CHAPTER 6

BRIDGES

NOTE: **All questions and comments should be directed to the Hydraulics Unit Supervisor, Environmental Section.**

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6.1 INTRODUCTION/PURPOSE

The purpose of this chapter is to provide guidance in the hydraulic design of a stream-crossing system through:

- Appropriate policy and design criteria.
- Technical aspects of hydraulic design.

Present non-hydraulic factors that influence design, including:

- Environmental concerns.
- Emergency access, traffic service.
- Consequence of catastrophic loss.

Present a design procedure which emphasizes hydraulic analysis using the computer program HEC-RAS.

A more in-depth discussion on design philosophy is presented in the AASHTO *Highway Drainage Guidelines*, Chapter VII (1).

6.2 DEFINITIONS

Bridges are defined as:

- Structures that transport traffic over waterways or other obstructions.
- Part of a stream-crossing system that includes the approach roadway over the floodplain, relief openings, and the bridge structure.
- Structures with a centerline span of 20 feet or more. However, structures designed hydraulically as bridges, as described above, are treated in this chapter regardless of length. Generally, structures less than 20 feet are considered culverts.

Backwater - The increase in water surface elevation induced upstream from such things as a bridge, culvert, dike, dam, another stream at a higher stage, or other similar structures or conditions that obstruct or constrict a channel relative to the elevation occurring under natural channel and floodplain conditions.

Floodplain - The land bordering a stream that is subject to inundation by floods.

Floodway - The channel of a river or a stream and those parts of the floodplain adjoining the channel which are reasonably required to carry a discharge, the flood water, or flood flow of any river or stream.

Harmful Interference - An unnaturally high stage or unnatural direction of flow on a river or stream that causes, or may cause, damage to property, a threat to life, a threat of personal injury, or a threat to water resources.

Spread - The accumulated flow in and next to the roadway gutter. This water often represents an interruption to traffic flow during rainstorms. The lateral distance, in feet, of roadway ponding from the curb.

Stream Channelization - Anything that straightens or changes the geometry to a fixed cross section.

Appendix 6-A contains a list of acronyms and symbols used in this chapter with their meanings.

6.3 POLICY AND DESIGN CRITERIA

6.3.1 General Policy and Design Criteria

Federal policies and state policies that broadly apply to drainage design are presented in Chapter 2, Legal Policy and Procedure. Policies that are unique to bridge crossings are presented in this section.

Below are the general AASHTO criteria related to the hydraulic analyses for the location and design of bridges as stated in the AASHTO *Highway Drainage Guidelines*, 1999.

- Backwater will not significantly increase flood damage to property upstream of the crossing.
- Velocities through the structure(s) will not damage either the highway facility or increase damage to adjacent property.
- Pier spacing, pier orientation, and abutments designed to minimize flow disruption and potential scour.
- Foundation and/or countermeasures designed to avoid failure by scour.
- Freeboard at structure(s) designed to pass anticipated debris and ice.
- Minimize disruption to ecosystems.
- Provide level of traffic service compatible with that commonly expected for the functional class of highway and projected traffic volumes.
- Design choices should consider costs for construction, maintenance, and operation. This includes probable repair, reconstruction, and potential liability.

6.3.2 MDOT Policy and Design Criteria

MDOT policies identify specific areas for which quantifiable criteria can be developed. See Part 31 of Public Act 451 of 1994, the State of Michigan Executive Order in Chapter 2, Legal Policy and Procedure, Appendix 2-F, for additional information.

Part 31 and FHWA technical advisories dictate the size of the bridge opening. They provide specific, quantifiable values that relate to local site conditions. Evaluation of various alternatives according to these design criteria shall be accomplished using water surface profile computer programs such as HEC-RAS.

Travelway

Overtopping of bridges is prohibited during the 2 percent chance (50-year) flood. The 1 percent chance (100-year) shall not cause harmful interference.

Design Flood

The design flood for the evaluation of backwater, clearance, and overtopping is the 1 percent chance (100-year) flood.

Backwater Increases Over Existing Conditions

Backwater impacts shall conform to Part 31 of Public Act 451 of 1994. The statute states that the project shall not produce a harmful interference to the watercourse, and it shall conform to FEMA regulations for sites covered by the National Flood Insurance Program (NFIP).

Clearance

Where practical, a minimum clearance of 2 feet between the water surface and low chord shall be provided during the design flood. Clearance should conform to Federal requirements based on normally expected flows during the navigation season. Navigation includes using canoes, small boats, and wading by fishermen.

Location

To establish the location of a new bridge crossing, the flow distribution must be determined. The proposed facility shall not cause any significant unwarranted change in the existing flow distribution.

Spill through Abutments

Provide minimum vertical clearance as noted above. Provide a minimum set back distance of 25 feet or ratio of 3 horizontal to 1 vertical (whichever is greater) from the Ordinary High Water Mark. The set back ratio is defined as the ratio of the set back distance to the hydraulic depth at the design flood in the main channel.

Scour

Design of bridge foundations shall be based on the estimated scour depth for the 1 percent chance (100-year) event. The resulting design should be checked using the estimated scour depth for the 0.2 percent chance (500-year) flood event and a geotechnical design practice safety factor of at least 1.0.

The following steps are used in the design of the bridge foundation for scour:

Step 1. Estimate the potential for long-term degradation of the channel.

Step 2. Evaluate the proposed bridge and road geometry for scour using the 1 percent chance (100-year) flood. Once the expected scour geometry has been assessed, the geotechnical engineer shall design the foundation. This foundation design shall use conventional foundation safety factors (between 2 and 3).

Step 3. Impose a 0.2 percent chance (500-year) flood on the proposed bridge and road geometry. This event shall be used to evaluate the proposed bridge opening to ensure that the resulting potential scour will produce no unexpected scour hazards. The foundation design for this flood shall be reviewed by the geotechnical engineer using a safety factor of 1.0.

6.3.3 Coordination, Permits, and Approvals

Please reference Chapter 2, Legal Policy and Procedure, for waterway environmental permits, policies, and regulations.

6.3.4 Bridge Deck Drainage

The design of pavement drainage on a bridge should use the same criteria as an approach roadway. However, it should be noted that an approach roadway with a rural typical section drains more easily than a bridge deck with parapets where the deck will confine the runoff in a manner similar to a curbed roadway section. Careful attention must be given to spread on bridge decks. See the Bridge Design Manual, Section 7.07.02.

Where it is necessary to intercept deck drainage at intermediate points along the bridge, the design of interceptors shall conform to HEC-21 procedures. When deck drainage interceptors are needed, a storm sewer collection system will be necessary to discharge the runoff. Some considerations for this system are:

- Environmental concerns when pavement runoff discharges directly into a waterway
- Extensive drain systems attached to the superstructure require design and maintenance
- Free drops from deck interceptors require erosion control underneath.
- Eight-inch minimum projection beyond lowest adjacent superstructure component.

6.4 DESIGN GUIDANCE AND PROCEDURE

6.4.1 Design Procedure Outline

The following design procedure outline may be used for hydraulically analyzing and sizing a bridge opening. Although the scope of the project and individual site characteristics make each design unique, this procedure may be applied unless indicated otherwise by MDOT.

I. Data Collection

A. Survey

- Topography
- Geology
- High-water marks
- History of debris accumulation, ice, and scour
- Review of hydraulic performance of existing structures
- Maps and aerial photographs
- Rainfall and stream gauge records
- Field reconnaissance

B. Studies by Other Agencies

- Federal Flood Insurance Studies
- Federal Floodplain Studies by the USACE, NRCS, etc.
- State and Local Floodplain Studies
- Hydraulic performance of existing bridges

C. Influences on Hydraulic Performance of Site

- Other streams, reservoirs, and water intakes
- Structures upstream or downstream
- Natural features of stream and floodplain
- Channel modifications upstream or downstream
- Floodplain encroachments
- Sediment types and bed forms (Also see Scour, Site Data, Level One Qualitative Analysis C, FHWA.)
- HEC-20 (Appendix 6-D)

D. Environmental Impact

- Existing bed or bank instability (Level 1,Qualitative Analysis, HEC-20)
- Floodplain land use and flow distribution
- Environmentally sensitive areas, MDOT Form 1775 (fisheries, wetlands, etc.)

E. Site-specific Design Criteria

- Preliminary risk assessment
- Application of agency criteria

II. Hydrologic Analysis - See Chapter 3, Hydrology.

III. Hydraulic Analysis

A. Computer model calibration and verification, when applicable

B. Hydraulic performance for existing conditions

C. Hydraulic performance of proposed designs

D. Scour computations

IV. Selection of Final Design

A. Measure of compliance with established hydraulic criteria

B. Consideration of environmental and social criteria

C. Design details such as riprap, scour abatement, and river training

V. Documentation

A. Complete project records, permit applications, etc.

B. Complete correspondence and reports

C. Plans

D. Specifications

6.4.2 Hydraulic Performance of Bridges

The stream-crossing system is subject to either open channel flow or pressure flow through one or more bridge openings with possible embankment overtopping. These hydraulic complexities shall be analyzed using the HEC-RAS computer program unless otherwise indicated by the Design Engineer - Hydraulics.

The hydraulic variables and flow types are defined in Figures 6-1, Bridge Hydraulics Definition Sketch, and 6-2, Bridge Flow Types. Note that:

- Symbols used in Figure 6-1 include:
  - Q_a, Q_b, and Q_c which are flows in the indicated regions of the channel
  - W and b which are the width of the channel and the width of the bridge, respectively
  - h_i indicates height (elevation) differences at various locations along the channel
  - $\alpha\ v^2/2g$ is the velocity head at various locations along the channel
  - S indicates slope along the channel

- Symbols used in Figure 6-2 include:
  - y_n which is the normal water depth
  - y_c which is the critical water depth (at various sections along the channel)

- Backwater (h_1) is measured relative to the normal water surface elevation without the effect of the bridge at the approach cross section (Section 1). The water surface elevation in FEMA Flood Insurance studies is the hydraulic grade line (HGL) through the floodway portion. It is the result of contraction and re-expansion head losses and head losses due to bridge piers. Backwater can also be the result of a "choking condition" in which critical depth is forced to occur in the contracted opening with a resultant increase in depth and specific energy upstream of the contraction. This is illustrated in Figure 6-2, Bridge Flow Types.
- Type I consists of subcritical flow throughout the approach, bridge, and exit cross sections and is the most common condition encountered in practice.
- Types IIA and IIB both represent subcritical approach flows which have been choked by the contraction resulting in the occurrence of critical depth in the bridge opening. In Type IIA, the critical water surface elevation in the bridge opening is lower than the undisturbed normal water surface elevation. In Type IIB, it is higher than the normal water surface elevation and a weak hydraulic jump immediately downstream of the bridge contraction is possible.
- Type III flow is supercritical approach flow and remains supercritical through the bridge contraction. Such a flow condition is not subject to backwater unless it chokes and forces the occurrence of a hydraulic jump upstream of the contraction.

