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Example TSL development report

PRELIMINARY BRIDGE TSL DEVELOPMENT REPORT Date:

I-35 (S.B.) & I-80 (W.B.) over U.S. 6 (Hickman
Road)
Project No. IM-080-3(267)125--13-77
PIN: 15-77-080-060
File No. 32251
Polk County – Design No. 0625
240'-0 x 88'-4 Welded Plate Girder (WPG)
Bridge Location: U.S. 6/Hickman Road
Interchange Station 419+51.02, 42.00' Lt.
(℄ I-35/80)
Maintenance No. 7725.1L080
FHWA No. 41311
Work Description: Bridge Replacement – WPG Bridge
Prepared for: Iowa DOT
Prepared by: Consultant

TSL DEVELOPMENT DETAILS

1. BDM 3.3 – Highway crossings
    a. Vertical clearance, for the proposed U.S. 6 DDI, was checked to ensure that the vertical clearance met or exceeded the required 16'-6" clearance over primary highways.
    b. Vertical clearance, to existing U.S. 6, was checked to ensure that the vertical clearance met or exceeded 14'-6" for the temporary condition during staged construction.
    c. Vertical clearance within the horizontal clear zone was checked to ensure that the vertical clearance met or exceeded 14'-6".
    d. The bridge is a single span structure with abutments placed behind MSE walls.

   e. Horizontal clearance to the MSE walls is 15'-0" from the back of curb. This 15'-0" of clearance will provide for snow storage when necessary.
   f. Pedestrian facilities under the bridge are currently in development. A shared use path alignment/location is currently under development. A sidewalk may also be included under the bridge. The sidewalk alignment/location/need are currently under development.
2. The roadway profile for I-35/80 is in a crest curve at the bridge location.
3. BDM 3.6.1.7 – Superstructure – CWPG
   a. The bridge length was determined by establishing the location of the MSE walls to provide the required clear zone. To reduce the length of the bridge, MSE walls are planned to retain the earth fill in front of the abutments and wrap around to retain approach fill.
   b. A single span 240'-0 x 88'-4 WPG bridge with a 2° skew (R.A.) was selected for the site.
   c. The final bridge roadway width consists of a 16'-4 inside shoulder, five 12'-0 lanes and a 12'-0 exterior shoulder as indicated in the approved Concept Statement.
   d. The proposed superstructure utilizes a steel girder with a depth of 6'-10, plus an 8.5" concrete deck; girders are spaced at 7'-1 5/16" and will likely utilize two field-bolted splices. The depth does not meet the traditional minimum depth shown in AASHTO LRFD Bridge Design Specifications, 8th Edition Table 2.5.2.6.3-1. However, a preliminary design of the girder indicates that all strength and serviceability requirements can be achieved. A shallower than traditional minimum depth steel girder at a tighter spacing was utilized to minimize the profile grade raise of I-35/80.
   e. The bridge staging and constructability was reviewed among the design team, DOT staff, and at an Iowa DOT / AGC of Iowa Structures Industry meeting. It was determined that the single-span bridge, with multiple bolted field splice locations, is feasible to build. Traffic can be fully maintained on U.S. 6 (Hickman Road) with small closure windows for bridge demolition and setting girders.
4. BDM 3.6.8 – Barrier Rails
   a. The barrier rails for all interstate mainline bridges shall require a TL-5 railing.
   b. Barrier rails for this project will be the TSS TL-5 rails.
5. BDM 3.6.9 – Staging
   a. The bridge will be constructed in two stages. The exterior (east side) of the bridge will be built in stage 1 and the median side of the bridge will be built in stage 2. Each stage will allow for 3 lanes of traffic with 11' lane widths. A portion of the existing bridge will be removed, and traffic will be maintained on the remaining existing bridge during stage 1. Traffic during stage 2 will shift to the new bridge previously constructed in stage 1. During stage 2, the remaining existing bridge will be demolished, and the remaining proposed bridge will be constructed.
   b. Temporary shoring will be required between the new MSE walls and the existing bridge embankment.
6. BDM 3.7.1 – Substructures – Skew
   a. The bridge abutments and MSE walls will be placed at a skew of 2° (R.A.) to match the skew of U.S. 6 (Hickman Road) to I-35/80.
7. BDM 3.7.2 – Abutments
   a. Semi-integral abutments will be placed approximately 1.5 feet behind the MSE walls.
   b. There will be two rows of vertical piles supporting the semi-integral abutments.
   c. A dead man will be installed behind the abutments to anchor the abutments and resist the longitudinal forces since battered piles are not feasible behind MSE walls.
8. BDM 3.7.3.7 – Mechanically Stabilized Earth (MSE) Walls adjacent to abutments
   a. The clear zone within the DDI is 10 feet from the edge of traveled way.
   b. The MSE wall is located outside of the 10-foot clear zone with deep foundations.
9. BDM 3.7.5 – Wing walls
   a. The abutments for the bridge will not have wing walls. The end of the bridge abutment will

