: Is the floodplain wide and the proposed hydraulic opening width comparatively narrow?
- Local scour at piers: High velocity and depth expected in the bridge opening?
- Local scour at abutments: Significant redirection of flow by road embankments? Abutments close to or within main channel?
- Local scour at culvert outlet: Is outlet velocity expected to be significantly higher than natural conditions?
- Summary of assessed potential under proposed design: Low, moderate or high potential for contraction scour and/or local scour
- Include streambed measurements from bridge inspection reports if available

8.3 Design Features to Minimize or Mitigate Scour Potential

- Anticipated foundation type
- Anticipated scour countermeasures at abutments
- Anticipated culvert outlet treatment or energy dissipator
- Additional measures to consider if foundation cost or scour countermeasures are excessive:
    - Consider longer bridge for wider hydraulic opening and thus lower velocities
    - Align piers with flood flow direction and consider designs that are not sensitive to attack angle
    - Align abutments with flow direction
    - Consider larger culvert barrel or flatter slope to reduce culvert outlet velocity

9 VARIANCES REQUIRED (if any)

If any variance from the standards and requirements of the ITD Bridge Hydraulics Manual is required, use this section to explain.

- What standard or requirement is the subject of the request for a variance?
- Justification for the variance
    - Why is it needed?
    - What adverse conditions and/or site constraints are driving the need?
    - What would it take to meet all criteria if a variance isn't granted (significantly greater cost, failure to meet permit requirements, unacceptable schedule for CLOMR approval, etc.)?
- What is the proposed variance?
- What risks are associated with the proposed variance and how will the design mitigate or minimize those risks?

Provide in this section of the HRAM a request and a signature line for the variance to be accepted by the Bridge Engineer.

APPENDIX C

FIELD RECONNAISSANCE FORM
(EXAMPLE DOCUMENT)

Field Reconnaissance Check List		
Bridge Structure #:		
Location:		
Highway: Milepost:		T:
County:		R:
Watercourse:		SEC:
Date:	Time: (Begin/End)	
Field Crew:		
NOTE: Photograph all structure elements and scour features identified		
1. General Site Characteristics (Road, Bridge, Watercourse, Reach)		
Sketch Site Reach (NTS): (Include road, bridge, guide banks, stream reach, floodplain, flow arrows, approximate angle of bridges skew to flow, north arrow)		

Sketch Typical Floodplain and Channel Cross Section (NTS): (include banks, typical vegetation, WSEL)
Sketch Profile View of Bridge: (include bridge structure, size and type of piers, abutments, high water marks, WSEL)

Digital Photography

☐ Bridge Identification

☐ Upstream face of bridge

☐ Downstream face of bridge

☐ Upstream channel

☐ Downstream channel

☐ Left and right abutments, upstream and downstream

☐ All piers (use enough photographs that each pier is clearly visible)

☐ Indicators of vertical channel instability (depositional areas, head cutting, incision terraces, exposed foundations of structures or bridge piers, etc.)

☐ Indicators of lateral channel instability (recent bank retreat, cut banks, fresh point bars, channel braids, etc.)

☐ Other features as necessary (bridge structural elements, scour features)

☐ High water marks

☐ Guidebanks, floodwalls, etc.

☐ Erosion and scour evidence (scour holes, etc.)

☐ Photographs of surveyed cross sections

☐ Photographs of debris blockages

☐ Photographs of hydraulic control features (crossings, diversions, critical depth sections, levees or other longitudinal floodplain encroachments, major obstructions, scour-resistant rock or other vertical grade controls, downstream water bodies, etc.)

2. Stream Hydraulic Characteristics
☐ General Description of Hydraulic Characteristics (uniform vs. varied flow, qualitative descriptions of velocity, etc.)
☐ Channel Slope (get from Survey, Section 5) Indicate how slope measured: (channel bottom or water surface)
☐ Channel and Floodplain Roughness (Manning's n values) LB: MC: RB:
☐ Visible High-Water Marks (describe)
☐ Hydraulic Controls (upstream, downstream, lateral)
☐ Estimated Velocity
☐ Likelihood of Pressure flow and/or roadway and bridge deck overtopping
Miscellaneous Notes:

3. Bridge Information

Abutments:

Left: Right:

☐ Configuration (type, side slope, location)

☐ Condition of abutments and slope protection (if intact riprap is in place, include the nominal size of the riprap)

☐ Signs of abutment scour

Piers:

☐ Type, dimensions and shape of piers

☐ Skew angle(s) (0° = flow aligned with pier axis) (estimate for low and high flows)

☐ Potential for debris accumulation (low, moderate, or high) (justify and measure debris width, if applicable)

☐ Signs of pier scour (visible scour holes, bed deposits downstream)

Contraction Scour:

☐ Qualitative assessment of potential for contraction scour

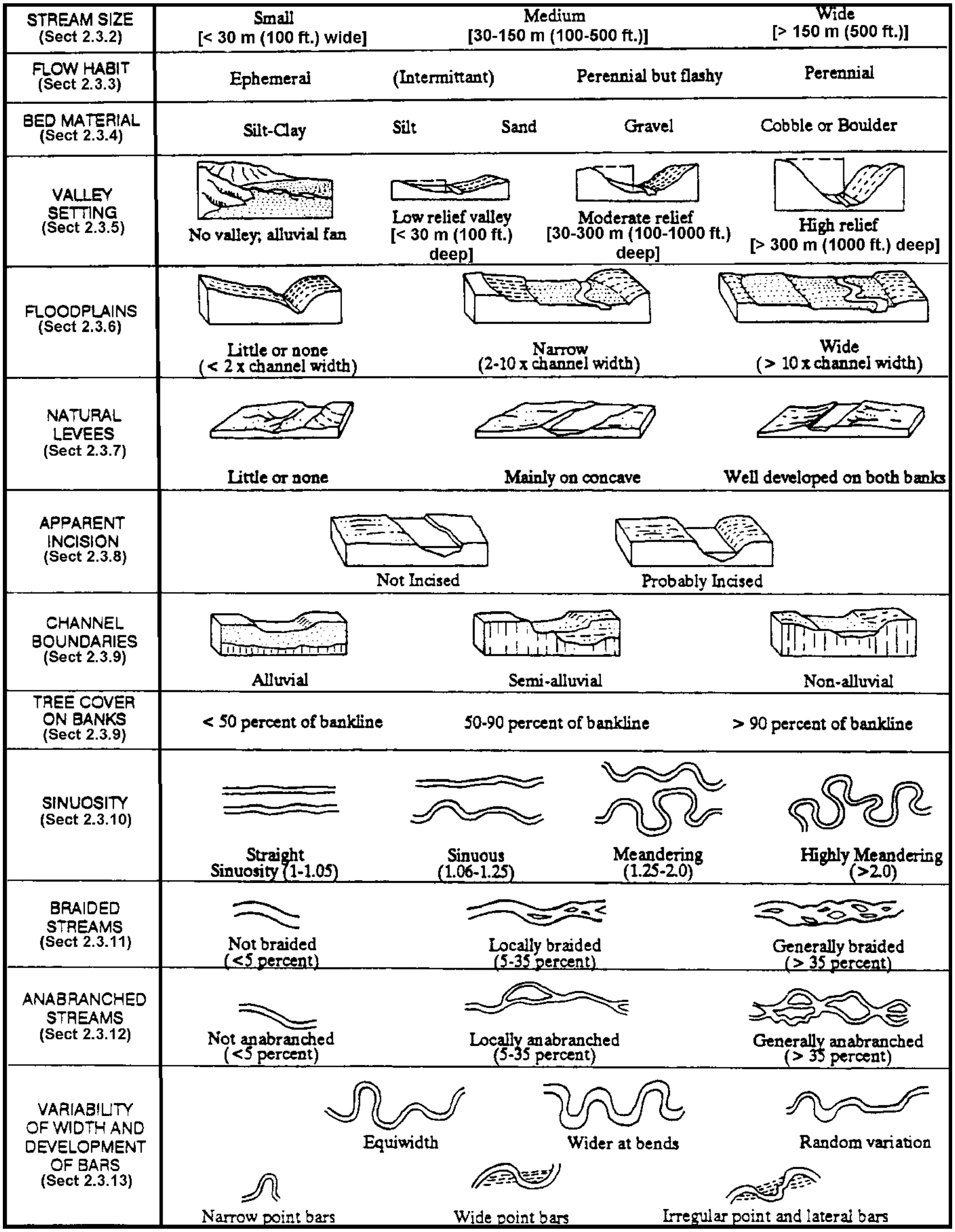
☐ Signs of past contraction scour (visible scour holes, bed deposits downstream)

