10 Bridges

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10 Bridges

10.1 INTRODUCTION

Bridges are legally defined (23 CFR 650.403) as structures with a centerline span of 20 feet or greater. However, structures designed hydraulically as bridges are treated in this chapter regardless of length.

Multiple considerations must be taken into account when deciding between a large culvert structure or a bridge structure to span a given waterway. These considerations include (but are not limited to) cost, aesthetics, environmental, and structural factors. The hydraulic performance of potential structures must be determined and analyzed to aid in this decision.

10.1.1 Design Goals

Proper hydraulic analysis and design of bridges is critical. Stream-crossing systems should be designed for:

- Minimum cost subject to design criteria;
- Desired level of hydraulic performance;
- Mitigation of impacts on the stream environment;
- Safe movement of desired traffic volume for an acceptable level of service; and
- Accomplishment of social, economic, and environmental goals.

10.1.2 Data Collection

Data collection is vital and requires gathering all necessary information for hydraulic analysis. This includes information such as topography and other physical features, land use and culture, any existing flood studies of the stream, historical flood data, basin characteristics, precipitation data, geotechnical data, historical highwater marks, existing structures, channel characteristics, and environmental data. A site plan showing the bridge location should be developed on which much of the data can be presented. Data of particular interest is discussed in this section. Refer to Chapter 6 - Data Collection, for more information.

10.2 DESIGN CRITERIA

10.2.1 General Criteria

The following general criteria must be considered in the hydraulic analysis and design of a bridge over a waterway:

- Sag vertical curves can cause deck drainage to pond and ice up on bridges, and should be avoided.
- Horizontal-curve transitions cause water to flow across lanes, and should not be located on a bridge due to the potential of icing and hydroplaning.

Photo 10.1

Photo 10.2

- Clearance or freeboard should be provided between the low-girder and the design water surface to allow for the passage of ice and debris.
- The idealized design discharge of any bridge over a waterway is the flow that will pass through the bridge with adequate freeboard, and without roadway or deck overtopping.
- Estimate all potential degradation, contraction scour, pressure scour and local scour for the hydraulic design-flood frequency, and for the 500-year event. In addition, estimate any potential aggradation at or near the bridge footprint. Indicate the total-scour envelope with a continuous line drawn such that the structural designer may adequately design substructure components. Consider local geology when estimating scour depths. The foundation-scour estimates should adhere to the methodologies found in the current version

of FHWA's Hydraulic Engineering Circular (HEC) and Hydraulic Design Series (HDS) documents, as well as the most recent research found in NCHRP publications.

- Flow velocities through the structure(s) must not damage the highway facility, or increase damages to adjacent properties in a manner not present previous to the structure's proposed configuration.
- Pier spacing, orientation, and type (wall, column, compound/complex piers, etc.), as well as abutment locations, must be designed to minimize flow disruption and potential scour. Efforts should be made to avoid placing bridge piers in the main channel area considering potential lateral channel migration.
- Foundation design and/or scour countermeasures must be established to avoid failure by scour for all discharges up to and including the scour-review discharge.
- Although appropriate in some debris-prone streams, connecting a discrete pier column to a debris-deflecting wall can significantly increase scour depths if the channel alignment shifts. A debris-deflecting wall can also greatly increase the stiffness of a pier, and reduces available design options. Preferably, a long-span bridge design reduces the number of piers, which reduces benefits derived from debris-deflecting walls. It is often more efficient for a designer to simply design a pier (and if necessary the superstructure) for increased stream loads due to debris.
- When two or more bridges are constructed in parallel over a channel, care should be taken to align the piers to the flowlines, and provide streamlined grading and protection for abutments. Streamlined abutment grading will minimize expansion or contraction of flow between the two bridges.
- Commercial mining of sands and gravel in streams is common because the material is clean and well graded, and the stream replenishes the supply. Borrow pits, either upstream or downstream of a highway-stream crossing, can cause or aggravate scour at the bridge. It is possible to consider this when calculating bridge scour, and may be estimated by sediment-transport modeling if the risk warrants such an analysis.
- Disruption of natural ecosystems should be minimized. Consideration must be given to the preservation of valuable ecological characteristics unique to the floodplain and stream system.
- Economic analysis of a design must include complete life-cycle costs and benefits. Factors that must be considered are construction, maintenance, and operation, as well as any potential liabilities. Such an analysis is multi-disciplinary in nature, but should include a hydraulics engineer's input.
- Adequate right-of-way must be provided upstream and downstream of a structure for maintenance operations.
- The final design selection should consider the maximum backwater allowed by the Colorado Water Conservation Board (CWCB), unless exceeding the limit can be justified by unusual hydraulic conditions.
- For sites outside of a regulatory floodplain or floodway, the backwater must not cause increased flood damage to insurable property upstream of the crossing, and should be limited to the allowable maximum rise determined by CWCB standards, or applicable local standards.
- The final design should not significantly alter the existing flow distribution in the floodplain.

10.2.2 Specific Criteria

Overtopping Flood

Inundation of the traveled way affects the level of traffic services provided by a facility. The potential failure of the roadway embankment during overtopping should be analyzed (see FHWA Report No. RD 86/126).

Per CFR 650.115(a)(2), a through lane of an Interstate highway must not experience overtopping in a 2-percent chance flood (the 50-year annual recurrence flood), nor should it experience overtopping for floods at a greater annual recurrence than the 2-percent chance flood. Structures over waterways, as well as road surfaces orientated within the floodplain fringe or floodway, must be carefully designed to ensure that this criteria is met.

Risk Evaluation

The selection of design frequency for determining the waterway opening, road grade, scour potential, riprap, and other features must consider the potential impacts to:

- Interruptions to traffic;
- Adjacent property;
- The environment; and
- The infrastructure of the highway.

The consideration of these potential impacts constitutes an assessment of risk for the specific site. HEC-17 (*Highways in the River Environment – Floodplains, Extreme Events, Risk, and Resilience*) provides guidance when producing a basis for comparison between alternatives developed in response to environmental, regulatory, and political considerations.

Backwater Increases Over Existing Conditions

A new structure must conform to FEMA regulations for sites covered by the NFIP. The maximum allowable rise in Base Flood Elevations (BFEs) is outlined in Chapter 2 – Policy.

Technical hydraulic evaluations must include channel conditions pertaining to the existing bridge and the proposed bridge. In some cases, a “natural-channel condition” prior to the construction of the existing bridge might be considered, though it is often difficult or impossible to ascertain the “natural” condition of the channel due to the lack of historical information.

Calculation of Maximum Backwater

The latest version of HDS-7 (*Hydraulic Design of Safe Bridges*, FHWA 2012) contains much of the hydraulic theory and practical illustrations and tips on methods of calculating backwater caused by normal, skewed, and eccentric bridge crossings in relation the flow path. The most current version of the *HEC-RAS Hydraulic Reference Manual* (USACE 2016) also contains valuable insights in relation to the accurate calculation of maximum backwater from an existing or proposed bridge crossing.

Freeboard

A minimum clearance, or freeboard, should be provided between the design approach water-surface elevation and the low girder of the bridge. A provided freeboard makes allowance for hydrologic uncertainties, wave action, ice, natural, and non-natural debris.

The recommended minimum freeboard for a bridge should be determined following these guidelines:

- For a high-debris stream, freeboard should be 4 feet or more. The definition of “high-debris” will often be site specific and should be arrived at through consultation with the Hydraulic Engineer and by maximizing use of historical information, contacts with local landowners and officials, and by a thorough investigation of the debris potential in the watershed.
- For low-to-moderate debris streams, an algebraic equation was previously utilized by CDOT (2004 *Drainage Design Manual*) to arrive at the recommended freeboard. For all current and future bridge designs under the guidance of this current manual, CDOT highly encourages that a minimum freeboard of 2 feet be provided, where practical, in lieu of the algebraic method.
- The elevation of the water surface 50 to 100 feet upstream of the face of the bridge should be the elevation to which the freeboard is added to get the bottom or low-girder elevation of the bridge as a rough estimate of the location of maximum backwater.
- A more complete analysis to determine the location of maximum backwater may be obtained from the hydraulic modeling and analysis, utilizing the physical principles discussed in HDS-7. Freeboard should be added to the water-surface elevation corresponding to this location to arrive at the bottom or low-girder elevation for the bridge.
- Another important consideration with freeboard is the location of freeboard on the structure. Requirements for locating freeboard on bridges with different profile-grade configurations are given in Figures 10.2 and 10.3.
- Debris-deflector walls used to divert debris around a pier may be considered for bridges on high-debris streams. A detail of a debris wall showing acceptable dimensions is shown in Figure 10.4. An alternative to a debris wall is to extend the upstream face of the wall pier out, flush with the deck. This design does not divert the debris, but moves the debris out in front of the bridge for easier removal by maintenance personnel. Basin characteristics such as snow melt, history of maintenance-debris problems, and the timber types present in the basin, should be taken into account for the design of debris deflectors and positions of the support columns and piers. The design of debris walls must carefully take scour potential and potential effectiveness into consideration.

Other issues that need to be addressed when designing a bridge for debris are how quickly maintenance equipment can reach the structure to remove debris, and how important the route is for emergencies. All these issues must be clearly addressed in the design documentation for the structure. For concrete rigid frames and concrete box culverts, freeboard is not as important unless debris causes reduced conveyance of flow.