abut the MSE wall.
    b. The MSE wall is needed to narrow the bridge embankment and allow the ramps to be closer to I-35/80 mainline.
10. BDM 3.7.6 – Foundation Conflicts
    a. The bridge abutments and MSE walls are located behind the existing abutments on each end of the bridge. There are no foundation conflicts with proposed abutment foundations.
    b. The removal of existing foundations will be developed during Final Design to determine removal extents to prevent interference to roadway construction.
11. Bridge aesthetics will be incorporated during Final Design. Discussions with team members and communities is ongoing as aesthetic features are currently being developed.
12. Under bridge deck lighting will be investigated during Final Design. Under bridge deck lighting will also be considered as a part of the aesthetic design for the bridge.

C3.2.1 Identification numbers

C3.2.2 Stream and river crossings

A certified Hydrology and Hydraulics (H&H) Report in pdf file format shall be prepared to document the design flowrates selected, design criteria, and proposed structure hydraulic design. A typical H&H Report might include the following information. More or less information may be needed depending on the complexity of the site.

- Purpose of Study and Introduction
- Site Description
- Existing conditions (ex: structure type, size, span arrangements, superstructure depth, low beam; low roadway location and elevation; review and document any existing scour, erosion, or channel shifting). History of overtopping at a water crossing site can be requested through the Iowa DOT District Maintenance Engineer.
- Document project datum, Iowa RCS Zone and datum correlations (LiDAR, existing plans, etc.)
- Listing of applicable hydraulic design and regulatory criteria. Identify required permits/approvals
    - Iowa DOT BDM policy
    - Iowa DNR Flood Plain Permit requirements
    - FEMA requirements (identify the FIRM Zone, site location panel number and date, document if there is a flood profile in the FIS and if no-rise is required)
    - Drainage District – slope, channel geometry, flowline requirements
    - Determination of upstream damage potential, and identification of possible high damage potential structures
- Identify needed coordination with DOT bureaus or outside entities that may affect the proposed preliminary design (Ex. DOT Location/Environment Bureau, Drainage district, US Coast Guard, etc.)
- Hydrology – Determination of recommended design discharges
    - Drainage Area (DA). Include StreamStats report with basin characteristics. Note if DA edits were needed.
    - AEPD spreadsheet, gage information (document if not applicable), USGS flood report excerpts
    - Proposed design discharges to be utilized for evaluating compliance with DOT and DNR Criteria. Include rationale for recommended method based on the DOT policy guidance.
    - The FIS published Q100, known as the base flood, may be different than the selected design discharge Q100 if a different methodology was used. The base flood is only utilized to evaluate compliance with FEMA requirements.
    - If an RIDB dataset is needed, a full range of discharges will need to be analyzed to create the stage/discharge relationships.
- Hydraulic Design- Verify compliance with Hydraulic Criteria