☐ Features that could aggravate scour problems (misaligned piers, obstructive abutments)

Miscellaneous Notes (include description of guidebanks, overtopping potential, flow relief, etc.):

4. Geomorphic Information

☐ Geomorphic factors that affect stream stability (Figure 2.6 from HEC-20, 3rd Edition)
Circle appropriate factors



STREAM SIZE (Sect 2.3.2)	Small [< 30 m (100 ft.) wide]	Medium [30-150 m (100-500 ft.)]		Wide [> 150 m (500 ft.)]
FLOW HABIT (Sect 2.3.3)	Ephemeral	(Intermittant)	Perennial but flashy	Perennial
BED MATERIAL (Sect 2.3.4)	Silt-Clay	Silt	Sand	Gravel	Cobble or Boulder
VALLEY SETTING (Sect 2.3.5)	No valley; alluvial fan	Low relief valley [< 30 m (100 ft.) deep]	Moderate relief [30-300 m (100-1000 ft.) deep]	High relief [> 300 m (1000 ft.) deep]
FLOODPLAINS (Sect 2.3.6)	Little or none (< 2 x channel width)	Narrow (2-10 x channel width)		Wide (> 10 x channel width)
NATURAL LEVEES (Sect 2.3.7)	Little or none	Mainly on concave		Well developed on both banks
APPARENT INCISION (Sect 2.3.8)	Not Incised	Probably Incised		
CHANNEL BOUNDARIES (Sect 2.3.9)	Alluvial	Semi-alluvial		Non-alluvial
TREE COVER ON BANKS (Sect 2.3.9)	< 50 percent of bankline	50-90 percent of bankline		> 90 percent of bankline
SINUOSITY (Sect 2.3.10)	Straight Sinuosity (1-1.05)	Sinuous (1.06-1.25)	Meandering (1.25-2.0)	Highly Meandering (>2.0)
BRAIDED STREAMS (Sect 2.3.11)	Not braided (<5 percent)	Locally braided (5-35 percent)		Generally braided (> 35 percent)
ANABRANCHED STREAMS (Sect 2.3.12)	Not anabranched (<5 percent)	Locally anabranched (5-35 percent)		Generally anabranched (> 35 percent)
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS (Sect 2.3.13)	Equiwidth	Wider at bends		Random variation
	Narrow point bars	Wide point bars		Irregular point and lateral bars

STREAM RECONNAISSANCE RECORD SHEETS

(Adapted from HEC-20, 3rd edition, Appendix C)

SECTION 2 - REGION AND VALLEY DESCRIPTION

PART 3: FLOOD PLAIN (VALLEY FLOOR)

Valley Floor Type
None ☐
Indefinite ☐
Fragmentary ☐
Continuous ☐

Valley Floor Data
None ☐
< 1 river width ☐
1 - 5 river widths ☐
5-10 river widths ☐
>10 river widths ☐

Flow Resistance*
Left Overbank Manning n value ___
Right Overbank Manning n value ___ (* note: n value for channel is recorded in Part 6)

Surface Geology
Bed rock ☐
Glacial Moraine ☐
Glacio/Fluvial ☐
Fluvial: Alluvium ☐
Fluvial: Backswamp ☐
Lake Deposits ☐
Wind Blown (Loess) ☐

Land Use
Natural ☐
Managed ☐
Cultivated ☐
Urban ☐
Suburban ☐
Industrial ☐

Vegetation
None ☐
Unimproved Grass ☐
Improved Pasture ☐
Orchards ☐
Arable Crops ☐
Shrubs ☐
Deciduous Forest ☐
Coniferous Forest ☐
Mixed Forest ☐

Riparian Buffer Strip
None ☐
Indefinite ☐
Fragmentary ☐
Continuous ☐

Strip Width
None ☐
< 1 river width ☐
1 - 5 river widths ☐
> 5 river widths ☐

PART 4: VERTICAL RELATION OF CHANNEL TO VALLEY

Terraces
None ☐
Indefinate ☐
Fragmentary ☐
Continuous ☐
Number of Terraces ___

Canyons
Channel in Canyon? ☐
Depth of Canyon ___

Overbank Deposits
None ☐
Silt ☐
Fine sand ☐
Medium sand ☐
Coarse sand ☐
Gravel ☐
Boulders ☐

Levees
None ☐
Natural ☐
Constructed ☐

Instability Status
Stable ☐
Degrading ☐
Aggrading ☐

Levee Data
Height (ft) ___
Side Slope (o) ___

Levee Condition
None ☐
Intact ☐
Local Failures ☐
Frequent failures ☐

Levee Description
None ☐
Indefinite ☐
Fragmentary ☐
Continuous ☐
Left Bank ☐
Right Bank ☐
Both Banks ☐

Trash Lines
Absent ☐
Present ☐
Height above flood plain (ft) ___

PART 5: LATERAL RELATION OF CHANNEL TO VALLEY

Planform
Straight ☐
Sinuous ☐
Irregular ☐
Regular meanders ☐
Irregular meanders ☐
Tortuous meanders ☐
Braided ☐
Anastomosed ☐

Lateral Activity
None ☐
Meander progression ☐
Increasing amplitude ☐
Progression+cut-offs ☐
Irregular erosion ☐
Avulsion ☐
Braiding ☐

Floodplain Features
None ☐
Meander scars ☐
Scroll bars+sloughs ☐
Oxbow lakes ☐
Irregular terrain ☐
Abandoned channel ☐
Braided Deposits ☐

Location in Valley
Left ☐
Middle ☐
Right ☐

Notes and Comments:

Field Reconnaissance/Bridge Structure # ______ Date: ______ Initials: ______

SECTION 3 - CHANNEL DESCRIPTION

PART 6: CHANNEL DESCRIPTION

Dimensions	Flow Type	Bed Controls	Bed Control Types	Width Controls	Width Control Types
Av. top bank width (ft) ___	None ☐	None ☐	None ☐	None ☐	None ☐
Av. channel depth (ft) ___	Uniform/Tranquil ☐	Occasional ☐	Solid Bedrock ☐	Occasional ☐	Bedrock ☐
Av. water width (ft) ___	Uniform/Rapid ☐	Frequent ☐	Weathered Bedrock ☐	Frequent ☐	Boulders ☐
Av. water depth (ft) ___	Pool+Riffle ☐	Confined ☐	Boulders ☐	Confined ☐	Gravel armor ☐
Reach slope ___	Steep + Tumbling ☐	Number of controls ☐	Gravel armor ☐	Number of controls ☐	Revetments ☐
Mean velocity (ft/s) ___	Steep + Step/pool ☐		Cohesive Materials ☐		Cohesive Materials ☐
			Bridge protection ☐		Bridge abutments ☐
Manning's n value ___	(Note: Flow type on day of observation)		Grade control structures ☐		Dykes or groines ☐

PART 7: BED SEDIMENT DESCRIPTION

Bed Material	Bed Armour	Surface Size Data	Bed Forms (Sand)	Bar Types	Bar Surface data
Clay ☐	None ☐	D50 (mm) ___	Flat bed (None) ☐	None ☐	D50 (mm) ___
Silt ☐	Static-armour ☐	D95 (mm) ___	Ripples ☐	Pools and riffles ☐	
Sand ☐	Mobile-armour ☐		Dunes ☐	Alternate bars ☐	
Sand and gravel ☐			Bed form height (ft) ___	Point bars ☐	
gravel and cobbles ☐	**Sediment Depth**	**Substrate Size Data**	**Island or Bars**	Mid-channel bars ☐	**Bar Substrate data**
cobbles + boulders ☐	Depth of loose	D50 (mm) ___	None ☐	Diagonal bars ☐	D50 (mm) ___
boulders + bedrock ☐	Sediment (ft) ___		Occasional ☐	Junction bars ☐	
Bed rock ☐			Frequent ☐	Sand waves + dunes ☐	

Channel Sketch Map (General planform, tributaries, controls, etc.)