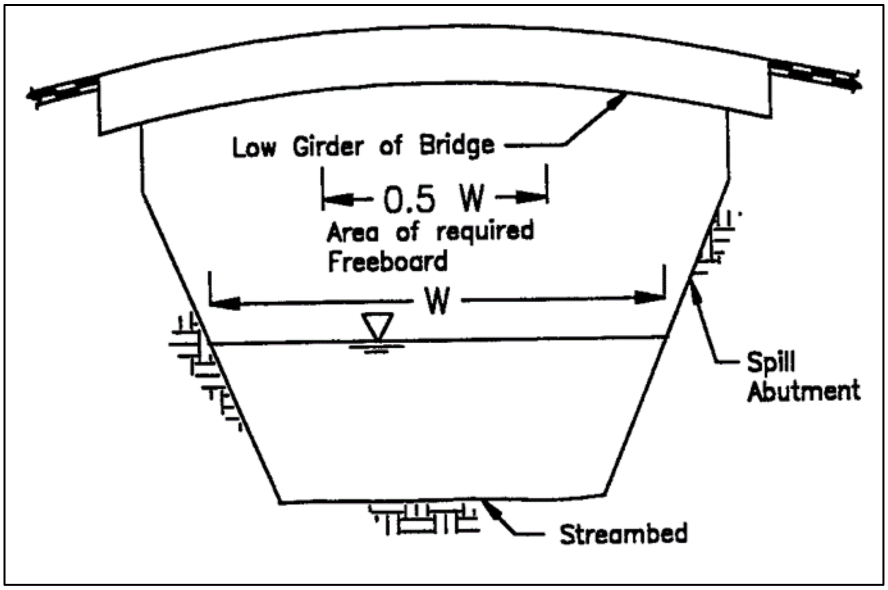


Figure 10.1 Freeboard for bridge with crest vertical curve

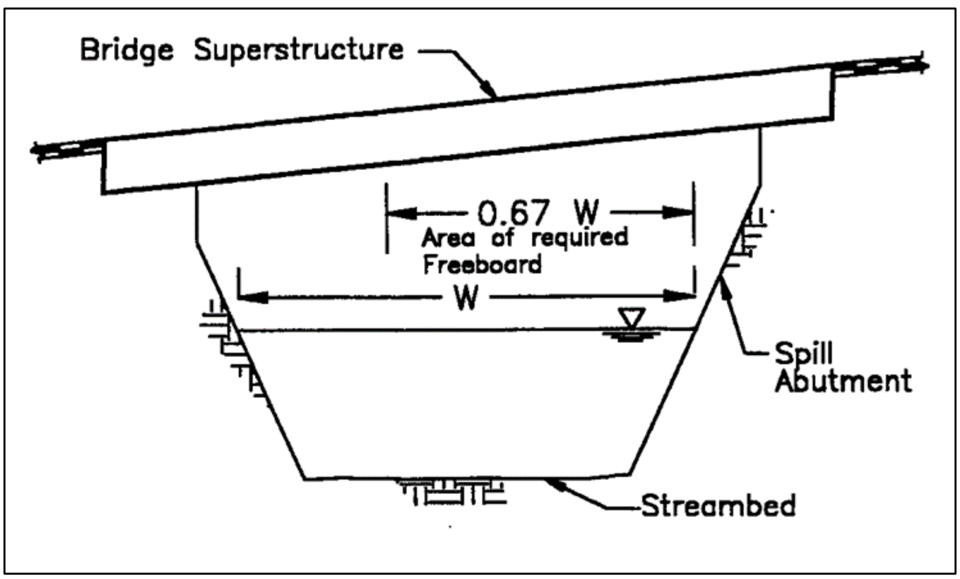


Figure 10.2 Freeboard for bridge on continuous grade

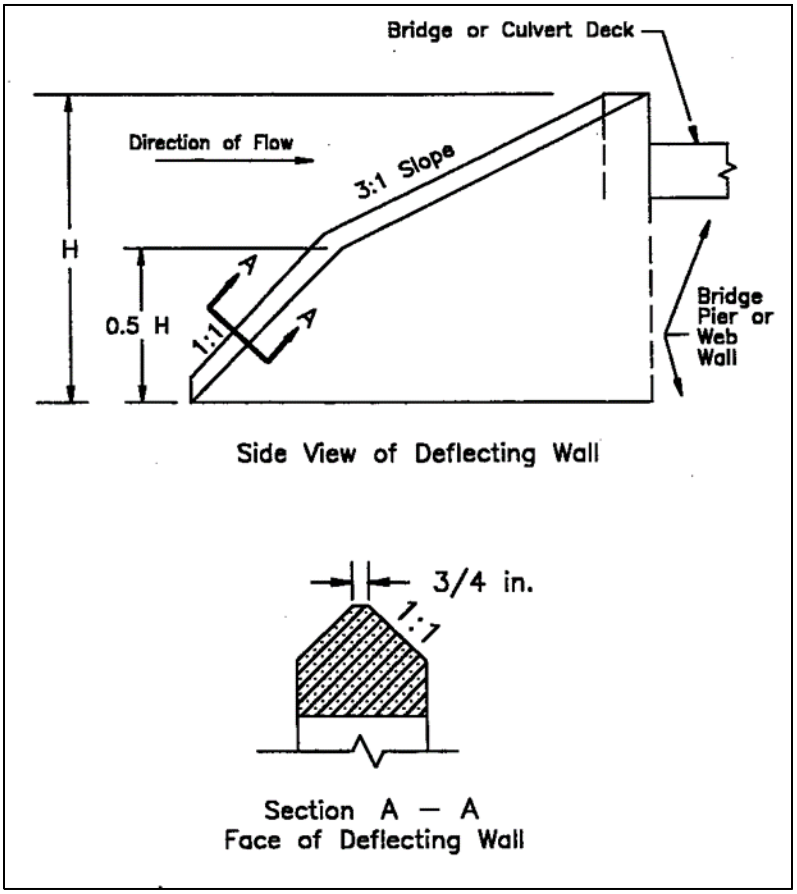


Figure 10.3 Debris deflecting wall on upstream face of bridge pier or concrete culvert web wall

Flow Distribution

An analysis should be made of the flow patterns at a proposed stream crossing to determine the flow distribution, and to establish the optimal location of the bridge opening(s). A graphical illustration of the stream flowlines going through the bridge may be made by utilizing a variety of hydraulic modeling software. A range of discharges and associated flow distributions should be investigated for any bridge design, as a bridge location may function well for one flood stage, but not at other flow stages.

10.3 HYDRAULIC ANALYSIS OF STREAM CHANNEL CROSSINGS

10.3.1 Hydraulic Nature of Stream Flow

Typical Assumptions for Natural Channels

Open channel flows are classified as steady or unsteady. Unsteady flow is further classified as rapidly or gradually varied. Additionally, flow through a stream-crossing system is subject to either free surface or pressure flow through one or more bridges, with possible roadway overtopping. An overview of hydraulic factors that affect stream crossings may be found in HEC-20 (2012), and a complete treatment is found in FHWA Hydraulic Design Series No. 6 (HDS-6, 2001) *River Engineering for Highway Encroachments*.

Most open channel flows in nature are unsteady with respect to some aspect of the flow (e.g., depth or velocity changing with time). Because unsteady-flow solutions can be very complicated and time consuming, these problems have typically been solved by assuming a steady-flow condition. The result is an approximate solution that is adequate for most types of planning or hydraulic design challenges, but may be inadequate for other types of problems (e.g., crossings of streams that have broad floodplains or highly skewed crossings).

Gradually-varied, unsteady flow creates a water-surface-profile wave with mild curvature and a gradual change in depth. In rapidly-varying unsteady flow, the change in depth is large and the curvature of the profile is very sharp. Typically, flow through a bridge is rapidly-varying, unsteady flow.

Cross Sections

In a one-dimensional hydraulic modeling approach, the geometry of streams is defined by cross-sectional coordinates of lateral distance and ground elevation that locate individual ground points. The cross sections are taken normal to the flow direction along a single, straight line where possible but, in wide floodplains or bends, it may be necessary to use a section along intersecting straight lines, i.e., a “dog-leg” section. A plot of each cross section is essential to reveal any inconsistencies or errors.

Locate cross sections to be representative of the subreaches between them. The following stream locations will require cross sections taken at shorter intervals to better model the change in conveyance:

- Major breaks in the bed profile;
- Abrupt changes in roughness or shape;
- Control sections, such as free overfalls, bends, and contractions; and
- Other abrupt changes in channel slope or conveyance.

Subdivide cross sections at vertical boundaries where there are abrupt lateral changes in geometry or roughness, or both, as for overbank flows. The conveyances of each subsection are computed separately to determine the flow distribution and are then added to determine the total flow conveyance. The subsection divisions must be chosen carefully so that the distribution of flow or conveyance is nearly uniform in each subsection.

More detailed guidance on cross-section location may be found in many technical documents, such as EM 1110-2-1003 (*Hydrographic Surveying*, USACE). A two-dimensional hydraulic model may help inform the location of 1D cross-section locations.

Calibration

A hydraulic model may be calibrated with historical highwater marks or gauged streamflow data, or both if available, to ensure that it accurately represents the local channel conditions. Use the following parameters for calibrations:

- Manning’s *n*;
- Slope;

- Discharge; and
- Cross section.

Both formal and informal data sources should be investigated to identify historical information that can be used to validate model calculations. Informal sources (e.g. bridge inspection reports and maintenance observations) should be used in addition to published gauge data and FEMA Flood Insurance Study reports. Proper calibration or model verification, when possible to perform, is ideal if accurate results are to be obtained.

In channels, the transverse variation of velocity in any cross section is a function of subsection geometry and roughness, and may vary considerably from one stage and discharge to another. It is important to know this variation for designing erosion-control measures and locating relief openings in highway fills. The best method of establishing transverse-velocity variations is by current meter measurements. If this is not feasible, the single-section method can be used by dividing the cross section into subsections of relatively uniform roughness and geometry. It is assumed that the slope of the energy grade line is the same across the cross section so that the total conveyance (K_t) of the cross section is the sum of the subsection conveyances. The total discharge is then $K_t S^{1/2}$, and the discharge in each subsection is proportional to its conveyance. The velocity in each subsection is obtained from the continuity equation, $V = Q/A$.

There may be locations where a stage-discharge relationship has already been measured in a channel. These usually exist at gauging stations on streams monitored by the United States Geological Survey (USGS). Measured stage-discharge curves will generally yield more accurate estimates of water surface elevation, and should take precedence over analytical methods.

10.3.2 Bridge Waterway Opening Analysis

The hydraulic design for a bridge waterway opening requires a comprehensive engineering approach that includes the consideration of:

- Alternatives;
- Data collection;
- Analysis;
- Selection of the most cost-effective alternative according to established criteria; and
- Documentation of the final design.

Manual calculations for the hydraulic analysis of a bridge waterway opening are impractical due to the flow complexities being simulated and the interactive, complex nature of the calculations involved. These analyses should be conducted using approved hydraulic software standard to the practice of hydraulic engineering.

Flow through bridges may be computed using a one-dimensional hydraulic model, two-dimensional models, computational fluid dynamics, or scale model flume studies. A one-dimensional approach determines the flow rate through the bridge based on the water-surface elevations at the upstream and downstream sides of the structure, and assumes steady, gradually-varied flow conditions.

Where conditions at the site deviate significantly from steady, gradually-varied flow conditions, a two-dimensional model should be considered. Examples include:

- Wide floodplains with multiple openings, particularly on skewed embankments;
- Floodplains with significant variations in roughness or complex geometry (e.g. ineffective flow areas, flow around islands, multiple channels);

- Sites where more accurate flow patterns and velocities are needed to design more cost-effective countermeasures (e.g. riprap along embankments or abutments); and
- High-risk or sensitive locations where losses and liability costs are high.