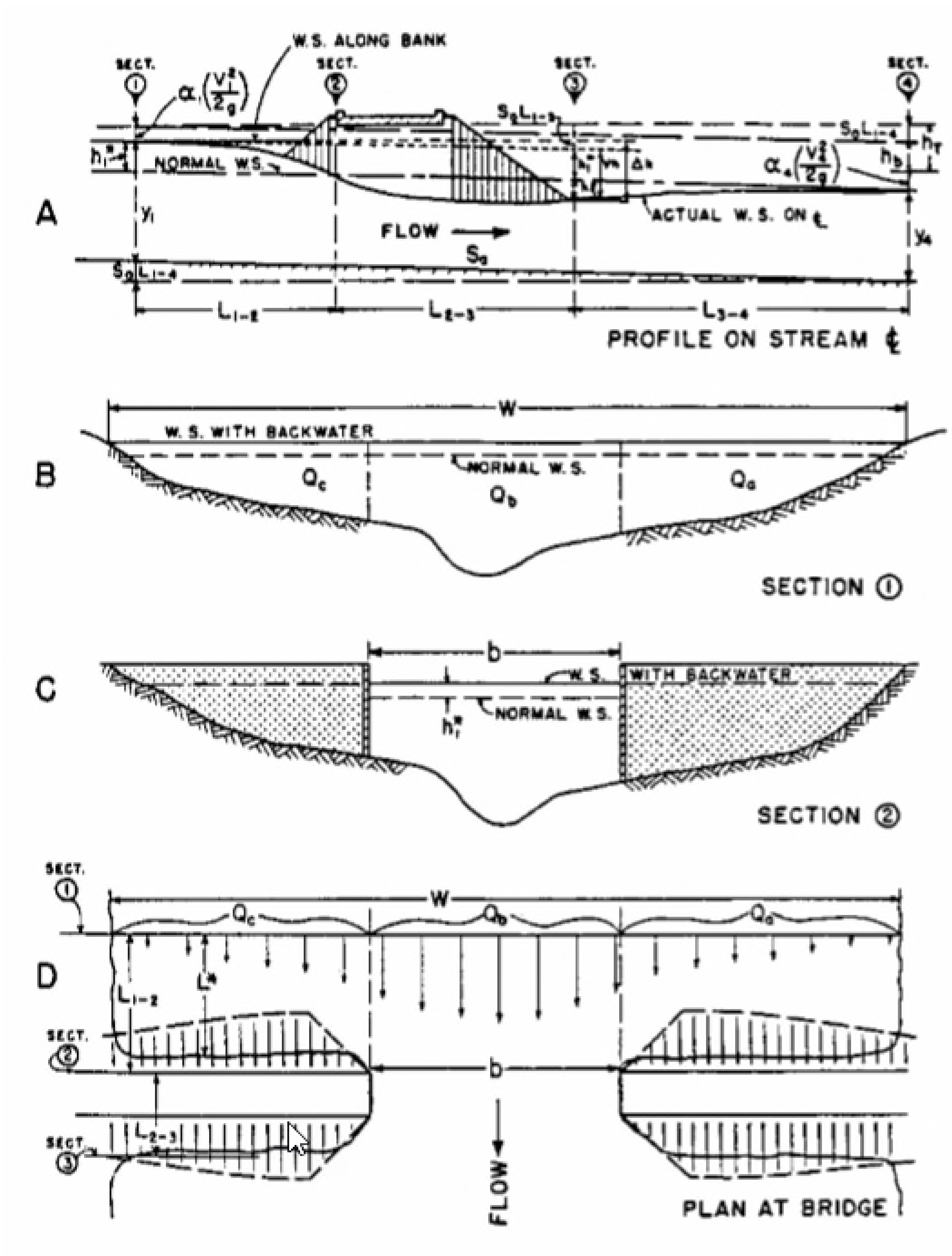


Figure 6 -1 Bridge Hydraulic Definition Sketch

Source: HDS-1

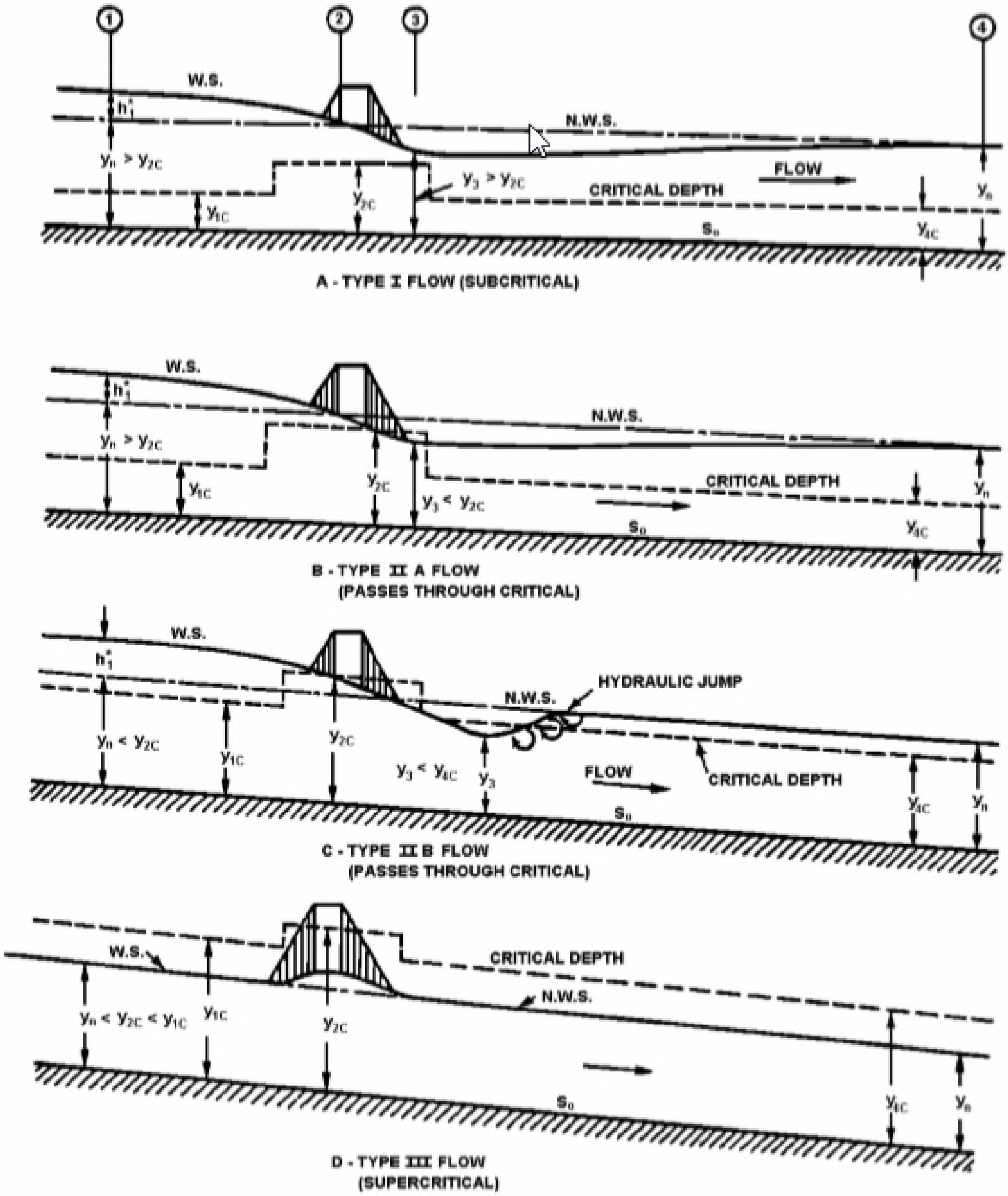


Figure 6-2 Bridge Flow Types

Source: HDS-1

6.4.3 Methodologies

The hydraulic design engineer shall use good engineering judgment and design methodologies. The hydraulic design engineer may contact the Design Engineer - Hydraulics for assistance. No single method is ideally suited for all situations. If a satisfactory computation cannot be achieved with a given method, an alternate method shall be attempted. It has been found that, with careful attention to the setup requirements of each method, essentially duplicate results can usually be achieved using both momentum and energy methods. Physical and two-dimensional modeling is not currently used by MDOT. However, a discussion is included for reference.

HEC-RAS (and its predecessor, HEC-2)

The U.S. Army Corps of Engineers Hydrologic Engineering Center has developed the HEC-RAS (River Analysis System) program. Although there are other one-dimensional analysis packages, HEC-RAS is the most widely used. HEC-RAS replaced the HEC-2 model. HEC-RAS operates under Microsoft Windows™ and has full graphic support. The finished package includes friction slope methods, mixed flow regime capability, automatic Manning n calibration, ice cover, quasi two-dimensional velocity distribution, super-elevation around bends, bank erosion, riprap design, stable channel design, sediment transport calculations, and scour at bridges. The HEC-2 model was used for the majority of the flood insurance studies performed under the National Flood Insurance Program (NFIP).

Two-Dimensional (2D) Modeling

The water surface profile and velocities in a section of river are often predicted using a computer model. While one-dimensional methods are adequate for many applications, these methods cannot provide a detailed determination of the cross-section water surface elevations, flow velocities, or flow distribution.

2D models are more complex and require more time to set up and calibrate. However, they require essentially the same field data as a one-dimensional model.

The use of 2D models is increasing. They use a finite element analysis to predict 2D flow components (within the horizontal plane). 2D models shall be used in specialized applications such as:

- Calculation of flow distribution across a channel (such as around bends).
- Contraction and expansion at bridges.
- Lateral inflow or outflow.
- Sheet flow patterns.

Physical Modeling

Complex hydrodynamic situations defy accurate or practical mathematical modeling. Physical models shall be considered when:

- Hydraulic performance data are needed that cannot be reliably obtained from mathematical modeling,
- Risk of failure or excessive over-design is unacceptable, and
- Research is needed.

The constraints on physical modeling are size (scale), cost, and time.

6.4.4 HEC-RAS Modeling

The cross sections that are necessary for the energy analysis through a bridge opening for a single opening bridge are shown in Figure 6-3. Additional cross sections should always be included upstream and downstream of the structure to compute the water surface profile.

Energy losses caused by structures such as bridges and culverts are computed in two parts. First, the losses due to expansion and contraction of the cross section on the upstream and downstream sides of the structure are computed in the standard step calculations. Second, the loss through the structure itself is computed by one of several different methods.

At low flows, the losses through a bridge may be computed by one of four methods: momentum balance, energy equation, Yarnell equation, or the FHWA WSPRO method. The HEC-RAS user may select any of all of these methods for comparison purposes.

At high flows that contact the low chord of the bridge, either the energy equation or separate hydraulic equations for pressure and weir flow may be selected. When selecting the pressure and weir flow method, the program will automatically switch to the energy equation when the weir becomes highly submerged (the program default value is 95 percent).

A user’s instruction manual for HEC-RAS shall serve as a source for more detailed information on using this computer model.

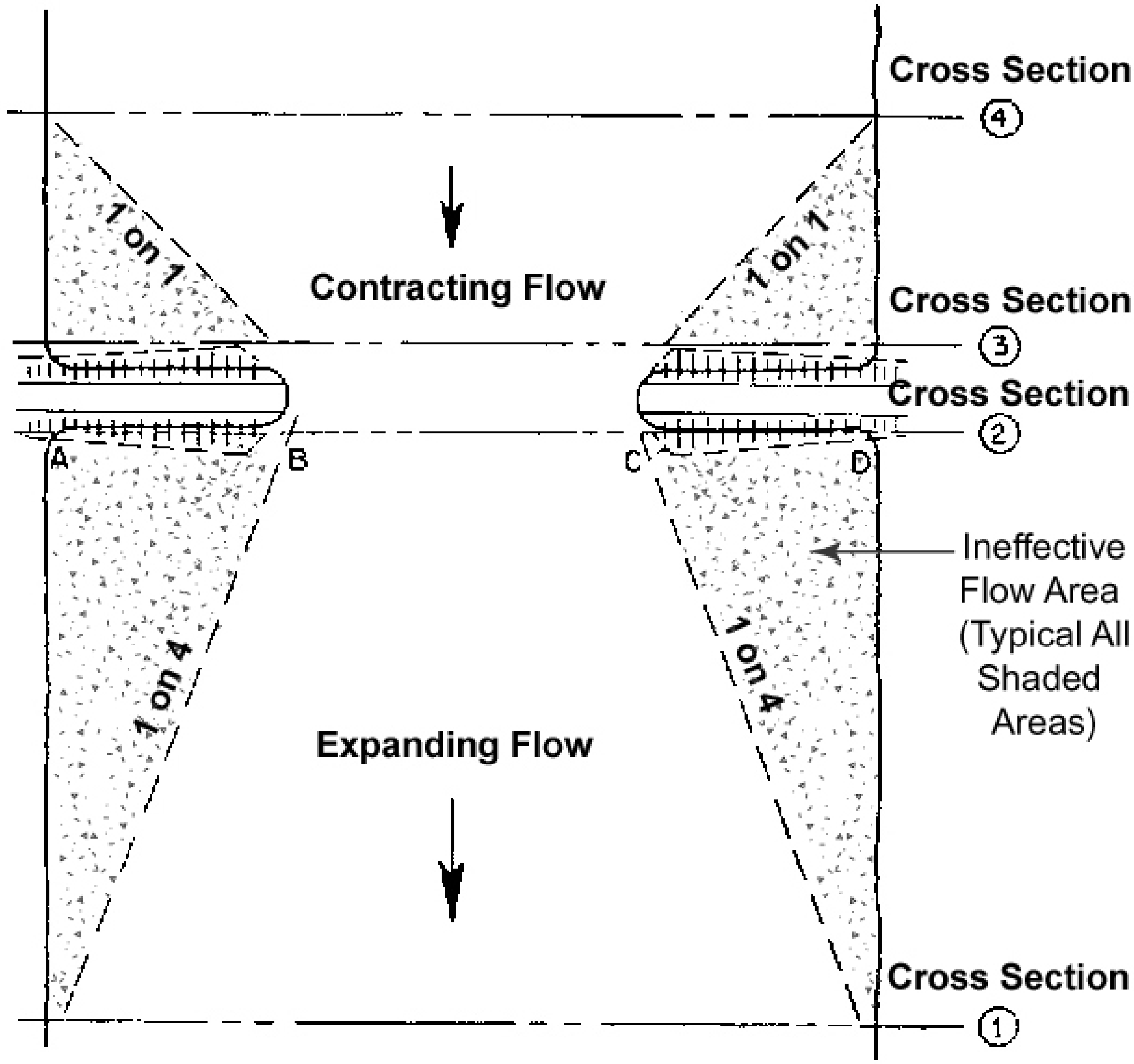


Figure 6-3 Cross-Section Locations in the Vicinity of Bridges

6.4.5 Bridge Scour

6.4.5.1 Introduction

Hydraulic analysis of a bridge design requires that an assessment be made of the proposed bridge's vulnerability to undermining due to potential scour. Because of the extreme hazard and economic hardships posed by a bridge collapse, special considerations must be given to selecting appropriate flood magnitudes for use in the analysis.

The FHWA issued a Technical Advisory (TA 5140.20) on bridge scour in September 1988. The document, "*Interim Procedures for Evaluating Scour at Bridges,*" was an attachment to the Technical Advisory. The interim procedures were replaced by HEC-18, *Evaluating Scour at Bridges* (1991, 1993, and 2001). Users of this manual shall consult HEC-18 for a more thorough treatise on scour and scour prediction methodology. Companion FHWA documents to HEC-18 are HEC-20, "*Stream Stability at Highway Structures,*" and HEC-23, "*Bridge Scour and Stream Instability Countermeasures.*" FHWA HEC documents can be obtained at www.fhwa.dot.gov\bridge\hydpub.htm

6.4.5.2 Information Needed

Items useful for completing a scour analysis include:

- Bridge inspection reports and maintenance records.
- Hydraulic analysis, including Flood Insurance Studies.
- Bridge construction drawings.
- Aerial photographs.
- Soil gradation analyses (stream and abutments).
- Topographic maps.

6.4.5.3 Scour Analysis Equations

Refer to HEC-18 for a thorough discussion of scour analysis procedures. A few important equations are included below for reference.

Present technology dictates that bridge scour be evaluated as interrelated components.

- Long-term profile changes (aggradation/degradation).
- Plan form change (lateral channel movement).
- Contraction scour/deposition.
- Local scour (piers and abutments).

Total scour is equal to the sum of these components, i.e., Total Scour = (aggradation/degradation) + (contraction scour) + (local pier or abutment scour).

