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Regions are from USGS Water Resources Investigation Report 87-4132, 1987. Q50 should be determined from that publication. Q500 can then be determined using this chart.
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Introduction

The most common cause of bridge failures in the nation is flooding, with bridge scour being the most common type of flood damage. Bridge scour is a complicated process and provides challenges to engineering analysis. Because of public safety and high replacement and repair costs, the need exists to evaluate or improve current design and maintenance practices concerning bridge foundations.

The objective in this document is to detail three items:

1. Factors that affect scour.
2. Recommendations to reduce or prevent scour effects on existing and proposed bridges.
3. Methods to estimate scour for existing and proposed structures.
Definition

A basic definition of scour is the result of erosive action of moving water as it excavates and carries away material from a streambed and banks. There are two types of scour:

1. General scour - the loss of material from most or all the bed and banks, usually caused by the road embankment encroaching onto the flood plain with resulting contraction of the flood flow (often called contraction scour).
2. Local scour – the loss of material around piers, abutments, spur dikes and embankments.

There are two conditions for contraction and local scour: clear-water and live-bed. Clear-water scour occurs when there is little to no movement of the bed material of the stream upstream of the crossing. Typical situations include most overflow bridges, coarse bed material streams, and flat gradient streams during low flow. Live-bed scour occurs when velocities are high enough to move the bed material upstream of the crossing. Most Iowa streams and rivers experience live-bed scour.

Streambed degradation, such as in the Western Iowa loess region, is considered in some documents to be scour. Even though degradation can affect structural stability like local or general scour does, the causes of degradation are of a different nature, and it will not be discussed in detail in this document.

The effects of scour are a complex problem involving geotechnical, hydraulic, and structural concerns, so decisions concerning scour should involve engineers in each of these disciplines.

Design guidelines and considerations

Numerous factors affect the stability of the bed and banks of a stream and are discussed below with some guidelines and considerations.

1. Soils

Soils with any combination of sand or silt have greater potential for scour: sand, silt, sandy silt, sandy silty clay, etc. As a general rule, according to IDOT's Soils Design Section, soils which have a blow count of ten or less are particularly susceptible.

Excessive loss of pile bearing due to scour is one cause for bridge damage or failure. However, perhaps a more common cause of failure is soil instability associated with the road embankment and bridge berm. Often a bridge berm or fill behind a high abutment has minimal factor of safety for stability. If this safety factor is reduced due to scour at the toe of the embankment, the soil may become unstable resulting in a slip failure. Damage to an abutment, pier or approach fill is a possible outcome.

For replacement structures, designing flatter berm slopes and/or placing the abutments farther from the channel will provide a greater safety factor. Then, when scour does occur, the embankment will more likely remain stable. For existing structures, protection of the berm, especially the toe, may be necessary.
2. Substructure

Generally, wider and longer piers have greater scour potential. Deeper footings and longer piles are more stable at greater scour depths. Spread footings should be used only on material highly resistant to scour such as limestone and some shales.

To maintain the integrity of the structure, do not allow scour to reduce pile bearing below a desirable safety factor that is selected by the structural or geotechnical engineer. Designing for this minimum safety factor may require designing longer piles for new bridges. For existing structures, protection of the piles may be necessary to maintain the safety factor.

New bridges should have sufficient length so that the abutments do not encroach on the channel but placed as far back from the streambank as practical. Vertical wall abutments (high abutments) have a greater potential for general and local scour as compared to the spill-through type (integral or stub abutments).

3. Flood discharge

In the publication “Evaluating Scour at Bridges, Fifth Edition”, Hydraulic Engineering Circular No. 18 (HEC-18), the FHWA recommends using scour flood frequencies that are larger than the hydraulic design flood frequencies. The rationale for this is that hydraulic design involves backwater and ensures that the bridge size will be adequate under normal flood conditions. In scour design, a higher discharge is used to ensure that the bridge will remain stable and will not fail or suffer severe damage during extreme flood events. Also, there is a reasonably high likelihood that the hydraulic design flood will be exceeded during the service life of the bridge.

Iowa DOT recommends using the Q_{200} or lesser discharge for scour analysis, depending on which results in the most severe scour conditions. Usually the overtopping flood results in the worst scour, so check this flood (if less than the Q_{200}) and the Q_{200}.

FHWA also recommends checking scour conditions for a superflood, such as a Q_{500}. If Q_{500} data is not available, HEC-18 recommends using 1.7 X Q_{100}. The safety factors for the bridge should remain above 1.0 under this flood condition. Similar to that mentioned above, Q_{overtopping} may be the worst-case flood and should be used if it is less than Q_{500}.

4. Interaction between road and flood plain

A highly skewed river crossing provides a less hydraulically efficient bridge opening and therefore has a greater contraction scour potential. Also, a high ratio of overbank flow to main channel flow will result in a greater contraction scour potential. For these situations, scour can be reduced by using wing dikes and/or riprap.

Road grade overflow or overflow structures may provide relief and reduce scour potential for the main channel bridge.

5. Interaction between piers and flood flow

The width, length and type of pier (e.g., pile bents, “tee” piers) all have an effect on local scour. Closely spaced piles in a pile bent pier can act similar to a solid wall. The angle of attack of flood flow to the pier can also significantly increase scour if this angle changes due to channel meandering during the life of the bridge. For example, if the angle of attack changes from 0° to 15°, the pier scour approximately doubles. The stream’s history of and future potential for meandering should be examined.
6. Debris and ice

Visual observation can be made and maintenance records can be checked to determine the history of debris and ice on the stream. Debris and ice can snag on the piers or superstructure, placing additional stresses on the bridge as well as promoting local scour. This scour can sometimes be quite significant although difficult to estimate. Therefore, for new designs, give consideration to raising the low superstructure above the low road grade elevation. This will allow hydraulic relief if the bridge opening becomes clogged.

Estimating scour

Procedures for estimating scour have been researched in the past 40 years in an attempt to develop reliable prediction equations. Some of these equations give reliable results, others do not. The Federal Highway Administration has attempted to find the best equations and published them in HEC-18.

HEC-18 contains equations for contraction scour, abutment scour and pier scour. The contraction scour equations are the best available equations of their type and sometimes provide reliable estimates, although these estimates still need to be evaluated considering soil types, site scour history, etc. The abutment scour equations frequently give questionable estimates. Because of comments similar to this from various states, FHWA is conducting additional research to develop new methods. At this time, IDOT recommends not using FHWA's abutment scour equations or, at most, use them with caution. However, be aware that abutment scour can occur.

Concerning pier scour, the equation in HEC-18 generally gives reliable results. However, a much simpler method that gives very similar results is found in Iowa Highway Research Board's Bulletin No. 4, “Scour Around Bridge Piers and Abutments,” by Emmett M. Laursen and Arthur Toch, May 1956. This method for estimating pier scour can be used in most cases instead of the methods in HEC-18.

1. Contraction scour estimation

See Chapter 4 of HEC-18 for detailed instructions on how to calculate contraction scour. To help explain this chapter, there are two determinations that must be made when estimating contraction scour:

- The appropriate case of contraction scour that depends on the flow interaction of the bridge to the channel and floodplain. There are four of these cases. See the figures later in this document for graphical illustrations of these cases.
- The appropriate sediment transport condition. There are two of these conditions and equations (live-bed and clear-water) that can occur in any of the four cases mentioned above.

Both determinations are explained below.

Four cases of contraction scour

Case 1 is overbank flow being forced back into the main channel due to the road fill. The majority of bridges in Iowa will be Case 1. There are three variations to Case 1, depending on the location of the abutments or abutment berms compared to the channel:

- **Case 1a** is normally used when the river channel width becomes narrower due to the bridge abutments (or berms) projecting into the channel.

- **Case 1b** does not involve any contraction of the channel itself, but the overbank flow area is completely obstructed by the embankment. In other words, the abutments or abutment berms are on the channel bank.

- **Case 1c** is when the abutments or abutment berms are set back from the channel. This case is more complex because there is both main channel flow and overbank flow in the bridge opening. Therefore, refer to discussion in Section 4.3.4 of HEC-18. More hydraulic analysis may be needed than in Cases 1a and 1b (such as WSPRO) to determine the distribution of flow in the bridge opening, i.e., what is the discharge in the main channel ($Q_2$) and the discharge in the overbank under the bridge ($Q_{overbank}$).
Most Case 1 streams in Iowa will have live-bed scour. However, if the streambed material has particles larger than a sand classification, calculate $V_c$ (see below) to determine if clear-water scour will occur instead of live-bed scour.

**Case 2** is when the stream has no overbank flow. This case will be common in Western Iowa streams that are severely degraded.

**Case 3** is an overflow (relief) bridge with no bed material transport, so use the clear-water scour equations. Hydraulic analysis (e.g., using WSPRO) is needed to determine the flood plain width associated with the relief opening and to determine the total flow going through the relief bridge.

**Case 4** is an overflow (relief) bridge similar to Case 3 except it does have sediment transport (live-bed scour), such as over a secondary channel on the flood plain of a larger stream. Hydraulically this case is no different than Case 1 except that analysis (e.g., using WSPRO) is needed to determine the flood plain width associated with the relief opening and the portion of the total flow going through the relief bridge.

**Sediment transport conditions: Live-bed scour versus clear-water scour**

Before an equation is selected to estimate contraction scour, it is necessary to determine if the flow is transporting bed material. If it is, the flow will create live-bed scour. If it is not, the flow will create clear-water scour. There are different scour equations for each of these sediment transport conditions.

Most Iowa stream channels will be live-bed. In other words, the velocities in the channel will be high enough to cause movement of the soil particles in the streambed. In order to be sure if the channel is live-bed, Chapter 2 in HEC-18 gives a simple equation to calculate the velocity needed to cause movement of the soil:

$$V_c = 10.95 y^{0.167} (D_{50})^{0.33}$$

where

- $V_c$ = critical velocity which will transport bed materials of size $D_{50}$ and smaller, ft/sec.
- $y$ = depth of upstream flow, feet
- $D_{50}$ = median diameter of the bed material, feet

If the velocity in the channel is greater than $V_c$, then the particles will move and the stream will have live-bed scour. If the velocity in the channel is less than $V_c$, then the particles will not move and the stream will have clear-water scour.

Most Iowa streambeds have sand or silt which results in a very low $V_c$. This means that even a low flood velocity will move the particles. Therefore, most Iowa streams will have live-bed scour. For example, for a medium sand with a $D_{50}$ of 0.0012 feet and a flow depth of 12 feet, $V_c$ is 1.8 ft/sec. Any flood with a channel velocity higher than this will cause sediment transport and therefore create live-bed scour. Even a medium gravel streambed with $D_{50}$ of 0.039 feet and depth of 12 feet results in $V_c$ of 5.7 ft/sec. Again, most Iowa streams will have a channel velocity higher than this.

In summary, as a rule of thumb, if the streambed material is larger than sand, calculate $V_c$ and compare to expected channel velocities to determine if live-bed or clear-water scour occurs. If the material is sand or smaller, assume live-bed scour occurs.