According to the recommendations given in HEC-18 (2012), almost all scour studies and bridge foundation designs for waterways should employ two-dimensional modeling or more sophisticated methods.

10.4 HYDRAULIC DESIGN

10.4.1 Design Sequence

The basic sequence for the hydraulic analysis of a bridge consists of the following:

1. Determine watershed hydrology per Chapter 7 - Hydrology.
2. Visit the site and obtain flood history from CDOT maintenance staff, bridge inspection files, and local residents. Check to see that the channel survey is adequate (see the CDOT Survey Manual). Investigate upstream and downstream for conditions affecting stream stability, such as man-made structures, significant hydraulic features, gravel-mining activities, etc.
3. Check for current effective floodplain studies and master plans, and use the effective 1D or 2D model, or model data from the study if appropriate. Often the Effective data is out of date and no longer represents the actual site conditions. The most recent survey should be used for sizing the bridge. FEMA and CWCB guidance should be consulted to determine the necessary model progression from Duplicate Effective to Corrected Effective, to Existing and finally Proposed conditions modeling, if the model is to be submitted for a Letter of Map Change (LOMC), Flood Hazard Area Delineation (FHAD), or regional or local master plan update.
4. Complete a water-surface profile analysis through the bridge reach for the existing conditions utilizing 1D or 2D hydraulic modeling, or other acceptable methodology identified in Chapter 8 – Channels.
5. The return period and design discharge for the water-surface profile analysis must be computed as discussed in Chapter 7 - Hydrology. Factors which contribute to the selection of the return period include the capacity and size of the highway, whether it is located in a rural or urban area, and the expected traffic levels. The review flood (including the scour design flood) should also be determined at this time.
6. A range of bridge-opening sizes smaller and larger than the existing channel should be analyzed and compared with the existing and natural conditions to choose the optimum bridge-channel width for the design flow.
7. Locate the bridge within the natural waterway and select a skew to best fit the alignment of the main channel and adjacent overbank. Keep skew to a minimum to reduce construction and maintenance costs. Be aware that flow patterns can change as the discharge changes – thus, historic aerial photography or ground surveys may be referenced at this stage to assist in determining the channel morphology and potential future shifts in lateral or vertical channel alignment in the vicinity of the proposed structure.
8. Assess impacts to the surrounding property and roadway for the overtopping, incipient overtopping, and 100-year flood for the various alternatives identified in step 4.
9. Conduct a preliminary scour analysis according to the guidance found in HEC-18 (2012), NCHRP publications, and the most current state of the practice.

10. Select and design any necessary revetment protection (riprap revetments, guidebanks, spur dikes, etc.) for the bridge and channel. Identify right-of-way needs if required for the construction and continued maintenance of this revetment protection.
11. For hydraulic crossings, in general and early in preliminary design, give: the preliminary channel width; elevation at excavated-channel width; skew; station at centerline of channel; recurrence interval for design event; drainage area; design discharge; 100-year discharge; 500-year discharge; minimum low-girder elevation; thalweg elevation; ordinary high-water elevation (*OHW*); design high-water elevation (*DHW*); 100-year high-water elevation; 500-year high-water elevation; design velocity (*V*); 100-year velocity; 500-year velocity; and, riprap dimensions to Staff Bridge on the Bridge Hydraulic Information Transmittal sheet. Examples of Bridge Hydraulic Information Transmittal sheets are shown in Figures 10.6 and 10.7. The bridge design engineer will use this information to evaluate how different bridge materials and configurations can be employed in order to best span the channel. From this, the bridge design engineer will complete the General Layout Sheet.
12. For hydraulic crossings at most major structures additional levels of hydraulics information are required after preliminary design. Typically, the hydraulics engineer draws in elevation view the maximum calculated total-scour envelope and differentiates the design-scour depth from the scour review discharge (often the 100- and 500-year scour depth, or incipient overtopping flood). The existing and proposed water-surface elevations are provided, determining the backwater associated with the profile and waterway opening, etc. These additional items of information must be provided to the bridge designer early in the final design. Figure 10.4 indicates the coordination required between Hydraulics, Geotechnical and Bridge Engineers.
13. After the FIR meeting and the geology report is received, the final scour profile should be completed. Refer to Section 10.4.1 for a more-detailed discussion of bridge-scour methods and requirements. The scour depth should be provided to the structural engineer and the geologist for final bridge design.
14. Complete all documentation, the Final Hydraulics Report, and the Bridge Hydraulic Information Sheets for the plans. The scour depths must be shown on the bridge-layout plan sheet. Figures 10.5 and 10.6 provide copies of the Bridge Hydraulic Information Transmittal Sheets for spill-through and vertical-wall abutments. Examples of bridge hydraulic-information plans are given at the end of this chapter.

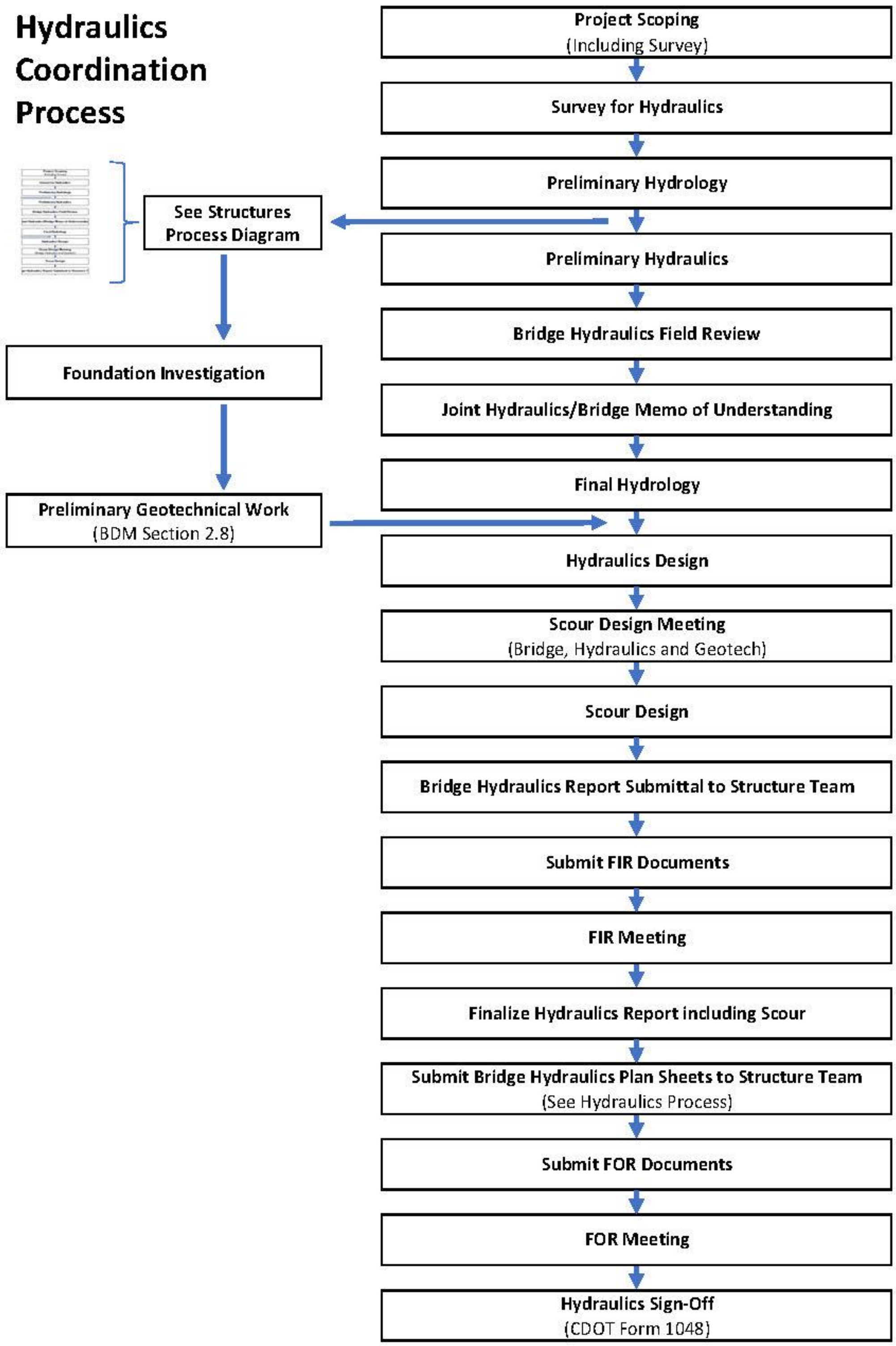


Figure 10.4 Coordination flowchart between Hydraulics, Geotechnical and Bridge Engineers.

Bridge Hydraulic Information Transmittal Sheet for Spill-Through Abutments

Below is the structure opening and hydraulic information required for the bridge across ________________ on SH _______ at/near ____.

Project Information

Date: ______________________________ Const. Project No.: ______________________________

To: ______________________________ P.E. Project No.: ______________________________

From: ______________________________ Project Name: ______________________________

Bridge Information

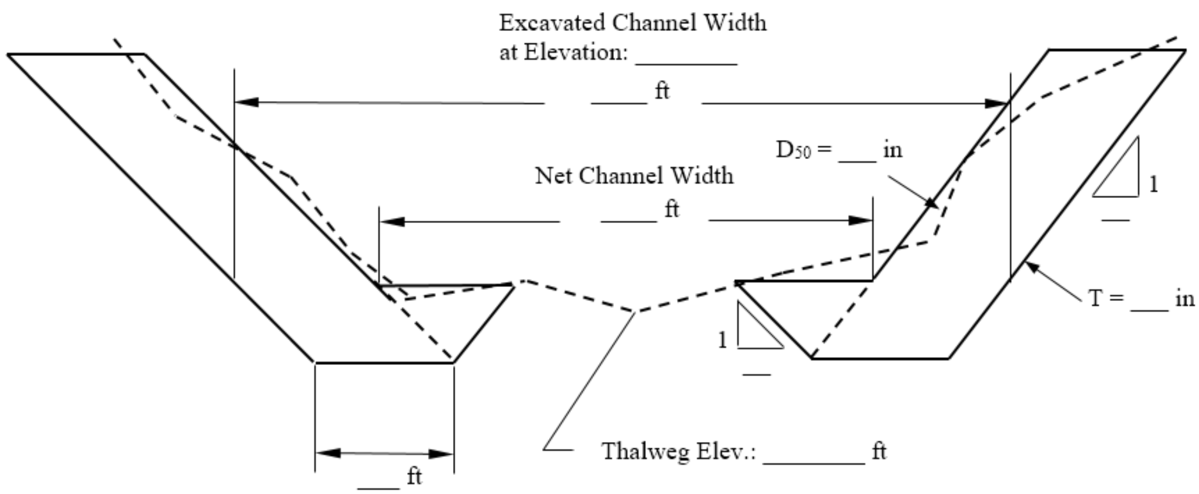
Existing Structure Number: ____

Station at Centerline of Channel: ____

Skew: ____

Minimum Low Girder Elevation: ____

Design Year Event: _____________ year recurrence.