Field Reconnaissance/Bridge Structure # ____________ Date: ____________ Initials: ____________

SECTION 4 - LEFT BANK SURVEY

PART 8: LEFT BANK CHARACTERISTICS

Type
Noncohesive ☐
Cohesive ☐
Composite ☐
Layered ☐
Even Layers ☐
Thick+thin layers ☐
Number of layers ___

Protection Status
Unprotected ☐
Hard points ☐
Toe protection ☐
Revetments ☐
Dyke Fields ☐

Bank Materials
Silt/clay ☐
Sand/silt/clay ☐
Sand/silt ☐
Sand ☐
Sand/gravel ☐
Gravel ☐
Gravel/cobbles ☐
Cobbles ☐
Cobbles/boulders ☐
Boulders/bedrock ☐

Layer Thickness
Material 1 (ft) ___
Material 2 (ft) ___
Material 3 (ft) ___
Material 4 (ft) ___

Ave. Bank Height
Average height (ft) ___

Ave. Bank Slope
angle (degrees) ___

Tension Cracks
None ☐
Occasional ☐
Frequent ☐

Crack Depth
Proportion of bank height ___

Distribution and Description of Bank Materials in Bank Profile

Material Type 1	Material Type 2	Material Type 3	Material Type 4
Toe ☐	Toe ☐	Toe ☐	Toe ☐
Mid-Bank ☐	Mid-Bank ☐	Mid-Bank ☐	Mid-Bank ☐
Upper Bank ☐	Upper Bank ☐	Upper Bank ☐	Upper Bank ☐
Whole Bank ☐	Whole Bank ☐	Whole Bank ☐	Whole Bank ☐
D50 (mm) ___	D50 (mm) ___	D50 (mm) ___	D50 (mm) ___

PART 9: LEFT BANK-FACE VEGETATION

Vegetation
None/fallow ☐
Artificially cleared ☐
Grass and flora ☐
Reeds and sedges ☐
Shrubs ☐
Saplings ☐
Trees ☐

Orientation
Angle of leaning (o) ___

Tree Types
None ☐
Deciduous ☐
Coniferous ☐
Mixed ☐

Tree species
(if known) ____________

Density + Spacing
None ☐
Sparse/clumps ☐
dense/clumps ☐
Sparce/continuous ☐
Dense/continuous ☐

Roots
Normal ☐
Exposed ☐
Adventitious ☐

Location
Whole bank ☐
Upper bank ☐
Mid-bank ☐
Lower bank ☐

Diversity
Mono-stand ☐
Mixed stand ☐
Climax-vegetation ☐

Health
Healthy ☐
Fair ☐
Poor ☐
Dead ☐

Age
Imature ☐
Mature ☐
Old ☐

Height
Short ☐
Medium ☐
Tall ☐
Height (m) ___

Lateral Extent
Wide belt ☐
Narrow belt ☐
Single row ☐

PART 10: LEFT BANK EROSION

Erosion Location
General ☐
Outside Meander ☐
Inside Meander ☐
Opposite a bar ☐
Behind a bar ☐
Opposite a structure ☐
Adjacent to structure ☐
Dstream of structure ☐
Ustream of structure ☐
Other (write in) ☐

Present Status
Intact ☐
Eroding:dormant ☐
Eroding:active ☐
Advancing:dormant ☐
Advancing:active ☐

Dominant Processes
Parallel flow ☐
Impinging flow ☐
Piping ☐
Freeze/thaw ☐
Sheet erosion ☐
Rilling + gullying ☐
Wind waves ☐
Vessel Forces ☐
Ice rafting ☐
Other (write in) ☐

PART 11: LEFT BANK GEOTECH FAILURES

Failure Location
General ☐
Outside Meander ☐
Inside Meander ☐
Opposite a bar ☐
Behind a bar ☐
Opposite a structure ☐
Adjacent to structure ☐
Dstream of structure ☐
Ustream of structure ☐
Other (write in) ☐

Present Status
Stable ☐
Unreliable ☐
Unstable:dormant ☐
Unstable:active ☐

Failure Scars+Blocks
None ☐
Old ☐
Recent ☐
Fresh ☐
Contemporary ☐

Apparent Failure Mode
Soil/rock fall ☐
Shallow slide ☐
Rotational slip ☐
Slab-type block ☐
Cantilever failure ☐
Pop-out failure ☐
Piping failure ☐
Dry granular flow ☐
Wet earth flow ☐
Other (write in) ☐

PART 12: LEFT BANK TOE SEDIMENT ACCUMULATION

Stored Bank Debris
None ☐
Individual grains ☐
Aggregates+crumbs ☐
Root-bound clumps ☐
Small soil blocks ☐
Medium soil blocks ☐
Large soil blocks ☐
Cobbles/boulders ☐
Boulders ☐

Vegetation
None/fallow ☐
Artificially cleared ☐
Grass and flora ☐
Reeds and sedges ☐
Shrubs ☐
Saplings ☐
Trees ☐

Age
Immature ☐
Mature ☐
Old ☐
Age in Years ☐

Health
Healthy ☐
Unhealthy ☐
Dead ☐

Tree species
(if known) ____________

Existing Debris Storage
No bank debris ☐
Little bank debris ☐
Some bank debris ☐
Lots of bank debris ☐

Roots
Normal ☐
Adventitious ☐
Exposed ☐

Notes and Comments:

SECTION 5 - RIGHT BANK SURVEY

PART 13: RIGHT BANK CHARACTERISTICS

Type
- Noncohesive ☐
- Cohesive ☐
- Composite ☐
- Layered ☐
- Even Layers ☐
- Thick+thin layers ☐
- Number of layers ___

Protection Status
- Unprotected ☐
- Hard points ☐
- Toe protection ☐
- Revetments ☐
- Dyke Fields ☐

Bank Materials
- Silt/clay ☐
- Sand/silt/clay ☐
- Sand/silt ☐
- Sand ☐
- Sand/gravel ☐
- Gravel ☐
- Gravel/cobbles ☐
- Cobbles ☐
- Cobbles/boulders ☐
- Boulders/bedrock ☐

Layer Thickness
- Material 1 (ft) ___
- Material 2 (ft) ___
- Material 3 (ft) ___
- Material 4 (ft) ___

Ave. Bank Height
- Average height (ft) ___

Ave. Bank Slope
- angle (degrees) ___

Tension Cracks
- None ☐
- Occasional ☐
- Frequent ☐

Crack Depth
- Proportion of bank height ___

Distribution and Description of Bank Materials in Bank Profile

Material Type 1	**Material Type 2**	**Material Type 3**	**Material Type 4**
Toe ☐	Toe ☐	Toe ☐	Toe ☐
Mid-Bank ☐	Mid-Bank ☐	Mid-Bank ☐	Mid-Bank ☐
Upper Bank ☐	Upper Bank ☐	Upper Bank ☐	Upper Bank ☐
Whole Bank ☐	Whole Bank ☐	Whole Bank ☐	Whole Bank ☐
D50 (mm) ___	D50 (mm) ___	D50 (mm) ___	D50 (mm) ___

PART 14: RIGHT BANK-FACE VEGETATION

Vegetation
- None/fallow ☐
- Artificially cleared ☐
- Grass and flora ☐
- Reeds and sedges ☐
- Shrubs ☐
- Saplings ☐
- Trees ☐

Orientation
- Angle of leaning (o)

Tree Types
- None ☐
- Deciduous ☐
- Coniferous ☐
- Mixed ☐

Tree species
(if known)

Density + Spacing
- None ☐
- Sparse/clumps ☐
- dense/clumps ☐
- Sparce/continuous ☐
- Dense/continuous ☐