**Live-bed scour**

From HEC-18, the equation for live-bed scour is as follows:

$$\frac{y_2}{y_1} = \left[ \frac{Q_2}{Q_1} \right]^{0.86} \left[ \frac{W_1}{W_2} \right]^{-k_1}$$

and

$$y_s = y_2 - y_1 = \text{average scour depth, ft}$$

where $y_1$ = average depth in the upstream main channel, ft
\[ W_2 = W_1 + Q_1 = Q_2 = \frac{Q_2}{W_2} \]

\[ W = W_1 + Q_1 = W_2 + Q_2 = \frac{Q_2}{W_2} \]

\[ W = W_1 + Q_1 = W_2 + Q_2 = \frac{Q_2}{W_2} \]

\[ \frac{W_1}{W_2} = \frac{Q_2}{Q_1} \]

\[ y_2 = \text{average depth in the contracted section (i.e., in the bridge opening), ft} \]

\[ W_1 = \text{top width of water in the upstream main channel, ft} \]

\[ W_2 = \text{top width of water in the main channel in the contracted section (i.e., in the bridge opening), ft} \]

\[ Q_1 = \text{discharge in the upstream main channel transporting sediment, cfs.} \]

\[ Q_2 = \text{discharge in the contracted channel (i.e., bridge opening), cfs} \]

\[ (Q_2) = \text{discharge in the upstream overbank flow} \]

\[ k_1 = \text{exponent. Assume } k_1 = 0.64 \text{ to simplify the calculations since the range for } k_1 \text{ in HEC-18 Section 4.3.4 makes very little difference on calculated scour depths.} \]

This results in the live-bed scour equation of:

\[ \frac{y_2}{y_1} = \left[ \frac{Q_2}{Q_1} \right]^{0.86} \left[ \frac{W_1}{W_2} \right]^{0.64} \]

Simply stated, the ratio \( W_1/W_2 \) reflects contraction or expansion in the channel. The ratio \( Q_2/Q_1 \) reflects the effect of forcing overbank flow through the bridge opening.

This equation is generally used for Case 1 (when streambed consists of sand-size particles or smaller) and Cases 2 and 4. In Case 1c, the live-bed scour equation is used for the main channel contraction scour and the clear-water scour equation is used for the contraction scour near the abutment on the overbank.

**Clear-water scour**

From HEC-18, the equation for clear-water scour is as follows:

\[ y_2 = \left[ \frac{Q^2}{139 (D_{50})^{0.67} (W_2)^2} \right]^{0.43} \]

\[ y_1 = y_2 - y_1 = \text{average scour depth, feet} \]

where

\[ y_2 = \text{depth in the bridge opening, ft} \]

\[ Q = \text{discharge through the bridge opening or on the overbank portion of the bridge opening, cfs} \]

\[ D_{50} = \text{median diameter of material in overbank, feet (see attached sediment size table from HEC-20)} \]

\[ W_2 = \text{top width of water in bridge opening or overbank width in bridge opening (set-back distance), feet} \]

\[ y_1 = \text{upstream depth, ft} \]

The average depths \( y_1 \) and \( y_2 \) are measured either in the channel for channel scour calculations or on the overbank for overbank/abutment-area scour calculations.

The clear-water scour equation is used for a few Case 1 bridges (when streambed particles are larger and, in Case 1c, when the abutment is set back a distance from the channel) and for all Case 3 bridges.

**Summary of estimating contraction scour**

- Determine which “case” is appropriate
- Determine if the channel has live-bed or clear-water scour
- Analyze the hydraulics
- Using the correct equation, estimate scour
- Evaluate the reasonableness of estimated scour
2. Abutment scour estimation

The equation given in Section 4.3.6 of HEC-18 is for the worst-case conditions. The equation will predict the maximum scour that could occur for an abutment projecting into a stream with velocities and depths upstream of the abutment similar to those in the main channel. In most cases, the equation will over-predict scour, especially the farther the abutment is from the channel. Do not calculate abutment scour at this time due to this questionable equation. Be aware, however, that scour at the abutments can occur. Site experience is very important in the engineering analysis, including known scour occurrences and settlement of approach pavement which indicates soil stability problems. It is important to note that high abutments may have up to twice the scour depths as spill-through abutments.

A conservative approach in determining effects of scour on the abutments is to assume that contraction scour is added to abutment scour when the abutment is near the channel.

Several questions should be considered for abutment stability. Is the soil scourable? What is the effect on berm stability? Are flatter berm slopes or a longer bridge needed? What is the effect on pile bearing? Are longer piles needed? Should riprap or wing dikes be used?

3. Pier scour estimation

Use “Scour Around Bridge Piers and Abutments”, Emmett M. Laursen and Arthur Toch, Iowa Highway Research Board, Bulletin No. 4, 1956, for most cases.

Figure 39 in Bulletin No. 4 is the basic design curve for pier scour. IDOT determined an equation from this curve:

\[
y_s = (K) (y'_s) = (K) (w_p)
\]

Equation 1

where

\[
y'_s, \text{ unfactored depth of scour, ft}
\]

\[
y_1, \text{ unscoured depth of flow, ft}
\]

\[
w_p, \text{ width of pier column, ft}
\]

Equation 1 is then substituted into the basic equation, resulting in Equation 2 below:

\[
y_s = 1.485 (K) (w_p) \left( \frac{y'_s}{w_p} \right)^{0.314}
\]

Equation 2

where \(y_s\) is depth of scour, ft

\(K\), a pier coefficient (either \(K_a\) or \(K_s\)),

\(K_s\), coefficient for pier nose shape (see below). Use only if angle of attack = 0.

\(K_a\), coefficient for angle of attack if angle is not zero (see table below).
Equation 2 should be used to calculate pier scour.

\[
\text{If angle of attack is zero, use one of the following values for } K_s, \text{ the coefficient for the shape of the upstream nose of the pier (adapted from Bulletin No. 4). Use this } K_s \text{ value in Equation 2 in place of } K. \text{ These values show that the better the “rounding” of the pier nose, the lower the pier scour.}
\]

<table>
<thead>
<tr>
<th>Shape</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rectangular</td>
<td>1.0</td>
</tr>
<tr>
<td>Semicircular</td>
<td>0.9</td>
</tr>
<tr>
<td>Elliptic</td>
<td>0.8</td>
</tr>
</tbody>
</table>

\[
\text{If angle of attack is not zero, use the following table adapted from Figure 39 in Bulletin No. 4 to determine } K_a. \text{ In this table, } L = \text{ length of pier, and } w_p = \text{ width of pier. Use this } K_a \text{ value in Equation 2 in place of } K. \text{ The values in the table show that as the angle of attack increases, the pier scour increases dramatically. For example, for a pier } L / w_p\text{ of 8, if the angle of attack changes from } 0^\circ \text{ to } 15^\circ, \text{ the factor } K_a \text{ changes from 1.0 to 2.0, doubling the calculated pier scour.}
\]

\[
\begin{array}{c|cccccc}
\text{Angle of Attack} & \text{4} & \text{6} & \text{8} & \text{10} & \text{12} & \text{14} \\
\hline
0^\circ & 1.0 & 1.0 & 1.0 & 1.0 & 1.0 & 1.0 \\
5^\circ & 1.2 & 1.3 & 1.3 & 1.5 & 1.6 & 1.6 \\
10^\circ & 1.4 & 1.5 & 1.7 & 1.9 & 2.1 & 2.3 \\
15^\circ & 1.5 & 1.8 & 2.0 & 2.2 & 2.5 & 2.7 \\
20^\circ & 1.7 & 2.0 & 2.3 & 2.5 & 2.8 & 3.0 \\
25^\circ & 1.8 & 2.2 & 2.5 & 2.8 & 3.1 & 3.5 \\
30^\circ & 1.9 & 2.4 & 2.7 & 3.1 & 3.4 & 3.8 \\
35^\circ & 2.0 & 2.5 & 2.9 & 3.3 & 3.7 & 4.0 \\
40^\circ & 2.1 & 2.7 & 3.1 & 3.6 & 4.0 & 4.3 \\
45^\circ & 2.2 & 2.8 & 3.3 & 3.8 & 4.2 & 4.6 \\
\end{array}
\]

See Scour Calculation Sheet to assist in pier scour estimation. Other subjects concerning pier scour discussed in more detail are found in Section 4.3.5 of HEC-18:

- Pier scour for exposed footings and exposed pile groups under a footing
- Pier footings that are above normal streambed
• Multiple columns in a pier (e.g., a pile bent pier)
• Pressure flow scour
• Scour from debris
• Width of pier scour holes

Summary of estimating pier scour:
• Analyze hydraulics
• Estimate scour
• Evaluate the reasonableness of the estimated scour
• Add pier scour to contraction scour to obtain total scour
• Determine action steps such as countermeasures or design features of the bridge

Coding for the Structure Inventory and Appraisal (SI&A)

See the attached pages from FHWA’s “Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridges” to determine what rating should be given to each bridge. All countermeasures (SI&A Item 113 coded as "7") should be monitored in future years by bridge inspectors.

Countermeasures: reducing the effects of scour

Generally, a new bridge should be designed to withstand scour without countermeasures, especially when the countermeasures cannot be easily inspected. For example, riprap protecting a pier in the channel is difficult to inspect, but a wing dike in the overbank is easily inspected and repaired. Countermeasures will be used most commonly on existing bridges that are scour critical. See HEC-18, Chapter 7, for an in-depth discussion of when and how to use countermeasures.

In summary, listed below are common considerations to reduce scour on the bridges. Some items may be relevant only to existing bridges; others may be relevant only in the design phase of a structure.
• Use longer piles.
• Set the pier or abutment footings lower. However, lengthening piles is generally preferred due to lesser cost.
• Place riprap around the pier, abutment, berm slope, or spur dike or across the entire streambed. Riprap is an easy and often inexpensive way to protect a bridge.
• Build abutments as far from the streambank as possible.
• Remove debris from piers.
• Wing dikes (a.k.a., spur dikes, guide banks) provide for a more hydraulically efficient bridge opening and force the scour to occur on the dike, which is expendable, rather than on the bridge itself.

More expensive solutions can be considered in some instances:
• Place sheet piling to protect existing piers or abutments.
• Underpin the foundation.
• Replace with a new bridge.
• Construct an additional span.
• Overflow (relief) bridges can be used on flood plains that have substantial overbank flow. This provides relief for the main channel bridge. However, be aware that these overflow structures are particularly susceptible to deep scour. Twenty to thirty feet of scour is not uncommon.
• Provide for road grade overflow which is a “relief valve” to the bridge opening during extreme flood events and can prevent or minimize damage to the bridge. A disadvantage to road grade overflow is potential hazard to the traveling public when water is over the road. These factors need to be weighed by the engineer when considering other factors such as traffic volumes, traffic speeds and costs.
Following are some design guidelines for sizing riprap and placing wing dikes as countermeasures. The recommendations concerning riprap are not intended to determine if it is needed, rather only how to properly size riprap.

1. Riprap at abutments.

Section 7.5.1 in HEC-18 gives several equations for sizing riprap at abutments. Considering these equations and past experience, IDOT recommends simplifying riprap design to the following:

When riprap is needed for countermeasure and the toe of the abutment berm or the vertical abutment is approximately 75 feet or less from the top of the bank, use the average velocity through the entire bridge opening to size the riprap. When the toe of the abutment berm or the vertical abutment is approximately 75 feet or more from the top of the streambank, use the average velocity in the overbank portion of the bridge opening.