Hydraulic Information

$D.A.$ = _____ mi^2	Q_{design} = _____ cfs	Q_{100} = _____ cfs	Q_{500} = _____ cfs
OHW = _____ ft	DHW_{design} = _____ ft	HW_{100} = _____ ft	HW_{500} = _____ ft
	V_{design} = _____ cfs	V_{100} = _____ fps	V_{500} = _____ fps

Please submit the information required by the *CDOT Drainage Design Manual* to Staff Bridge so they may proceed with design. Bridge Layout requested: yes ☐ no ☐

Comments:

____

____

____

Figure 10.5 Transmittal of bridge hydraulic information sheet for spill-through abutments

Bridge Hydraulic Information Transmittal Sheet for Vertical Wall Abutments

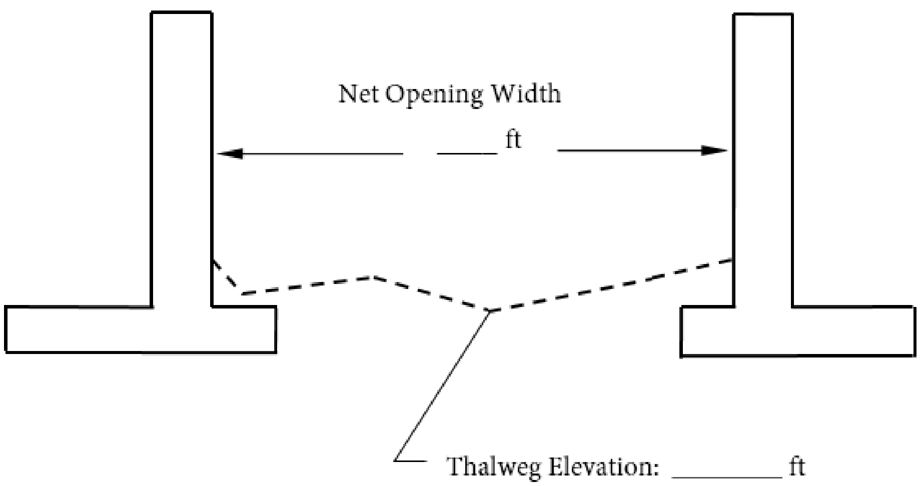
Below is the structure opening and hydraulic information required for the bridge across ________________ on SH _______ at/near ____.

Project Information

Date: ________________________________ Const. Project No.: ______________________________
To: ________________________________ P.E. Project No.: ______________________________
From: ______________________________ Project Name: ________________________________

Bridge Information

Existing Structure Number: ____
Station at Centerline of Channel: ____
Skew: ____
Minimum Low Girder Elevation: ____
Design Year Event: _____________ year recurrence.



Hydraulic Information

$D.A.$ = _____ mi^2	Q_{design} = _____ cfs	Q_{100} = _____ cfs	Q_{500} = _____ cfs
OHW = _____ ft	DHW_{design} = _____ ft	HW_{100} = _____ ft	HW_{500} = _____ ft
	V_{design} = _____ cfs	V_{100} = _____ fps	V_{500} = _____ fps

Please submit the information required by the *CDOT Drainage Design Manual* to Staff Bridge so they may proceed with design. Bridge Layout requested: yes ☐ no ☐

Comments:

____

____

____

Figure 10.6 Transmittal of bridge hydraulic information sheet for vertical-wall abutments

10.4.2 Parallel Bridges

Arrangement

Parallel bridges of nearly identical design that are placed parallel and only a short distance apart are commonly encountered in highway systems. The backwater produced by these dual bridge systems is naturally larger than that for a single bridge. Yet, it is less than the value which would result by considering the two bridges separately. Additional calibration of the hydraulic modeling of dual bridges is suggested. The *HEC-RAS Hydraulic Reference Manual* and HDS-7 (*Hydraulic Design of Safe Bridges*) both give excellent guidance on best design practices for parallel bridges.

10.5 DESIGN CONSIDERATIONS

10.5.1 Introduction

Streams are dynamic, natural systems which, as a result of the encroachment caused by elements of a stream crossing system, will respond in a way that may challenge even an experienced hydraulic engineer. The complexities of the stream response to encroachment require that:

- Hydraulic engineers must be involved from the outset in the choice of alternate stream-crossing locations; and
- At least some of the members of the engineering design team must have extensive experience in hydraulic design of stream crossing systems. Hydraulics engineers should also be involved in the solution of stream-stability problems at existing structures.

This section discusses qualitatively some of the design issues which contribute to the overall complexity of spanning a stream with a stream crossing system. A much more thorough discussion of design philosophy and design considerations is found in HDS-7 (2012).

10.5.2 Location of Stream Crossing

All bridges crossing streams must have two signs showing the name of the stream. Although many factors, including nontechnical ones, influence the final location of a stream crossing system, the hydraulics of the proposed location must have a high priority.

10.5.3 Coordination Permits/Approvals

The interests of other government agencies must be considered in the evaluation of a proposed stream crossing system, and cooperation and coordination with these agencies, especially water resources planning agencies, must be undertaken.

Designers of stream crossing systems must be aware of relevant local, State, and Federal laws and permit requirements. Permits for construction activities in waters of the United States are under the jurisdiction of the U.S. Army Corps of Engineers. Applications for Federal permits may require environmental impact assessments under the National Environmental Policy Act of 1969 (NEPA). In Colorado, provisions of Senate Bill 40 must be addressed on any stream crossing (see Chapter 2 – Legal Aspects for more information).

Colorado Parks and Wildlife often has interests in CDOT activities where an active fish population is present, and may have requirements or suggestions on how best to accommodate the fauna of a given site.

10.5.4 Deck Drainage

Bridge hydraulic design should include deck drainage in order to protect public safety and maximize bridge service life. Improperly-drained bridge decks can cause numerous problems including hydroplaning, icing, and corrosion.

A bridge deck drainage system includes the bridge deck, sidewalks, railings, gutters, and inlets (or scuppers). The primary objective of the drainage system is to remove runoff from the bridge deck before it collects in the gutter to a point that exceeds the allowable design spread. Proper bridge deck drainage provides many other benefits, including:

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

Spread width criteria for bridges are presented in Chapter 13 – Storm Drains. Superelevation transitions may cause cross-flow and ponding problems on a bridge deck, and should be avoided. If a superelevation transition cannot be avoided it should be mitigated by placing inlets upslope of the transition.

Whenever possible, bridge decks should be watertight, and all deck drainage should be carried to the ends of the bridge. It is usually desired to capture deck drainage before reaching the expansion joints. Drains at the end of the bridge should have sufficient inlet capacity to carry all of the minor drainage. A curb roll is required from the bridge ends to the end of the guard rail. At the end of the curb roll an inlet and pipe (preferred design), or a well-depressed rundown with a transition from the curb roll is required to convey the drainage down the fill slope.

HEC-21, *Design of Bridge Deck Drainage* (FHWA), should be referenced for additional design considerations.

Even short-span bridges should provide storm drains at both ends of the bridge, to minimize flow onto the bridge. Combination curb opening and grated inlets should be used.

10.5.5 Auxiliary/Relief Opening

The need for auxiliary waterway openings, or relief openings, arises on streams with wide floodplains. The purpose of openings on the waterway overbank is to pass a portion of the flood flow without contracting it through the main channel opening when the stream reaches flood stage. They do not provide relief for the principal waterway opening in the manner that an emergency spillway at a dam does, but they have predictable design capacity during flood events. However, the hydraulics engineer should be aware that the presence of overtopping or relief openings may not result in a significant reduction in flow through the principal bridge opening and may concentrate flow at undesirable locations.

Basic objectives in choosing the location of auxiliary openings include:

- Maintenance of flow distribution and flow patterns;
- Accommodation of relatively large-flow concentrations on the overbanks;

- Avoidance of floodplain flow along the roadway embankment for long distances;
- Including crossings of significant tributary channels;
- Accomodation of eccentric stream crossings to provide for drainage; and
- Accommodation of requests from Colorado Parks and Wildlife to minimize flows for the benefit of wildlife.

The technological weakness in modeling auxiliary openings is in the use of one-dimensional models to analyze two-dimensional flow. The use of two-dimensional models is more appropriate for analysis of complex stream crossing systems.

The most complex factor in designing auxiliary openings is determining the division of flow between two or more structures. If incorrectly proportioned, one or more of the structures may be overtaxed during a flood event, with possible damage to the structure and downstream property. Auxiliary openings should be designed conservatively large to guard against that possibility, though the employment of two-dimensional modeling should inform the design to minimize the need for an overly conservative design.

10.5.6 Bridge Rehabilitation

Often an existing bridge over a drainageway only needs widening and rehabilitation. For these types of bridges the hydraulic engineer must consider the same design criteria as for a completely new structure. A multi-disciplinary team should evaluate whether it is cost effective to rehabilitate the existing bridge, or replace it with a new bridge.

The most important issues to be addressed are very similar to those present for a full bridge replacement, as noted above. The decision to replace or rehabilitate a bridge is based on engineering judgement, including a detailed cost benefit analysis if warranted by the complexity of the situation. Any hydraulic analysis on rehabilitated bridges should be well-documented and coordinated with the Structural Engineer or Geotechnical Engineer. Bridges to be rehabilitated which are over drainageways may have a detailed hydraulic analysis performed as part of the design process, based in part on the scope of the rehabilitation.

10.6 BRIDGE SCOUR

10.6.1 Introduction

Hydraulic analysis of a bridge requires an assessment of the proposed bridge's vulnerability to scour. Because of the unacceptable hazard to life and property, in addition to the economic hardships, posed by a catastrophic bridge collapse, special consideration must be given to the scour and foundation analysis of any proposed bridge. Since the study of scour prediction and analysis is relatively new and constantly changing, the hydraulic engineer should always be aware of and use the most current scour-predicting methodologies. Designers should consult FHWA Hydraulic Engineering Circular No. 18 (HEC 18) *Evaluating Scour at Bridges*, Hydraulic Engineering Circular No. 20 (HEC 20), *Stream Stability at Highway Structures*, and Hydraulic Engineer Circular No. 23 (HEC 23), *Bridge Scour and Stream Instability Countermeasures: Experience, Selection and Design Guidance*, for a more thorough treatment of scour-prediction methodologies and countermeasure design. The NCHRP publications are also excellent supplementary sources where recent advances in scour estimation are reported upon.