        - DOT policy and DNR requirements- design discharges shall be used to analyze backwater, freeboard, average bridge velocity, scour, level of service for overtopping, etc.
        - FEMA requirements –analysis based on the FIS base flood. It is most commonly used to document a proposed no-rise as compared to existing conditions, or for preparing an FIS map revision. The base flood may be higher or lower than the project design Q100.
    - Review of the site morphology (review historical maintenance reports, existing bridge plans, aerial photography, etc.). Document the estimated future degradation to be considered in the scour analysis.
    - Document the streambed profile, design streambed elevation, and methods for determination [BDM 3.2.2.10]
    - Model selection for hydraulic analysis
        - Document the hydraulic model and version/date used for the analysis. Include rationale for hydraulic model selection. If a site is within a detailed FIS, use of the FEMA model may be preferred.
        - Document input data, boundary conditions and assumptions.
        - For sites with shallow bedrock, follow guidance within BDM 3.7.4 regarding assumed pier widths for hydraulic analysis
    - Proposed structure and site features
        - Bridge size, type, span arrangements, wing dikes (when applicable)
        - Calculation breakdown from profile grade to operational and channel low beam
        - Note whether the roadway profile grade will have a proposed change or stay the same
        - Low roadway overtop location and condition (any change from existing?)
        - Overflow structures, if so, what is the type, size, and location
        - Proposed revetment recommendations for bridges (class of revetment, thickness, locations/extent)
    - Summary of hydraulic results
        - Identify any hydraulic model calibration and data utilized (example flood report stream profiles, on-site gage data)
        - Documentation relative to design hydraulic criteria compliance (ex: freeboard, average bridge velocity, backwater
        - Documentation relative to FEMA or other criteria compliance (ex: no-rise)
    - Describe any necessary actions or mitigations required for non-compliance with design policy or regulatory criteria (Ex: DNR variance request, drainage/flowage easement needed)
    - Provide a summary of scour calculations (clear water vs. live bed) and results (contraction scour, pier scour, and degradation components and total scour depth/elevation). Review and document the potential for abutment scour and recommended mitigation. Review the proposed structure to ensure global effective slope stability or provide mitigation/protection for an extreme event.