When riprap is needed and the determined average velocity is less than approximately 8 feet per second, use IDOT’s Class E riprap (D50 of 90 pounds). When the determined average velocity is greater than approximately 8 feet per second, use the Class B gradation which is heavier than Class E (D50 of 275 pounds).

2. Riprap at piers.

From Section 7.5.1 in HEC-18, the equation for sizing riprap at piers reduces to the following (assuming specific gravity of 2.65 for riprap):

\[ D_{50} = \frac{(K \cdot V)^2}{153.6} \]

where

- \( D_{50} \) = median stone diameter, feet
- \( K \) = coefficient for pier shape (1.5 for round-nose pier, 1.7 for square-nose pier)
- \( V \) = average velocity approaching pier, ft/sec

To determine \( V \), multiply the average channel velocity \((Q/A)\) by a coefficient that ranges from 0.9 for a pier near the bank in a straight uniform reach of the stream to 1.7 for a pier in the main current of flow around a bend.

The \( D_{50} \) for IDOT’s Class E riprap is 90 pounds or approximately 1.0-foot diameter and will be adequate for many situations. From the above equation, this diameter will tolerate a velocity of 8.3 ft/sec for round-nose piers and 7.3 ft/sec for square-nose piers.

When the adjusted velocity exceeds this and riprap is needed as a countermeasure, consider using Class B riprap. This has a \( D_{50} \) of 275 pounds which is approximately 1.5 feet in diameter and will tolerate a velocity of approximately 10 ft/sec for round-nose piers and 9 ft/sec for square-nose piers. This gradation should be adequate in almost all situations where the standard gradation is not adequate.

According to HEC-18, the width of the riprap around the pier should at least twice the pier column width. However, on several countermeasure projects, IDOT has placed a much wider layer (25’) around the entire pier. The riprap should be placed at or below the streambed so as not to create a greater obstruction to flow. HEC-18 recommends a thickness for the pier scour protection layer of 3 x \( D_{50} \) or greater. IDOT has used thicknesses of three and four feet on previous projects. Either guideline seems reasonable.

3. Wing dikes

Use Design Bureau’s Standard Road Plan EW-210. See C3.2.2.7.5.3 for a table to determine the length of wing dikes. See also HEC-20 or HDS No. 1 for further guidance.
References

# SCOUR CALCULATION SHEET

## LOCATION
County_________________ Hwy. No._________ Des. No.__________________
Maint. No.________________ FHWA No.________________
Stream_________________ Drain. Area______sq. mi.
Twp_____ Range_____ Section______
Prepared by____________ Date_________________

## BRIDGE DESCRIPTION
Size and Type______________________________________________________

### Pier
Type_________________ Width_______ft  Shape Coeff (Ks)_______
Angle of Attack _____ Coeff (Kal)_______
Pile Type___________ Pile Length below Str.Bed____  Pile Tip Elev._____

### Abutment
Type_______________ Pile Type_______Pile Length
Pile Tip Elev._______ Berm Slope_______(proposed or existing)

## STREAM INFORMATION
Exist. Streambed Elev.______ Stream Slope______ ft/mi
n-values:  LOB__________ Channel_____________ROB________________
Soils: Type __________________ Depth* ________ D50 __________ft
       Type _____________ Depth* ________
       Type _____________ Depth* ________
       Type _____________ Depth* ________ *below streambed

### Streambed Degradation
At this site __________________ feet since _______ year
At other known sites _____________ feet since _______ year
Estimated future degradation _______feet

## HYDROLOGIC/ HYDRAULIC INFORMATION
Low road elev.
Methodology used to determine:  Q _____________ Water surface elev. ___________

<table>
<thead>
<tr>
<th>Discharge (Q), cfs</th>
<th>Q200</th>
<th>Q500 or Qovertopping</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Water surface elev.</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>y1, depth in main channel, ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vel. in main channel, fps</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
CONTRACTION SCOUR

\( V_c = 10.95 \ y^{0.167} \ D_{50}^{0.33} = \text{__________} \ \text{ft/sec.} \) If \( V_c < \text{average channel velocity, use live-bed scour equation. If} V_c > \text{average channel velocity, use clear-water scour equation.} \)

**Live-bed scour**

Generally, used for Cases 1a, 1b, 2, and 4, and also for the main channel scour portion of Case 1c. See Section 4.3.4 in HEC-18.

\[
\frac{y_2}{y_1} = \left( \frac{Q_2}{Q_1} \right)^{0.86} \left( \frac{W_1}{W_2} \right)^{0.64}
\]

- \( Q_2 \), discharge in the contracted channel, cfs
- \( Q_1 \), discharge in the upstream main channel, cfs
- \( W_1 \), top width of the upstream main channel, ft
- \( W_2 \), top width of the main channel in contracted section (i.e., bridge opening), ft
- \( y_1 \), ave. depth in upstream main channel, ft
- \( y_2 \), ave. depth in contracted section, ft
- \( y_s = y_2 - y_1 = \text{ave. scour depth, ft} \)

**Clear-water scour**

For Case 3 and the overbank area of the bridge opening for Case 1c. Occasionally used for Cases 1a, 1b, 1c (main channel portion), and 2. See Section 4.3.4 in HEC-18.

\[
y_2 = \left( \frac{Q^2}{139 \ (D_{50})^{0.67} \ (W_2)^2} \right)^{0.43}
\]

- \( y_2 \), depth in bridge opening, ft
- \( Q \), discharge through bridge opening or on overbank portion of bridge opening, cfs
- \( D_{50} \), median diameter of material in overbank, ft
- \( W_2 \), top width of bridge opening or overbank width in bridge opening, ft
- \( y_1 \), upstream depth, ft
- \( y_s = y_2 - y_1 = \text{ave. scour depth, ft} \)

Is this contraction scour depth realistic?
Is the soil scourable?
What is the effect on berm stability (including any abutment scour)?
Are longer abutment piles or a flatter abutment berm needed?
Should riprap or wing dikes be used?
Other comments?
PIER SCOUR

Use “Scour Around Bridge Piers and Abutments”, Emmett M. Laursen and Arthur Toch, Iowa Highway Research Board Bulletin No. 4, 1956, for most cases. Use Equation 2 below and previous discussion in the text. Also, see Section 4.3.5 in HEC-18 for more discussion on estimating pier scour.

\[ y_s = 1.485 \left( K \right) \left( \frac{y_1}{w_p} \right)^{0.314} \]

where:
- \( y_s \), depth of scour, ft
- \( y_1 \), unscoured depth of flow, ft
- \( w_p \), width of pier column, ft
- \( K \), a pier coefficient (either \( K_s \) or \( K_a \)),
  - \( K_s \), coefficient for pier nose shape (see values in text). Use only if angle of attack = 0.
  - \( K_a \), coefficient for angle of attack if angle is not zero (see table in text).

**TOTAL SCOUR AT PIER** = pier scour \( (y_s) \) + contraction scour \( (y_s) \)

<table>
<thead>
<tr>
<th>( y_1 ), ft</th>
<th>( Q_{200} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( w_p ), ft</td>
<td>( Q_{500} ) or ( Q_{overtopping} )</td>
</tr>
<tr>
<td>( K ) (either ( K_s ) or ( K_a ))</td>
<td>( y_s ), ft (from Equation 2)</td>
</tr>
</tbody>
</table>

Is \( y_s \) or the total scour depth at the pier realistic?
Is the soil scourable?
What is the effect on pile stability?
Should riprap or other countermeasures be used?
What is the rating for SI&A Item 113?
Other comments?

<table>
<thead>
<tr>
<th>Size</th>
<th>Approximate Sieve Mesh Openings (per inch)</th>
<th>Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Millimeters</td>
<td>Microns</td>
<td>Inches</td>
</tr>
<tr>
<td>4000-2000</td>
<td>---</td>
<td>180-160</td>
</tr>
<tr>
<td>2000-1000</td>
<td>---</td>
<td>80-40</td>
</tr>
<tr>
<td>1000-500</td>
<td>---</td>
<td>40-20</td>
</tr>
<tr>
<td>500-250</td>
<td>---</td>
<td>20-10</td>
</tr>
<tr>
<td>250-130</td>
<td>---</td>
<td>10-5</td>
</tr>
<tr>
<td>130-64</td>
<td>---</td>
<td>5-2.5</td>
</tr>
<tr>
<td>64-32</td>
<td>---</td>
<td>2.5-1.3</td>
</tr>
<tr>
<td>32-16</td>
<td>---</td>
<td>1.3-0.6</td>
</tr>
<tr>
<td>16-8</td>
<td>---</td>
<td>0.6-0.3</td>
</tr>
<tr>
<td>8-4</td>
<td>---</td>
<td>0.3-0.16</td>
</tr>
<tr>
<td>4-2</td>
<td>---</td>
<td>0.16-0.08</td>
</tr>
<tr>
<td>2.00-1.00</td>
<td>2000-1000</td>
<td>---</td>
</tr>
<tr>
<td>1.00-0.50</td>
<td>1000-500</td>
<td>---</td>
</tr>
<tr>
<td>0.50-0.25</td>
<td>500-250</td>
<td>---</td>
</tr>
<tr>
<td>0.25-0.125</td>
<td>250-125</td>
<td>---</td>
</tr>
<tr>
<td>0.125-0.062</td>
<td>125-62</td>
<td>---</td>
</tr>
<tr>
<td>0.062-0.031</td>
<td>62-31</td>
<td>---</td>
</tr>
<tr>
<td>0.031-0.016</td>
<td>31-16</td>
<td>---</td>
</tr>
<tr>
<td>0.016-0.008</td>
<td>16-8</td>
<td>---</td>
</tr>
<tr>
<td>0.008-0.004</td>
<td>8-4</td>
<td>---</td>
</tr>
<tr>
<td>0.004-0.0020</td>
<td>4-2</td>
<td>---</td>
</tr>
<tr>
<td>0.0020-0.0010</td>
<td>2-1</td>
<td>---</td>
</tr>
<tr>
<td>0.0010-0.0005</td>
<td>1-0.5</td>
<td>---</td>
</tr>
<tr>
<td>0.0005-0.0002</td>
<td>0.5-0.24</td>
<td>---</td>
</tr>
</tbody>
</table>

Case 1A: Abutments project into channel

Case 1B: Abutments at edge of channel

Case 1C: Abutments set back from channel

Case 2A: River narrows

Case 2B: Bridge abutments constrict flow

Case 3: Relief bridge over flood plain

Case 4: Relief bridge over secondary stream

ITEM 113--SCOUR CRITICAL BRIDGES

Use a single-digit code as indicated below to identify the current status of the bridge regarding its vulnerability to scour. Scour analyses shall be made by hydraulic/geotechnical/structural engineers. Details on conducting a scour analysis are included in the FHWA Technical Advisory 5140.23 titled, “Evaluating Scour at Bridges”. Whenever a rating factor of 4 or below is determined for this item, the rating factor for “Item 60 – Substructure” may need to be revised to reflect the severity of actual scour and resultant damage to the bridge. A scour critical bridge is one with abutment or pier foundations which are rated as unstable due to (1) observed scour at the bridge site or (2) a scour potential as determined from a scour evaluation study.