Roots
- Normal ☐
- Exposed ☐
- Adventitious ☐

Location
- Whole bank ☐
- Upper bank ☐
- Mid-bank ☐
- Lower bank ☐

Diversity
- Mono-stand ☐
- Mixed stand ☐
- Climax-vegetation ☐

Health
- Healthy ☐
- Fair ☐
- Poor ☐
- Dead ☐

Age
- Imature ☐
- Mature ☐
- Old ☐

Height
- Short ☐
- Medium ☐
- Tall ☐
- Height (m) ___

Lateral Extent
- Wide belt ☐
- Narrow belt ☐
- Single row ☐

PART 15: RIGHT BANK EROSION

Erosion Location
- General ☐
- Outside Meander ☐
- Inside Meander ☐
- Opposite a bar ☐
- Behind a bar ☐
- Opposite a structure ☐
- Adjacent to structure ☐
- Dstream of structure ☐
- Ustream of structure ☐
- Other (write in) ☐

Present Status
- Intact ☐
- Eroding:dormant ☐
- Eroding:active ☐
- Advancing:dormant ☐
- Advancing:active ☐

Dominant Processes
- Parallel flow ☐
- Impinging flow ☐
- Piping ☐
- Freeze/thaw ☐
- Sheet erosion ☐
- Rilling + gullying ☐
- Wind waves ☐
- Vessel Forces ☐
- Ice rafting ☐
- Other (write in) ☐

PART 16: RIGHT BANK GEOTECH FAILURES

Failure Location
- General ☐
- Outside Meander ☐
- Inside Meander ☐
- Opposite a bar ☐
- Behind a bar ☐
- Opposite a structure ☐
- Adjacent to structure ☐
- Dstream of structure ☐
- Ustream of structure ☐
- Other (write in) ☐

Present Status
- Stable ☐
- Unreliable ☐
- Unstable:dormant ☐
- Unstable:active ☐

Failure Scars+Blocks
- None ☐
- Old ☐
- Recent ☐
- Fresh ☐
- Contemporary ☐

Apparent Failure Mode
- Soil/rock fall ☐
- Shallow slide ☐
- Rotational slip ☐
- Slab-type block ☐
- Cantilever failure ☐
- Pop-out failure ☐
- Piping failure ☐
- Dry granular flow ☐
- Wet earth flow ☐
- Other (write in) ☐

PART 17: RIGHT BANK TOE SEDIMENT ACCUMULATION

Stored Bank Debris
- None ☐
- Individual grains ☐
- Aggregates+crumbs ☐
- Root-bound clumps ☐
- Small soil blocks ☐
- Medium soil blocks ☐
- Large soil blocks ☐
- Cobbles/boulders ☐
- Boulders ☐

Vegetation
- None/fallow ☐
- Artificially cleared ☐
- Grass and flora ☐
- Reeds and sedges ☐
- Shrubs ☐
- Saplings ☐
- Trees ☐

Age
- Immature ☐
- Mature ☐
- Old ☐
- Age in Years ☐

Health
- Healthy ☐
- Unhealthy ☐
- Dead ☐

Tree species
(if known)

Roots
- Normal ☐
- Adventitious ☐
- Exposed ☐

Existing Debris Storage
- No bank debris ☐
- Little bank debris ☐
- Some bank debris ☐
- Lots of bank debris ☐

Notes and Comments:

Summary of geomorphic observations
☐ The size and stability of the channel, and where applicable, the floodplain (signs of bank erosion, bar deposition, scour holes, terraces)
☐ Signs of ongoing aggradation or degradation
☐ Signs of lateral migration or channel widening
☐ Presence of vertical or lateral controls (such as diversion structures, grade controls, culverts)
☐ Evidence of channelization, levees, etc.
☐ Gravel mining nearby, upstream or downstream

Quantitative assessment of the channel bed material	
☐ Collect representative bed material samples for laboratory sieve analysis or perform Wolman counts on site for coarse-grained streams. Note sample locations on the Channel Sketch Map.	
Overall Geomorphic Assessment	
Long-term aggradation/degradation potential	High 5 4 3 2 1 Low
Bank erosion potential	High 5 4 3 2 1 Low
Thalweg migration potential	High 5 4 3 2 1 Low

5. Survey

General Notes:

☐ Vertical datum

☐ Bench mark data

☐ Horizontal datum

☐ Basis of bearings

Survey Point Codes:

Description	Code
Bench Mark	BM
Overbank Topo	OB
Top of Slope	TS
Bottom of Slope	BS
Top of Revetment	TR
Bottom of Revetment	BR
Channel	CH
Thalweg	T
Left Top of Bank	LTB
Right Top of Bank	RTB
Left Edge of Water	LEW
Right Edge of Water	REW
Top of Bridge/Road	HC
Parapet or Rail used for soundings	TPR
Low Chord	LC
Top of Abutment	TA
Bottom of Abutment	BA
Top of Pier	TP
Bottom of Pier	BP
Top of Weir or Hydraulic Structure	VC

Surveyed Features:

☐ Bridge corners

☐ Profile along overtopping crest (if applicable)

☐ Profile of parapet or rail used for bridge soundings

☐ Low chord elevation or profile

☐ Top and bottom of abutment wingwalls at the corners and ends

☐ Locations of pier noses and abutment toes

☐ Elevation(s) of top of exposed footings at piers and abutments

☐ Channel cross sections of the channel bed, banks, and overbank area at four locations: (1) just upstream of the bridge, (2) just downstream of the bridge, (3) some distance upstream of the bridge, and (4) some distance downstream of the bridge. (Extent into overbanks determined prior to field visit based on floodplain width and available mapping.)

☐ Channel slope (Survey points at least 1000' apart. Indicate if slope based on channel bed or water surface.)

☐ Elevation of vertical controls downstream (i.e., weir crests or gate inverts of downstream diversion structures)

APPENDIX D

OUTLINE FOR
PRELIMINARY AND FINAL BRIDGE
HYDRAULICS REPORT

BRIDGE HYDRAULICS REPORT (OUTLINE)

1 INTRODUCTION

1.1 Project Description

- Sponsor/authorization
- Purpose of study
- Project location
- Route and stream/floodplain crossed
- Location (*Figures: Location map; vicinity map*)
- Work to be done (e.g. rehabilitation, replacement, new crossing)

1.2 Previous Hydrology and Hydraulic Studies

- ITD Studies
- FEMA
- USGS
- Others
- Include links to the studies if they are accessible online
- Provide reference citations

2 SUMMARY OF EXISTING CONDITIONS

2.1 River and Floodplain Description

- General description summarized from Hydraulic Risk Assessment Memorandum (HRAM) (*Figures: Aerial photograph of river and floodplain in vicinity of the crossing; field photos of channel*)

2.2 Site Reconnaissance

- Hydraulic controls and features observed (*Figures: annotated field photos*)
- Measurements taken of structure dimensions (*Figures: annotated field photos of structure*)
- High-water marks observed (*Figures: photos of noticeable high-water marks*)
- Scour holes observed at bridge or culvert (*Figures: photos of scour holes*)
- Presence and condition of existing scour countermeasures (*Figures: annotated field photos of countermeasures*)

- Wolman counts or sediment samples taken (*Figure: aerial image showing locations of samples or Wolman counts*)
- Observations of different areas of hydraulic roughness/Manning's n values (*Figures: photos of the different roughness categories in the model area [e.g. willow riverbanks, river bottom, farm fields, overbank forest, etc.]*)

2.3 FEMA Regulatory Status

- FEMA-regulated floodplain encroached upon by the project (if any)
- Type of floodplain (Zone AE-Detailed vs Zone A-Approximate) with or without a regulatory floodway (*Figure: excerpt of FIRM panel showing crossing site, flood zones*)
- Community having jurisdiction (may be more than one)

2.4 Existing Crossing (if there is one)

- Hydraulic opening width (length of bridge or span of culvert), positions of abutments with respect to channel banks, length of culvert, wingwalls and aprons (*Figures: photos of existing bridge and roadway*)
- Flow alignment (opening, piers, abutments)
- General description of road and bridge profile, height of low chord above the channel bed (*Appendix: Plan and elevation drawings of road and structure from as-built plans*)

3 PROJECT DESIGN STANDARDS AND CRITERIA

3.1 Design Flood Frequency

- Capacity design and check flood frequency
- Freeboard or headwater constraints
- Backwater considerations
- Scour design and check flood frequency
- Scour countermeasure design flood frequency

3.2 Floodplain permit requirements

- If within a FEMA regulated floodplain, a Floodplain Development Permit will be required
- If project encroaches on floodway: no-rise
- Consider local jurisdiction requirements that may exceed FEMA regulations