After the bridge waterway opening has been established, the hydraulics engineer should evaluate the estimated scour that will occur at each of the bridge elements. This section discusses this evaluation. For most new bridges, pier scour will be accommodated by adjusting the pier design in coordination with the geotechnical and structural engineers. Abutment scour can be mitigated with scour countermeasures. However, the most cost-effective design may be to modify the opening to reduce the amount of scour and the cost of the scour countermeasures. Considerable judgment and consideration will be necessary to make this determination. For existing bridges, pier and abutment scour are mitigated with hydraulic or structural-scour countermeasures and monitoring.

A complete analysis of stream stability requires a multilevel solution procedure involving hydraulics, bridge, and geotechnical staff. The evaluation and design of a highway stream crossing or encroachment should begin with a qualitative assessment of stream stability. This involves application of geomorphic concepts to identify potential problems and alternate solutions. This analysis should be followed by a quantitative analysis using hydrologic, hydraulic, and sediment-transport engineering concepts. Such analyses should include evaluation of flood history, channel hydraulic conditions (water-surface profile analysis), and basic sediment-transport analyses (watershed-sediment transport, incipient-motion analysis, and scour calculations). An analysis of this type is adequate for most locations in Colorado. If not, a more complex quantitative analysis based on detailed mathematical modeling and/or physical hydraulic models should be considered. This multilevel approach is presented in HEC-20.

Additional geotechnical data needs, aside from the typical abutment and channel borings, must be identified by the hydraulics engineer.

10.6.2 Scour Types

Bridge scour must be evaluated as interrelated components. The major components of scour are:

- Contraction scour;
- Pressure scour;
- Local scour (pier and abutment);
- Aggradation and degradation; and
- Planform changes (bendway scour, vertical wall scour, etc.).

Contraction Scour

Contraction scour results from a constriction of the flow area caused by approach fills in the floodplain or, to a lesser extent, by bridge piers in the waterway. This contraction generally causes flow to accelerate, increasing the flow's erosive strength. Contraction scour is calculated using a contracted section and fully uncontracted upstream approach-section method in HEC-18 (2012), and contributes to the calculation of total scour at a bridge.

Pressure Flow Scour

Pressure flow, also known as orifice or sluice gate flow, occurs when the water surface elevation at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure. Pressure flow under a bridge results from a buildup of water on the upstream bridge face and plunging of the flow downward and under the bridge. At higher approach-flow depths, a bridge can be entirely submerged with the resulting flow being a complex combination of plunging flow under the bridge and weir flow over the bridge. Guidance on determining fully-sealed pressure flow and submerged backwater flow is available in the HEC-RAS *Hydraulics Reference Manual* (2016).

With pressure flow, the contraction-scour depths beneath the bridge deck are larger than for free-surface flow with similar depths and approach velocities. The increase in contraction scour subject to pressure flow results from the flow being directed downwards, toward the bed, by the superstructure, and by increasing the intensity of the horseshoe vortex. The vertical contraction of flow is a more significant cause of the increase in scour depths, and contributes to the calculation of contraction scour (and therefore total scour) at a bridge.

However, in many cases, when a bridge becomes submerged, the average velocity under it is reduced due to a reduction of discharge that must pass under the bridge as a result of weir flow over the bridge and approach embankments. As a consequence, increases in local scour attributed to pressure-flow scour at a particular site may be offset slightly. The effects of this type of condition should be considered in the design process. Refer to HEC-18 and NCHRP published documentation for more information pertaining to the determination of pressure flow scour. It should be noted that in the case of pressure flow conditions, larger of the pressure scour or contraction scour is used in total scour computations.

Local Scour

All abutments and piers located within the flood-flow prism increase the potential scour hazard at a bridge site. The potential scour caused by these features is termed local scour, and is a function of the geometry of these features as they relate to the flow geometry. Local scour occurs around piers, abutments, the end of guidebanks, and any other obstructions to flow. It results from turbulence and changes in local velocity vectors caused by these obstructions.

Aggradation and Degradation

Long-term profile changes can occur from aggradation, degradation, or both. Aggradation is the deposition of bed load due to a decrease in stream-sediment transport capacity that results from a reduction in the energy gradient, or an increase in the sediment load. Degradation is the removal of bed material due to increased stream-sediment transport capacity that results from an increase in the energy gradient, or a decrease in the sediment load. Degradation and aggradation can occur over long periods of time and during non-flood events and should be considered as imposing a future permanent change for the stream bed elevation at a bridge site (see HEC-20). For most bridges, this determination will be made as a part of the stream stability assessment.

History of streambed elevation changes at existing bridge sites is documented in the inspection records maintained by the Bridge Inspection Unit, Staff Bridge Branch. These inspection records should always be obtained and taken into consideration when a scour analysis is being conducted. Published flood studies will occasionally include information regarding the tendency of streams to aggrade and/or degrade in certain areas.

Plan Form Changes

Plan form changes are lateral morphological changes such as meander migrations or bank widening. The lateral movement of meanders can threaten bridge approaches, as well as increase scour by changing flow patterns approaching a bridge opening, especially angles of attack at piers. The comparison of past and present aerial photographs over a series of decades is helpful in understanding the plan-form behavior of a river.

Channel shifting can cause a significant change in the distribution of flow in the main channel upstream from a bridge. The change in flow pattern may alter the angle of attack on bridge piers and abutments, which will cause additional local scour.

10.6.3 Scour Design Flood Philosophy and Concepts

Bridge foundations for new structures should be designed to withstand scour caused by floods larger than the design flood. Typically, bridges are designed to accommodate scour associated with 100-year flood hydraulic conditions and checked with the 500-year flood event, incipient overtopping, or maximum scour causing event.

FHWA supports use of a risk-based and data-driven approach to bridge program and other infrastructure initiatives as implemented in the FHWA Scour Program. A risk-based approach considers economic consequences of failure, while providing safe, reliable waterway crossings.

Table 10.1 provides recommended **minimum-scour** design-flood frequencies, and scour-design check-flood frequencies based on hydraulic-design flood frequencies. These guidelines may be used for off-system bridges which are often designed to pass smaller design frequency flood events.

Table 10.1 Hydraulic design, scour design, and scour design check flood frequencies

Hydraulic Design Flood Frequency, Q_D	Scour Design Flood Frequency, Q_S	Scour Design Check Flood Frequency, Q_C
Q_{10}	Q_{25}	Q_{50}
Q_{25}	Q_{50}	Q_{100}
Q_{50}	Q_{100}	Q_{200}
Q_{100}	Q_{200}	Q_{500}

Source: FHWA HEC 18

The hydraulic-design flood frequencies above contain an inherent level of risk. There is a direct relation between risk assumed to be acceptable at the bridge (defined by FHWA standards), and the frequency of the floods the bridges are designed to accommodate.

Scour-design flood frequencies are larger than hydraulic-design flood frequencies because there is a high probability that the hydraulic-design flood will be exceeded during the service life of the bridge. As such, a bridge must be designed to a higher level for scour than for hydraulic design. If the hydraulic-design flood is exceeded, greater scour may occur and lead to bridge failure. Designing for a higher level of scour than the hydraulic-design flood provides a measure of redundancy after a hydraulic-design flood occurs. Scour-design check flood frequencies are larger than scour-design flood frequencies for the same reasons.

If there is a flood event greater than the hydraulic-design flood, but less than the scour-design flood, that causes greater scour than the scour-design flood (i.e. emergent roadway overtopping flow) this event should be used as the scour-design flood. Under these conditions, the scour-design check flood should still be analyzed and included in the results in addition to the event causing maximum scour.

Similarly, if there is a flood event greater than the scour-design flood but smaller than the scour-design check flood and it causes greater scour than the scour-design flood, this event should be used as the scour-design check flood.

Scour-design considerations include:

- For all designs, scour must not cause failure of a bridge structure for the 500-year flood.

- All designs should be cross-checked for pressure-flow and incipient overtopping-flood conditions, since these are commonly the highest total-scour events by calculation and by historic observation.
- Prior to the completion of the final geology report, hydraulics, geotechnical, and structural engineers must meet after borings are taken to assess the validity of scour-depth calculations.
- Riprap only mitigates or lessens the amount of scour. Scour is calculated in the absence of riprap. For design, riprap cannot be assumed to completely eliminate scour and should not be used as a substitute for adequate bridge capacity.
- Riprapped guidebanks are acceptable to mitigate abutment scour and some cases of pier scour. With a guidebank, the abutment scour will move upstream from the bridge to the upstream toe of the guidebank. For a guidebank to be effective, riprap on the bank must be continuously maintained. The predicted-scour depth will dictate the elevation and amount of riprap to be used (see Chapter 17 and HEC-23).
- Under certain situations, a flood less than a 500-year flood could cause the worst-case scour conditions. Overtopping and incipient overtopping floods should be evaluated along with the 500-year scour as these are commonly the highest total-scour events by calculation and by historic observation.. The worst-case scour condition with no mitigation should use the discharge of greatest potential total scour for the bridge foundation design.

10.6.4 Assessing and Plotting Scour

The procedures and guidelines outlined in HEC-18 (2012) should be used to compute and assess bridge scour. Examples of scour calculations and a procedure to plot scour depths are included in the FHWA document.

A plot of scour depths corresponding to the design flow and the 500-year discharge (or maximum total-scour flood event) must be included in the design plans. Scour is usually plotted as part of the Bridge Hydraulic Information sheet and the Bridge General Layout Sheet.

10.6.5 Natural Armoring

Natural armoring occurs because a stream or river is unable during a major flood to move the more-coarse material comprising either the bed, or if some bed-scour occurs, its underlying material. Scour may occur initially, but later become arrested by armoring before the full scour potential is reached for a given flood magnitude. When armoring occurs, the coarser bed material will tend to remain in place or quickly redeposit, forming a layer of riprap like armor on the stream bed or in the scour holes, limiting further scour for a particular discharge. This armoring effect can decrease scour-hole depths which have been predicted based on formulae developed for sand or other fine-material channels. When a larger flood occurs than used to cause the previous scour-hole depths, resultant scour will likely penetrate deeper until armoring again occurs at some lower threshold.

If armoring of a streambed occurs, the stream may widen its banks to maintain continuity of sediment transport. Bank widening encourages rivers or streams to form a more unstable, braided regime. Such instabilities may pose serious problems for bridges as they encourage stream migration. Bank widening also spreads the approach-flow distribution, which results in a more-severe bridge-opening contraction.