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>Bridge not over waterway.</td>
</tr>
<tr>
<td>U</td>
<td>Bridge with “unknown” foundation that has not been evaluated for scour. Since risk cannot be determined, flag for monitoring during flood events and, if appropriate, closure.</td>
</tr>
<tr>
<td>T</td>
<td>Bridge over “tidal” waters….</td>
</tr>
<tr>
<td>9</td>
<td>Bridge foundations (including piles) on dry land well above floodwater elevations.</td>
</tr>
<tr>
<td>8</td>
<td>Bridge foundations determined to be stable for assessed or calculated scour conditions; calculated scour is above top of footing. (Example A)</td>
</tr>
<tr>
<td>7</td>
<td>Countermeasures have been installed to correct a previously existing problem with scour. Bridge is no longer scour critical</td>
</tr>
<tr>
<td>6</td>
<td>Scour calculation/evaluation has not been made. (Use only to describe cases where bridge has not yet been evaluated for scour potential.)</td>
</tr>
<tr>
<td>5</td>
<td>Bridge foundations determined to be stable for calculated scour conditions; scour within limits of footing or piles. (Example B)</td>
</tr>
<tr>
<td>4</td>
<td>Bridge foundations determined to be stable for calculated scour conditions; field review indicates action is required to protect exposed foundations from effects of additional erosion and corrosion.</td>
</tr>
</tbody>
</table>
| 3    | Bridge is scour critical; bridge foundations determined to be unstable for calculated scour conditions:
  --Scour within limits of footing or piles. (Example B)
  --Scour below spread-footing base or pile tips. (Example C) |
| 2    | Bridge is scour critical; field review indicates that extensive scour has occurred at bridge foundations. Immediate action is required to provide scour countermeasures. |
| 1    | Bridge is scour critical; field review indicates that failure of piers/abutments is imminent. Bridge is closed to traffic. |
| 0    | Bridge is scour critical. Bridge has failed and is closed to traffic. |
ITEM 113--SCOUR CRITICAL BRIDGES (CONT’D)

<table>
<thead>
<tr>
<th>Example</th>
<th>Calculated Scour Depth</th>
<th>Action Needed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Spread Footing</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(not founded in rock)</td>
<td></td>
</tr>
<tr>
<td>A. Above top of footing</td>
<td>None--indicate rating of 8 for this item</td>
<td></td>
</tr>
<tr>
<td>B. Within limits of footing or piles</td>
<td>Conduct foundation structural analysis</td>
<td></td>
</tr>
<tr>
<td>C. Below pile tips or spread footing base</td>
<td>Provide for monitoring and scour countermeasures as necessary.</td>
<td></td>
</tr>
</tbody>
</table>

Calculated Scour Depth = ---------------
BULLETIN NO. 4
IOWA HIGHWAY RESEARCH BOARD

Scour Around Bridge Piers And Abutments

by
Emmett M. Laursen and Arthur Toch
Iowa Institute of Hydraulic Research
State University of Iowa

Prepared by the
Iowa Institute of Hydraulic Research
in cooperation with
THE IOWA STATE HIGHWAY COMMISSION
and
THE BUREAU OF PUBLIC ROADS
May 1956
Fig. 38. Basic design curve for depth of scour.
Fig. 39. Design factors for piers not aligned with flow.

**TABLE V**

Shape coefficients $K_e$ for nose forms
(To be used only for piers aligned with flow)

<table>
<thead>
<tr>
<th>Nose form</th>
<th>Length-width ratio</th>
<th>$K_e$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rectangular</td>
<td></td>
<td>1.00</td>
</tr>
<tr>
<td>Semicircular</td>
<td></td>
<td>0.90</td>
</tr>
<tr>
<td>Elliptic</td>
<td>2:1</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>3:1</td>
<td>0.75</td>
</tr>
<tr>
<td>Lenticular</td>
<td>2:1</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>3:1</td>
<td>0.70</td>
</tr>
</tbody>
</table>
C3.2.2.7.1 Types
C3.2.2.7.2 Design conditions
C3.2.2.7.3 Evaluating existing structures
C3.2.2.7.4 Depth estimates
C3.2.2.7.5 Countermeasures
C3.2.2.7.5.1 Riprap at abutments
C3.2.2.7.5.2 Riprap at piers
C3.2.2.7.5.3 Wing dikes

Determining Wing Dike Lengths

The use of wing dikes (also called spur dikes or guide banks) shall be considered at any bridge site that has appreciable overbank discharge. Wing dikes help minimize backwater and scour effects. Refer to IDOT’s Design Bureau Standard EW-210 for specific details on slopes, dimensions and other notes. Items that need to be specified for EW-210 include Length and Station Location.

Generally, the top of dike elevation will be the same as the abutment berm elevation. However, if this berm elevation is much higher than the Q_{50} or Q_{100} elevations, a lower wing dike elevation may be specified.

The following guidelines provide assistance in determining appropriate wing dike lengths. “Long” and “Short” refer to the longer and shorter wing dikes necessary on skewed bridges as shown on EW-210. If obtaining right of way for the recommended length is a problem at a bridge site, a shortened wing dike is preferred over no dike.
### Wing Dike Lengths, in feet (meters)

<table>
<thead>
<tr>
<th>Bridge Length, feet (meters)</th>
<th>Bridge Skew</th>
<th>0 deg.</th>
<th>15 deg.</th>
<th>30 deg.</th>
<th>45 deg.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Equal</td>
<td>Long</td>
<td>Short</td>
<td>Long</td>
<td>Short</td>
</tr>
<tr>
<td>&lt; 150 (45)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>150-180 (45-55)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>180-210 (55-65)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>210-240 (65-75)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt; 240 (75)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### C3.2.2.7.6 Coding

#### C3.3 Highway crossings

#### C3.3.1 Clearances

#### C3.3.2 Ditch drainage

#### C3.4 Railroad crossings

#### C3.4.1 BNSF and UP overhead structures

#### C3.4.1.1 Vertical clearance

#### C3.4.1.2 Horizontal clearance

#### C3.4.1.3 Piers

#### C3.4.1.4 Bridge berms

#### C3.4.1.5 Drainage

#### C3.4.1.6 Barrier rails and fencing
C3.4.2 Non-BNSF and -UP overhead structures

C3.4.2.1 Vertical clearance

C3.4.2.2 Horizontal clearance

C3.4.2.3 Piers

C3.4.2.4 Bridge berms

C3.4.2.5 Drainage

C3.4.2.6 Barrier rails and fencing

C3.4.3 Underpass structures

C3.4.4 Submittals

1 December 2008

In discussions with the BNSF and UP railroads, the Bureau has agreed to provide the new standard sheet 1067 and the information listed below. This information will be provided by Preliminary Design Section on the Plan View and Elevation View on the TS & L sheet of all bridge projects that involve BNSF and UP railroad except the items noted with an asterisk (*). These items will be provided by the Final Design Sections. Final Design Sections should review the list to make sure all information is provided.

Plan View
1. Centerline of bridge and/or centerline of project.
2. Track layout and limits of railroad right-of-way with respect to centerline of main lines.
3. Future tracks, access roadways and existing tracks as main line, siding, spur, etc.
4. Horizontal clearance at right angle from centerline of nearest existing or future track to the face of obstruction such as substructure above grade.
* 5. Horizontal clearance at right angle from centerline of nearest existing or future track to the face of nearest foundation below grade.
6. Horizontal spacing at right angle between centerlines of existing and/or future tracks.
* 7. Limits of shoring and minimum distance at right angle from centerline of nearest track.
8. All existing facilities and utilities.
9. Existing ground shots and proposed grading.
10. Railroad Milepost and direction of increasing Milepost (Provided by Railroad).
11. Direction of flow for all drainage systems within project limits.
* 12. Limits of barrier rail and fence with respect to centerline of track.
* 13. Location of deck drains (Note drains shall not be located over the railroad right-of-way).
* 14. Total width of superstructure.
15. Width of shoulder and/or sidewalk.
16. North arrow
17. Footprint of proposed superstructure and substructure including existing structure if Applicable

Elevation View
1. Future tracks, access roadways and existing tracks as main line, siding, spur, etc.
2. Point of minimum vertical clearance and distance within the vertical clearance envelope, measured perpendicular from the centerline of nearest track.
* 3. Limits of shoring and minimum distance at right angle from centerline of nearest track.
4. Toe of slope and/or limits of retaining wall.
* 5. Limits of barrier rail and fence with respect to centerline of track.
6. Depth of foundation from top of tie / base of rail.
* 7. Top and bottom of pier protection wall elevation relative to top of rail elevation.
8. Controlling dimensions of drainage ditches and/or drainage structures.
9. Top of rail elevations for all tracks.
10. Minimum permanent vertical clearance above the top of high rail to the lowest point under the bridge.
11. Existing and proposed groundline and roadway profile.
12. Show slope and specify type of slope paving. Toe of slope shall be shown relative to drainage ditch and top of subgrade.

Note: Items denoted with an asterisk shall be provided by Final Design.

The new 1067 CADD standard shows details of:
1. Railroad General Notes
2. General Shoring Notes
3. General Excavation Zones detail
4. Minimum Construction Clearance Envelope detail
5. Top of Rail Elevations chart.

For additional information, see BNSF Railway – Union Pacific Railroad, Guidelines for Railroad Grade Separation Projects.

C3.5 Pedestrian and shared use path crossings
C3.6 Superstructures
C3.6.1 Type and span
C3.6.1.1 CCS J-series
C3.6.1.2 Single-span PPCB HSI-series
C3.6.1.3 Two-span BT-series
C3.6.1.4 Three-span PPCB H-series
C3.6.1.5 Three-span RSB-series
C3.6.1.6 PPCB

Preliminary haunch for all Prestressed Beam Bridges

Note: The calculations provide a haunch thickness estimate (X) value, which does not include the nominal haunch thickness.

\[ S := 111.5 \text{ ft} \quad \text{Longest Span (feet)} \]

\[ e := 0.03 \quad \text{Superelevation (feet/feet)} \]

\[ G_1 := -1.68 \quad \text{Grade 1 vertical curve [+ increasing, - decreasing] (\%)} \]

\[ G_2 := 2.10 \quad \text{Grade 2 vertical curve [+ increasing, - decreasing] (\%)} \]

\[ A := \frac{G_2 - G_1}{100} \quad A = 0.038 \]

\[ L := 984 \text{ ft} \quad \text{Length vertical curve (feet)} \]

\[ D_c := 1.75 \text{deg} \quad \text{Degree of Horizontal Curvature (degree)} \]

\[ C := 0.337 \text{ ft} \quad \text{Final Beam Camber (feet) - From prestressed concrete beam standards} \]

\[ D := 0.19 \text{ ft} \quad \text{Dead load deflection - Elastic + 1/2 Plastic (feet) - From prestressed concrete beam standards} \]

\[ T := 1.667 \text{ ft} \quad \text{Top flange width (feet)} \]

\( X = \text{Haunch estimate along the centerline of the beam.} \)

\[ X := (C - D) + \frac{S \cdot e}{2} \left( \frac{1}{\sin \left( \frac{D_c}{2} \right)} - \frac{1}{\tan \left( \frac{D_c}{2} \right)} \right) + \left( \frac{S}{L} \right)^2 \cdot A \cdot \frac{L}{8} \]

\[ X = 0.219 \text{ ft} \quad X = 66.894 \text{ mm} \]

\[ T \cdot e = 0.6 \text{ in} \]

If \( T \cdot e < 1 \) then \( X < 4 \text{ in} \). If \( T \cdot e > 1 \) then \( X < 3 \text{ in} \).