C3.2.2.1 Hydrology

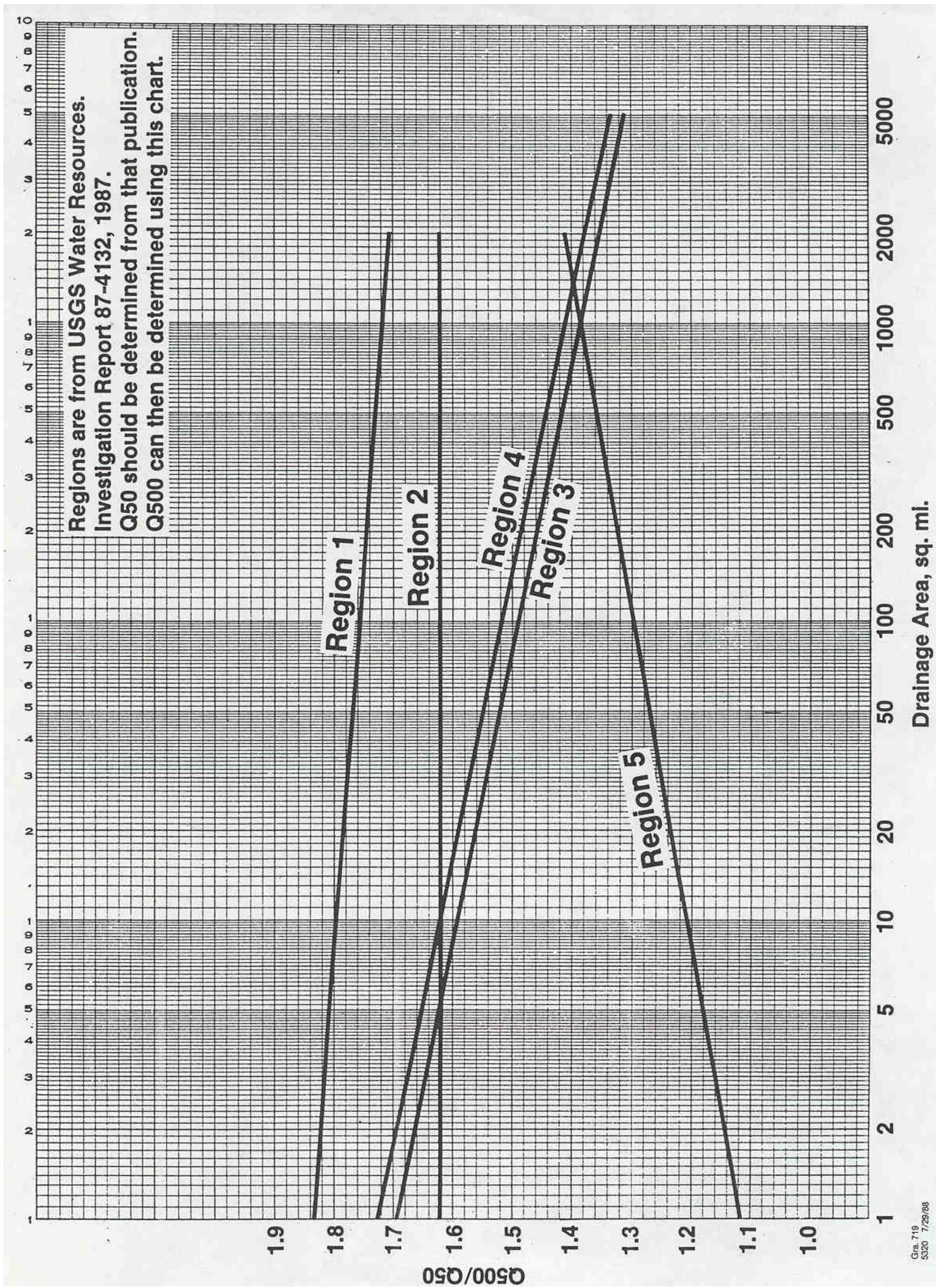

Regions are from USGS Water Resources.
Investigation Report 87-4132, 1987.
Q50 should be determined from that publication.
Q500 can then be determined using this chart.
Region 1
Region 2
Region 3
Region 4
Region 5
Drainage Area, sq. mi.
Q500/Q50

Table 1. Chronology of U.S. Geological Survey reports documenting flood profiles of streams in Iowa, 1963-2012.