6.4.5.3.1 Contraction Scour

Contraction scour occurs from a contraction of the natural stream's flow area (which might occur at a bridge). One contraction scour equation is used for live bed (bed sediment moving) and another for clear water (no bed sediment moving). Live bed scour occurs when the shear velocity, V^*, exceeds the fall velocity, ω (found in Figure 6-4, Fall Velocity of Sand-Sized Particles).

$$\mathbf{V_* = (g\ y_1\ S_1)^{1/2}} \qquad \textbf{(6.1)}$$

Where: y_1 = average flow depth in upstream channel
S_1 = slope of energy grade line or main channel
g = acceleration due to gravity, 32.2 feet/s^2

Live Bed Equation

$$\mathbf{y_2/y_1 = (Q_2/Q_1)^{6/7}\ (W_1/W_2)^{K_1}} \qquad \textbf{(6.2)}$$

$y_s = y_2 – y_0$ (average scour depth)

Where: y_s = average contraction scour depth
y_1 = average flow depth in the upstream main channel, feet
y_0 = existing flow depth in the contracted section before scour, feet
y_2 = average flow depth in the contracted section, feet
W_1 = bottom width of the upstream main channel, feet
W_2 = bottom width of the main channel in the contracted section, feet
Q_1 = flow in the upstream channel transporting sediment, cfs
Q_2 = flow in the contracted channel, cfs
K_1 = exponent determined below in Table 6-1

Table 6-1 Bed Material Transport Exponent

V_*/ω	K_1	Mode of Bed Material Transport
< 0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
> 2.0	0.69	Mostly suspended bed material discharge

$V_* = (\tau/\rho)^{1/2} = (gy_1\ S_1)^{1/2}$, shear velocity in the upstream section, fps
ω = fall velocity of bed material based on the D_{50} fps (see Figure 6-4, Fall Velocity of Sand-Sized Particles)
g = acceleration due to gravity (32.2 feet/s^2)
S_1 = slope of energy grade line of main channel, feet/foot
τ = shear stress on the bed, lb./feet^2
ρ = density of water (1.94 slugs/feet^3)

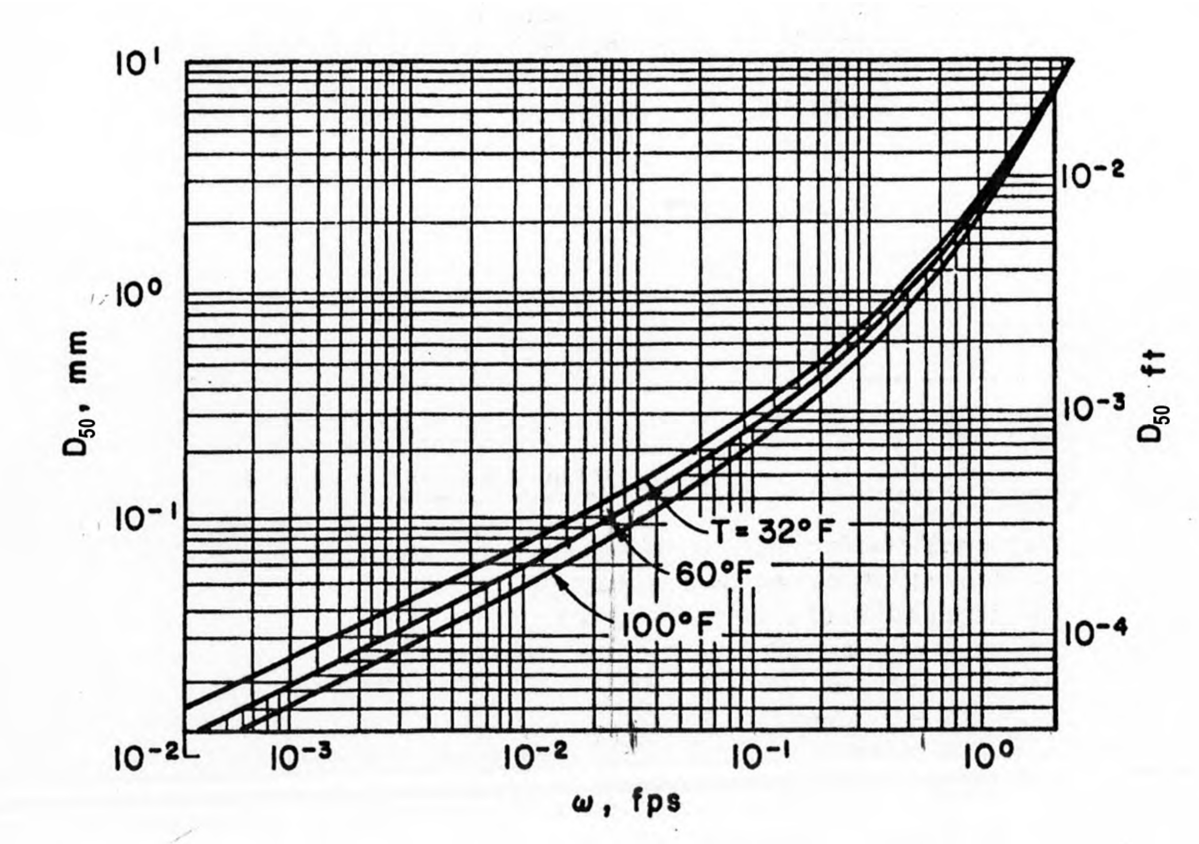


Figure 6-4 Fall Velocity of Sand-Sized Particles

<u>Clear Water Contraction Scour</u>

$$\mathbf{y_2 = [(K_uQ^2)/(D_m^{2/3}\ W^2)]^{3/7}} \qquad \textbf{(6.3)}$$

$y_s = y_2 - y_0$ (average contraction scour depth)

Where:
- y_0 = existing depth in the contracted section before scour, feet
- y_2 = average equilibrium depth in the contracted section after contraction scour, feet
- y_s = depth of scour, feet
- Q = discharge through the bridge or on the overbank at the bridge, cfs
- D_m = diameter of the smallest nontransportable particle in the bed material ($D_m = 1.25\ D_{50}$) in the contracted section, feet
- D_{50} = median diameter of bed material in the bridge opening or on the floodplain, ft.
- W = bottom width of the bridge, less pier widths or overbank width (set back distance), feet
- K_u = 0.0077 (English units)

6.4.5.3.2 Pier Scour

Pier scour occurs due to vortices produced by the obstruction. The pier scour equation takes the form of:

$$\mathbf{y_s/y_1 = 2.0\ K_1\ K_2\ K_3\ K_4\ (a/y_1)^{0.65}\ Fr_1^{0.43}} \qquad \textbf{(6.4)}$$

Where:
- y_s = scour depth, feet
- y_1 = flow depth directly upstream of the pier, feet
- K_1 = correction factor for pier nose shape from Table 6-2
- K_2 = correction factor for angle of attack of flow from Table 6-3
- K_3 = correction factor for bed condition from Table 6-2
- K_4 = correction factor for armoring by bed material size from the equation $K_4 = 0.4V_R^{0.15}$ discussed below
- a = pier width, feet
- L = length of pier, feet
- Fr_1 = Froude number = $V_1/(gy_1)^{1/2}$
- V_1 = mean velocity of flow directly upstream of the pier, fps

Table 6-2 Correction Factor K_1 for Pier Nose Shape

Shape of Pier Nose	K_1
(a) Square Nose	1.1
(b) Round Nose	1.0
(c) Circular Cylinder	1.0
(d) Sharp Nose	0.9
(e) Group of Cylinders	1.0

Table 6-3 Correction Factor K_2 for Angle of Attack of Flow

Angle	L/a = 4	L/a = 8	L/a = 12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = Skew angle of flow L = Length of pier			

Note: The correction factor K_1 for pier nose shape should be determined using Table 6-2 for angles of attack up to 5 degrees. For greater angles, K_2 dominates and K_1 should be considered as 1.0. If L/a is larger than 12, use the values for L/a = 12 as a maximum.

Table 6-4 Increase in Equilibrium Pier Scour Depths (K_3) for Bed Condition

Bed Condition	Dune Height H feet	K_3
Clear Water Scour	N/A	1.1
Plane Bed and Antidune Flow	N/A	1.1
Small Dunes	10 > H < 2	1.1
Medium Dunes	30 > H < 10	1.1 to 1.2
Large Dunes	H > 30	1.3

In Michigan, K_3 = 1.1 is most common. For very large rivers, K_3 = 1.2 may be warranted. K_3 = 1.3 is not usually used for Michigan rivers.

The correction factor K_4 decreases scour depths for armoring of the scour hole for bed materials that have a D_{50} equal to or larger than 0.079 inch (2.0 mm) and D_{95} equal to or larger than 0.79 inch (20 mm). The correction factor results from recent research by Molinas and Mueller. Molinas' research for FHWA showed that when the approach velocity (V_1) is less than the critical velocity (V_{C90}) of the D_{90} size of the bed material and there is a gradation in sizes in the bed material, the D_{90} will limit the scour depth. Mueller and Jones developed a K_4 correction coefficient from a study of 384 field measurements of scour at 56 bridges. The equation developed by Jones given in HEC-18 Third Edition should be replaced with the following:

- If $D_{50} < 0.079$ inch (2 mm) or $D_{95} < 0.79$ inch (20 mm), then $K_4 = 1$
- If $D_{50} \geq 0.079$ inch (2 mm) or $D_{95} \geq 0.79$ inch (20 mm)

Then:

$$\mathbf{K_4 = 0.4\,(V_R)^{0.15}} \quad \textbf{(6.5)}$$

Where:

$$\mathbf{V_R = (V_1 - V_{icD50})/(V_{cD50} - V_{icD95}) > 0} \quad \textbf{(6.6)}$$

and:

V_{icDx} = The approach velocity, fps, required to initiate scour at the pier for the grain size D_x feet

$V_{icDx} = 0.645\,(D_x/a)^{0.053}\,V_{cDx}$ **(6.7)**

V_{cDx} = the critical velocity, fps, for incipient motion for the grain size D_x , feet

$V_{cDx} = K_u\,(y_1)^{1/6}\,(D_x)^{1/3}$ **(6.8)**

y_1 = depth of flow just upstream of the pier, excluding local scour, feet

V_1 = velocity of the approach flow just upstream of the pier, n/s fps

D_x = grain size for which x percent of the bed material is finer, feet (mm)

K_u = 11.17 (English units)

The minimum value of K_4 is 0.4, and it should only be used when $V_1 < V_{icD50}$.

6.4.5.3.3 Abutment Scour

Abutment scour is caused by the constriction of flow at the sides of a channel created by the abutments of the bridge.

The abutment scour equation is:

$$\mathbf{y_s/y_a = 2.27K_1K_2(a'/y_a)^{0.43}\,Fr^{0.61} + 1} \quad \textbf{(6.9)}$$

Where: K_1 = coefficient for abutment shape (see Table 6-5)

K_2 = coefficient for angle of embankment to flow (see Table 6-3)

= $(\theta/90)^{0.13}$ (see Figure 6-5, Adjustment of Abutment Scour Estimate for Skew, for definition of θ)

$\theta < 90$ degrees if embankment points downstream

$\theta > 90$ degrees if embankment points upstream

a' = length of abutment projected normal to flow, feet

A_e = flow area of the approach cross section obstructed by the embankment, sf

Fr = Froude number of approach flow upstream of the abutment = $V_e/(gy_a)^{1/2}$

V_e = Q_e/A_e, ft./s

Q_e = flow obstructed by the abutment and approach embankment, cfs

y_a = average depth of flow in the overbank, feet

y_s = scour depth, feet

Table 6-5 Abutment Shape Coefficients

Description	K_1
Vertical-wall Abutment	1.00
Vertical-wall Abutment with Wing Walls	0.82
Spill-through Abutment	0.55

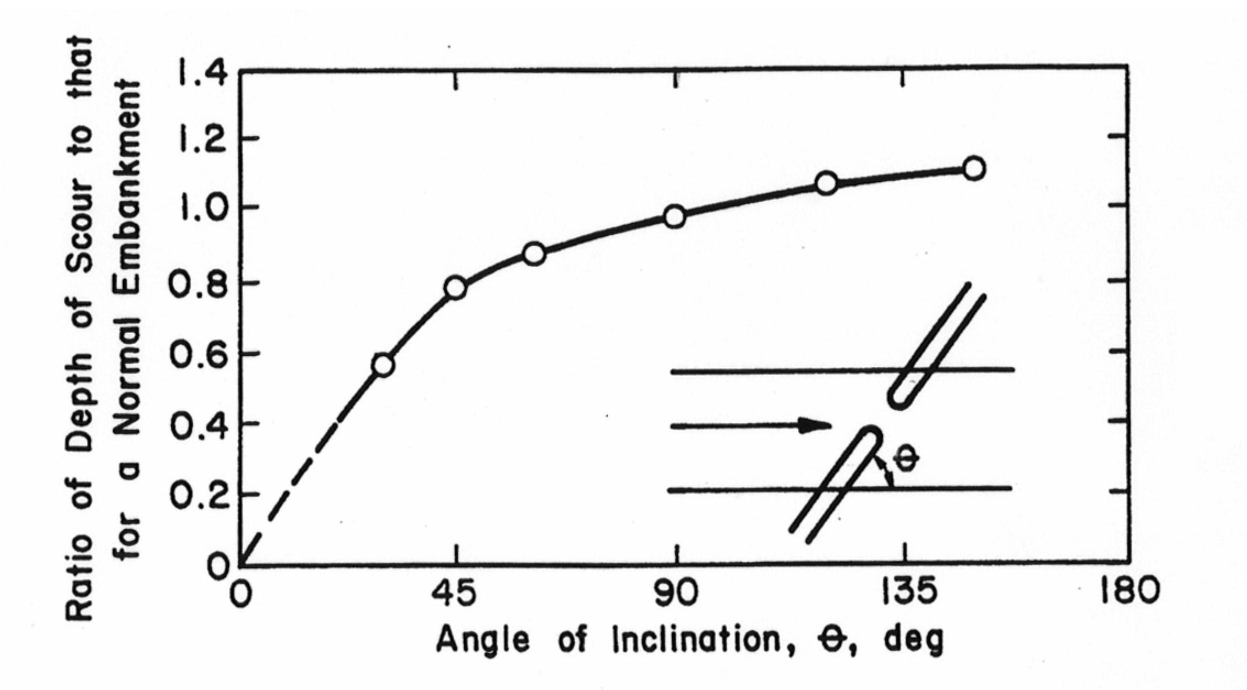


Figure 6-5 Adjustment of Abutment Scour Estimate for Skew

6.4.5.4 Assessment Steps

Scour analysis procedure for design of a new bridge or evaluation of an existing bridge uses worksheets based on HEC-20 and NBIS. These worksheets, both Level One and Level Two, are included in Appendix 6-D.