3.3 Approved variances

- If any of the above described standards or criteria represent a variance from ITD Bridge Hydraulics Manual Section 2
- Variance must have been requested and approved earlier
- Summarize the history and status of any variance request (detail will be in the HRAM)

4 HYDROLOGIC ANALYSIS

4.1 Watershed

- Size in square miles (*Figure: Drainage basin map*)
- Basin terrain and slope
- Land use and ground cover

4.2 Flood History

- Notable flood events: Date, cause of flood (intense rainfall, long duration rainfall, snowmelt, rain on snow), peak discharge
- Damage to the crossing or maintenance issues associated with notable flood events

4.3 Analysis Methods

- Which methods were used to estimate flood-frequency relationship?
- Reasons for the methods selected

4.4 Relevant Data

- Available stream gage data
- Discharge-frequency relationship for site from FEMA Flood Insurance Study (if it exists)
- Regional regression equations (if used)
- Basin parameters (area in sq. miles, annual precipitation in inches, basin slope, ground cover and land use, time of concentration, etc.)
- Design rainfall data (if rainfall-runoff analysis was performed)

4.5 Analysis Results

- Flood-frequency relationship from each method (*Figure: Plot of flood-frequency curves from each method)*
- Adopted flood-frequency relationship: rationale for adopting it (*Figure: Plot of adopted flood-frequency curve. Table: Peak discharge rates for range of recurrence intervals including 1.5-year, 10-year, 25-year, 50-year, 100-year and 500-year floods*)

5 MODEL DEVELOPMENT

5.1 Modeling Method and Program Used

- 2D Modeling with SRH-2D/SMS (preferred)
- 1D Modeling with HEC-RAS (requires variance request and approval within the HRAM)

5.2 Topographic and Bathymetric Data

- Sources (include specifics such as method of data collection and when data was surveyed/acquired)

- Vertical Datum (should be NAVD 88 except in special circumstances)
- Horizontal Coordinate System
- Key topographic and structural hydraulic controls (*Figure: Plot of terrain surface contours with elevation labels and controls pointed out*)

5.3 Model Geometry Existing Conditions

- Downstream and upstream model limits and why chosen
- If 2D modeling was performed
    - Model domain *(Figure: Plot of full mesh)*
    - Total number of elements
    - Areas of high density and why *(Figure: plots of mesh in specific high-density areas)*
    - Mesh quality (discuss general compliance with mesh quality recommendations and justify areas of exception) (*Figure: mesh quality plot*)
    - Representation of terrain (*Figure: Elevation contours from mesh with contours labeled and hydraulic controls pointed out*)
- If 1D modeling was performed
    - Reaches
    - Cross section locations (*Figures: Cross sections plotted over topographic contours and aerial imagery of model domain*)

5.4 Boundary Conditions Existing Conditions

- Inflow boundaries: locations and discharge values (related to hydrologic analysis)
- Exit boundaries: locations and value assignments (justify)
- Sensitivity analysis and verification that model extents are adequate (e.g. that results in the areas of interest are not highly sensitive to the boundary condition values)
- Internal boundary conditions if present *(Figure: Annotated plot showing all boundary conditions)*

5.5 Roughness Assignments Existing Conditions

- Designation of roughness value zones (*Figures: Aerial image with roughness zones mapped superimposed over plot of 2D mesh or 1D cross sections*)
- Assigned Manning's coefficient values in each zone (and justification)
- Discussion of calibration or validation data, approach, and results
- Justification if calibration or validation was not done

5.6 Model Run Control Parameters

- If 2D modeling was performed
    - Computational time step and run duration
    - Initial conditions settings
    - Turbulence options selected
    - Output frequency
- If 1D modeling was performed
    - Computational tolerance for water surface elevation

- Computational tolerance for split flows
- Friction slope averaging method selected

5.7 Modeling of Structures Existing Conditions

- If 2D modeling was performed (*Figures: Plots of mesh showing the modeling of the crossing and structures, also annotated to show use of HY-8 module, weirs, gates, or pressure flow*)
  - Representation of bridge or culvert in 2D model
  - Representation road embankment and abutments
  - Representation of piers
  - Handling of pressure flow if relevant
  - Use of 1D features in 2D model: HY-8 module; weirs; gates
- If 1D modeling was performed *(Figures: Plan view plot of cross sections and ineffective flow areas; cross section plot showing road embankment and bridge structure [e.g. cross section BU])*
  - Cross section placement for contraction and expansion
  - Ineffective flows
  - Road profile
  - Bridge modeling approach
  - Bridge data or culvert data editor input

5.8 Description of Proposed Conditions

- Structure: hydraulic width, piers, abutments, superstructure, culvert barrel shape, length of culvert, wingwall *(Appendix: Plan and elevation drawing of proposed structure; culvert profile drawing)*
- Road profile (*Appendix: Road profile*)
- Grading of channel or overbank areas *(Appendix: Grading plan)*
- Changes to ground cover/roughness

5.9 Modeling of Proposed Conditions

- For each design alternate that was modeled, describe the modifications from the existing conditions model to represent the new: (*Figures: plots of revised mesh layout and elevations in modified areas (if 2D); cross section plots showing new crossing geometry (if 1D))*
  - Road alignment, profile, and embankment geometry (Appendices:
  - Abutments and piers
  - Superstructure and/or pressure flow conditions
  - Manning’s n changes if relevant
  - Boundary condition changes if required (this is not expected)

6 Existing Condition Model Results

6.1 Water Surface Elevation Results

- Determine freeboard or headwater for design flood and check flood
- Identify recurrence interval of pressure flow (if any) (*Figure: plan view showing overtopping locations*)
- Identify locations and recurrence intervals of overtopping (if any)
- Resulting water surface profile along the river and if 2D modeling was performed, along the edges of the floodplain (*Figure*)
- If 2D modeling was performed: Water surface elevation contour plots for each recurrence interval modeled (*Multiple figures*)
- If 2D modeling was performed: Summary table of the minimum, maximum and average values of water surface elevation along various transects across the model width located strategically throughout the model domain (e.g. at FEMA model cross section locations if applicable)
- If 1D modeling was performed: Table of water surface elevations at each cross section for each recurrence interval modeled.

6.2 Depth Results

- If 2D modeling was performed: Depth contour plots for each recurrence interval modeled (*multiple figures*)
- If 2D modeling was performed: Summary table of the minimum, maximum and average values of depth along various transects across the model width located strategically throughout the model domain
- If 1D modeling was performed: Table of maximum main channel depth, main channel hydraulic depth and average overbank depths at each cross section for each recurrence interval modeled

6.3 Velocity Results

- If 2D modeling was performed: Velocity magnitude contour plots including vectors for each recurrence interval modeled (*multiple figures*)
- If 2D modeling was performed: Summary table of the minimum, maximum and average values of velocity along various transects across the model width located strategically throughout the model domain
- If 1D modeling was performed: Table of maximum main channel velocity, main channel average velocity and average overbank velocities at each cross section for each recurrence interval modeled

6.4 Discharge Distribution and Flow Splits

- If 2D modeling was performed: Check for model continuity
- Determine flow distribution at flow splits, multiple openings, etc. (*Figure: Plot showing discharge observations lines; Table: Summary table of discharge across observation lines*)

7 Proposed Condition Model Results

7.1 Water Surface Elevation Results

- Determine freeboard or headwater for design flood and check flood and compare to design standards and criteria
- Identify recurrence interval of pressure flow (if any) (*Figure: plan view showing overtopping locations*)
- Identify locations and recurrence intervals of overtopping (if any)
- Comparative water surface profile plots showing proposed condition and existing condition water surface profile along the river and if 2D modeling was performed, along the edges of the floodplain (*Figure*)
- If 2D modeling was performed: Water surface elevation contour plots for each recurrence interval modeled (*Multiple figures*)
- If 2D modeling was performed: Water surface elevation difference contours (proposed condition water surface elevation minus existing condition water surface elevation at all points) (*Multiple figures*)
- If 2D modeling was performed: Summary table of the minimum, maximum and average values of water surface elevation along various transects across the model width located strategically throughout the model domain (e.g. at FEMA model cross section locations if applicable)
- If 1D modeling was performed: Table of proposed condition and existing condition water surface elevations at each cross section for each recurrence interval modeled