Report number	Report citation	Report URL
1	Myers, R.E. 1963, Floods at Des Moines, Iowa: U.S. Geological Survey Hydrologic Investigations Atlas HA-53, 1 sheet, scale 1:24,000, included in Open-File Report 67-37 (listed below).	https://doi.org/10.3133/ofr6737
2	Schwob, H.H., 1963, Cedar River Basin floods: Ames, Iowa Department of Transportation, Iowa Highway Research Board Bulletin No. 27, 59 p.	https://pubs.er.usgs.gov/publication/70168617
3	Schwob, H.H., and Meyers, R.E., 1965, The 1965 Mississippi River flood in Iowa: Iowa City, U.S. Geological Survey Open-File Report 65-145, 46 p. Sponsored cooperatively by the Iowa Geological Survey.	https://doi.org/10.3133/ofr65145
4	Schwob, H.H., 1966b, Little Sioux River Basin floods: U.S. Geological Survey Open-File Report 67-196, 60 p.	https://doi.org/10.3133/ofr67196
5	Carpenter, P.J., and Appel, D.H., 1966, Water-surface profiles of Raccoon River at Des Moines, Iowa: U.S. Geological Survey Open-File Report 67-37, 12p., includes Hydrologic Investigations Atlas HA-53 (listed above). Sponsored cooperatively by the Iowa Institute of Hydraulic Research and City of Des Moines.	https://doi.org/10.3133/ofr6737
6	Schwob, H.H., 1967, Floods on Otter Creek in Linn County, Iowa: U.S. Geological Survey Open-File Report 67-195, 22 p. Sponsored cooperatively by Linn County, Iowa.	https://doi.org/10.3133/ofr67195
7	Carpenter, P.J., 1967, Floods in Rock River Basin: U.S. Geological Survey Open-File Report 67-36, 28 p.	https://doi.org/10.3133/ofr6736
8	Schwob, H.H., 1968, Flood of June 7, 1967, in the Wapsinonoc Creek Basin, Iowa: U.S. Geological Survey Open-File Report 68-b, 21 p.	https://doi.org/10.3133/ofr68b
9	U.S. Geological Survey, 1968, Flood profile study, Squaw Creek, Linn County, Iowa, U.S. Geological Survey Open-File Report 68-302, 13 p. Sponsored cooperatively by the City of Cedar Rapids, Iowa.	https://doi.org/10.3133/ofr68302
10	Schwob, H.H., 1970d, Floods in the upper Des Moines River Basin, Iowa: U.S. Geological Survey Open-File Report 70-296, 49 p.	https://doi.org/10.3133/ofr70296
11	Schwob, H.H., 1970c, Flood profile study, Morgan Creek, Linn County, Iowa: U.S. Geological Survey Open-File Report 70-295, 16 p. Sponsored cooperatively by the City of Cedar Rapids, Iowa.	https://doi.org/10.3133/ofr70295
12	Schwob, H.H., 1970a, Flood of March 3, 1970, on Old Mans Creek, Johnson County, Iowa: U.S. Geological Survey Open-File Report 70-293, 9 p.	https://doi.org/10.3133/ofr70293
13	Schwob, H.H., 1970b, Flood profile study, Hoosier Creek, Linn County, Iowa: U.S. Geological Survey Open-File Report 70-294, 18 p. Sponsored cooperatively by Linn County, Iowa.	https://doi.org/10.3133/ofr70294
14	Schwob, H.H., 1971, Floods in the Wapsipinicon River Basin, Iowa: U.S. Geological Survey Open-File Report (unnumbered), 52 p.	https://doi.org/10.3133/70006260
15	Heinitz, A.J., 1973a, Floods in the Iowa River Basin upstream from Coralville Lake, Iowa: U.S. Geological Survey Open-File Report 73-106, 75 p.	https://doi.org/10.3133/ofr73106
16	Heinitz, A.J., 1973b, Floods in the Rock River Basin, Iowa: U.S. Geological Survey Open-File Report 74-1047, 74 p.	https://doi.org/10.3133/ofr741047
17	Lara, O.G., and Heinitz, A.J., 1976, Flood of June 27, 1975, in city of Ames, Iowa: U.S. Geological Survey Open-File Report 76-728, 56 p.	https://doi.org/10.3133/ofr76728
18	Heinitz, A.J., 1977, Floods in the Big Creek Basin, Linn County, Iowa, U.S. Geological Survey Open-File Report 77-209, 35 p. Sponsored cooperatively by Linn County, Iowa.	https://doi.org/10.3133/ofr77209
19	Heinitz, A.J., and Wiitala, S.W., 1978, Floods in the Skunk River Basin, Iowa: U.S. Geological Survey Open-File Report 79-272, 80 p.	https://doi.org/10.3133/ofr79272
20	Heinitz, A.J., 1979, Supplement to floods in the upper Des Moines River Basin, Iowa: U.S. Geological Survey Open-File Report 79-1486, 6 p.	https://doi.org/10.3133/ofr791486
21	Heinitiz, A.J., 1980, Floods in the Raccoon River Basin, Iowa: U.S. Geological Survey Open-File Report 80-162, 110 p.	https://doi.org/10.3133/ofr80162
22	Heinitz, A.J., and Riddle, D.E., 1981, Floods in the English River Basin, Iowa: U.S. Geological Survey Open-File Report 81-67, 61 p.	https://doi.org/10.3133/ofr8167
23	Heinitz, A.J., 1986a, Floods in south-central Iowa: U.S. Geological Survey Open-File Report 85-100, 95 p.	https://doi.org/10.3133/ofr85100
24	Heinitiz, A.J., 1986b, Floods of June-July, 1982, in Iowa: U.S. Geological Survey Open-File Report 85-151, 18 p.	https://doi.org/10.3133/ofr85151
25	Heinitz, A.J., 1986c, Floods in the Floyd River Basin, Iowa: U.S. Geological Survey Open-File Report 86-476, 61 p.	https://doi.org/10.3133/ofr86476
26	Eash, D.A., and Heinitz, A.J., 1991, Floods in the Nishnabotna River Basin, Iowa: U.S. Geological Survey Open-File Report 91-171, 118 p.	https://doi.org/10.3133/ofr91171