Conduct a site visit to complete the Level One scour worksheet. Proceed to Level Two analysis for all new bridges. Complete Level Two analysis for existing bridges if warranted from Level One analysis. The general procedure for Levels One and Two analyses can be found in HEC-20.

6.4.5.5 Preventative/Protection Measures

Based on an assessment of potential scour provided by the hydraulic design engineer, the geotechnical and structural designers can incorporate design features that will prevent or mitigate scour damage at piers. In general, circular piers or elongated piers with circular noses and an alignment parallel to the flood flow direction are a possible alternative. Spread footings should be used only where the streambed is extremely stable below the footing and where the spread footing is founded at a depth below the maximum scour computed. Drilled shafts or drilled piers are possible where pilings cannot be driven. Protection against general streambed degradation can be provided by drop structures, grade-control structures, or downstream of the bridge opening.

Riprap is often used, where stone of sufficient size is available, to armor abutment and pier footings. Riprap design information is presented in HEC-23. For assistance, contact the –Hydraulics Unit Supervisor – Environmental Section.

Whenever possible, clearing of vegetation upstream and downstream of the toe of the embankment slope should be avoided. Embankment overtopping may be incorporated into the design but should be located well away from the bridge abutments and superstructure. When embankments encroach on wide floodplains, guide banks are recommended to align the approach flow with the bridge opening and to prevent scour around the abutments. Guide banks are usually elliptical shaped, with a major-to-minor axis ratio of 2.5 to 1. A length of approximately 150 feet provides a satisfactory standard design and can be verified using HEC-23. Guide banks, embankments, and abutments shall be protected by riprap with geotextile liner or other revetments approved by MDOT.

6.4.5.6 Pressure Flow Scour

Pressure 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. At higher approach flow depths, the bridge can be entirely submerged with the resulting flow being a complex combination of the plunging flow under the bridge and the flow over the bridge.

With pressure flow, the local scour depths at a pier or abutment are larger than for free surface flow with similar depths and approach velocities. The increase in local scour at a pier 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 the flow is a more significant cause of the increase in scour depth. However, in many cases when a bridge becomes submerged, the average velocity under it is reduced due to a combination of additional backwater caused by the bridge superstructure impeding the flow, and a reduction of discharge which must pass under the bridge due to weir flow over the bridge and approach embankments. As a consequence of this, increases in local scour attributed to pressure flow scour at a particular site, may be offset to a degree by lesser velocities through the bridge opening due to increased backwater and a reduction in discharge under the bridge due to overtopping.

Additional procedures and equations for predicting scour under pressure flow conditions is contained in HEC-18.

6.4.6 Hydraulic and Scour Analysis Documentation

A summary of the hydraulic analysis is included on bridge construction drawings in tabular form (Appendix 6-B). The results of the hydraulic analyses shall be documented in MDOT's Standard Report Form (Appendix 6-C). Typically, this report is included with the MDEQ permit application. MDOT has developed guidelines and worksheets to be used for Level One and Level Two scour analyses (Appendix 6-D).

6.4.7 Example Problem - Bridge Hydraulic and Scour Analysis

Given: An existing bridge is being replaced to accommodate a road widening project. The existing bridge is a two-span bridge with a total span of 60 feet. The proposed bridge will be a two-span bridge with a total span of 80 feet.

XY coordinates for eight cross sections are given in Table 6-6. For guidance in choosing cross sections, see the MDEQ report in Chapter 4, Natural Channels and Roadside Ditches, Appendix 4-B. Manning's n is 0.050 on the banks and 0.020 in the center. The 2 percent chance (50-year), 1 percent chance (100-year), and 0.2 percent chance (500-year) storms flows are:

Storm	Flows (cfs)
2 percent (50-year) chance	3,000
1 percent (100-year) chance	3,500
0.2 percent (500-year) chance	6,000

Figure 6-6, Plan View of Bridge and River Reach, shows a plan view of the bridge and river reach. There is a dam downstream that keeps the river at Station 0 at an elevation of 817.5 feet.

Find: Verify that the proposed bridge has an adequate waterway area. Find the scour depths at the abutments and the pier for the proposed design.

Solution: All the above data can be entered into HEC-RAS and the model can be run with the three different flows: 2 percent chance (50-year), 1 percent chance (100-year), and 0.2 percent chance (500-year) storms.

Hydraulic Analysis

Figure 6-7, Existing Conditions Water Surface Profile During 1 Percent Chance (100-year) Storm (Printout from HEC-RAS), and Figure 6-8, Proposed Conditions Water Surface Profile During 1 Percent Chance (100-year) Storm (Printout from HEC-RAS), show the existing and proposed profiles. Tables 6-7 and 6-8 are the HEC-RAS profile output tables for the existing and proposed conditions respectively. Tables 6-9 and 6-10 show detailed cross section data for the proposed conditions during the 1 percent chance (100-year) and 0.2 percent chance (500-year) storms respectively. Table 6-11 is a Standard Drainage Tabulation Form. A blank copy of this form is included in Appendix 6-B.

Consult the FHWA's publication HEC-18 for further guidance in evaluating scour. Equations for scour analysis are explained in Section 6.4.5.3.

The MDOT Hydraulic Report Format is included in Appendix 6-C.

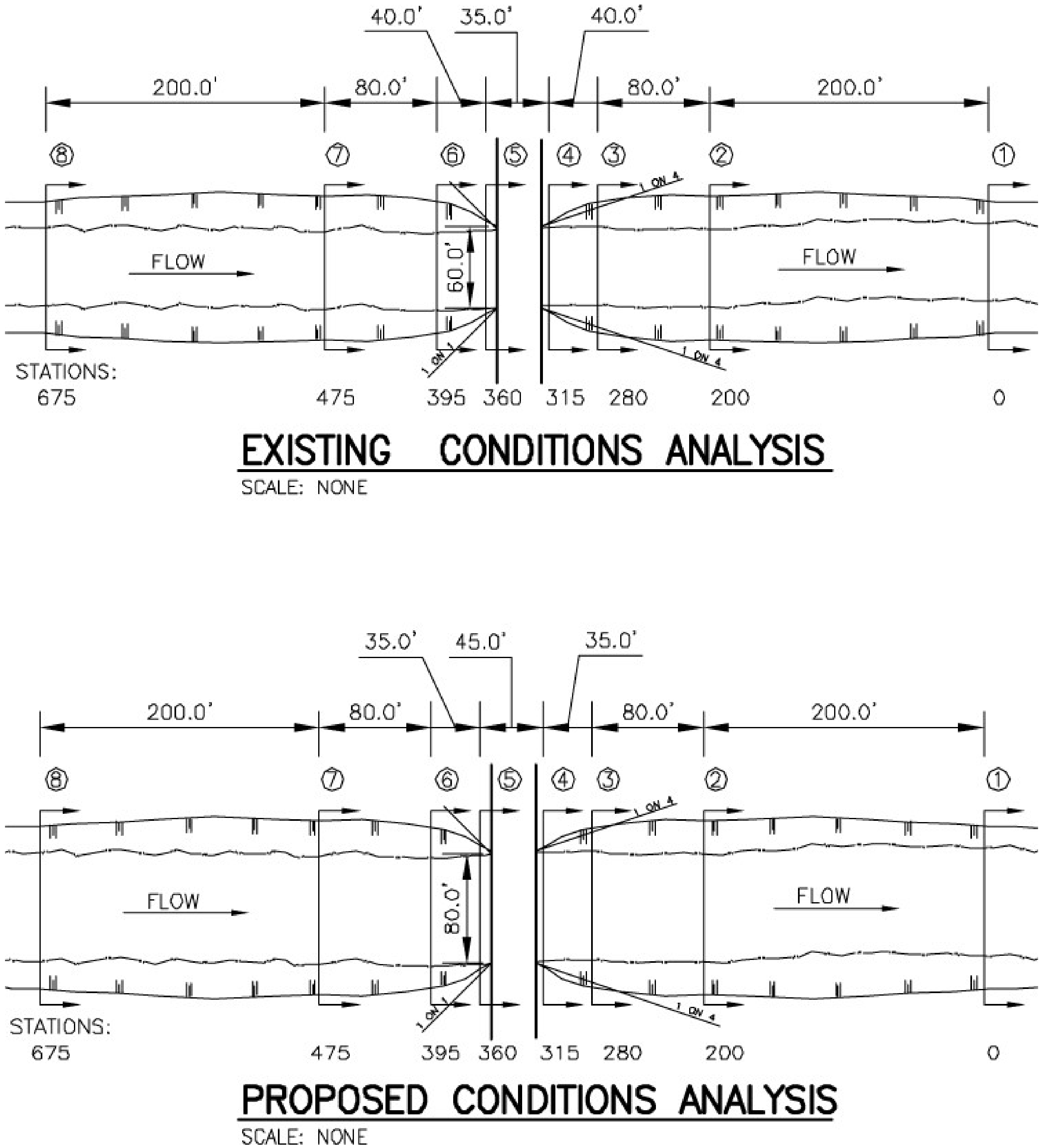
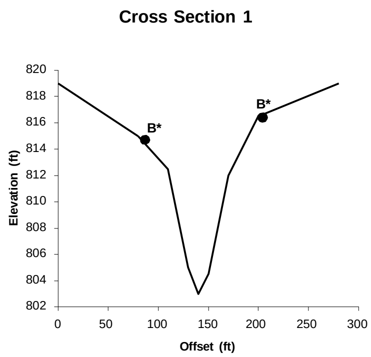


Figure 6-6 Plan View of Bridge and River Reach

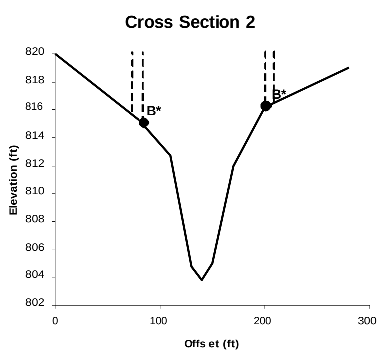
Table 6-6 Cross-Section Data

CROSS-SECTION		EXISTING		PROPOSED	
		Station 0 (Cross Section 1)			
		Offset	***Elevation***	***Offset***	***Elevation***
		0	819	unchanged	unchanged
	B*	80	815		
		110	812.5		
		130	805		
		140	803		
		150	804.5		
		170	812		
	B*	200	816.5		
		280	819		
		Ineffective Flow Offsets			
		N/A		N/A	



B: indicates bank offset locations*

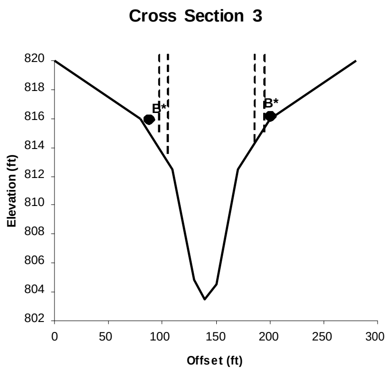
CROSS-SECTION		EXISTING		PROPOSED	
		Station 200 (Cross Section 2)			
		Offset	***Elevation***	***Offset***	***Elevation***
		0	820	unchanged	unchanged
	B*	80	815.3		
		110	812.7		
		130	804.8		
		140	803.8		
		150	805		
		170	812		
	B*	200	816.2		
		280	819		
		Ineffective Flow Offsets			
		80 left		71 left	
		200 right		209 right	



B: indicates bank offset locations*

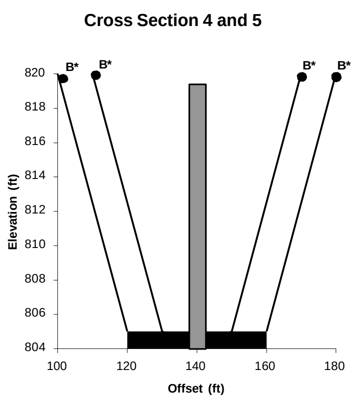
Table 6-6 Cross-Section Data (continued)

CROSS-SECTION **EXISTING** **PROPOSED**



Station 280 (Cross Section 3)				
	Offset	***Elevation***	***Offset***	***Elevation***
	0	820	unchanged	unchanged
B*	80	816		
	110	812.5		
	130	804.8		
	140	803.5		
	150	804.5		
	170	812.5		
B*	200	816		
	280	820		
Ineffective Flow Offsets				
100 left			91 left	
180 right			189 right	