Also check maximum offset for horizontal curve < or = 9 in.
C3.6.1.7 CWPG

The table below extracted from the AASHTO LRFD Specifications [AASHTO-LRFD 2.5.2.6.3] can be used as a guide to establish minimum girder depths, when 1/25 of the span is not possible due to vertical clearance or profile grade issues.

**Traditional Minimum Depths for Constant Depth Superstructures**

<table>
<thead>
<tr>
<th>Material</th>
<th>Type</th>
<th>Simple Spans</th>
<th>Continuous Spans</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>Overall Depth of Composite I-Beam</td>
<td>0.040L</td>
<td>0.032L</td>
</tr>
<tr>
<td></td>
<td>Depth of I-Beam Portion of Composite I-Beam</td>
<td>0.033L</td>
<td>0.027L</td>
</tr>
<tr>
<td></td>
<td>Trusses</td>
<td>0.100L</td>
<td>0.100L</td>
</tr>
</tbody>
</table>


C3.6.1.8 Cable/Arch/Truss

C3.6.2 Width

C3.6.2.1 Highway

C3.6.2.2 Sidewalk, separated path, and bicycle lane

When placing sidewalks on bridges, the following policy should be used for determining whether to use raised sidewalks or sidewalks at grade.

1. Raised sidewalks, which allow water to drain through slots in the separation barrier curb to the bridge gutterline, shall be used on highway and railroad overpasses.
2. All other situations may use an at grade sidewalk which allows the water to drain over the slab edge.

At grade sidewalks, which drain the water back towards the gutter line, shall not be used. The reason the Bureau would like to avoid this condition is that it would require the exterior girder to be placed higher than the adjacent interior girder. In addition, in situations of excessive rainfall the sidewalks may be temporarily flooded because of water from the roadway. Superelevated bridges may require special considerations. Check with your section leader in this case.

Regardless of the sidewalk type, the top of the slab where the chain link fence is attached shall be made level and drip grooves shall be used on the underside of the slab.

C3.6.3 Horizontal curve

C3.6.3.1 Spiral curve

C3.6.4 Alignment and profile grade

For situations where the profile grade line is not at the centerline of approach roadway, elevations for the bridge deck will be established taking the bridge deck crown into account. The elevations will be noted on the TS&L as
“TOP OF BRIDGE DECK AT CENTERLINE ROADWAY IS ‘X’ ABOVE (OR BELOW) THE PROFILE GRADE TO ACCOUNT FOR DECK CROSS SLOPE AND PARABOLIC CROWN.

For situations where the profile grade line is at the centerline of approach roadway, elevations for the bridge deck will be established in accordance with BDM 1.7.1.

C3.6.5 Cross slope drainage
C3.6.6 Deck drainage
C3.6.7 Bridge inspection/maintenance accessibility
C3.6.8 Barrier rails
Flow Chart for determining Bridge Barrier Rail Height for New Bridges on Interstate and Primary Highways
Revised 5 December 2016

Interstate Bridge
No
Bridge over BNSF or UP RR
No
Coordinate with Systems Planning

Heavy Truck Volume > 7,500 Annual Average Daily Truck Traffic for Design Year
Yes

Fracture Critical Elements within the zone of intrusion for truck roll
No

Fly over Bridge
No

Unfavorable site conditions - see guidelines below
Yes

Coordinate With Design

Coordinate With Design

Frequent Transitions between Mainline roadway 44” Rail and Bridge Rail
No

Coordinate with Assistant District Engineer

Based on past maintenance experience and current snow removal policies Is snow pile up a concern?
Yes

Coordinate with Assistant District Engineer

Have special concerns been raised about headlight glare or ramping due to snow pile up?
Yes

Coordinate with Assistant District Engineer

Is plowed snow spilling over roadways, Railroad track or waterways below, a concern?
Yes

Coordinate with Assistant District Engineer

Design for TL-4 Barrier Rail (34")
No

Design for TL-5 Barrier Rail (44")
Guidelines for unfavorable site conditions (see flow chart above):
- Reduced radius of curvature
- Steep downgrades on curvature
- Variable cross slopes
- Adverse weather conditions

C3.6.9  Staging

C3.7  Substructures

C3.7.1  Skew

C3.7.2  Abutments

C3.7.3  Berms

C3.7.3.1  Slope

C3.7.3.2  Toe offset

C3.7.3.3  Berm slope location table
  See also the RBLT example C3.2.7.3.4.
SLOPE PROTECTION LOCATION FOR BSLT GRADING SURFACES

NOTES:
1. BSLT POINTS GIVEN AT THE GRADING SURFACE = TOP OF SLOPE PROTECTION.
2. THE GRADING SURFACE IS DEFINED BY THE BRIDGE OFFICE SLOPE PROTECTION STANDARD.
3. WING ARMORING DETAILS ARE DEFINED BY THE BRIDGE OFFICE WING ARMORING STANDARDS.
4. SLOPE PROTECTION AND WING ARMORING QUANTITIES WILL BE CALCULATED IN FINAL DESIGN.

CONCRETE OR MACADAM SLOPE PROTECTION

NOTES:
1. BSLT POINTS GIVEN AT GRADING SURFACE = TOP OF EROSION STONE AND TOP OF EMBEDDED REVETMENT.
2. THE GRADING SURFACE SHALL BE LAYERED ON THE TRL REVETMENT TYPICAL SECTION, TOP OF REVETMENT ELEVATION SHALL BE DEFINED.
3. ADDITIONAL EROSION STONE DETAILS ARE COVERED BY THE BRIDGE OFFICE SLOPE PROTECTION STANDARD.
5. WING ARMORING DETAILS ARE DEFINED BY THE BRIDGE OFFICE WING ARMORING STANDARD, FINAL DESIGN WILL CALCULATE QUANTITIES RELATED TO THE WING ARMORING.

EMBEDDED REVETMENT

NOTES:
1. BSLT POINTS GIVEN AT GRADING SURFACE = BASE OF EROSION STONE AND BASE OF NON-EMBEDDED REVETMENT.
2. THE GRADING SURFACE SHALL BE LAYERED ON THE TRL REVETMENT TYPICAL SECTION, TOP OF REVETMENT ELEVATION SHALL BE DEFINED.
3. ADDITIONAL EROSION STONE DETAILS ARE COVERED BY THE BRIDGE OFFICE SLOPE PROTECTION STANDARD.
4. THE BERM ARMORING QUANTITIES TABLE SHALL INCLUDE ENGINEERING FABRIC, EROSION STONE, AND REVETMENT, BERM ARMORING QUANTITIES GENERALLY WILL INCLUDE ARMORING WORK UP TO THE FACE OF ABUTMENT.
5. WING ARMORING DETAILS ARE DEFINED BY THE BRIDGE OFFICE WING ARMORING STANDARD, FINAL DESIGN WILL CALCULATE QUANTITIES RELATED TO THE WING ARMORING.

REVETMENT (NOT EMBEDDED)
FOR DUAL BRIDGES A BERM GRADING CONTROL LINE WILL BE PROVIDED.

THE BERM GRADING CONTROL LINE IS A CONTINUOUS LINE FROM 3 FT. BEYOND THE OUTSIDE BRIDGE FASCIA’S SET AT A CONSTANT ELEVATION. THE GRADING CONTROL LINE WILL RESULT IN A PLANAR BERM SURFACE BETWEEN AND UNDER THE BRIDGES.

FOR DUAL BRIDGES WHERE BOTH BERMS HAVE THE SAME ELEVATION AND THE EDGE OF THE 3 FT. BRIDGE BERM FORMS A CONTINUOUS LINE OUT-OF-THE 90° POINTS DEFINE THE BERM GRADING CONTROL LINE. FOR MOST DUAL BRIDGE SITES THIS CAN BE ACCOMPLISHED BY ADJUSTMENT OF THE LOW BRIDGE BERM AND/OR ELIMINATION OF A SLOPING BERM.

TO ATTAIN LEVEL/EQUAL BERM ELEVATIONS THE BERM CAN BE ELEVATED UP TO THE FOLLOWING LIMITS (ELEVATED FROM THE 2 FT. TYPICAL FROM BOTH FACES)

INTEGRAL – 0.5 FT.
STUB – 0.25 FT.

THE PROVISIONS OF ARTICLE 3.3.7.2 (SLOPING OF ABUT, FOOTING/BERM) SHOULD BE REVIEWED FOR APPLICABILITY. THE PROVISIONS OF THE ABOVE ARTICLE SHALL GOVERN.

FOR SITES WHERE THE 90° POINTS CANNOT BE ADJUSTED TO FORM A CONTINUOUS LINE AT A CONSTANT ELEVATION, 90° POINTS WILL BE UTILIZED TO DEFINE THE BERM GRADING CONTROL LINE.

THE CONTROL LINE WILL BE SET AT AN ELEVATION 1 FT. BELOW THE LOW BERM ELEVATION. THE ALIGNMENT WILL BE SET SUCH THAT THE SLOPE BETWEEN ADJACENT 90° AND 90° POINTS MATCHES OR IS FLATTER THAN THE BERM SLOPE BELOW THE GRADING CONTROL LINE.

BERM SLOPE LOCATION TABLE

<table>
<thead>
<tr>
<th>POINTS</th>
<th>LEFT BERM</th>
<th>ELEV.</th>
<th>STATION</th>
<th>OFFSET ELEV.</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>881+04.80</td>
<td>23.40 LT</td>
<td>1200.80</td>
<td>899+54.40</td>
</tr>
<tr>
<td>A2</td>
<td>899+95.40</td>
<td>12.50 LT</td>
<td>1200.60</td>
<td>895+58.50</td>
</tr>
<tr>
<td>D1</td>
<td>889+51.40</td>
<td>23.40 LT</td>
<td>1245.28</td>
<td>896+70.50</td>
</tr>
<tr>
<td>D2</td>
<td>889+50.00</td>
<td>23.40 LT</td>
<td>1249.28</td>
<td>896+70.00</td>
</tr>
<tr>
<td>B1</td>
<td>889+52.25</td>
<td>23.40 LT</td>
<td>1250.28</td>
<td>896+68.00</td>
</tr>
<tr>
<td>B2</td>
<td>889+49.67</td>
<td>23.40 LT</td>
<td>1250.28</td>
<td>896+49.58</td>
</tr>
<tr>
<td>W1</td>
<td>889+32.20</td>
<td>23.40 LT</td>
<td>1257.74</td>
<td>896+36.05</td>
</tr>
<tr>
<td>W2</td>
<td>889+37.25</td>
<td>12.50 LT</td>
<td>1257.74</td>
<td>897+37.00</td>
</tr>
</tbody>
</table>

BERM SLOPE ELEVATIONS REFLECT THE GRADING SURFACE, BERM GRADING BELOW BERM GRADING CONTROL LINE, DEFINED BY CONTROL LINE.

BERM GRADING CONTROL LINE IS DEFINED BY "9" POINTS IN ABOVE TABLE.