27	Baebenroth, R.W., and Schaap, B.D., 1992, Floods of 1986 and 1990 in the Raccoon River Basin, west-central Iowa: U.S. Geological Survey Open-File Report 92-94, 144 p.	https://doi.org/10.3133/ofr9294
28	Barnes, K.K., and Eash, D.A., 1994, Flood of June 17, 1990, in the Clear Creek Basin, east-central Iowa: U.S. Geological Survey Open-File Report 94-78, 21 p.	https://doi.org/10.3133/ofr9478
29	Einhellig, R.F., and Eash, D.A., 1996, Floods of June 17, 1990, and July 9, 1993, along Squaw Creek and the South Skunk River in Ames, Iowa, and vicinity: U.S. Geological Survey Open-File Report 96-249, 34 p.	https://doi.org/10.3133/ofr96249
30	Eash, D.A., 1996b, Flood of May 19, 1990, along Perry Creek in Plymouth and Woodbury Counties, Iowa: U.S. Geological Survey Open-File Report 96-476, 39 p.	https://doi.org/10.3133/ofr96476
31	Eash, D.A., and Koppensteiner, B.A., 1996, Floods of July 12, 1972, March 19, 1979, and June 15, 1991, in the Turkey River Basin, northeast Iowa: U.S. Geological Survey Open-File Report 96-560, 55 p.	https://doi.org/10.3133/ofr96560
32	Eash, D.A., and Koppensteiner, B.A., 1997a, Floods of September 15-16, 1992, in the Thompson, Weldon, and Chariton River Basins, south-central Iowa: U.S. Geological Survey Open-File Report 97-122, 68 p.	https://doi.org/10.3133/ofr97122
33	Eash, D.A., and Koppensteiner, B.A., 1997b, Flood of July 9-11, 1993, in the Raccoon River Basin, west-central Iowa: U.S. Geological Survey Open-File Report 97-557, 117p.	https://doi.org/10.3133/ofr97557
34	Schaap, B.D., and Harvey, C.A., 1995, Delineation of flooding within the upper Mississippi River Basin, 1993--Flood of June 29-September 18, 1993, in Iowa City and vicinity, Iowa: U.S. Geological Survey Hydrologic Investigations Atlas HA735-B, 1 sheet, scale 1:24,000.	https://doi.org/10.3133/ha735B
35	Schaap, B.D., 1996a, Delineation of flooding within the upper Mississippi River Basin--Flood of June 19-July 31, 1993, in Davenport, Iowa, and vicinity: U.S. Geological Survey Hydrologic Investigations Atlas HA735-C, 1 sheet, scale 1:24,000.	https://doi.org/10.3133/ha735C
36	Schaap, B.D., 1996b, Delineation of flooding within the upper Mississippi River Basin--Flood of June 18 through August 4, 1993, in Des Moines and vicinity, Iowa: U.S. Geological Survey Hydrologic Investigations Atlas HA735-D, 2 sheets, scale 1:24,000.	https://doi.org/10.3133/ha735D
37	Fischer, E.E., 1999, Flood of June 15-17, 1998, Nishnabotna and East Nishnabotna Rivers, southwest Iowa: U.S. Geological Survey Open-File Report 99-70, 15 p.	https://doi.org/10.3133/ofr9970
38	Ballew, J.L., and Fischer, E.E., 2000, Floods of May 17-20, 1999, in the Volga and Wapsipinicon River Basins, northeast Iowa: U.S. Geological Survey Open-File Report 00-237, 36 p.	https://doi.org/10.3133/ofr00237