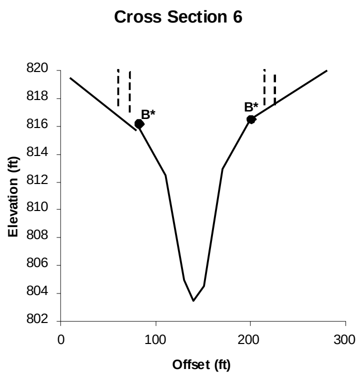
B: indicates bank offset locations*



Stations 320 and 355			Stations 315 and 360		
	Offset	***Elevation***		***Offset***	***Elevation***
B*	110	820	B*	100	820
	130	805		120	805
	139	804		139	804
	141	804		141	804
	150	805		160	805
B*	170	820	B*	180	820
	139	820		139	820
	139	804		139	804
	141	804		141	804
	141	820		141	820
Low Chord Bridge Elevation					
		820			820

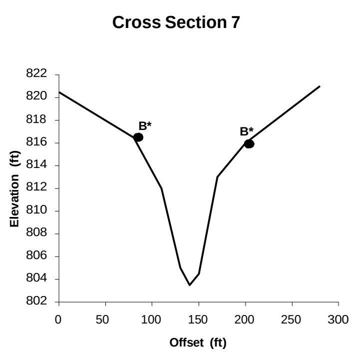
B: indicates bank offset locations*

Table 6-6 Cross-Section Data (continued)



CROSS-SECTION	EXISTING		PROPOSED	
Station 395				
	Offset	***Elevation***	***Offset***	***Elevation***
	0	820.5	unchanged	unchanged
B*	80	816.2		
	110	812.5		
	130	805		
	140	803.5		
	150	804.5		
	170	812.9		
B*	200	816.5		
	280	820		
Ineffective Flow Offsets				
	70 left		65 left	
	210 right		215 right	

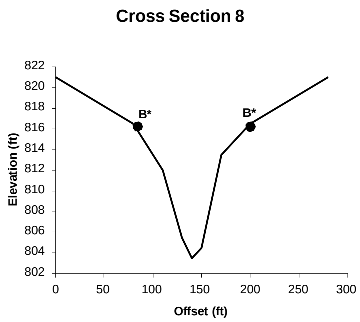
B: indicates bank offset locations*



CROSS-SECTION	EXISTING		PROPOSED	
Station 475				
	Offset	***Elevation***	***Offset***	***Elevation***
	0	820.5	unchanged	unchanged
B*	80	816.5		
	110	812		
	130	805		
	140	803.5		
	150	804.5		
	170	813		
B*	200	816		
	280	821		
Ineffective Flow Offsets				
	N/A		N/A	

B: indicates bank offset locations*

Table 6-6 Cross-Section Data (continued)

CROSS-SECTION		EXISTING		PROPOSED	
		Station 675			
		0	821	unchanged	unchanged
	B*	80	816.5		
		110	812		
		130	805.5		
		140	803.5		
		150	804.5		
		170	813.5		
	B*	200	816.5		
		280	821		
		Ineffective Flow Offsets			
		N/A		N/A	

B: indicates bank offset locations*

Table 6-7 Existing Profile Output Tables

2 Percent (50-year) Chance Storm

River Station	Q Total (cfs)	Min Ch El (feet)	W.S.Elev (feet)	E.G. Elev (feet)	Vel Chnl (ft./s)	Flow Area (sq ft.)	Top Width (feet)	Froude # Channel
0	3,000	803	817.50	817.68	3.47	921	202	0.23
200	3,000	803.80	817.67	817.87	3.56	842.65	200.67	0.24
280	3,000	803.50	817.70	817.96	4.11	729.47	187.82	0.24
320	3,000	804	817.48	818.12	6.40	468.46	53.29	0.38
337.5	Bridge							
355	3,000	804	817.64	818.26	6.29	476.92	53.71	0.37
395	3,000	803.50	818.15	818.32	3.39	905.85	193.91	0.22
475	3,000	803.50	818.22	818.39	3.28	965.92	190.02	0.21
675	3,000	803.50	818.37	818.54	3.33	948.79	186.44	0.22

1 Percent (100-year) Chance Storm

River Station	Q Total (cfs)	Min Ch El (feet)	W.S. Elev (feet)	E.G. Elev (feet)	Vel Chnl (ft./s)	Flow Area (sq ft.)	Top Width (feet)	Froude # Channel
0	3,500	803	817.50	817.75	4.05	921	202	0.27
200	3,500	803.80	817.73	818	4.12	850.07	203.54	0.27
280	3,500	803.50	817.76	818.12	4.76	734.93	190.55	0.28
320	3,500	804	817.47	818.34	7.48	467.67	53.25	0.45
337.5	Bridge							
355	3,500	804	817.70	818.52	7.29	479.88	53.86	0.43
395	3,500	803.50	818.39	818.61	3.82	939.83	203.98	0.25
475	3,500	803.50	818.48	818.69	3.68	1,016.90	199.44	0.23
675	3,500	803.50	818.66	818.87	3.71	1,004.53	196.78	0.24

0.2 Percent (500-year) Chance Storm

River Station	Q Total (cfs)	Min Ch El (feet)	W.S. Elev (feet)	E.G. Elev (feet)	Vel Chnl (ft./s)	Flow Area (sq ft.)	Top Width (feet)	Froude # Channel
0	6,000	803	817.50	818.23	6.95	921	202	0.46
200	6,000	803.80	818.20	818.88	6.62	905.90	225.10	0.42
280	6,000	803.50	818.25	819.18	7.76	773.47	209.82	0.44
320	6,000	804	817.16	819.91	13.29	451.61	52.44	0.80
337.5	Bridge							
355	6,000	804	818.61	820.60	11.32	530.01	56.29	0.65
395	6,000	803.50	820.42	820.81	5.06	1224.38	278.57	0.29
475	6,000	803.50	820.60	820.91	4.62	1519.96	273.64	0.26
675	6,000	803.50	820.80	821.11	4.65	1507.54	272.93	0.26

Note: Shaded numbers are used in Standard Drainage Tabulation Form (Table 6-11).

Table 6-8 Proposed Profile Output Tables

2 Percent (50-year) Chance Storm

River Station	Q Total (cfs)	Min Ch El (feet)	W.S. Elev (feet)	E.G. Elev (feet)	Vel Chnl (ft./s)	Flow Area (sq ft.)	Top Width (feet)	Froude # Channel
0	3,000	803	817.50	817.69	3.52	921	202	0.23
200	3,000	803.80	817.53	817.73	3.62	851.85	193.99	0.24
280	3,000	803.50	817.52	817.75	3.87	775.49	180.86	0.24
315	3,000	804	817.51	817.77	4.11	729.82	73.35	0.23
337.5	Bridge							
360	3,000	804	817.53	817.79	4.10	731.45	73.41	0.23
395	3,000	803.50	817.60	817.81	3.68	835.32	171.13	0.25
475	3,000	803.50	817.69	817.89	3.57	870.29	170.94	0.24
675	3,000	803.50	817.88	818.08	3.59	862.48	169.18	0.24

1 Percent (100-year) Chance Storm

River Station	Q Total (cfs)	Min Ch El (feet)	W.S. Elev (feet)	E.G. Elev (feet)	Vel Chnl (ft./s)	Flow Area (sq ft.)	Top Width (feet)	Froude # Channel
0	3,500	803	817.50	817.76	4.11	921	202	0.27
200	3,500	803.80	817.54	817.81	4.21	853.31	194.48	0.28
280	3,500	803.50	817.53	817.85	4.51	776.28	181.18	0.28
315	3,500	804	817.51	817.87	4.79	729.98	73.36	0.27
337.5	Bridge							
360	3,500	804	817.54	817.90	4.78	732.43	73.45	0.27
395	3,500	803.50	817.64	817.92	4.26	841.60	172.87	0.29
475	3,500	803.50	817.77	818.03	4.11	883.30	173.66	0.27
675	3,500	803.50	818.02	818.28	4.1	885.61	173.97	0.27

0.2 Percent (500-year) Chance Storm

River Station	Q Total (cfs)	Min Ch El (feet)	W.S. Elev (feet)	E.G. Elev (feet)	Vel Chnl (ft./s)	Flow Area (sq ft.)	Top Width (feet)	Froude # Channel
0	6,000	803	817.50	818.26	7.04	921	202	0.47
200	6,000	803.80	817.62	818.41	7.13	865.20	198.47	0.48
280	6,000	803.50	817.59	818.51	7.67	782.43	183.69	0.48
315	6,000	804	817.52	818.57	8.21	731.04	73.40	0.46
337.5	Bridge							
360	6,000	804	817.69	818.70	8.07	743.17	73.83	0.45
395	6,000	803.50	818.05	818.77	6.85	903.30	189.92	0.45
475	6,000	803.50	818.40	819.02	6.39	1000.27	196.42	0.41
675	6,000	803.50	818.96	819.52	6.08	1066.16	207.62	0.38

Note: Shaded numbers are used in Standard Drainage Tabulation Form (Table 6-11).

Table 6-9 Proposed 1 Percent Chance (100-year) Storm Cross Section Data

River Station: 475					
E.G. Elev (ft.)	818.03	Element	Left OB	Channel	Right OB
W.S. Elev (ft.)	817.77	Reach Len. (ft.)	80	80	80
Crit W.S. (ft.)		Flow Area (sq ft.)	16.09	842.20	25.02
E.G. Slope (ft./ft.)	0.001246	Area (sq ft.)	16.09	842.20	25.02
Q Total (cfs)	3,500	Flow (cfs)	12.45	3,463.41	24.14
Top Width (ft.)	173.66	Top Width (ft.)	25.37	120	28.29
Vel Total (ft./s)	3.96	Avg. Vel. (ft./s)	0.77	4.11	0.97

River Station: 337.5 (Bridge Upstream Face)					
E.G. Elev (ft.)	817.89	Element	Left OB	Channel	Right OB
W.S. Elev (ft.)	817.51	Reach Len. (ft.)	35	35	35
Crit W.S. (ft.)	810.56	Flow Area (sq ft.)		702.34	
E.G. Slope (ft./ft.)	0.000366	Area (sq ft.)		702.34	
Q Total (cfs)	3,500	Flow (cfs)		3,500	
Top Width (ft.)	69.35	Top Width (ft.)		69.35	
Vel Total (ft./s)	4.98	Avg. Vel. (ft./s)		4.98	

River Station: 337.5 (Bridge Downstream Face)					
E.G. Elev (ft.)	817.89	Element	Left OB	Channel	Right OB
W.S. Elev (ft.)	817.51	Reach Len. (ft.)	10	10	10
Crit W.S. (ft.)	810.56	Flow Area (sq ft.)		702.34	
E.G. Slope (ft./ft.)	0.000366	Area (sq ft.)		702.34	
Q Total (cfs)	3,500	Flow (cfs)		3,500	
Top Width (ft.)	69.35	Top Width (ft.)		69.35	
Vel Total (ft./s)	4.98	Avg. Vel. (ft./s)		4.98	

Note: Shaded numbers are used in scour calculations.

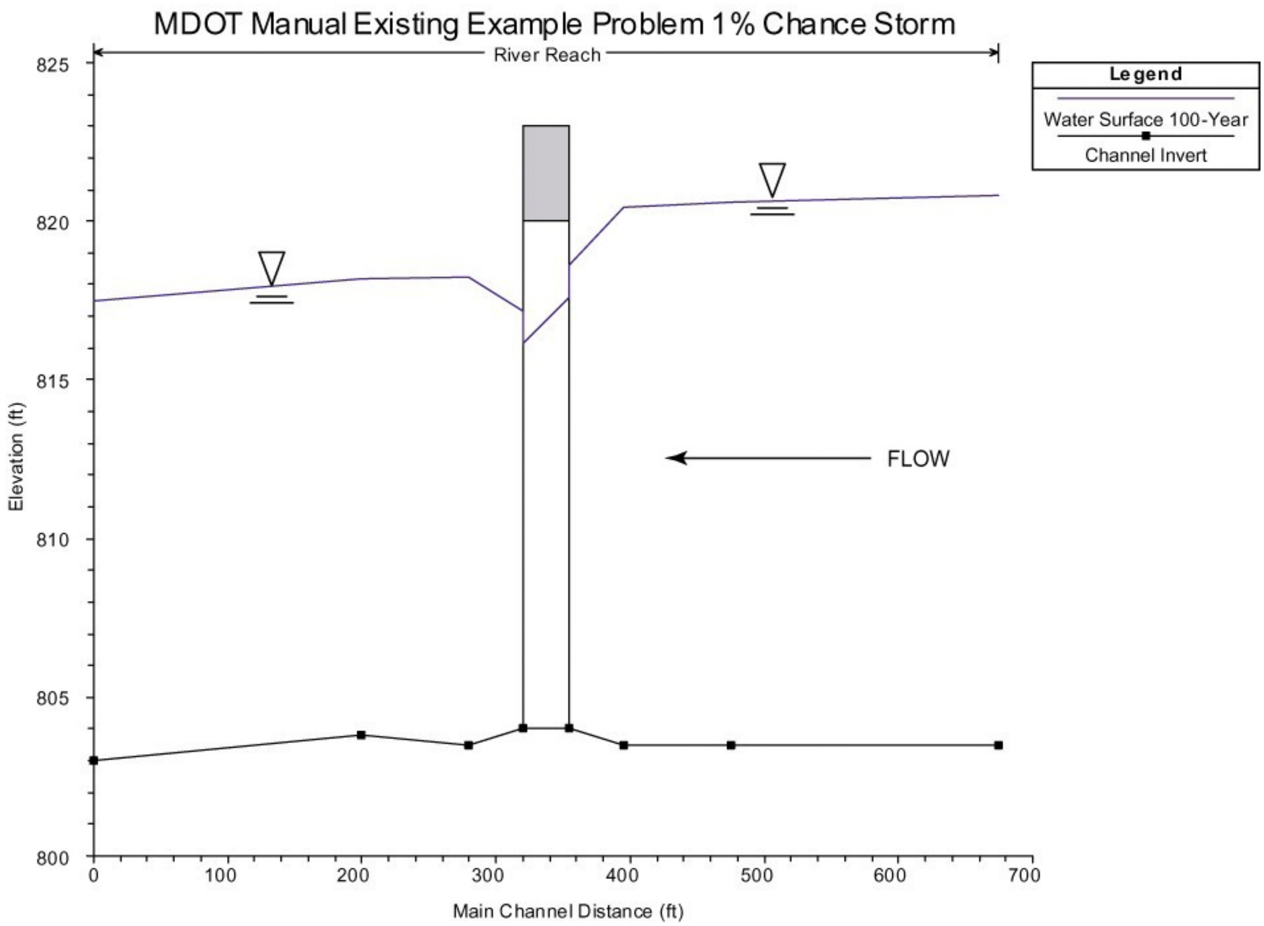
Table 6-10 Proposed 0.2 Percent Chance (500-year) Storm Cross Section Data