DUAL BRIDGES

BERM SLOPE DEFINITION

REV. DATE: 5-01-13

January 2020
C3.7.3.4 Recoverable berm location table

See also the BSLT example in C3.2.7.3.3.
C3.7.3.5 Slope protection
C3.7.3.6 Grading control points
C3.7.3.7 Mechanically Stabilized Earth (MSE) Walls adjacent to abutments

C3.7.4 Piers and pier footings
Ref: 2013 AASHTO LRFD Intermediate Revisions
3.6.5—Vehicular Collision Force: \textit{CT}

3.6.5.1—Protection of Structures

Unless the Owner determines that site conditions indicate otherwise, abutments and piers located within a distance of 30.0 ft to the edge of roadway shall be investigated for collision. Collision shall be addressed by either providing structural resistance or by redirecting or absorbing the collision load. The provisions of Article 2.3.2.2.1 shall apply as appropriate.

Where the design choice is to provide structural resistance, the pier or abutment shall be designed for an equivalent static force of 600 kip, which is assumed to act in a horizontal plane, at a distance of 5.0 ft above ground.

Where the design choice is to redirect or absorb the collision load, protection shall consist of one of the following:

- An embankment;
- A structurally independent, crashworthy ground-mounted 54.0-in. high barrier, located within 10.0 ft from the component being protected; or
- A 42.0-in. high barrier located at more than 10.0 ft from the component being protected.

Such barrier shall be structurally and geometrically capable of surviving the crash test for Test Level 5, as specified in Section 13.

In cases where substructures are found to be inadequate to resist the increased longitudinal forces, consideration should be given to design and detailing strategies which distribute the braking force to additional substructure units during a braking event.

C3.6.5.1

Where an Owner chooses to make an assessment of site conditions for the purpose of implementing this provision, input from highway or safety engineers and structural engineers should be part of that assessment. The equivalent static force of 600 kip is based on the information from full-scale crash tests of rigid columns impacted by 80.0-kip tractor trailers at 50 mph. For individual column shafts, the 600-kip load should be considered a point load. Field observations indicate shear failures are the primary mode of failure for individual columns and columns that are 30.0 in. in diameter and smaller are the most vulnerable. For wall piers, the load may be considered to be a point load or may be distributed over and area deemed suitable for the size of the structure and the anticipated impacting vehicle, but not greater than 5.0 ft wide by 2.0 ft high. These dimensions were determined by considering the size of a truck frame.

Requirements for train collision load found in previous editions have been removed. Designers are encouraged to consult the AREMA Manual for Railway Engineering or local railroad company guidelines for train collision requirements.

For the purpose of this Article, a barrier may be considered structurally independent if it does not transmit loads to the bridge.

Full-scale crash tests have shown that some vehicles have a greater tendency to lean over or partially cross over a 42.0-in. high barrier than a 54.0-in. high barrier. This behavior would allow a significant collision of the vehicle with the component being protected if the component is located within a few ft of the barrier. If the component is more than about 10.0 ft behind the barrier, the difference between the two barrier heights is no longer important.

One way to determine whether site conditions qualify for exemption from protection is to evaluate the annual frequency of impact from heavy vehicles. With the approval of the Owner, the annual frequency for a bridge pier to be hit by a heavy vehicle, \( F_{\text{Imp}} \), can be calculated by:

\[
AF_{\text{Imp}} = 2(\text{ADTT}) (P_{\text{Imp}}) 365
\]

where:

- \( \text{ADTT} \) = the number of trucks per day in one direction
- \( P_{\text{Imp}} \) = the annual probability for a bridge pier to be hit by a heavy vehicle

*Revised AASHTO LRFD 2013

Intermediate Revisions

January 2020
Table C3.6.1.4.2-1 may be used to determine ADTT from available ADT data.

\[ P_{ADT} = 3.457 \times 10^{-4} \text{ for undivided roadways in tangent and horizontally curved sections} \]
\[ 1.090 \times 10^{-5} \text{ for divided roadways in tangent sections} \]
\[ 2.184 \times 10^{-5} \text{ for divided roadways in horizontally curved sections} \]

Design for vehicular collision force is not required if \( AFE_{BRP} \) is less than 0.0001 for critical or essential bridges or 0.001 for typical bridges.

The determination of the annual frequency for a bridge pier to be hit by a heavy vehicle, \( AFE_{BRP} \), is derived from limited statistical studies performed by the Texas Transportation Institute. Due to limited data, no distinction has been made between tangent sections and horizontally curved sections for undivided roadways. The target values for \( AFE_{BRP} \) mirror those for vessel collision force found in Article 3.14.5.

Table C3.6.5.1-1 provides typical resulting values for \( AFE_{BRP} \).

<table>
<thead>
<tr>
<th>ADT (Both Directions)</th>
<th>ADTT* (One Way)</th>
<th>Undivided</th>
<th>Divided Curved</th>
<th>Divided Tangent</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000</td>
<td>50</td>
<td>0.0001</td>
<td>0.0001</td>
<td>0.0000</td>
</tr>
<tr>
<td>2000</td>
<td>100</td>
<td>0.0003</td>
<td>0.0002</td>
<td>0.0001</td>
</tr>
<tr>
<td>3000</td>
<td>150</td>
<td>0.0004</td>
<td>0.0002</td>
<td>0.0001</td>
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<tr>
<td>4000</td>
<td>200</td>
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<td>0.0002</td>
</tr>
<tr>
<td>6000</td>
<td>300</td>
<td>0.0008</td>
<td>0.0005</td>
<td>0.0002</td>
</tr>
<tr>
<td>8000</td>
<td>400</td>
<td>0.0010</td>
<td>0.0006</td>
<td>0.0003</td>
</tr>
<tr>
<td>12000</td>
<td>600</td>
<td>0.0015</td>
<td>0.0010</td>
<td>0.0005</td>
</tr>
<tr>
<td>14000</td>
<td>700</td>
<td>0.0018</td>
<td>0.0011</td>
<td>0.0006</td>
</tr>
<tr>
<td>16000</td>
<td>800</td>
<td>0.0020</td>
<td>0.0013</td>
<td>0.0006</td>
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<tr>
<td>18000</td>
<td>900</td>
<td>0.0023</td>
<td>0.0014</td>
<td>0.0007</td>
</tr>
<tr>
<td>20000</td>
<td>1000</td>
<td>0.0025</td>
<td>0.0016</td>
<td>0.0008</td>
</tr>
<tr>
<td>22000</td>
<td>1100</td>
<td>0.0028</td>
<td>0.0018</td>
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</tr>
<tr>
<td>24000</td>
<td>1200</td>
<td>0.0030</td>
<td>0.0019</td>
<td>0.0010</td>
</tr>
<tr>
<td>26000</td>
<td>1300</td>
<td>0.0033</td>
<td>0.0021</td>
<td>0.0010</td>
</tr>
<tr>
<td>28000</td>
<td>1400</td>
<td>0.0035</td>
<td>0.0022</td>
<td>0.0011</td>
</tr>
</tbody>
</table>

*Assumes ten percent of ADT is truck traffic.

3.6.5.2—Vehicle Collision with Barriers

The provisions of Section 13 shall apply.
WING WALL CHECK/DESIGN GUIDELINE

METHOD:
CALCULATE THE CRITICAL SLOPE IN TRANSITION AREAS NEAR WINGS. DETERMINE ACCEPTABILITY BASED ON CHECK OR DESIGN CRITERIA.

CRITERIA:
CHECK - CRITICAL SLOPE 2:1 OR FLATTER IS ACCEPTABLE.
DESIGN - DESIGN TO MEET CRITICAL SLOPE OF 2.5:1 (NOTE: IF MORE THAN 5 FEET BEYOND THE STANDARD WING EXTENSION IS REQUIRED, THE DESIGN CRITERIA MAY BE STEEPENED TO 2:1 PENDING APPROVAL BY THE SECTION LEADER.)

DEFINITIONS:
FOR "W" AND "B" POINTS, SEE BDM ARTICLE 3.2.7.3.3.
CRITICAL SLOPE = ΔH/ΔL
ΔH (FT) = ELEV. AT "W" - ELEV. AT "B"
ΔL (FT) = SHORTEST HORIZONTAL DISTANCE BETWEEN "W" AND THE LINE REPRESENTING THE TOP OF BERM ELEVATION AS MEASURED IN CAD PLAN VIEW (SEE SKETCH).

DETAILED STEPS:
1. DETERMINE "W" AND "B" POINT ELEVATIONS.
2. DRAW A LINE IN CAD PLAN VIEW REPRESENTING THE TOP OF BERM ELEVATION BETWEEN THE ROADWAY EMBANKMENT AND POINT "B".
4. FOR BRIDGES ON ZERO DEGREE SKEW AND FOR THE SIDE WITH THE LONGEST DISTANCE BETWEEN "W" AND "B", THE CRITICAL SLOPE IS ALONG THE LINE "W-B".
5. WING LENGTH IS ACCEPTABLE IF CRITERIA CAN BE MET ON BOTH SIDES.
6. IF WING LENGTH IS NOT ACCEPTABLE, TRY A LONGER WING. IT IS RECOMMENDED THAT ONLY EVEN FOOT INCREMENTS BE USED. IN GENERAL, KEEP WING LENGTHS THE SAME UNLESS THERE WOULD BE GREATER THAN A 5 FOOT DIFFERENCE.
7. NON-STANDARD WINGS SHALL BE NOTED AS SUCH ON THE TSL.

WING WALL CHECK/DESIGN GUIDELINE

REV. DATE: 11-14-14
C3.8 Cost estimates

C3.9 Type, Size, and Location (TS&L) plans

PRELIMINARY BRIDGE DESIGN
TS&L PLAN SHEET(S) LAYOUT GUIDELINES

Refer to the PLAN REVIEW CHECKLIST or PRELIMINARY DESIGN GUIDELINES available on the Bridge Web Site which include required information for the TS&L Plan sheet(s). The following guidelines are intended to provide consistency for placing information when additional plan sheet(s) are needed.

The first sheet shall show a typical bridge layout per guidelines and be labeled SITUATION PLAN below the plan view and in the title block.

Bridge sites typically have areas of interest such as stream meanders, interchanges, etc. which do not fit on a single Situation Plan sheet. To show these areas, a SITE PLAN sheet shall be created. This second plan sheet shall be labeled as SITE PLAN below the plan layout and the title block shall be labeled as SITUATION PLAN - SITE. The scale of the site plan layout may be changed (labeled with a Scale Legend) to adequately show conditions outside of the proposed structure area. Typically, the SITE PLAN shall be shown on one sheet. The SITE PLAN sheet may also be used to place information when insufficient room remains on the SITUATION PLAN sheet.

Any additional sheet(s) showing details or other preliminary information shall be labeled as MISCELLANEOUS DETAILS and the title block(s) should be labeled as SITUATION PLAN - MISC.

In general, additional plan sheets shall be created except for relatively small bridges where limited additional information is needed.

All items required by the PLAN REVIEW CHECKLIST or PRELIMINARY DESIGN GUIDELINES which are not listed in the mandatory or preferred item guidelines shall be placed at the designer’s discretion. The designer shall follow the guidelines of the mandatory and preferred items listed for both situation plan layout and site plan layout sheets when placing information.

Topography is defined as information typically obtained from the project survey such as ground features and utilities, excluding ground shots and contours.

The mandatory items listed below shall be shown on the situation plan layout sheet(s).