7.2 Depth Results

- If 2D modeling was performed: Depth contour plots for each recurrence interval modeled (*multiple figures*)
- If 2D modeling was performed: Summary table of the minimum, maximum and average values of depth along various transects across the model width located strategically throughout the model domain
- If 1D modeling was performed: Table of maximum main channel depth, main channel hydraulic depth and average overbank depths at each cross section for each recurrence interval modeled

7.3 Velocity Results

- If 2D modeling was performed: Velocity magnitude contour plots including vectors for each recurrence interval modeled (*multiple figures*)
- If 2D modeling was performed: Velocity magnitude difference contours (proposed condition velocity minus existing condition velocity at all points) (*Multiple figures*)
- If 2D modeling was performed: Summary table of the minimum, maximum and average values of velocity along various transects across the model width located strategically throughout the model domain
- If 1D modeling was performed: Table of maximum main channel velocity, main channel average velocity and average overbank velocities at each cross section for each recurrence interval modeled

7.4 Discharge Distribution and Flow Splits

- If 2D modeling was performed: Check for model continuity
- Determine flow distribution at flow splits, multiple openings, etc. (*Figure: Plot showing discharge observations lines; Table: Summary table of discharge across observation lines*)
- Discuss any differences in flow distribution comparing proposed to existing conditions

7.5 Other Results as Needed

- Froude number contour plots for recurrence intervals of interest (*multiple figures*)
- Shear stress contour plots for recurrence intervals of interest (*multiple figures*)
- Other variables as might be useful on a project specific basis

8 FEMA ANALYSIS (If located within a FEMA-regulated floodplain)

8.1 FEMA Flood Zone Classification

8.2 Compliance with Regulations

- State whether the comparison of proposed conditions to existing conditions represents an acceptable no-rise condition in the 100-year flood
- If a no-rise is not achieved, discuss changes to the design that could mitigate the rise (e.g. increase structure capacity, allow more overtopping in the 100-year flood)
- Provide information needed for Floodplain Development Permit (if required)
- State whether Conditional Letter of Map Revision (CLOMR) is required and whether approval is required before construction

8.3 Additional FEMA Modeling (if required for no-rise certificate or CLOMR)

- Effective Model
    - Duplicate Effective Model
    - Corrected Effective Model
- Existing Condition Model
- Proposed Condition Model
- Water surface comparison table for 100-year flood for all conditions modeled

9 STREAM STABILITY AND SCOUR EVALUATION

9.1 Stream Stability Assessment

- Refer to Geomorphic Assessment in the Hydraulic Risk Assessment Memorandum: (*Figure: Include the figure summarizing channel characteristics from the Geomorphic Assessment [annotated version of HEC-20 Figure 2.6]*)

- Summarize the assessed potential for lateral and vertical instability from the Geomorphic Assessment (*Figures: Areal imagery and/or field photos depicting lateral stability or instability; time history of cross section plots depicting lateral stability or instability*)
- Discuss any known site-specific sediment transport issues (e.g. high sediment inflow leading to aggradation or sediment deficiency leading to degradation)

9.2 Scour History of Existing Structure

- Inspection reports
- Include streambed measurements from bridge inspection reports if available
- Item 113 code (for bridge-sized structures)
- Maintenance records
- Previous scour studies
- Comparison of current bridge opening cross section with historic data
- Description and condition of existing scour countermeasures
- Discuss history of and potential for debris collection at the structure and its possible effects on scour

9.3 Scour Evaluation for Design

- Bed material
- Contraction scour
- Local scour at piers
- Local scour at abutments *(Appendix: Scour calculations for each type of scour)*
- Total scour *(Figure: Scour profile plot; Table: Scour summary table)*
- Possible adjustments to scour depth due to site-specific soils (e.g. cohesive soil or bedrock)

9.4 Scour and Stream Instability Countermeasures

- Explain countermeasures needed and why
  Abutment protection
- Guide banks
- Streambank protection
- Road embankment protection
- Countermeasure type (e.g. riprap, concrete armor units, articulating concrete block, turf reinforcing mat, etc.)
- Sizing calculations
- Design (*Figures or Appendix: Sketches of layout extent, typical sections, termination requirements, etc.)*

10 OTHER PROJECT SPECIFIC TOPICS

- Aquatic Organism Passage (AOP) requirements and design accommodation
- Stream Restoration or Other Habitat Enhancements
- Temporary Crossing Structures during Construction
- Temporary Diversions or In-Channel Work Areas during Construction

11 CONCLUSIONS

Concisely summarize the results that are directly relevant to the design of the structure, including:

- Design recurrence interval and design discharge
- Summary of variances to requirements that have been requested and approved
- Required freeboard height above design event flood profile and the low chord elevation required to meet that requirement. Also the actual freeboard provided by actual low chord profile, if known
- Impact (increase or decrease and by how much maximum) on the 100-year water surface profile vs. existing conditions
- Refer to Floodplain Development Permit in Appendix
- Refer to scour profile and scour summary table in Section 9

12 **REFERENCES** (may include but not be limited to the following)

If information is taken directly from a published reference document or book it should be cited in the report text and listed in this section. Examples of documents that are often cited are provided below.

- ITD Manuals and Requirements
    - *Bridge Hydraulics Manual*
    - *Idaho Manual for Bridge Evaluation*
- FHWA Reference Documents
    - *Two-Dimensional Hydraulic Modeling for Highways in the River Environment*
    - HDS 5 *Hydraulic Design of Highway Culverts*
    - HDS 7 *Hydraulic Design of Safe Bridges*
    - HEC-14 *Hydraulic Design of Energy Dissipators for Culverts*
    - HEC-17 *Highways in the River Environment-Floodplains, Extreme Events, Risk and Resilience*
    - HEC-18 *Evaluating Scour at Bridges*
    - HEC-20 *Stream Stability at Highway Structures*
    - HEC-23 *Bridge Scour and Stream Instability Countermeasures*
- USGS Data and Reports
    - Stream Gage Data (links to online gage data)
    - Stream Stats and associated USGS research reports
- AASHTO Guidance
    - *LRFD Bridge Design Specification*
    - *Highway Drainage Guidelines*
- FEMA Floodplain Information
    - Relevant Flood Insurance Rate Map (FIRM) panels
    - Relevant Flood Insurance Study (FIS) reports
- Local Government Floodplain Ordinances

APPENDICES

- ITD Form 211
- Hydraulic Risk Assessment Memorandum (HRAM)
- As-builts of existing structure
- Situation and Layout drawings of proposed structure

- Proposed road profile drawings
- Proposed grading plan
- FEMA FIRM maps, FIS reports and relevant Letters of Map Revision
- Floodplain Development Permit Application
- Scour calculations and scour countermeasure sizing calculations
- Scour countermeasure design layout

APPENDIX E

QA COVER FORM AND MODEL REVIEW CHECKLISTS

QA COVER FORM LINK
HYDRAULIC 2D MODEL CHECKLIST LINK
HYDRALIC 1D MODEL CHECKLIST LINK

Idaho Transportation Department, Bridge Section

Hydraulic Quality Assurance Form
per ITD Bridge LRFD Manual Article 0.12

Structure:
Route & Feature Intersected:
Project No.:
Project Key:
Bridge Key:
County:

Report Preparer (does not require a engineers seal):

Model Preparer (does not require a engineers seal):

Report Checker:

Model Checker:

Senior Independent Reviewer:

Project: ______________________ **Reviewer:** __________

River: ______________________

Project Purpose: ______________________ **Date:** __________

Project File Name: ______________________

Additional Information: ______________________ **Modeler:**

***** Blank comment entries below indicate that the item was not reviewed.***

Item	Comment	Action Needed (blank=none)	Response to Comment/ Resolution	Screen Shot	Link
Model Background Data					
Version of SMS/SRH-2D documented?				☐	
Project vertical datum?				☐	
Project horizontal datum?				☐	
Documentation of techniques and procedures?				☐	
Meta data included in model files?				☐	
Topography					
Source/Date				☐	
If Lidar data, has it been filtered to removed vegetation and structures?				☐	
Stated Accuracy				☐	
Datums verified				☐	
Data type (Scatter set or 3D Raster image)				☐	
Number of points / average spacing				☐	
Bathymetry					
Source/Date				☐	
Datums verified				☐	
Additional Survey					
Source/Date				☐	
Datums verified				☐	
Bridge/Culvert/Structure Data					
Source/Date				☐	
Datums verified				☐	
Topographic Data review					
Were multiple data sources merged to create a terrain map? If so, which sources?				☐	
Data consistency - Are the transitions between data sets smooth?				☐	
Does final surface accurately represent site (are hydraulic controls represented)?				☐	
Confirm breaklines used where necessary				☐	
2D Mesh					
How many mesh elements?				☐	
Are the number and size of mesh elements appropriate?				☐	
What is the range of element sizes and is it appropriate for this project application?				☐	

What is the length of the modeled reach?				❑	
What are the approximate floodplain widths (upstream/downstream)?				❑	
Is the upstream mesh limit sufficient?				❑	
Is the downstream mesh limit sufficient?				❑	
Are the lateral extents sufficient?				❑	
Are key project features correctly represented?				❑	
Are all slope features (channel banks, embankments, etc.) represented by at least 2 or more elements?				❑	
Is mesh quality acceptable?				❑	

Boundary Conditions					
Are unsteady or steady simulations performed?				☐	
Do boundary conditions have descriptive names?				☐	
What is the source for the inflow data?				☐	
Upstream Boundary - Verify correct inflow(s) amount, type, and location				☐	
How were downstream tailwater boundaries computed (normal depth, critical depth, known water surface, other?)				☐	
Downstream Boundary - Verify correct stage, type, and location				☐	
Are boundary conditions applied (mapped) to mesh correctly?				☐	
Are monitoring lines used?				☐	
Material Roughness					
How many materials types are used?				☐	
What is the source of material coverage and values?				☐	
Do the materials definiition extend to the limits (or beyond) the mesh domain limits?				☐	
Are material types correctly assigned?				☐	
Are the appropriate Manning's n-values used?				☐	
Hydraulic Structures					
How many structures are represented? What types?				☐	
Bridge					
Is the geometry beneath the bridge represented correctly?				☐	
For detailed hydraulics, piers should be represented as holes in the mesh. The dimensions of the hole should represent the average dimensions that are				☐	
Pressure BC arcs should be parallel and form rectangular zone betweeen them.				☐	
The ceiling elevation should represent the average low chord elevation of the bridge, or the span represented.				☐	
If the upstream WSEL exceeds the deck elevation, the overtopping option should be selected and parameters defined.				☐	
If the deck is overtopping, the Internal#.dat file should be reviewed for stable WSEL and flow				☐	
Culvert					
The mesh elements should generally align with the culvert and have element faces that are located close to the culvert inverts				☐	
Culvert BC arcs should be placed at the culvert invert locations and should generally represent the width of the culvert(s)				☐	
Is the culvert modeled in the 2D mesh or as a HY-8 culvert?				☐	
HY-8 Culvert BC arcs should be located at the culvert invert locations and the HY-8 elevations should be consistent with the mesh elevations at the invert locations.				☐	
Is culvert correctly represented				☐	
Obstructions					
Are obstructions used in the model?				☐	
The elevation of the obstruction arc should be set to the bottom elevatoin of the obstruction.				☐	
The obstruction arc should align with the centerline of the obstruction, with the appropriate dimentions and coefficients enterred in the obstructions dialog.				☐	
Other Structures					
What other structures are represented?				☐	
Is structure correctly represented?				☐	

Model Controls and Simulations					
How many simulations are included?				❑	
Are they labeled appropriately and do they include the correct components.				❑	
Review time step used for each simulation				❑	
Review simulation times				❑	
Turbulence model should be set to the Parabolic Method with a coefficient of 0.7				❑	
Initial Condition used				❑	

Model Results					
Are monitoring points used?				❑	
Confirm model stability at monitoring points				❑	
Confirm continuity at monitoring lines				❑	
Confirm stable results through the domain				❑	
Froude Number - Are results reasonable?				❑	
Shear Stress				❑	
Water Elevations				❑	
Velocity				❑	
Water Depth				❑	
Additional Notes:					
Model Calibration					
Was calibration performed? If so, does the model data match the calibration data?				❑	
If no calibration, were any sensitivity analyses performed?				❑	
General Comments					

PROJECT:
PROJECT NO:
LOCATION:
DESIGNER:

STEADY FLOW DATA **Boundary Conditions** Upstream Downstream	**Yes**	**No**	**N/A**
☐ Normal Depth S= S= ☐ Known WS WSE = WSE_{50} = Q_{50} = WSE = WSE_{100} = Q_{100} = WSE = WSE_{500} = Q_{500} = ☐ Critical Depth ☐ Rating Curve source = source =			
Sensitivity analysis of normal depth slopes all converge downstream of bridge?	☐	☐	☐
Known WS used from the FEMA Flood Insurance Study (NAVD)?	☐	☐	☐
Backwater Influence ☐ Structure ☐ Reservoir ☐ River Drainage Area Ratio: Frequency for Coincidental Occurrences: Downstream distance: Known water surface:	☐	☐	☐
Flow Regime ☐ Subcritical (*Froude < 1.0 at all sections, DS boundary conditions*) ☐ Supercritical (*Froude > 1.0 at all sections, US boundary conditions*) ☐ Mixed (*Froude > 1.0 at some sections, US and DS boundary conditions*)			
Discharge ☐ Closed bottom pipes (*spans less than 12 feet*) 25-year Design (*HW/D≤1.25*): _______ cfs 100-year Base/riprap/overtopping: _______ cfs ☐ Closed bottom rectangular culverts (*spans less than 12 feet*) 50-year Design (*HW/D≤1.25*): _______ cfs 100-year Base/riprap/overtopping: _______ cfs ☐ All open bottom structures and culverts (*spans 12-20 feet*) 50-year Design (*1' freeboard*): _______ cfs 100-year Base/riprap/overtopping: _______ cfs ☐ Bridges (*spans more than 20 feet*) 50-year Design (*2' freeboard*): _______ cfs 100-year Base/riprap/girder: _______ cfs 500-year Scour/overtopping: _______ cfs ☐ Bridges/Culverts over controlled-flow canals Average flow Design (*1' freeboard*): _______ cfs Maximum flow Low chord,riprap/scour: _______ cfs *(stormwater or infiltration added to max flow?)*			
Comments:			

GEOMETRIC DATA **Cross Sections**	**Yes**	**No**	**N/A**
Background pictures (.jpg) on the Schematic?	☐	☐	☐
Stations increase from downstream to upstream?	☐	☐	☐
Data is entered from left to right looking downstream (*XS direction arrows*)?	☐	☐	☐
Extend across 100-year floodplain without “vertical walls”?	☐	☐	☐
Perpendicular to anticipated flow direction in channel and overbanks?	☐	☐	☐
Each cross section represents a single water surface elevation (*stage*)?	☐	☐	☐
Do any overlap?	☐	☐	☐
Are interpolated cross sections used?	☐	☐	☐
If Geo-referenced, cross sections are **green** (*GIS Tools > XS Cut Lines Table*)?	☐	☐	☐
If not Geo-referenced, cross sections are **brown**?	☐	☐	☐
If Geo-referenced, “Display Ratio of Cut Line Length to XS Length” are 1.0, except at optional skewed bridge and Bounding sections?	☐	☐	☐
If not Geo-referenced, the “Scale Cut Lines to Reach Lines” is checked?	☐	☐	☐
Schematic and river stations match downstream reach lengths table?	☐	☐	☐
Left and right bank stations are reasonable with consistent elevations?	☐	☐	☐
Contraction/Expansion Coefficients (Steady Flow) adjusted for bridge effects?	☐	☐	☐
Manning’s n-values used were calibrated or a sensitivity analysis performed?	☐	☐	☐
Manning's Roughness Coefficients (‘n’ values) Left Overbank Min. Max. Channel Min. Max. Right Overbank Min. Max.			
Backwater influence downstream of bridge? Backwater length: L = 0.7*D/S (*Paul Samuel equation)* Spacing distance: dx = 0.15*D/S *(Paul Samuel equation)* *(D is bank full depth, S is bed slope)*	☐	☐	☐
Ineffective Flow areas (*no wetted perimeter, storage but no conveyance*) in overbanks are appropriate?	☐	☐	☐
Levees (*wetted perimeter, no storage until overtopped*) are appropriate?	☐	☐	☐
Obstructions (*wetted perimeter, no storage or conveyance*) are appropriate?	☐	☐	☐
FEMA lettered cross sections used in the model?	☐	☐	☐
No-Rise Certification measured from same cross section (Exist-vs-Proposed)?	☐	☐	☐
Error Warnings and Notes are reasonable for each Profile?	☐	☐	☐
Ineffective Flow Area			
Ineffective flow limits modeled within contraction/expansion reaches?	☐	☐	☐
Elevations for ineffective flow correspond to weir flow over bridge US & DS?	☐	☐	☐
(4) Approach XS: *(d= avg. length constriction from road abutment)*			
(3) Bounding XS: *(Ineffective Flow 1:1 from US bridge face)*			
(2) Bounding XS: *(Ineffective Flow 1:1 from DS bridge face)*			
(1) Exit XS:			
Bounding XS’s are at or beyond the roadway embankment toe?	☐	☐	☐
Bounding XS’s are parallel to each other?	☐	☐	☐
Length of contraction L_c =			
Length of expansion, L_e =			
Contraction Ratio, CR =			
Expansion Ratio, ER =			
Comments:			