River Station: 475					
E.G. Elev (ft.)	819.02	Element	Left OB	Channel	Right OB
W.S. Elev (ft.)	818.40	Reach Len. (ft.)	80	80	80
Crit W.S. (ft.)		Flow Area (sq ft.)	36.12	918.05	46.1
E.G. Slope (ft./ft.	0.002682	Area (sq ft.)	36.12	918.05	46.1
Q Total (cfs)	6,000	Flow (cfs)	53.68	5,866.31	80.02
Top Width (ft.)	196.42	Top Width (ft.)	38.01	120	38.41
Vel Total (ft./s)	6	Avg. Vel. (ft./s)	1.49	6.39	1.74

River Station: 337.5 (Bridge Upstream Face)					
E.G. Elev (ft.)	818.65	Element	Left OB	Channel	Right OB
W.S. Elev (ft.)	817.52	Reach Len. (ft.)	35	35	35
Crit W.S. (ft.)	812.93	Flow Area (sq ft.)		703.34	
E.G. Slope (ft./ft.)	0.001073	Area (sq ft.)		703.34	
Q Total (cfs)	6,000	Flow (cfs)		6,000	
Top Width (ft.)	69.34	Top Width (ft.)		69.34	
Vel Total (ft./s)	8.53	Avg. Vel. (ft./s)		8.53	

River Station: 337.5 (Bridge Downstream Face)					
E.G. Elev (ft.)	818.65	Element	Left OB	Channel	Right OB
W.S. Elev (ft.)	817.52	Reach Len. (ft.)	10	10	10
Crit W.S. (ft.)	812.92	Flow Area (sq ft.)		703.34	
E.G. Slope (ft./ft.)	0.001073	Area (sq ft.)		703.34	
Q Total (cfs)	6,000	Flow (cfs)		6,000	
Top Width (ft.)	69.34	Top Width (ft.)		69.34	
Vel Total (ft./s)	8.53	Avg. Vel. (ft./s)		8.53	

Note: Shaded numbers are used in scour calculations.



Note: Abrupt change in water surface at bridge depicts rapidly varied transition at existing bridge, which is corrected by proposed bridge (See Figure 6-8).

Figure 6-7 Existing Conditions Water Surface Profile During 1 Percent Chance (100-year) Storm (Printout from HEC-RAS)

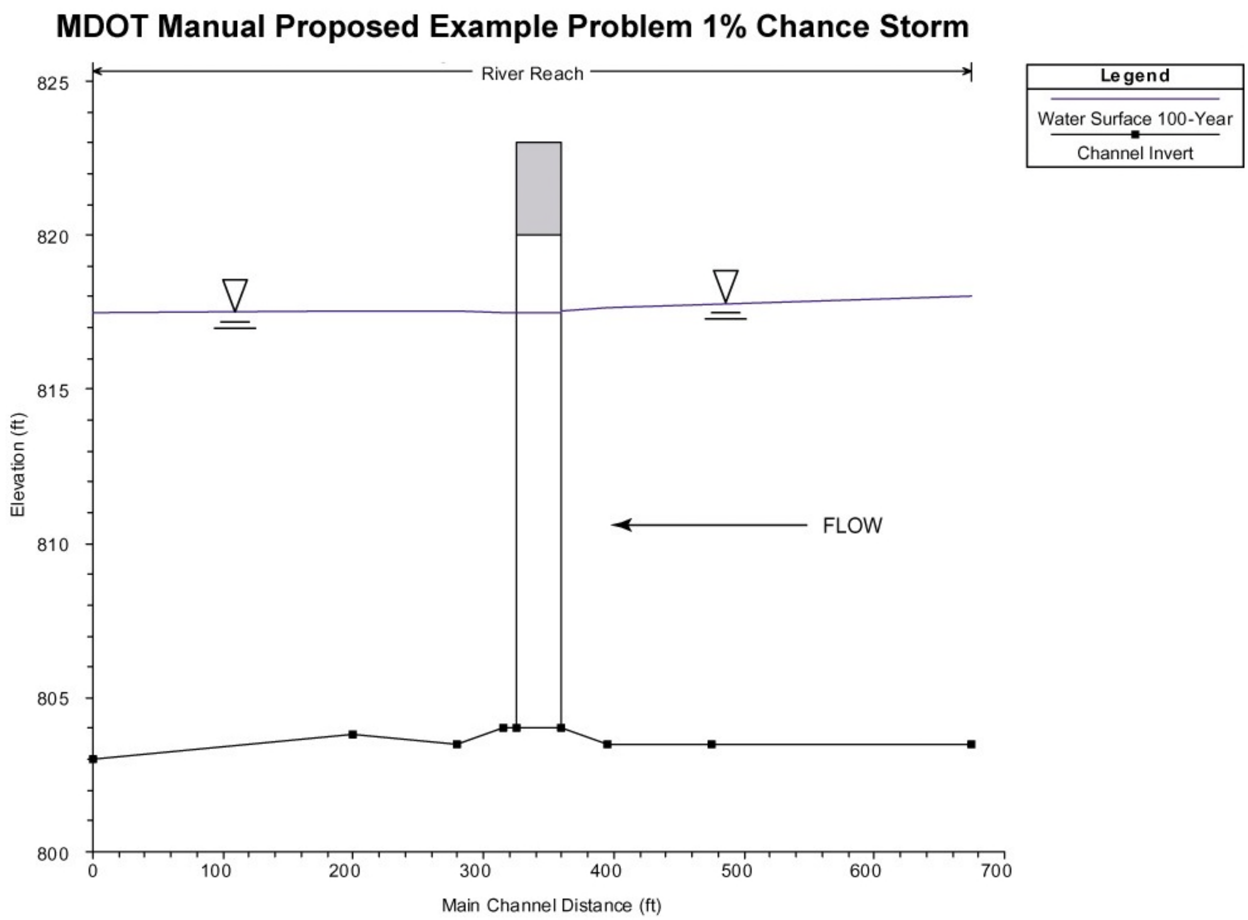


Figure 6-8 Proposed Conditions Water Surface Profile During 1 Percent Chance (100-year) Storm (Printout from HEC-RAS)

Table 6-11 Standard Drainage Tabulation Form

Flood Data	EXISTING CONDITIONS			PROPOSED CONDITIONS			
	Discharge (cfs)	Water Surface Elev. at Upstream Face of Structure (feet)	Velocity at Down-stream Face (fps)	Water Surface Elev. at Upstream Face of Structure (feet)	Velocity at Down-stream Face (fps)	Waterway Area at Down-stream Face (sf)	Change in Water Surface Elev. 35 feet. Upstream of Proposed Structure (feet)
2 Percent Chance (50-year) Storm	3,000	817.64	6.40	817.53	4.11	729.82	-0.55*
1 Percent Chance (100-year) Storm	3,500	817.70	7.48	817.54	4.79	729.98	-0.75**
0.2 Percent Chance (500-year) Storm	6,000	818.61	13.29	817.69	8.21	731.04	-2.37***

Note: 1. The drainage area contributing to this crossing is 30 square miles.

2. The water surface and/or energy grade elevations shown on the above hydraulic table are to be used for comparison purposes only and are not to be used for establishing a regulatory floodplain.

* Change in WSEL during 2 percent (50-year) chance storm
= (proposed conditions WSEL) - (existing conditions WSEL)
= 817.60 - 818.15 = -0.55 foot.

** Change in WSEL during 1 percent (100-year) chance storm
= 817.64 - 818.39 = -0.75 foot.

*** Change in WSEL during 0.2 percent (500-year) chance storm
= 818.05 - 820.42 = -2.37 feet.

Hydraulic Analysis Findings

The analysis shows that the proposed bridge will create a lower water surface (and energy grade line elevation) than the existing bridge. The stream velocities will also be reduced to reasonable levels (during smaller flows). There also appears to be reasonable freeboard between the water surface and low chord of the bridge. It can be concluded that the proposed bridge does not create a harmful interference and the bridge opening is reasonable.

Options the designer can consider if a further reduction in water surface is desired include:

- Widening the structure.
- Relief bridge.
- Improving flow transitions between the channel and bridge.

<u>Scour Analysis - 1 Percent Chance (100-year) Flood</u>

A Level One assessment shows that there is little reason to believe the channel will aggrade or degrade. Completed worksheets for this example problem are included at the end of the example. Blank forms are located in Appendix 6-D. Furthermore, the channel appears horizontally stable. Therefore, there is no need to assume long-term profile or plan form changes.

<u>Contraction Scour</u>

First find depth of scour for 1 percent chance (100-year) storm. From sieve analysis, the D_{50} in the channel is:

D_{50} = 0.000328 ft. = 1 x 10^{-1} mm

V_c = $11.17\ y_1^{1/6}\ D_{50}^{\ 1/3} = 11.17\ (14.14)^{1/6}\ (0.000328)^{1/3} = 1.20$ fps
V_c = Critical velocity which will transport bed materials of size D_{50} and smaller
y_1 = Depth of upstream flow, feet
(Found from HEC-RAS model run at upstream side of bridge, Station 395, 1 percent chance (100-year) storm, water surface elevation. Subtract the minimum channel elevation (803.5 feet), Table 6-8)

The velocity at the upstream side of the bridge is 4.78 fps (from HEC-RAS model run, Table 6-8, Station 360).

Because 4.78 > 1.20, consider the channel to have live bed conditions. Equation 6.2 for calculation of live bed contraction scour is:

$$y_2/y_1 = (Q_2/Q_1)^{6/7}\ (W_1/W_2)^{K_1}$$
$$y_s = y_2 - y_0$$

Use Station 475 as the first fully expanded upstream cross section (see Table 6-9).

Q_1 = 3,463 cfs
A_1 = 842.2 sf
V_1 = Q_1/A_1 = 3,463/842.2 = 4.1 fps
W_1 = 120 ft.
y_1 = A_1/W_1 = 842.2/120 = 7.01 feet
y_0 = y_1 = 7.01 feet

Use Section 337.5 as the station in the contracted channel (see Table 6-9).

Q_2 = 3,500 cfs
A_2 = 702.34 sf
W_2 = 69.35 feet
ω = 0.025 (Figure 6-4, Fall Velocity of Sand-Sized Particles)

$V^* = (g\ y_1\ S_1)^{1/2} = [(32.2)\ (7.01)\ (0.001246)]^{1/2} = 0.53$ fps

Note: S_1 is the energy grade line slope at Station 475, given from HEC-RAS model, Table 6-9. Also, y_1 is the average depth at Station 475 calculated above.

$V^*/\omega = 0.53/0.025 = 21.2$
(ω = 0.025, from Figure 6-4, Fall Velocity of Sand-Sized Particles)

Therefore: $K_1 = 0.69$ (Table 6-1)

Using the live bed equation, find y_s

$y_2/y_1 = (Q_2/Q_1)^{6/7}\ (W_1/W_2)^{K1}$
$y_2/y_1 = (3{,}500/3{,}463)^{6/7}\ (120/69.35)^{0.69}$
$y_2/y_1 = 1.47$
$y_2 = 1.47\ y_1$
$y_2 = 1.47\ (7.01)$
$y_2 = 10.3$ feet
$y_s = y_2 - y_0$
$y_s = y_2 - y_0$
$= 10.3 - 7.01$
y_s = 3.3 feet (Contraction scour)

<u>Pier Scour</u> (updated January 2004)

Find the scour at the pier using the pier scour equation:

$y_s/y_1 = 2.0\ K_1\ K_2\ K_3\ \ K_4\ (a/y_1)^{0.65}\ Fr_1^{\ 0.43}$

y_1 = 7.01 feet
K_1 = 1.0, round nose pier (Table 6-2)
K_2 = 1.0, angle zero (Table 6-3)
K_3 = 1.1, clear water scour (Table 6-4)
K_4 = 1, (D_{50} = 1 x 10^{-1} mm)< 2 mm (Section 6.4.5.3.2)
a = 2.0 feet
V_1 = 4.1 fps
Fr = $V/(g\ y_1)^{1/2} = 4.1/(32.2 \times 7.02)^{1/2} = 0.27$

$y_s/7.01 = 2.0\ (1.0)\ (1.0)\ (1.1)\ (2/7.01)^{0.65}\ (0.27)^{0.43}$

y_s = 3.9 feet (Pier scour)

Pile caps will be set below contraction scour depth. Therefore, no adjustments to calculated depth are needed.