Mandatory Items for the Situation Plan layout sheet(s)

1. Situation Plan
   - SITUATION PLAN heading under plan view layout
   - Dimensions of Proposed Structure(s)
   - North Arrow
   - Centerline Roadway Alignments and labels
   - Centerline Stationing labels
   - Profile Grade Line labels
   - Existing Structure(s) (A)
   - Proposed Grading Slope Lines (not proposed contours) (A)
   - Revetment (A)
   - Slope Protection Note (A)
   - Guardrail Indicated
   - Topography (A)
Minimum Vertical Clearance Location (overhead bridges)
- Scale Legend
- Horizontal Clearance to Piers (overhead bridges)

2. Longitudinal Section
3. Typical Approach Section
4. Location Data
5. Bench Mark

(A) These items to be edited as required prioritizing clarity of other mandatory items or text. More comprehensive treatment of these items can be made on the site plan sheet in cases where extensive editing is required on the situation plan layout sheet(s).

The preferred items listed are expected to be shown on the situation plan layout sheet(s) but due to space restrictions may be shown on the site plan layout sheet.

Preferred Items for the Situation Plan layout sheet(s) (In order of preference)

1. Proposed Grade
2. Hydraulic Data
3. Traffic Estimate
4. Utilities Legend
5. Spiral Curve Data
6. Horizontal Curve Data
7. Minimum Vertical Clearance note
8. Staging Widths

The mandatory items listed below shall be shown on the site plan layout sheet. Some duplication is necessary for references between the multiple SITUATION PLAN sheets.

Mandatory Items for the Site Plan layout sheet

1. Site Plan
   - SITE PLAN heading under plan view layout
   - North Arrow
   - Centerline Roadway Alignments and labels
   - Centerline Stationing labels
   - Proposed Structure(s) (B)
   - Existing Structure(s) (B)
   - Proposed Grading Slope Lines (not proposed contours) (B)
   - Revetment (B)
   - Guardrail Indicated
   - Topography (B)
   - Scale Legend
   - Beginning & End Bridge Stations at Centerline Abutment Bearings

(B) These items should not be edited extensively on the site plan layout sheet and a more comprehensive treatment of these items should be shown on this sheet where extensive editing may have been necessary on the situation plan layout sheet(s).

The preferred items listed are expected to be shown on the site plan layout sheet but due to space restrictions may be shown on the situation plan layout sheet(s).

Preferred Items for the Site Plan layout sheet

1. Berm Slope Location Table & Associated Point I.D. Labels (Show together on the sheet)
2. Revetment Limits & Typical Section Details
3. Survey Ground Shots or Contours of existing ground supplemented with Ground Shots (not proposed contours)
### REVESTMENT QUANTITIES

<table>
<thead>
<tr>
<th>Revestment Type - Location</th>
<th>Revetment Type</th>
<th>Erosion Control</th>
<th>Engineering/Excavation</th>
<th>Sand Fabic</th>
</tr>
</thead>
<tbody>
<tr>
<td>BERM LINED STONE TOE - SOUTH</td>
<td>234</td>
<td>269</td>
<td>146</td>
<td></td>
</tr>
<tr>
<td>SLOPE PROTECTION - SOUTH</td>
<td>24.0</td>
<td>61</td>
<td>15.5</td>
<td></td>
</tr>
<tr>
<td>SLOPE PROTECTION - NORTH</td>
<td>34.2</td>
<td>60</td>
<td>15.1</td>
<td></td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td><strong>40.2</strong></td>
<td><strong>1.2</strong></td>
<td><strong>60.6</strong></td>
<td></td>
</tr>
</tbody>
</table>

Revestment excision quantity calculated from grading surface. Revestment and excision stone estimated at 16 TOT/20.
C3.10 Permits and approvals

C3.10.1 Waterway

Department of Natural Resources List of Meandered Streams
22 December 2006

Iowa Department of Natural Resources Sovereign Lands Construction Permits are required for work on or over meandered streams. (This is a different permit than a Floodplain Development Permit.) The term “meandered stream” for this permit is a legal description where the State of Iowa owns the stream bed and banks of certain reaches of rivers. A meandered stream is one which at the time of the original government survey was so surveyed as to mark, plat and compute acreage of adjacent fractional sections. DNR is responsible for this state-owned land and therefore issues a Construction Permit. The following is a list of the descriptions of the limits of these rivers in the state of Iowa.

1. Des Moines River. From Mississippi River to the junction of the east and west branches. The west branch to west line T95N, R32W, Palo Alto County, due south of Emmetsburg. The east branch to north line T95N, R29W, Kossuth County, near the north edge of Algona.

2. Iowa River. From Mississippi River to west line T81N, R11W, Iowa County, due north of Ladora.

3. Cedar River. From Iowa River to west line T89N, R13W, Black Hawk County, at the east edge of Cedar Falls.

4. Raccoon River. From Des Moines River to west line of Polk County.

5. Wapsipinicon River. From Mississippi River to west line T86N, R6W, Linn County northwest of Central City.

6. Maquoketa River. From Mississippi River to west line T84N, R3E Jackson County, due north of Maquoketa.

7. Skunk River. From Mississippi River to north line of Jefferson County, at the southwest edge of Coppock.

8. Turkey River. From Mississippi River to west line T95N, R7W, Fayette County, northwest of Clermont.


10. Upper Iowa River. From Mississippi River to west line Section 28, T100N, R4W, Allamakee County, about two and one-half miles upstream from its mouth.

11. Little Maquoketa River. From Mississippi River to west line Section 35, T90N, R2E, Dubuque County, about one mile upstream from its mouth.

12. Mississippi River, Missouri River, Big Sioux River.
C3.10.2 Railroad

C3.10.3 Highway

C3.11 Forms
Examples of forms to follow:
Bridge Cost Estimate for Concept Statement

Location:
County: Lucas
Des. No.: 1054
Maint. No.: 5927.38014
On IA 14 over English Creek
Section 13, T73N, R21W
Functional Class: ADT: 2580 vpd
By: D. Claman
Date: 5/17/2010

Existing Bridge:
Type: I-Beam
Pier Type: N/A
Spans: 60
Skew: 0
Length x Width: 60’ x 30’
Abut. Type: Stub
Approach Pavement Width: 30
Design Loading:
Drainage Area: 7.8 sq. mi.
Existing Bridge Width Acceptable: No
New/Reconstructed Roadway Width: 44.0’
Repair/Remodel by Staging Traffic: Yes

General Comments: Existing bridge is a 4-beam single span structure that could be staged. Stage 1 lane width would be 15’ wide and Stage 2 lane width would be approximately 12 feet wide with an additional 2’ wide bridge. Staging a slab bridge may create constructability issues due to deflection and falsework.

Option A - Stage 110’ x 46’ CCS Bridge
Type: CCS
Pier Type: Pile Bent
Spans: 1 @ 35’, 2@27.5’
Stage Traffic: Yes, One 15’ Lane - Stage 1, One 12’ Lane - Stage 2

Costs:
Bridge - 110’ x 46’ @ $755/sf = $375,500
Remove Exist. Bridge -60’ x 30’ @ $7.00/sf = $12,600
Riprap Berms - $50,000
Staged Construction (10%) = $44,210
Mobilization (10%) = $44,210
Contingency (15%) = $66,315
---------
Total Option A = $596,835

Comments: Staged CCS bridges may have constructability issues depending upon the contractor.
Bridge Concept Statement 4/12/2011

Lucas County
EEF-014-2(34)-38-59

Option B - 110' x 44' CSS Bridge - Detour

Type: CSS  Length x Width: 110' x 44'
Pier Type: Pile Bent  Abutment Type: Integral
Spans: 1@35.0, 2@27.5'  Skew: 0.0
Stage Traffic: No

Costs:
Bridge - 110 x 44' @ $75/sf  = $363,000
Remove Exist. Bridge 60' x 30' @ $7.00/sf  = $12,600
Riprap Berms  = $50,000
Mobilization (15%)  = $42,560
Contingency (15%)  = $63,840

Total Option B  = $532,000

Comments: Detour reduces construction time and eliminates constructability issues staging slab bridges.

Revisions:
None
# RECORD OF COORDINATION
## FLOODPLAIN DEVELOPMENT

The purpose of this form is to document Iowa Department of Transportation coordination with the local community for projects which are not within the Iowa Department of Natural Resources' permitting jurisdiction and which are in a community that is participating in the National Flood Insurance Program.

1. **Highway Number:** US 69  
   **Stream:** Kelgley Branch  
   **Project Location:**  
   **File No.:** 31080  
   **Design No.:** 116  
   **Project Location:**  
   **Description of Location:** On US 69 over Kelgley Branch, 1.1 Miles South of Co. Rd. E18

---

2. **Flood Insurance Rate Map/Floodway Map:**  
   **Panel Number:** 1916900040E  
   **Effective Date of Map:** February 20, 2008

3. **Type of Development:**  
   - [ ] Filling  
   - [ ] Grading  
   - [ ] Excavation  
   - [ ] Bridge Construction  
   - [ ] Road Construction  
   **Channel Improvement:** Lining upstream bank with riprap on outside of bend  
   **Description of Development:** Remove existing bridge. Replace with a new 120' x 44' Continuous Concrete Slab bridge.  
   Line upstream channel on outside of bend for channel migration and to protect the roadway embankment.

---

4. **Is project located in a designated 100-year floodplain?**  
   - [ ] Yes (check the appropriate zone:  
     - [ ] A  
     - [ ] A1-30  
     - [ ] AE  
     - [ ] AD  
     - [ ] AH)  
   - [ ] No

5. **Has a detailed Flood Insurance Study (FIS) been published?**  
   - [ ] Yes  
   - [ ] No  
   **If yes, what is the Base Flood Elevation (BFE) at project site?**  
   **If no, what is the estimated BFE at project site?** 578.9 (includes the bridge backwater)

---

6. **Is project located in designated floodway?**  
   - [ ] Yes  
   - [ ] No

7. **Does FIS need to be revised?**  
   - [ ] Yes  
   - [ ] No  
   **If yes, describe type and extent of revision:**

---

**David R. Claman, P.E.**  
IDOT Preliminary Bridge Design Engineer  
**Signature**  
**Date**

**Scott Dockslater, P.E.**  
IDOT District Engineer  
**Signature**  
**Date**

**Community Official Concurrence:**

---

**Community Official**  
**Signature**  
**Date**

**NOTE:** Office of Bridges and Structures to submit copy to:  
Bill Capuzzo  
NFIP State Coordinator  
Iowa Department of Natural Resources  
Wallace State Office Building  
502 East Ninth Street  
Des Moines, IA 50319  
515-281-8642

---

January 2020
EXEMPLARY

Iowa Department of Transportation
FIELD NOTES FOR BRIDGES AND LARGE CULVERTS (20' SPAN)
PRIMARY ROAD SYSTEM

LOCATION
1. County: Phone: Skill: WORTH
   NE 23
   829-624-6088
2. Over: Highway No. 400
   State Park
   Hwy 28
3. Mile: 19.28
   Rd: 5AC 22
   Range: 28E
   Sec: 61