10.6.6 Scour-Resistant Materials

Caution is necessary in determining the scour resistance of bed materials and underlying strata. With non-cohesive material, the passage of a single flood may result in the predicted scour depths. Conversely, in scour-resistant material the maximum predicted depth of scour may not be realized during the passage of a particular flood, although some scour-resistant material may be lost. Commonly, this material is replaced with more-easily scoured material. At some later flood the predicted-scour depth then may be reached. Serious scour has been observed to occur in materials commonly perceived to be scour-resistant, such as consolidated soils and glacial till, as well as bedrock streams, and streams with gravel and boulder beds. Additional guidance on scour effects on materials can be found in HEC-18. Additional guidance may be found in NCHRP Report 717, *Scour at Bridge Foundations on Rock*.

Where bedrock is above the calculated-scour depth, an evaluation must be made of the bedrock's scour-resistance (i.e. rock-life) by a multidisciplinary design team, consisting of the hydraulic engineer, the geotechnical engineer, and the structural engineer. The mutidisciplinary design team must determine the resistance to scour considering the following:

- Experience in the project area;
- Uniformity of the bedrock material;
- Type of foundation and its effect on the bedrock. Blasting for excavation of spread footers and driving piling may fracture the bedrock and should be avoided;
- Evaluation of undisturbed core samples considering:
    i. Rock quality designation;
    ii. Unconfined compressive strength; and
    iii. Orientation and condition of natural jointing or fractures in the core sample;
- Relative duration of the scouring flood. A 500-year snowmelt flood may last for weeks, large-basin rainfall floods may last days, a rainfall flood may only last several hours; and
- Depth of bedrock to channel invert, and frequency of bedrock exposure to scour and air. Wet dry cycles in shale can reduce it to highly erodible particles.

10.6.7 Preventive/Protection Measures

Based on an assessment of potential scour provided by the hydraulic engineer, scour countermeasures can be incorporated to prevent or mitigate scour damage at the superstructure foundations. In designing the superstructure, spread footings should be used only where the stream bed is extremely stable below the footing, or where the spread footing is founded at a depth below the maximum scour. Rock riprap can be used, if sufficient size and density is available, to armor abutment fill slopes and the area around the base of piers. Riprap, and additional scour countermeasure design information is presented in HEC-23 (2009).

Whenever possible, clearing of vegetation upstream and downstream of the toe of the embankment slope should be avoided. Embankment overtopping creates a hazardous condition for the traveling public and is the lead cause of flood deaths in the United States. It should be avoided in design whenever possible. Guidebanks are recommended to align the approach flow with the bridge opening, and to prevent scour around the abutments. Guidebanks, embankments, and abutments must be protected by adequately-sized rock riprap, or with other revetment types approved by CDOT.

10.6.8 The Countermeasure Matrix

Selecting scour countermeasures for a bridge requires evaluating several alternatives. These may include hydraulic, structural and biotechnical countermeasures, or monitoring, individually or in combination. When selecting appropriate scour-countermeasure alternatives, a useful matrix that describes various countermeasures and their attributes has been developed by FHWA. Table 10.2 contains the Stream Stability and Bridge Scour Countermeasures Matrix.

The countermeasure matrix highlights groups of commonly-used scour countermeasures and their individual characteristics. It lists information functional applicability, suitability to specific river environments, general level of maintenance required, and state DOTs that experience with specific countermeasures.

For additional information on scour countermeasures, refer to HEC-23 (2009).

10.7 SOFTWARE FOR ANALYZING BRIDGE HYDRAULICS

Software is available for computing the hydraulics of bridge waterways using both one-dimensional and two-dimensional models, and is detailed in Chapter 8 – Channels.

Table 10.2 Stream instability and bridge scour countermeasures matrix

Countermeasure Group	Countermeasure Characteristics: FUNCTIONAL APPLICATIONS: Local Scour: Abutments	Local Scour: Piers[4]	Contraction Scour: Floodplain and Channel	Stream Instability: Vertical	Stream Instability: Lateral	Overtopping Flow: Approach Embankments	SUITABLE RIVER ENVIRONMENT: River Type (B = braided, M = meandering, S = straight)	Stream Size (W = wide, M = moderate, S = small)	Bend Radius (L = long, M = moderate, S = short)	Velocity (F = fast, M = moderate, S = slow)	Bed Material (C = coarse bed, S = sand bed, F = fine bed)	Ice/Debris Load (H = high, M = moderate, L = low)	Bank Slope (V = very steep, S = steep, M = mild)	Floodplain (W = wide, M = moderate, N = narrow/none)	MAINTENANCE: Estimated Allocation of Resources (H = high, M = moderate, L = low)	INSTALLATION EXPERIENCE BY STATE	DESIGN GUIDELINE REFERENCE*
GROUP 1. HYDRAULIC COUNTERMEASURES																	
GROUP 1.A. RIVER TRAINING STRUCTURES																	
TRANSVERSE STRUCTURES																	
Impermeable spurs (jetties, groins, wing dams)	◗	◗	○	○	●	○	B, M	W, M	L, M	✓	✓	✓	✓	✓	M - L	Widely Used	DG 2
Permeable spurs (fences, netting)	◗	◗	○	○	●	○	B, M	W, M	L, M	M, S	S, F	L	✓	✓	H - M	Widely Used	DG 2
Transverse dikes	○	○	○	○	●	○	B, M	W, M	✓	✓	✓	✓	✓	✓	M - L	NE	
Bendway weirs/Stream barbs[1]	◗	◗	○	○	●	○	M	✓	M, S	✓	✓	✓	✓	✓	L	CO, ID, IL, MO, MT, OR, WA	DG 1
Hardpoints	○	○	○	○	●	○	✓	✓	✓	✓	✓	✓	✓	✓	L	CA, ND, NE, SD	CH 8
Drop structures (check dams, grade control)	◗	◗	◗	●	○	○	✓	✓	✓	✓	✓	✓	✓	✓	M	Widely Used	DG 3
Embankment Spurs	◗	○	◗	○	○	○	✓	✓	✓	✓	✓	✓	✓	W	L	AK, OK	
LONGITUDINAL STRUCTURES																	
Longitudinal dikes (crib/rock toe/embankments)	◗	○	○	○	●	◗	✓	✓	L, M	✓	✓	M, L	✓	✓	M - L	AK, AZ, CA, OK, OR, MS	CH 8
Retards	◗	○	○	○	●	○	✓	✓	L, M	✓	S, F	L	✓	✓	H - M	Widely Used	CH 8
Bulkheads	●	○	○	○	●	○	✓	✓	✓	✓	✓	✓	V, S	✓	M	Widely Used	CH 8
Guide banks	●	◗	◗	○	◗	◗	✓	W, M	✓	✓	✓	✓	✓	W, M	M - L	Widely Used	DG 15
AREAL STRUCTURES/TREATMENTS																	
Jacks/tetrahedron jetty fields	○	○	○	○	●	○	B, M	W, M	L	M, S	S, F	M, L	✓	W, M	M	Widely Used	CH 8
Vanes	○	◗	○	○	●	○	B, M	W, M	L, M	M, S	S, F	L	✓	✓	H - M	IA	
Channelization	◗	◗	○	○	●	○	B, M	✓	✓	✓	✓	✓	✓	✓	M	MS, MO, MT, TX	CH 8
Flow relief (overflow, relief bridge)	◗	◗	●	○	○	●	✓	✓	✓	✓	✓	✓	✓	W	M	Widely Used	
Sediment detention basin	○	○	○	●	○	○	✓	✓	✓	✓	C, S	✓	✓	✓	H - M	Widely Used	
GROUP 1.B. ARMORING COUNTERMEASURES																	
REVETMENTS AND BED ARMOR																	
Rigid																	
Soil cement	●	●	◗	◗	●	●	✓	✓	✓	✓	S, F	✓	✓	✓	L	AZ, CO, NM	DG 7
Roller compacted concrete	●	◗	●	●	●	●	✓	✓	✓	✓	S, F	✓	✓	✓	L	Widely Used	
Concrete pavement	◗	○	●	◗	●	◗	✓	✓	✓	✓	✓	✓	S, M	✓	M	Widely Used	
Rigid grout filled mattress/concrete fabric mat	◗	○	◗	◗	●	◗	✓	✓	✓	✓	✓	✓	S, M	✓	M	GA, MA, MD, ME, SD, WA	
Fully grouted riprap	○	○	○	○	◗	○	✓	✓	✓	✓	✓	✓	S, M	✓	M	AZ, CA, CT, ME, MI, TN	CH 5
Flexible/articulating																	
Riprap	●	●	◗	◗	●	◗	✓	✓	✓	✓	✓	✓	S, M	✓	M	Widely Used	DG 4
Self launching riprap (windrow)	○	○	○	○	◗	○	✓	✓	✓	✓	C,S	✓	V, S	✓	H - M	GA, CA, IL, PA	DG 4
Riprap fill-trench	◗	○	○	○	●	○	✓	✓	✓	✓	✓	✓	✓	✓	M	Widely Used	DG 4
Gabions/gabion mattress[2]	●	●	◗	◗	●	◗	✓	✓	✓	✓	S, F	M, L	✓	✓	M	Widely Used	DG 11
Wire enclosed riprap mattress (rail bank/sausage)	●	○	○	○	●	○	✓	✓	✓	M, S	S, F	M, L	S, M	✓	M	AZ, CO, NM	DG 6
Articulated blocks (interlocking and/or cable tied)	●	●	●	◗	●	●	✓	✓	✓	✓	✓	✓	S, M	✓	M - L	Widely Used	DG 9
Concrete/grout mattress (fabric-formed)	●	◗	●	◗	●	◗	✓	✓	✓	M, S	✓	✓	S, M	✓	M - L	OR, CA, IA, IL, AZ	DG 10
Partially grouted riprap	●	●	◗	◗	●	○	✓	✓	✓	✓	✓	✓	S, M	✓	L	European practice	DG 12
LOCAL SCOUR ARMORING																	
Riprap (fill/apron)	●	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	S, M	✓	H - M	Widely Used	DG 8, DG 14
Fully grouted riprap	◗	○	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	S, M	✓	H - M	Widely Used	CH 5
Concrete armor units (Toskanes, tetrapods, etc.)[3]	◗	◗	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	M	✓	M - L	AZ, PA, NY, VA	CH 5
Grout filled bags/sand cement bags	●	◗	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	M, L	M	✓	H - M	Widely Used	DG 13
Gabions/gabion mattress[2]	●	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	S, F	M, L	S, M	✓	M	FL, WA, TN, OR	DG 11
Articulated blocks (interlocking and/or cable tied)	●	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	S, M	✓	M - L	Widely Used	DG 9
Sheet pile/cofferdam	◗	◗	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	M - L	CA, CT, FL, NH, WA	
Partially grouted riprap	●	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	S, M	✓	L	European practice	DG 12