<u>Abutment Scour</u>

Find scour at the abutments; begin with the left side (looking downstream). The equation for live bed scour at abutments is:

$y_s/y_a = 2.27\ K_1\ K_2\ (a')^{0.43}\ Fr^{\ 0.61} + 1$

At Section 475:

K_1 = 0.55, spill through abutment (Table 6-5)
K_2 = 1.0, angle zero (Table 6-3)
Q_e = 12.45 cfs
A_e = 16.09 sf
V_e = 0.77 fps
a’ = 25.37 feet
y_a = 16.09/25.37 = 0.63 foot
Fr = $0.77/(32.2 \times 0.63)^{1/2} = 0.17$

$y_s/0.63 = 2.27\ (0.55)\ (1.0)\ (25.37/0.63)^{0.43}\ (0.17)^{0.61} + 1$

y_s = 1.9 feet (Left abutment scour)

Find the scour for the right abutment:

$y_s/y_a = 2.27\ K_1 K_2\ (a'/y_a)^{0.43}\ Fr^{\ 0.61} + 1$

K_1 = 0.55 (Table 6-5)
K_2 = 1.0 (Table 6-4)
Q_e = 24.14 cfs
A_e = 25.02 sf
V_e = 0.97 fps
a' = 28.29 feet
y_a = 25.02/28.29 = 0.88 foot
Fr = $0.97/(32.2 \times 0.88)^{1/2} = 0.18$

$y_s/0.88 = 2.27\ (0.55)\ (1.0)\ (28.29/0.88)^{0.43}\ (0.18)^{0.61} + 1$

y_s = 2.6 feet (Right abutment scour)

Total scour is the sum of long-term bed change, contraction scour, and local scour. These are also tabulated on the completed worksheets (at the end of this example). Note: long-term bed change is assumed to be zero.

Left abutment scour = (3.3) + (1.9) = 5.2 feet

Right abutment scour = (3.3) + (2.6) = 5.9 feet

Pier scour = (3.3) + (4.0) = 7.3 feet

Scour Analysis - 0.2 Percent Chance (500-year) Flood

Contraction Scour

Find the contraction scour in the channel.

$V_c = 11.17\ y_1^{1/6}\ D_{50}^{1/3} = 11.17\ (14.55)^{1/6}\ (0.000328)^{1/3} = 1.20$

y_1 = 818.05 – 803.50 = 14.55 feet (Table 6-8, 0.2 percent, Station 395)

Velocity at upstream side of the bridge (Station 355) is 11.32 fps.

11.32 > 1.20 therefore live bed conditions

$y_2/y_1 = (Q_2/Q_1)^{6/7}\ (W_1/W_2)^{\ K_1}$

Use Station 475 as the first fully expanded cross section (Table 6-10).

Q_1 = 5,866.31 cfs
A_1 = 918.05 sf
V_1 = 6.39 fps
W_1 = 120 feet
y_1 = 918.05/120 = 7.65 feet
y_0 = y_1 = 7.65 feet

Use Station 337.5 as the section in the contracted channel.

Q_2 = 6,000 cfs
A_2 = 703.34 sf
W_2 = 69.34 feet
ω = 0.025 (Figure 6-4, Fall velocity of Sand-Sized Particles)

$V_* = (g\ y_1\ S_1)^{1/2} = [(32.2)\ (7.65)\ (0.002682)]^{1/2} = 0.81$ fps

Note: S_1 is the EGL slope at Station 475 given from HEC-RAS Model Table 6-10. Y_1 is the average depth at Station 475, calculated above.

$V_*/\omega = 0.81/0.025 = 32.4$

Therefore: $K_1 = 0.69$ (Table 6-1)

Using the live bed equation:

$$y_2/y_1 = (Q_2/Q_1)^{6/7}(W_1/W_2)^{0.69}$$

$$y_2/7.65 = (60000/5866.3)^{6/7}\ (120/69.34)^{0.69}$$

$$y_2 = 11.35 \text{ feet}$$

$$y_s = y_2 - y_0 = 11.35 - 7.65$$

y_s = 3.7 feet (Contraction scour)

Pier Scour (updated January 2004)

Find the scour at the pier:

$$y_2/y_1 = 2.0\ K_1\ K_2\ K_3\ K_4\ (a/y_1)^{0.65}\ Fr_1^{0.43}$$

y_1 = 7.65 feet
K_1 = 1.0 (Table 6-3)

K_2 = 1.0 (Table 6-4)
K_3 = 1.1 (Table 6-2)
K_4 = 1.0 (Section 6.5.6.2)
a = 2
$Fr_1 = V_1/(g\ y_1)^{1/2} = 6.39/(32.2 \times 7.65)^{1/2} = 0.40$

$y_s/7.65 = 2.0\ (1.0)\ (1.0)\ (1.1)\ (2/7.65)^{0.65}\ (0.40)^{0.43}$

y_s = 4.8 feet (Pier scour)

Find scour at the abutments; begin with the left side.

$y_s/y_a = 2.27\ K_1 K_2\ (a'/y_a)^{0.43}\ Fr^{0.61} + 1$

At Section 475 (Table 6-10):

Q_e = 53.68 cfs
A_e = 36.12 sf
V_e = 1.49 fps
a' = 38.01 feet
y_a = (36.12/38.01) = 0.95 feet
$Fr = 1.49/(32.2 \times 0.95)^{1/2} = 0.27$
K_1 = 0.55 (Table 6-5)
K_2 = 1.0 (Table 6-4)

$y_s/0.95 = 2.27\ (0.55)\ (1.0)\ (38.01/0.95)^{0.43}\ (0.27)^{0.61} + 1$

y_s = 3.6 feet (Left abutment scour)

Find the scour for the right abutment:

$y_s/y_a = 2.27\ K_1 K_2\ (a'/y_a)^{0.43}\ Fr^{0.61} + 1$

At Section 475:

Q_e = 80.02 cfs
A_e = 46.1 sf
V_e = 1.74 fps
a' = 38.41 feet
y_a = (46.1/38.41) = 1.20 feet
$Fr = 1.74/(32.2 \times 1.20)^{1/2} = 0.28$
K_1 = 0.55 (Table 6-5)
K_2 = 1.0 (Table 6-4)

$y_s/1.20 = 2.27\ (0.55)\ (1.0)\ (38.41/1.20)^{0.43}\ (0.28)^{0.61} + 1$

y_s = 4.3 feet (Right abutment scour)

Total scour is the sum of long-term bed change, contraction scour, and local scour. These are also tabulated in the completed worksheets (at the end of this example). Note: long-term bed change is assumed to be zero.

Left abutment scour = (3.7) + (3.6) = 7.3 feet

Right abutment scour = (3.7) + (4.3) = 8.0 feet

Pier scour = (3.7) + (4.8) = 8.5 feet

Summary

Plot the bridge section and show the scour for the 1 percent (100-year) chance storm and the 0.2 percent (500-year) chance storm (Figures 6-9, 1 Percent Chance (100-year) Storm Scour, and Figure 6-10, 0.2 Percent Chance (500-year) Storm Scour). Design the foundation using this data.

Fill out the scour analysis worksheets provided in Appendix 6-D. For additional guidance, see the following forms filled out for this example problem.

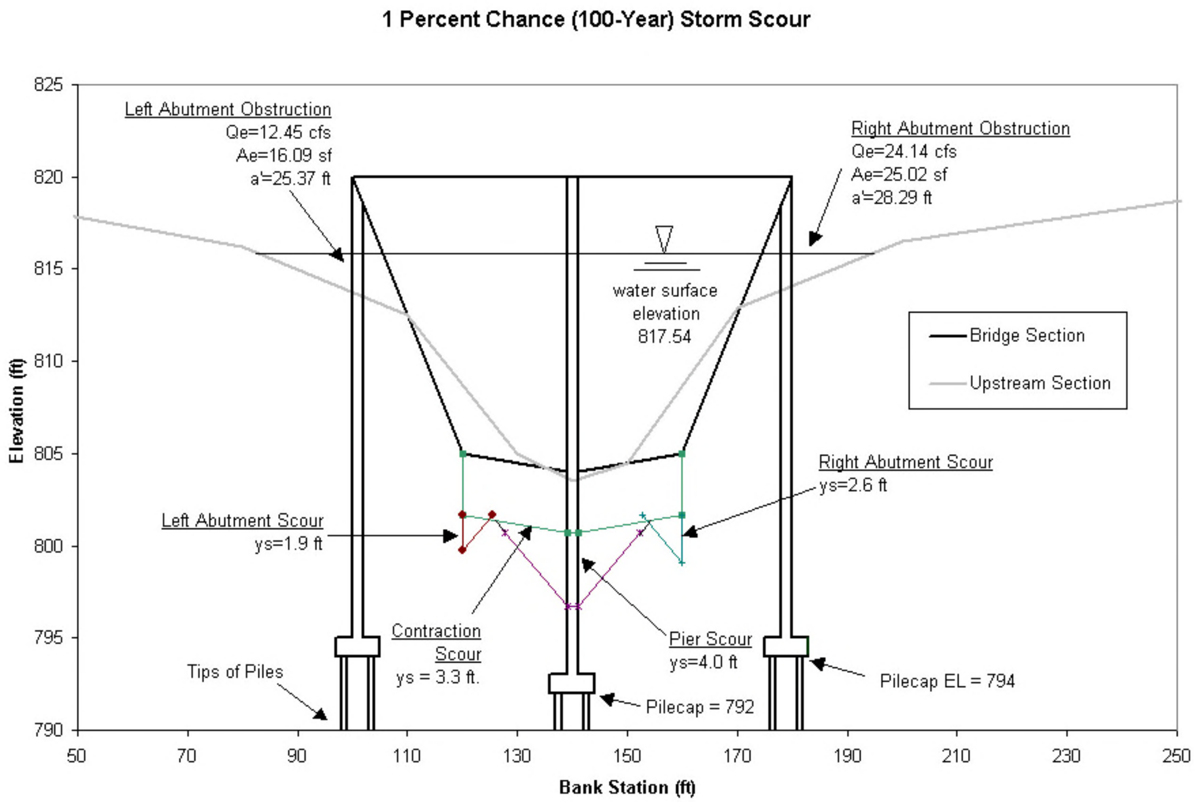


Figure 6-9 1 Percent Chance (100-year) Storm Scour

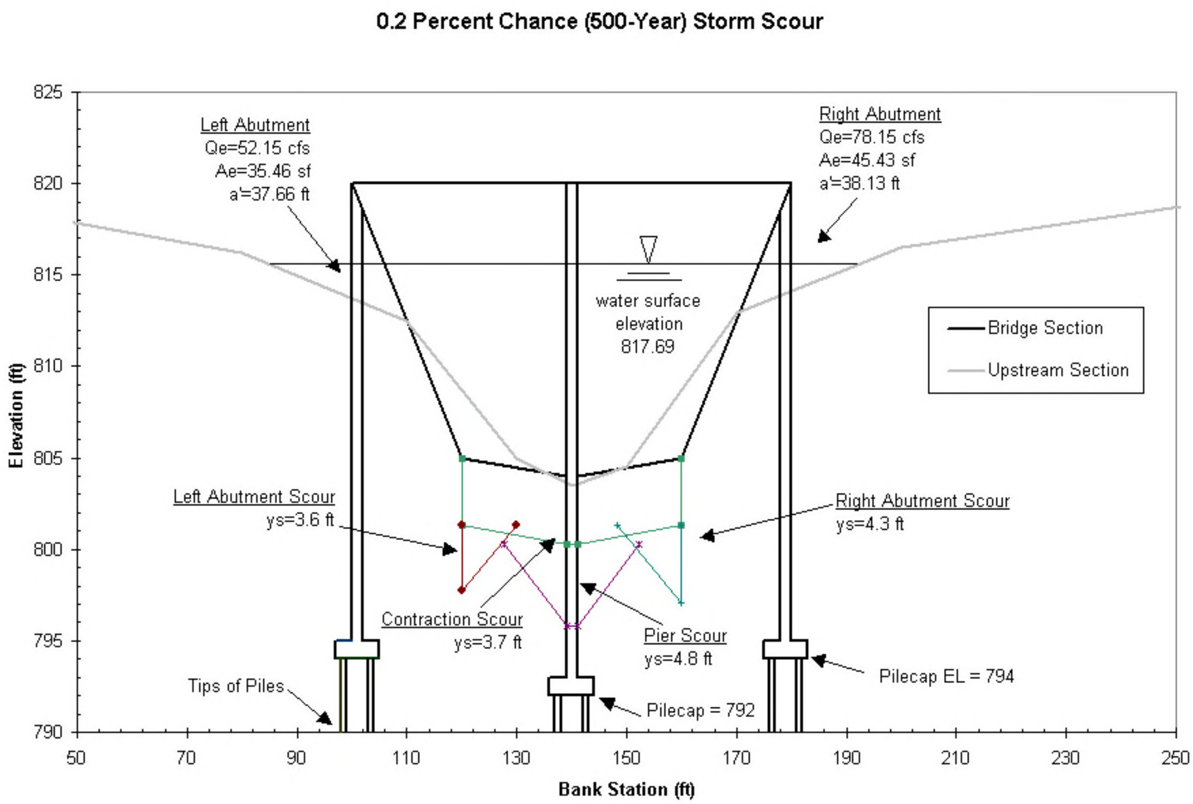


Figure 6-10 0.2 Percent Chance (500-year) Storm Scour

MICHIGAN DEPARTMENT OF TRANSPORTATION

LEVEL ONE SCOUR ANALYSIS WORKSHEET

Date: May 2003 By: MJH Structure No: 1 Control Section: 81104

Job No. 48847C Route: 94 Watercourse: Icicle Creek

All references are to HEC-20, 3rd Edition.

Data Collection

x Plans
x Bridge Inspection Reports (Maintenance Division)
x Underwater Inspection Reports (Maintenance Division)
x Review existing items 60, 61, 71, 92, 93, and 113 of the NBIS
x Review available construction, design, and maintenance files for repair and maintenance work done on structure

Field Investigation Date: April 2003

x Channel bottom width approximately one bridge span upstream = 120 feet

x Overbank and channel Manning's roughness coefficients

0.050 Left 0.020 Channel 0.050 Right

x Is there sufficient riprap? Abutments yes Piers yes

x Photographs

x Cross sections at upstream and downstream faces of bridge

Comments:

Stream Characteristics

x Complete the attached Figure 2.6 from HEC-20 (see Appendix 6-D).