GENERAL DATA (FIELD)
4. Derivation Area: 8.25 sq mi
   Designers: Inc.
   Approx. length and width: 8.4 mi x 2.8 mi
5. Extreme highwater: Date of occurrence: 1993
   Water level from: Ledges State Park Bridge Headroom Location
   (Gave the elev. 822.5 Location STA 6+01.21, KT 152.41') (Rev. Upstream)
   (Rev. downstream) Location
6. Typical highwater: Sec: 863.5
   Occurred every 7 Years. Date of last occurrence: 1993
   (Water level: 863.50 Location: 862.77 (Diluvial 862.47 on Atlas of survey)
   (Water level: 863.50 Location: 862.77 (Gave the elv 862.47 on Atlas of survey)
   (Water level: 863.50 Location: 862.77 (Gave the elv 862.47 on Atlas of survey)
   (Water level: 863.50 Location: 862.77 (Gave the elv 862.47 on Atlas of survey)
   (Water level: 863.50 Location: 862.77 (Gave the elv 862.47 on Atlas of survey)
   (Water level: 863.50 Location: 862.77 (Gave the elv 862.47 on Atlas of survey)
7. Limiting buildings in flood plain: Note: Location
   (Rev. Upstream) Location
   (Rev. Downstream) Location
   (Rev. Downstream) Location
8. Upstream land use: State Park
   Anticipate any changes? No
9. Is stream disappearing or filling? Flowing
   Approx. amount per year: Unknown
   (Rev. Downstream) Location
10. Is stream flowing? No
    (Rev. Upstream) Location
11. Does stream carry appreciable amount of dirt? Yes
    (Rev. Downstream) Location
12. Does stream carry appreciable amount of debris? Yes
    (Rev. Downstream) Location
13. Bank Mark No. BM050 BB: Spike in West Face of Flood Pole Northwest of G001 STA 6+47.21, KT 152.41

PRESENT OR OLD STRUCTURE
14. Superstructure Type: Uplift 20' x 7.75' Aluminum Box Culvert
    (Dek angle 27.42° L/A)
15. Substructure Type: N/A
16. Span lengths N/A: Roadway with 22'
    Type of roof: N/A
17. Culvert B&B Piles: Material: B & B Pile
    Nominal: 100"
18. Culvert Open: 28.8 X 28.8
    Material: Rt.
19. Grade: 100
    Design No. 204-8690 (Q-8690)
20. Condition of superstructure: Damaged beyond repair
21. Condition of substructure

PROPOSED STRUCTURE (OFFICE)
23. Superstructure Type: 130' x 30' Continuous Concrete Slab Bridge
    Span angle 30' L/A.
24. Substructure Type: Pile, Integral Abutments
25. Span lengths(Bridges): 26.57', 47.05', 46.5'
    Culvert B&B Piles: Material: B&B Pile
26. Culvert Open: IR
    Material: L/A
27. Roadway width: 30'
    Type of roof: Concrete
    Class of Roading: HL-83
28. Type of subgrade: TL-4, Open Road Design
    Type of cut
29. Grade: 871.96
    Actual: Filling above: 965.66
    Material: B&B Pile
30. Length and types of pile: Abutts: P1110X62 - 49'
    Class: 1110X62 - 53' (P1), 53' (P2)
31. Design flood: Basis: 80 YR
    Frequency 50 Year Area: 8,775 sq ft
    Discharge: 3,877 cfs
32. What provisions are made for maintenance? None
33. Can channel be closed to provide more drainage? No
    Are any existing structures to be preserved? No
34. Is excess water to be provided? No
    Probable max. depth of water above streambed 4.40
35. Date of estimated structure: Removable
36. 2017: Nov 30 VPD
37. Remarks:

County: Bureau
Project No.: 8K-624-03-28-08
File No.: 36586
File No.: 7-86-624-608
Design No.: 311
Main No.: 8600-35014

File Notes by: Adam Balcerek, P.E.
Date: 2/25/11
This Project Engineer:

January 2020
VALLAY CROSS SECTION DATA

The submittal of a bridge type structure will include a right angle valley section. This section should be taken downstream from the crossing. It shall be noted whether to be an average section or a control section. Enough ground truth will be taken to such the valley to an elevation and above extreme Nighthawks. Special care will be taken to accurately outline the main channel. Each shot should be identified by a (F) Flood point, (W) top of zone, (E) edge of stream, etc. shoreline, equation roughness factors will be assigned each shot. Include site photos with this information.

Remarks: Refer to HSC-RAS model for valley cross section data

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<thead>
<tr>
<th>Distance</th>
<th>Elevation</th>
<th>(b) Roughness</th>
<th>Remarks</th>
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PLAT OF DRAINAGE AREA

The plat map area is to be plotted as completely and accurately as possible and to the largest practicable scale on a separate sheet. Use a definite scale, as 1" equals 1/10, 1/2, or 1/4 mile, and indicate what scale has been used. In addition to the sections of the watershed, indicate the position of the streams and major tributaries, location of the bridge and all points. Wherever practicable, the above information should be secured by going over the area either by foot or in a car. For most waterways, this information may be secured from the best existing data, GOO maps, U.S.G.S. maps, and Section 10 of this map. No plat is necessary if the area is listed in Table Number 7.

Remarks:

Give additional information by reference to marginal number on reverse side of this sheet.

<table>
<thead>
<tr>
<th>Measure</th>
<th>Description</th>
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<tbody>
<tr>
<td>5</td>
<td>Storm highwater due to backwater from Sayreville Lake</td>
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<tr>
<td>10</td>
<td>Excessive silt deposition at this site is due to backwater from Sayreville Lake</td>
</tr>
<tr>
<td>18</td>
<td>Culvert flowing data based on construction year since flow data could not be obtained due to culvert damage</td>
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</tbody>
</table>

IMPORTANT NOTE

The information given on the form must in all cases be supplemented by complete plot and profile of the site, drawn to a convenient scale on a separate sheet.

The information as shown on this form is essential and must be supplied in detail before the plans can be prepared or approved. It will be necessary to return this form for correction unless the data supplied is complete.

January 2020
C3.12  Noise walls

Excerpts from AASHTO LRFD Bridge Design Specifications, 8th Edition, Section 15: Design of Sound Barriers, Copyright 2017, by the American Association of State Highway and Transportation Officials, Washington, DC. Used by permission:

In lieu of crash-testing, the resistance of components and connections to Extreme Event II force effects may be determined based on a controlled failure scenario with a load path and sacrificial elements selected to ensure desirable performance of a structural system containing the soundwall. Vehicular collision forces shall be applied to sound barriers located within the clear zone as follows:

Case 1: For sound barriers on a crashworthy traffic railing and for sound barriers mounted behind a crashworthy traffic railing with a sound barrier setback no more than 1.0 ft; vehicular collision forces specified in Section 15 shall be applied to the sound barrier at a point 4.0 ft above the surface of the pavement in front of the traffic railing for Test Levels 3 and lower and 6.0 ft above the surface of the pavement in front of the traffic railing for Test Levels 4 and higher.

Case 2: For sound barriers behind a crashworthy traffic railing with a sound barrier setback of 4.6 ft; vehicular collision force of 4.0 kips shall be...
applied. The collision force shall be assumed to act at a point 4.0 ft above the surface of the pavement in front of the traffic railing for Test Levels 3 and lower and 14.0 ft above the surface of the pavement in front of the traffic railing for Test Levels 4 and higher.

Case 3: For sound barriers behind a crashworthy traffic railing with a sound barrier setback between 1.0 ft and 4.0 ft: vehicular collision forces and the point of application of the force shall vary linearly between their values and locations specified in Case 1 and Case 2 above.

Case 4: For sound barriers behind a crashworthy traffic railing with a sound barrier setback more than 4.0 ft: vehicular collision forces need not be considered, herein are meant to be applied to the sound barriers portion of such systems.

Crash Test Levels 3 and lower are performed using small automobiles and pick-up trucks. Crash Test Levels 4 and higher include single unit, tractor trailer trucks, or both. The difference in height of the two groups of vehicles is the reason the location of the collision force is different for the two groups of sound barriers.

For crash Test Levels 3 and lower, the point of application of the collision force on the sound barriers is assumed to be always 4.0 ft above the pavement.

During crash testing of traffic railings for crash Test Level 4 and higher, trucks tend to tilt above the top of the railing and the top of the truck cargo box may reach approximately 4.0 ft behind the traffic face of the traffic railing. For such systems, the point of application of the collision force is expected to be as high as the height of the cargo box of a truck, assumed to be 14.0 ft above the pavement surface.

For sound barriers mounted on crashworthy traffic barriers or with a small setback assumed to be less than 1.0 ft, the full crash force is expected to act on the sound barrier. The point of application of this force is assumed to be at the level of the cargo bed, taken as 6.0 ft above the surface of the pavement.

For a sound barrier mounted with a setback more than 1.0 ft behind the traffic face of the traffic railing, it is expected that the truck cargo box, not the cargo bed, will impact the sound barrier. It is expected that the top of the cargo box will touch the sound barrier first. Due to the soft construction of cargo boxes, it is assumed that they will be crushed and will soften the collision with the sound barrier. The depth of the crushed area will increase with the increase of the collision force, thus lowering the location of the resultant of the collision force. The magnitude of the collision force and the degree to which the cargo box is crushed are expected to decrease as the setback of the sound barrier increases.

In the absence of test results, it is assumed that a collision force of 4.0 kips will develop at the top of the cargo box when it impacts sound barriers mounted with a setback of 4.0 ft.

The collision force and the point of application are assumed to vary linearly as the sound barrier setback varies between 1.0 ft and 4.0 ft.

The setback of the sound barrier, S, shall be taken as shown in Figure 15.8.4-1.
C3.13 Submittals

Collision forces on sound barriers shall be applied as a line load with a length equal to the longitudinal length of distribution of collision forces, \( L_c \), specified in Appendix A.13.

For sound barriers prone to vehicular collision forces, the wall panels and posts and the post connections to the supporting traffic barriers or footings shall be designed to resist the vehicular collision forces at the Extreme Event II limit state.

For post-and-panel construction, the design collision force for the wall panels shall be the full specified collision force placed on one panel between two posts at the location that maximizes the load effect being checked. For posts and post connections to the supporting components, the design collision force shall be the full specified collision force applied at the point of application specified in Cases 1 through 3 above.

The vehicular railing part of the sound barrier system does not need to satisfy any additional requirements beyond the requirements specified in Section 13 of the Specifications for the stand-alone railings, including the height and resistance requirements.

Unless otherwise specified by the Owner, vehicular collision forces shall be considered in the design of sound barriers.

In some cases, the wall panel is divided into a series of horizontal elements. In these situations, each horizontal strip should be designed for the full design force.

Owners may select to ignore vehicular collision forces in the design of sound barriers at locations where the collapse of the sound barrier or portions of thereof has minimal safety consequences.
C3.14 Zone of Intrusion


*Figure 1: Guideline for Desired Clearance (Adapted from Zone of Intrusion for TL-4 Barriers per NCHRP Report 350, Ref. AASHTO RDG Figure 5-31)*

*Figure 2: Guideline for Minimum Clearance (Adapted from Zone of Intrusion Guidelines for TL-3 Concrete Barriers Ref. AASHTO RDG Figure 5-28)*

Note: The 34 inch tall and 44 inch tall Iowa Standard F-shape barrier rails meet National Cooperative Highway Research Program (NCHRP) Report 350 Test Level 4 (TL-4) and Test Level 5 (TL-5) respectively. Note that the Iowa Standard F-shape barrier rails are 2 inches taller than the minimum heights required for TL-4 and TL-5 barrier rails in order to account for the possibility of a 2 inch thick future overlay.