● well suited/primary use
◗ possible application/secondary use
○ unsuitable/rarely used
N/A not applicable

✓ suitable for the full range of the characteristic

DG# HEC-23 Design Guideline
CH# HEC-23 Chapter

Table 10.2 Stream instability and bridge scour countermeasures matrix (continued)

Countermeasure Group	Countermeasure Characteristics															INSTALLATION EXPERIENCE BY STATE	DESIGN GUIDELINE REFERENCE*
	FUNCTIONAL APPLICATIONS						SUITABLE RIVER ENVIRONMENT								MAINTENANCE		
	Local Scour		Contraction Scour	Stream Instability		Overtopping Flow	River Type	Stream Size	Bend Radius	Velocity	Bed Material[5]	Ice/Debris Load	Bank Slope	Floodplain	Estimated Allocation of Resources		
	Abutments	Piers[4]	Floodplain and Channel	Vertical	Lateral	Approach Embankments	B = braided M = meandering S = straight	W = wide M = moderate S = small	L = long M = moderate S = short	F = fast M = moderate S = slow	C = coarse bed S = sand bed F = fine bed	H = high M = moderate L = low	V = very steep S = steep M = mild	W = wide M = moderate N = narrow/none	H = high M = moderate L = low		
GROUP 2. STRUCTURAL COUNTERMEASURES																	
FOUNDATION STRENGTHENING																	
Crutch bents/Underpinning	○	●	●	●	◗	N/A	✓	✓	✓	✓	✓	✓	✓	✓	L	FL, NC, OR, TX	
Cross bracing	○	●	●	●	○	N/A	✓	✓	✓	✓	✓	✓	✓	✓	L	NC, FL, LA	
Continuous spans	○	●	●	●	○	N/A	✓	✓	✓	✓	✓	✓	✓	✓	L	NC	
Pumped concrete/grout under footing	●	●	◗	◗	◗	N/A	✓	✓	✓	✓	✓	✓	✓	✓	M	Widely Used	DG 13
Lower foundation	●	●	●	●	●	N/A	✓	✓	✓	✓	✓	✓	✓	✓	L	CA, OR, TX	
PIER GEOMETRY MODIFICATION																	
Extended footings	N/A	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	L	Widely Used	
Pier shape modifications	N/A	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	M	FL	
Debris deflectors	N/A	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	H - M	CA, FL, NM, OR	
Sacrificial piles/dolphins	N/A	●	N/A	N/A	N/A	N/A	✓	✓	✓	✓	✓	✓	✓	✓	H - M		
GROUP 3. BIOTECHNICAL COUNTERMEASURES[5]																	
Vegetated geosynthetic products	○	○	○	○	●	◗	M, S	M, S	✓	M, S	✓	M, L	M, S	✓	H - M	Widely Used	CH 6
Fascines/woody mats	○	○	○	○	●	○	✓	M, S	✓	M, S	✓	L	M, S	✓	H - M	Widely Used	CH 6
Vegetated riprap	○	○	○	○	●	◗	✓	✓	✓	✓	✓	✓	M, S	✓	M - L	Widely Used	CH 6
Root wads	○	○	○	○	●	○	✓	M, S	✓	M, S	✓	L	M	✓	H - M	Widely Used	CH 6
Live staking	○	○	○	○	●	○	✓	M, S	✓	M, S	✓	M, L	M, S	✓	H - M	Widely Used	CH 6
GROUP 4. MONITORING																	
FIXED INSTRUMENTATION																	
Sonar scour monitor	◗	●	●	●	◗	○	✓	✓	✓	✓	✓	L	✓	✓	M	CO, FL, IN, NY, VA, TX	CH 9
Magnetic sliding collar	●	●	●	●	◗	○	✓	✓	✓	✓	S, F	✓	✓	✓	M	Widely Used	CH 9
Float out device	●	●	●	●	●	●	✓	✓	✓	✓	S, F	✓	✓	✓	L	AZ, CA, NV	CH 9
Sounding rods	◗	●	●	●	◗	○	✓	✓	✓	M, S	C	M, L	✓	✓	H	AR, IA, NY	CH 9
PORTABLE INSTRUMENTATION																	
Physical probes	●	●	●	●	●	○	✓	✓	✓	M, S	✓	M, L	✓	✓	L	Widely Used	CH 9
Sonar probes	●	●	●	●	●	○	✓	✓	✓	M, S	✓	L	✓	✓	L	Widely Used	CH 9
VISUAL MONITORING																	
Periodic Inspection	●	●	●	●	●	●	✓	✓	✓	✓	✓	M, L	✓	✓	H	Widely Used	CH 2
Flood watch	●	●	●	●	●	●	✓	✓	✓	✓	✓	M, L	✓	✓	H	Widely Used	CH 2

● well suited/primary use
◗ possible application/secondary use
○ unsuitable/rarely used
N/A not applicable

✓ suitable for the full range of the characteristic

DG# HEC-23 Design Guideline
CH# HEC-23 Chapter

NOTES:

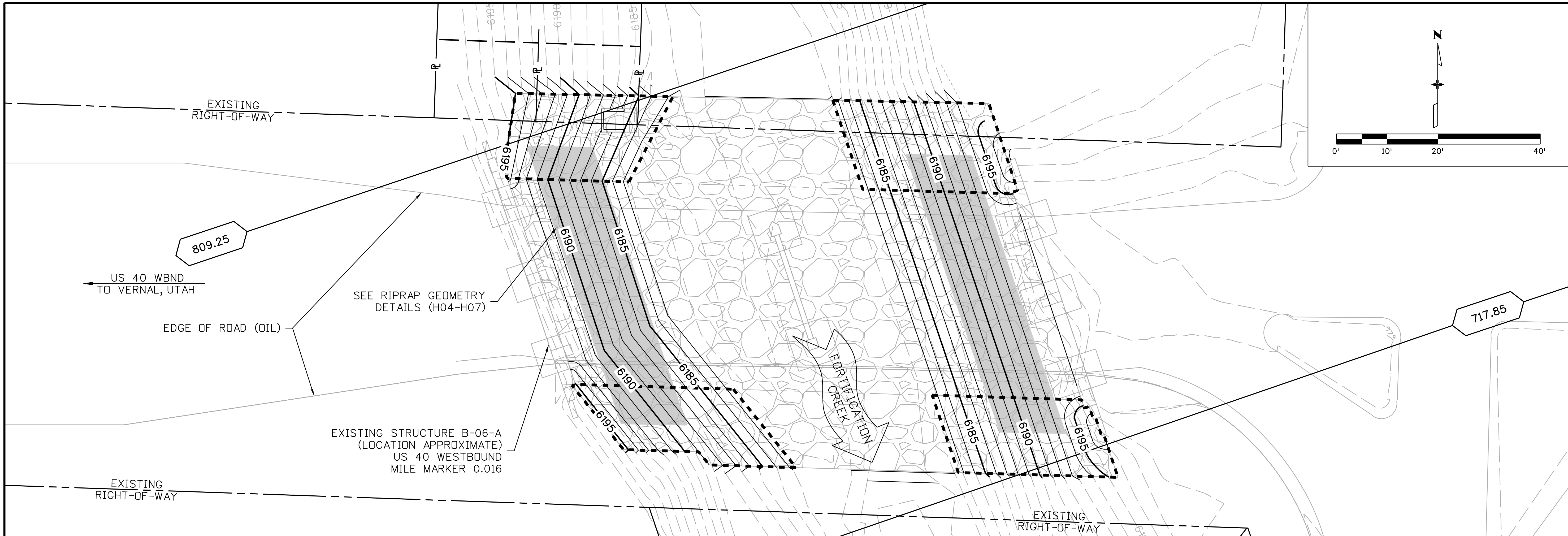
1. There is limited but successful field experience using bendway weirs/stream barbs as stream instability countermeasures.

2. Performance of welded vs. twisted wire, and PVC coated vs. uncoated wire gabions is not distinguished in the matrix.

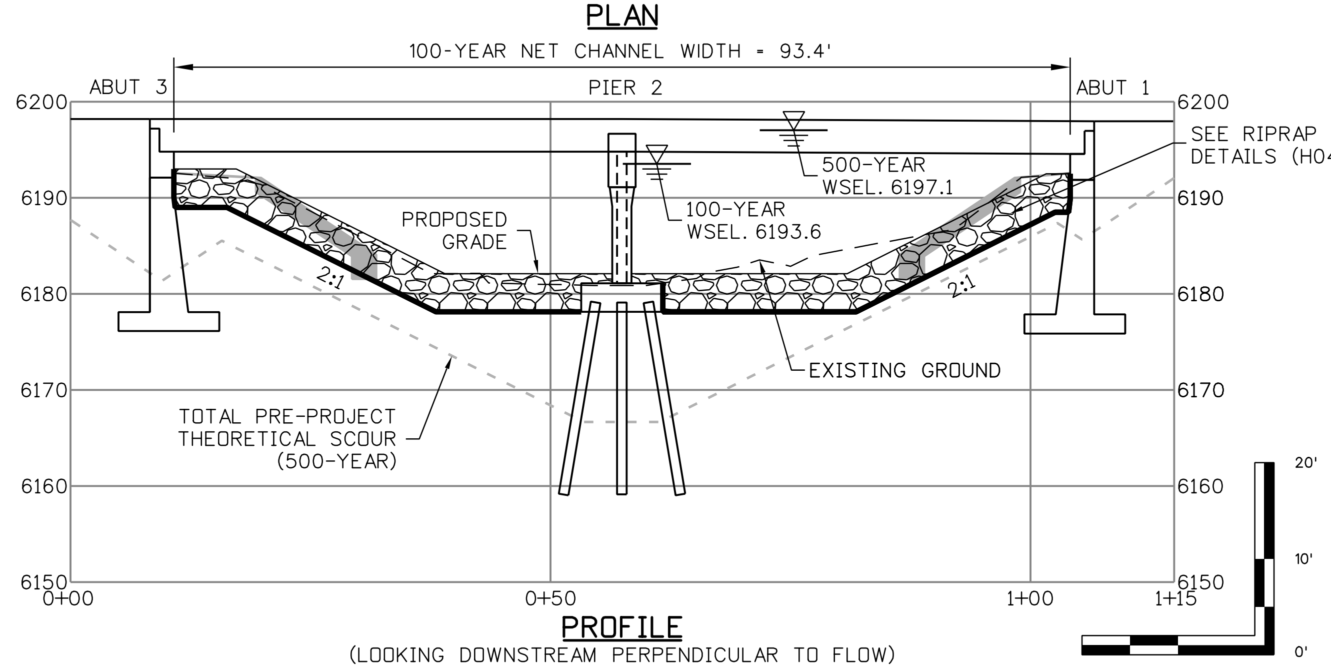
3. There is limited but successful field experience using concrete armor units for scour protection at bridge piers.