Bridge Geometry	**Yes**	**No**	**N/A**
Bridge River Sta.:			
"Distance" between upstream XS and deck/roadway: ______ ft			
Deck/roadway "width" along the stream: ______ ft			
Deck/roadway "width" matches the report/drawings?	☐	☐	☐
High chord elevation (*top of road*): ____________ ft			
Low chord elevation (*min. low*): ____________ ft			
High/low chords match the report/drawings?	☐	☐	☐
Bridge span based on Deck/Roadway stations (*skew angle "0"*):			
Bridge span based on Bridge Data profiles (*skew angle "0"*):			
Bridge span matches the report/drawings?	☐	☐	☐
Embankment side slopes (*display purposes only Profile Plot*): _____ (H:V)			
Minimum weir flow elevation is blank? *(defaults to lowest high chord elevation on the US side of the bridge to start checking for weir flow)*	☐	☐	☐
Roadway profile grade modeled along the deck/roadway high chord?	☐	☐	☐
Abutments created with Sloping Abutment Editor or editing Bounding XS's?	☐	☐	☐
Abutment side slopes at BRU and BRD (w/skew angle): ______ (H:V)			
Abutment side slopes match the report/drawings?	☐	☐	☐
Deck roadway and abutments skewed?	☐	☐	☐
Skew angle based on angle of flow path as it goes through the bridge compared with a line perpendicular to the Bounding XS's?	☐	☐	☐
Number of spans:			
Number of piers:			
Pier centerline station distances match pier spans?	☐	☐	☐
Pier widths are correct?	☐	☐	☐
Pier elevations are correct (entered lowest to highest values)?	☐	☐	☐
Piers match the report/drawings?	☐	☐	☐
Pier skewed?	☐	☐	☐
Bridge Modeling Approach Low Flow Methods (*flow below the maximum low chord)*: ☐ Energy (Standard Step) ☐ Momentum Pier drag coefficient, C_d: ☐ Yarnell (Class A only) Pier shape coefficient, K: ☐ WSPRO Method (Class A only) High Flow Methods (*flow contacts the maximum low chord*): ☐ Energy Only (Standard Step) ☐ Pressure and/or Weir			

Culvert Geometry	**Yes**	**No**	**N/A**
Culvert River Sta.:			
“Distance” between upstream XS and deck/roadway: ______ ft			
Deck/roadway “width” along the stream: ______ ft			
Deck/roadway matches the report/drawings?	☐	☐	☐
High chord elevation (*top of road*): ______ ft			
High chord matches the report/drawings?	☐	☐	☐
Embankment side slopes (*display purposes only Profile Plot*): _____ (H:V)			
Minimum weir flow elevation is blank *(defaults to lowest high chord elevation on the upstream side of the culvert)*?	☐	☐	☐
Shape: ☐ Circular ☐ Box ☐ Pipe Arch ☐ Ellipse ☐ Arch ☐ Semi-Circle ☐ Low Arch ☐ High Arch ☐ Conspan Arch			
Span: ______ ft			
Rise: ______ ft			
Length: ______ ft			
Span, rise, and length match the report/drawings?	☐	☐	☐
FHWA Chart # (*Table 6-6*):			
FHWA Scale # (*Table 6-6*):			
Chart and scale description matches the report/drawings?	☐	☐	☐
Distance to upstream XS: ______ ft			
Entrance Loss Coefficient (*Tables 6-3, 6-4, and 6-5*):			
Exit Loss Coefficient:			
Manning’s n value for Top (*Tables 6-1 and 6-2*):			
Manning’s n value for Bottom:			
Depth (*above invert*) to use bottom n value:			
Depth blocked (*from passing flow*):			
Upstream Invert Elevation:			
Downstream Invert Elevation:			
Inverts match the report/drawings?	☐	☐	☐

CONTRACTION SCOUR (Q_{500})	Yes	No	N/A
Streambed Particle Size D_{50}: ______ mm = ______ in. = ______ ft. Method used to determine D_{50}? ☐ Visual inspection ☐ Woman pebble count ☐ Sieve analysis ☐ Core boring			
Critical Velocity HEC-18, Equation 6.1 used? ☐ Clear-water contraction scour ☐ Live-bed contraction scour	☐	☐	☐
Live-Bed Scour HEC-18, Equations 6.2-6.3 used?	☐	☐	☐
HEC-RAS output tables included?	☐	☐	☐
Clear-Water Scour HEC-18, Equations 6.4-6.5 used?	☐	☐	☐
HEC-RAS output tables included?	☐	☐	☐
LOCAL PIER SCOUR (Q_{500}) **Local Pier Scour for Simple Pier Substructure**			
HEC-18, Equation 7.3 (CSU equation) used?	☐	☐	☐
L, length of pier:			
a, pier width:			
θ, angle of attack:			
K_1 correction factor for pier nose shape is correct? (HEC-18, Table 7.1)	☐	☐	☐
K_2 correction factor for angle of attack is correct? (HEC-18, Table 7.2)	☐	☐	☐
K_3 correction factor for bed condition is correct? (HEC-18, Table 7.3)	☐	☐	☐
K_4 correction factor set to 1.0 (*removed from HEC-18, Fifth edition, 2012*)?	☐	☐	☐

REFERENCES

1. Hydraulic Engineering Center. 2010. HEC-RAS, River Analysis System Hydraulic Reference Manual. U.S. Army Corps of Engineers, Davis, CA.
2. Hydraulic Engineering Center. 2010. HEC-RAS, River Analysis System Hydraulic Reference Manual. Appendix B – Flow Transitions in Bridge Backwater Analysis, U.S. Army Corps of Engineers, Davis, CA.
3. Federal Highway Administration. 2012. Hydraulic Engineering Circular No. 18, Evaluating Scour At Bridge (Fifth Edition).
4. Samuels, P.G., 1989. “Backwater lengths in rivers”, Proceedings – Institution of Civil Engineers, Part 2, Research and Theory, 87, 571-582.
5. Hydraulic Engineering Center. 1995. RD-42 “Flow Transitions in Bridge Backwater Analysis”.

APPENDIX F

NO-RISE CERTIFICATION FORM

ENGINEERING "NO-RISE" CERTIFICATION

This is to certify that I am a duly qualified engineer licensed to practice in the State of ______________________________.

It is to further certify that the attached technical data supports the fact that proposed ____ _will

(Name of Development)

not impact the 100-year flood elevations, floodway elevations and floodway widths on _____________________________________ at published sections

(Name of Stream)

in the Flood Insurance Study for ____________________________________,

(Name of Community)

dated _____________________________ _ and will not impact the 100-year flood elevations, floodway elevations, and floodway widths at unpublished cross-sections in the vicinity of the proposed development.

Attached are the following documents that support my findings:

____

____

____

____

____

____

(Date) ___________________

(Signature) (Title)

____

____

____

(Address)

(Seal)