Comments:

Land Use: Identify the existing and past land use of the upstream watershed:

Urban Area	Yes x	No __	Comments: Residential
Sand and Gravel Mining	Yes __	No x	Comments:
Undeveloped Land	Yes x	No __	Comments:

Lateral Stability: Refer to HEC-20, Section 2.3.9 on Channel Boundaries and Vegetation for channel bank stability. Comment: Banks appear stable

Vertical Stability:

	Yes	No
-streambed elevation change from as-built plans?		x
-exposed pier footings (degradation)?		x
-exposed abutment footings (degradation)?		x
-channel bank caving in (degradation)?		x
-eroding floodplain (aggradation)?		x
-crossing at confluence or tributaries?		x
-bridge sites upstream and downstream?	x	
-grade or hydraulic controls, i.e., dams, weirs, diversions?		x
-foundation on rock		x
-channel armoring potential		x

Comments:

Stream Stability: Make a qualitative assessment of the overall stream stability by referring to the above information and Figure 2.6 and Table 3.2 from HEC-20 (attach copies of figures).

Stable_x_ Unstable_____ Degrading _____ Aggrading _____

Comments:

RECOMMENDED NBIS ITEM 113 CODE: 6

LEVEL TWO ANALYSIS NEEDED: YES_X_ NO___

Worksheet approved by:___Smith___P.E. License # _11111_ Date_4/1/2003_

MICHIGAN DEPARTMENT OF TRANSPORTATION
LEVEL TWO SCOUR ANALYSIS WORKSHEET

Date: May 2003 By: MJH

Structure No: 1 Control Section: 81104 Job No. 48847C

Route: I-94 Watercourse: Icicle Creek

Page numbers refer to HEC-20, 3rd Edition and HEC-18, 4th Edition. Attach water surface profile modeling printouts with pertinent variables highlighted. Scour calculations automatically done by HEC-RAS are not acceptable. All calculations must be attached or on the back of their respective pages.

Hydrology:

Method of Analysis: DEQ estimate, SCS, Regression, DAR to gauge, other

Drainage Area: 20 square miles

$Q_{50\ (2\%)}$ = 3,000 cfs $Q_{100\ (1\%)}$ = 3500 cfs $Q_{500\ (0.2\%)}$ = 6,000 cfs

Hydraulics: Water surface profiles by: HEC-RAS x OTHER ____

Geotechnical: Bed and overbank material values:

D_{50} 3.28×10^{-4} D_{84} ___ (feet) Left Overbank

D_{50} 3.28×10^{-4} D_{84} ___ (feet) Right Overbank

D_{50} 3.28×10^{-4} D_{84} ___ (feet) Main Channel

Source of information: Sieve analysis

Incipient motion analysis: For gravel and cobble streams only. Refer to Page 6.14 of HEC-20.

Armoring potential: Refer to Page 6.16 of HEC-20.

Scour calculations

LONG-TERM BED ELEVATION CHANGES – AGGRADATION/DEGRADATION

_x_ Use information from **Level One** Analysis

___Use information from bridge inspection reports

___ Estimate change during the next 100 years if enough information exists

Estimated aggradation/degradation = _____ feet

*** Do not adjust fixed bed hydraulics for contraction scour and local scour. If channel has aggraded, do not adjust the estimated scour depth.

CONTRACTION SCOUR (Section 5.2, HEC-18)

Bridge Site Condition:

CASE: 1a__ 1b__ 1c__ 2 x_ 3__ 4__

Compare critical velocity V_c to the mean velocity V.

$V_c = 11.17\ y^{1/6}\ D^{1/3}$ (p. 5.2, HEC-18)

y = 14.14 ft.

D_{50} = 3.28×10^{-4}

V_c = 1.20 fps

_x_ If $V_c<V$, use Laursen's Live-Bed contraction scour.

____ If $V_c>V$, use Laursen's Clear-Water contraction scour.

____ If coarse sediments in bed material, see Page 5.12, HEC-18.

Laursen's live-bed scour equation (p 5.10, HEC-18):

$y_2/y_1 = (Q_2/Q_1)^{6/7}(W_1/W_2)^{k1}$ and

$y_s = y_2 - y_0$ = average contraction scour depth (feet)

y_1 =	7.01 feet	V^* =	0.53 ft./s
y_2 =	10.13 feet	ω =	0.53 ft./2
y_0 =	7.01 feet	S_1 =	1.246×10^{-3} ft./ft.
W_1 =	120 feet	V^*/ω =	21.2
W_2 =	69.35	k_1 =	0.69
Q_1 =	3,463	y_s =	3.3 feet
Q_2 =	3,500		

Laursen's Clear-Water Contraction Scour (p. 5.12, HEC-18)

$y_2 = (0.0077\, Q^2/(D_m^{\ 2/3}\, W^{\ 2}))^{3/7}$

$y_s = y_2 - y_0$ = average scour depth (feet)

y_0 = ___________ feet		D_m = ___________ feet	
y_2 = ___________ feet		D_{50}= ___________ feet	
Q = ___________ cfs		y_s = ___________ feet	
W = ___________ feet			

LOCAL SCOUR

ABUTMENTS

Froehlich's live-bed scour equation: (If $L'/y_1 > 25$, use HIRE equation, p. 7.8, HEC-18.)

Froehlich's equation: $y_s / y_a = 2.27\ K_1\ K_2\ (L'/y_a)^{.43}\ (Fr)^{0.61} + 1$ (p. 7.8, HEC-18)

	Left Abutment	Right Abutment
K_1 =	0.55	0.55
K_2 =	1.0	1.0
L' =	25.37 feet	28.29 feet
L' =	16.09 $feet^2$	16.09 $feet^2$
Q_e =	12.45 cfs	24.14 cfs
V_e =	0.77 ft./s	0.97 ft./s
Fr =	0.17	0.18
y_a =	0.63 feet	0.88 feet
y_s =	1.9 feet	2.6 feet

PIER(S)

Colorado State University equation (p. 6.2, HEC-18):

$$y_s/y_1=2.0\ K_1\ K_2\ K_3\ K_4\ (a/y_1)^{0.65}(Fr_1)^{0.43}$$

Pier #: 1

y_1	=	7.01 feet	feet	feet
K_1	=	1.0		
K_2	=	1.0		
K_3	=	1.1		
K_4	=	1.0		
a	=	2.0 ft.		
V_1	=	4.1 ft./s	ft./s	ft./s
Fr_1	=	0.27		
y_s	=	4.0		

Note: If there is a possibility of channel migration, use the worst case condition for all piers. For complex pier foundations, see Section 6.4, HEC-18.

SUMMARY

1 Percent Chance (100-year)

Element	Long-term (ft.)	Contraction (ft.)	Local (ft.)	Total (ft.)
Left Abutment	0	3.3	1.9	5.2
Right Abutment	0	3.3	2.6	5.9
Pier # 1	0	3.3	4.0	7.3
Pier #				
Pier #				

Adjust total scour depth as needed if scour holes overlap.

0.2 Percent Chance (500-year)

Element	Long-term (ft.)	Contraction (ft.)	Local (ft.)	Total (ft.)
Left Abutment	0	3.7	3.6	7.3
Right Abutment	0	3.7	4.3	8.0
Pier # 1	0	3.7	4.8	8.5
Pier #				
Pier #				

_x_ Attach sketch or marked copy of existing design plan showing 100-year and 500-year total scour depths in relation to foundation. Foundation elevations must be shown.

Geotechnical Evaluation of scour results by: ___________________

Structural Evaluation of scour results by: _____________________

Is the structure stable under the estimated scour depth presented in this scour evaluation?

Yes _x_ No ____

RECOMMENDED NBIS ITEM 113 CODE: ___8___ (p. J.14, HEC-18)

ATTACHMENTS:

1. Calculations
2. Water surface profile computer output with pertinent values highlighted
3. Sketch of bridge with scour depths in relation to foundation
4. Scour countermeasure calculations with plans showing limits of countermeasures
5. Recommended plan of action

Worksheet approved by: __Smith____ Date: _5/1/2003____

P.E. LICENSE# __11111__

Additional comments:

6.4.8 Bridge Deck Drainage

Drainage of bridge decks is similar to curbed roadway sections. It is often less efficient because cross slopes are flatter, parapets collect large amounts of debris, and small drainage inlets or scuppers have a higher potential for clogging by debris. Bridge deck construction usually requires a constant cross slope. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal.

The gutter spread should be checked to ensure compliance with the design criteria in Chapter 7, Road Storm Drainage Systems, Section 7.2. Zero gradients and sag vertical curves should be avoided on bridges. The minimum desirable longitudinal slope for bridge deck drainage should be 0.5 percent. In many areas of the country, scuppers are the recommended method of deck drainage because they can reduce the problems of transporting a relatively large concentration of runoff in an area of generally limited right-of-way. They also have a low initial cost and are relatively easy to maintain. However, the use of scuppers should be evaluated for site-specific concerns. Scuppers should not be located over embankments, slope pavement, slope protection, navigation channels, driving lanes, or railroad tracks. Runoff collected and transported to the end of the bridge should generally be collected by inlets and downdrains, although sod flumes may be used for extremely minor flows in some areas. Runoff should also be handled in compliance with applicable stormwater quality regulations.

There may be no need for inlets on the bridge if the necessary length between inlets is longer than the bridge (FHWA, HEC-21, 1993). To compute the inlet spacing, follow procedures in HEC-21.

6.5 MAINTENANCE

6.5.1 Introduction

Bridges must be kept free of obstructions. Debris and vegetative growth under a bridge may contribute to scour, create a potential fire hazard, and reduce freeboard for ice and debris during high-water flows, resulting in a serious threat to the bridge. A reduced effective flow area under the bridge may also result in excessive bridge backwater damage, more frequent roadway overtopping, and a hazard to the traveling public.

Maintaining a channel cross-section record and revising it as significant changes occur, provides an invaluable record of the tendency toward scour, channel shifting, and degradation or aggradation. A study of these characteristics can help predict when protection of pier and abutment footings may be required.

6.5.2 Inspection

Maintenance inspection must be commensurate with the risk involved. Where probing and or diving are necessary, the inspection should be scheduled at the season of lowest water elevation. High water, high ice, and debris marks with the date of occurrence should be recorded for future reference. Bridge cross sections should be taken every 4 to 6 years. Cross sections are used to track long-term changes in the channel.

6.5.3 Maintenance Problems

Following are some of the maintenance problems that can be encountered.

- Clogging of bridge deck drains and scuppers, which may create a hazard to traffic and contribute to deck deterioration.
- Discharges of bridge deck drains that are detrimental to other members of the bridge and those spilling onto a traveled way below. In addition, discharges that may cause fill and bank erosion.
- Clogging of air vents in the superstructure or deck of bridges subject to overtopping which may increase buoyancy forces and the possibility of bridge washouts.
- Accumulation of debris in the open space between the handrails of bridges subject to overtopping which may induce additional lateral forces on the bridges and increase the risk of washouts.
- Channel aggradation or degradation.
- Scour at piers and abutments caused by accumulation of debris and or excessive velocities.
- Damage to bridge approach embankment caused by channel encroachment.
- Loss of riprap due to erosion, scour, and wave action.

- Damage to bridge elements due to debris, ice jams, and excessive velocities.
- Missing navigational signs and lights over navigable channels.

6.5.4 Maintenance Measures

Maintenance measures include the following:

- Repair of damaged bridge elements.
- A schedule for removal of debris after major floods.
- Removal of sand and gravel bars in the channel that may direct stream flow in such a manner as to cause harmful scour at piers and abutments.
- Cleaning bridge deck drains and keeping their outlets away from traffic underneath. Also providing riprap or other means of protection at outlets to avoid fill and bank erosion.
- Removal of debris caught between bridge handrails and opening vent holes designed to reduce buoyancy.
- Making a channel change when necessary to redirect the flow away from bridge approaches and in line with the bridge skew.
- Dredging of channels that are subjected to a high degree of aggradation in order to maintain waterway adequacy.
- Construction of scour counter-measures where needed.
- Constructing cutoff walls to reduce or stop progressive channel degradation.
- Replacing lost dirt in scour holes and constructing riprap mats or other means of protection for undermined piers and abutments.
- Replacing missing riprap on embankment slopes, channel banks, spur dikes, etc.
- Replacing missing or damaged navigational signs and lights.
- Construction of additional openings to accommodate increased urbanization in the drainage area upstream from the bridge.
- Modifying or increase existing protective measures when needed.

References

American Association of State Highway and Transportation Officials. Volume VII-Highway Drainage Guidelines, Hydraulic Analyses for the Location and Design of Bridges. AASHTO Task Force on Hydrology and Hydraulics. 1999.

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U.S. Army Corps of Engineers. HEC-RAS River Analysis System. User's Manual-Draft, Version 3.0. January 2001.

U.S. Army Corps of Engineers. Accuracy of Computed Water Surface Profiles. December 1986.

Note: References in bold type are recommended reading for the engineer's library.

Weblinks

MDOT Bridge Design Manual

http://mdotcf.state.mi.us/public/design/englishbridgemanual/

FHWA Hydraulic Engineering Circulars (HEC Reports)

http://www.fhwa.dot.gov/engineering/hydraulics

- HEC-18, Evaluating Scour at Bridges
- HEC-12, Drainage of Highway Pavements
- HEC-20, Stream Stability at Highway Structures
- HEC-21, Design of Bridge Deck Drainage
- HEC-23, Bridge Scour and Stream Instability Countermeasures
- Other FHWA publications

MDEQ, Michigan Department of Environmental Quality

http://www.michigan.gov/deq

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