4. Piers at new bridges cannot rely on countermeasures to reduce the design depths of foundation elements (Federal guidance).

5. Biotechnical countermeasures are only intended for stream banks, not stream beds. This matrix assumes that any biotechnical treatments are fully grown, with well-established root systems. The toe of <u>any</u> streambank treatment should be reinforced with rock riprap or other armor material, as discussed in Chapter 6 of this document.



PLAN



PROFILE

(LOOKING DOWNSTREAM PERPENDICULAR TO FLOW)
(DEVELOPED USING CDOT AS-BUILT PLANS)

NOTES:

1. REFER TO FORTIFICATION CREEK SCOUR MITIGATION PLANS FOR RIPRAP LAYOUT AND DETAILS.
2. SEE UTILITY PLANS FOR POTHOLE AND CONTACT INFORMATION.
3. IF EXISTING RIPRAP MEETS THE SPECIFICATIONS AND IS IN GOOD CONDITION, AS APPROVED BY THE ENGINEER, THEN IT MAY BE USED FOR PROPOSED RIPRAP.

FINAL (DRAFT) AD SUBMITTAL - AUGUST 17, 2018

Print Date: 08/07/2018	
File Name: B-06-A BHIS 1.dgn	
Horiz. Scale: As Noted	Vert. Scale: As Noted
Unit Information	Unit Leader Initials
HYDRAU-TECH, INC. ENGINEERING AND SOFTWARE	363 W. DRAKE RD. SUITE 3B FORT COLLINS, CO 80526 PHONE (970) 223-3984

Sheet Revisions		
Date:	Comments	Init.

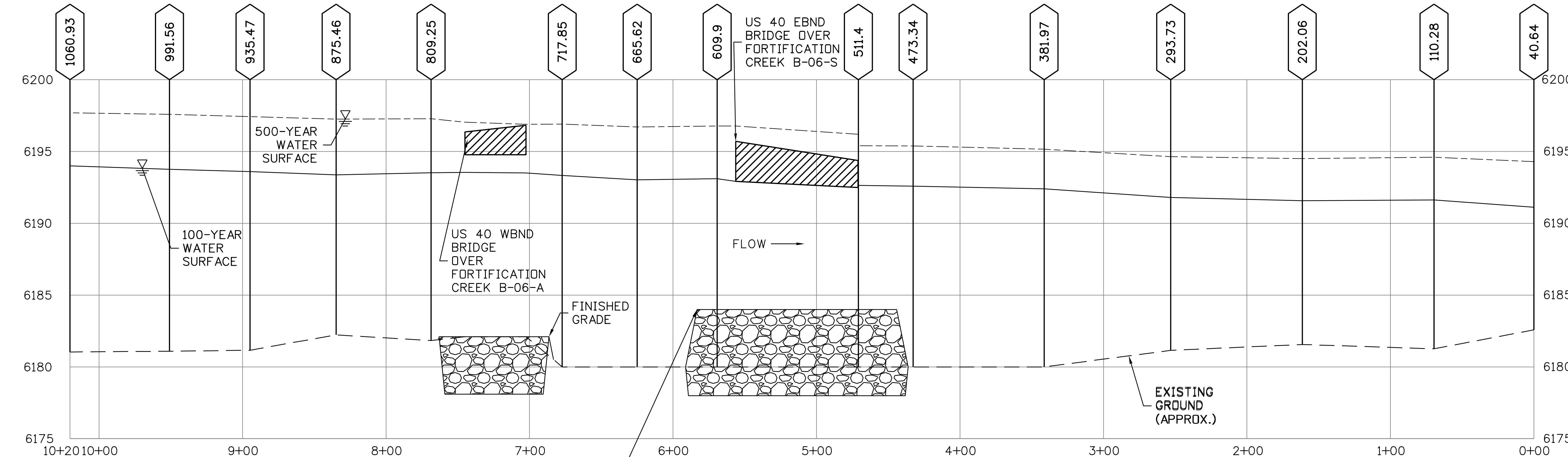
Colorado Department of Transportation
CDOT
270 Ranney Street
Craig, CO 81625
Phone: 970-826-5189 FAX: 970-826-5209
Region 03 CPM

As Constructed
No Revisions:
Revised:
Void:

FORTIFICATION CREEK
US 40 WBND MM 0.016
BRIDGE HYDRAULIC INFORMATION (1 OF 2)

Designer:	AM	Structure Numbers	B-06-A
Detailer:	MS		
Sheet Subset:	BRDG	Subset Sheets: B04 of B30	

Project No./Code
NHPP 0402-090
20753
Sheet Number 37



FINISHED GRADE (RIPRAP PLACED ALONG ABUTMENTS AND PIERS, NOT ALONG CENTERLINE OF CHANNEL)

LEGEND

XXX.XX — HEC-RAS ID

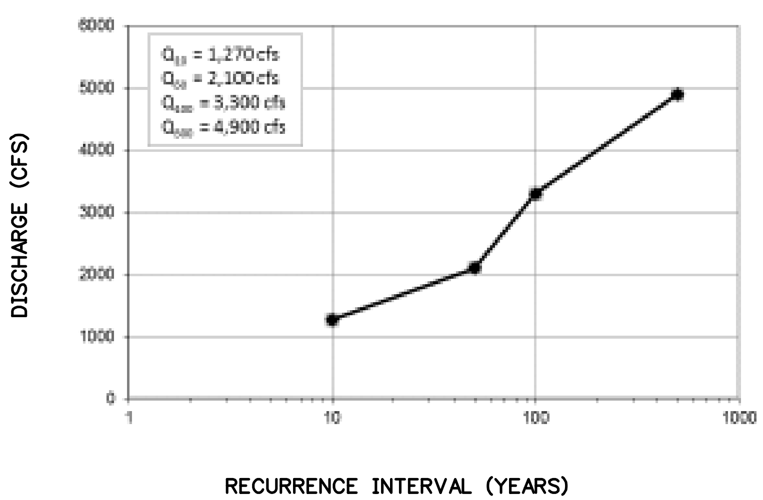
RIPRAP AND SOIL RIPRAP (24 INCH)

CL CHANNEL (FORTIFICATION CREEK)

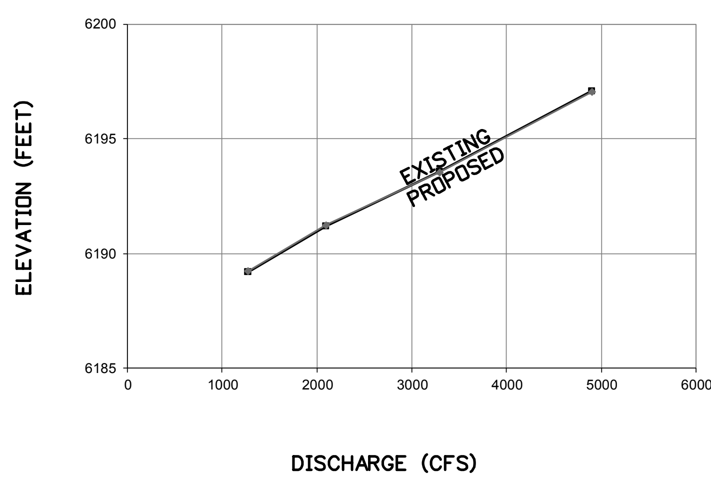
HORIZ. SCALE: 1"=80'
VERT. SCALE: 1"=8'

BRIDGE HYDRAULIC INFORMATION

FLOOD FREQUENCY CURVE



STAGE - DISCHARGE CURVE



COMPARISON OF HYDRAULICS DURING 100-YEAR EVENT AT UPSTREAM BRIDGE FACE

TYPE	VELOCITY (FPS)	FREEBOARD (FT)*	WS ELEV. (FT)
EXISTING	4.3	1.1	6193.56
PROPOSED	4.4	1.1	6193.52

*MINIMUM 4' FREEBOARD FOR HIGH DEBRIS STREAMS
REQUIRED FREEBOARD = 1.3' PER BACKWATER CALCULATION AT RIVER STA. 809.25
PROPOSED FREEBOARD = 1.0' PER BACKWATER CALCULATION AT RIVER STA. 809.25

COMPARISON OF HYDRAULICS DURING 500-YEAR EVENT AT UPSTREAM BRIDGE FACE

TYPE	VELOCITY (FPS)	FREEBOARD (FT)*	WS ELEV. (FT)
EXISTING	5.5	0.0	6197.29
PROPOSED	5.7	0.0	6196.89

100-YEAR RECURRENCE INTERVAL

FLOW UPSTREAM OF BRIDGE = 3,300 CFS

SCOUR ANALYSIS RESULTS

TYPE	Depth (FT) 100-YEAR	500-YEAR
CONTRACTION FOR PRESSURE FLOW	N/A	3.7
CONTRACTION	7.4	N/A
ABUTMENT 1	0.0	3.3
PIER 2	9.4	10.5
ABUTMENT 3	0.0	7.6

CHANNEL DESCRIPTION

Bottom Material – Cohesive ☐ Non-Cohesive ☒
Bottom Material – Size – Clay ☐ Silt ☒ Sand ☒ Gravel ☒ Cobbles ☐ Other ☐
Stream Form – Straight ☒ Meandering ☐ Braided ☐
Mannings "n" For Design – Channel = 0.035-0.04
Overbanks = 0.045
Debris – Brush ☒ Trees/Logs ☒ Ice ☐ Other ☒
Drainage Area = 258 sq. mi.

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FORT COLLINS, CO 80526
PHONE (970) 223-3984
FAX (970) 223-8484

Sheet Revisions		
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As Constructed
No Revisions:
Revised:
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FORTIFICATION CREEK
US 40 WBND MM 0.016
BRIDGE HYDRAULIC INFORMATION (2 OF 2)

Designer:	AM	Structure Numbers	B-06-A
Detailer:	MS		
Sheet Subset:	BRDG	Subset Sheets: B05 of B30	

Project No./Code
NHPP 0402-090
20753
Sheet Number 38

REFERENCES

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