39	Ballew, J.L., and Eash D.A., 2001, Floods of July 19-25, 1999, in the Wapsipinicon and Cedar River Basins, northeast Iowa: U.S. Geological Survey Open-File Report 01-13, 45 p.	https://doi.org/10.3133/ofr0113
40	Eash, D.A., 2004a, Flood of June 4, 2002, in the Indian Creek Basin, Linn County, Iowa: U.S. Geological Survey Open-File Report 2004-1074, 31 p.	https://doi.org/10.3133/ofr20041074
41	Eash, D.A., 2004b, Flood of June 4-5, 2002, in the Maquoketa River Basin, east-central Iowa: U.S. Geological Survey Open-File Report 2004-1250, 29 p.	https://doi.org/10.3133/ofr20041250
42	Eash, D.A., 2006, Flood of May 23, 2004, in the Turkey and Maquoketa River Basins, Northeast Iowa: U.S. Geological Survey Open-File Report 2006-1067, 35 p.	https://doi.org/10.3133/ofr20061067
43	Fischer, E.E., and Eash, D.A., 2008, Flood of May 6, 2007, Willow Creek, West-Central Iowa: U.S. Geological Survey Open-File Report 2008-1229, 11 p. with appendix.	https://doi.org/10.3133/ofr20081229
44	Fischer, E.E., and Eash, D.A., 2010, Flood of June 8-9, 2008, Upper Iowa River, northeast Iowa: U.S. Geological Survey Open-File Report 2010-1087, 17 p. with appendix.	https://doi.org/10.3133/ofr20101087
45	Linhart, S.M., and Eash, D.A., 2010, Floods of May 30 to June 15, 2008, in the Iowa and Cedar River Basins, eastern Iowa: U.S. Geological Survey Open-File Report 2010-1190, 99 p. with appendixes.	https://doi.org/10.3133/ofr20101190
46	Eash, D.A., 2012, Floods of July 23-26, 2010, in the Little Maquoketa River and Maquoketa River Basins, northeast Iowa: U.S. Geological Survey Open-File Report 2011-1301, 45 p. with appendix.	https://doi.org/10.3133/ofr20111301
47	Barnes, K.K., and Eash, D.A., 2012, Flood of August 11-16, 2010, in the South Skunk River Basin, central and southeast Iowa: U.S. Geological Survey Open-File Report 2012-1202, 27 p. with appendix.	https://doi.org/10.3133/ofr20121202
48	Linhart, S.M., and O’Shea, P.S., 2018, Flood of August 24–25, 2016, Upper Iowa River and Turkey River, northeastern Iowa: U.S. Geological Survey Open-File Report 2017–1128, 20 p., with appendix.	https://doi.org/10.3133/ofr20171128

49	O'Shea, P.S., Vegrzyn, J.C., and Barnes, K.K., 2021, Flood of June 30–July 1, 2018, in the Fourmile Creek Basin, near Ankeny, Iowa: U.S. Geological Survey Open-File Report 2021–1044, 18 p.	https://doi.org/10.3133/ofr20211044
50	O'Shea, P.S., Wilson, J.L., Vegrzyn, J.C., and Barnes, K.K., 2022, Floods of June 21–July 1, 2018, in the Floyd River and Little Sioux River Basins, northwestern Iowa: U.S. Geological Survey Open-File Report 2022–1015, 35 p.	https://doi.org/10.3133/ofr20221015

C3.2.2.2 Hydraulics

C3.2.2.3 Backwater

C3.2.2.4 Freeboard

C3.2.2.5 Road grade overflow

C3.2.2.6 Streambank protection

Bank Toe Protection Example

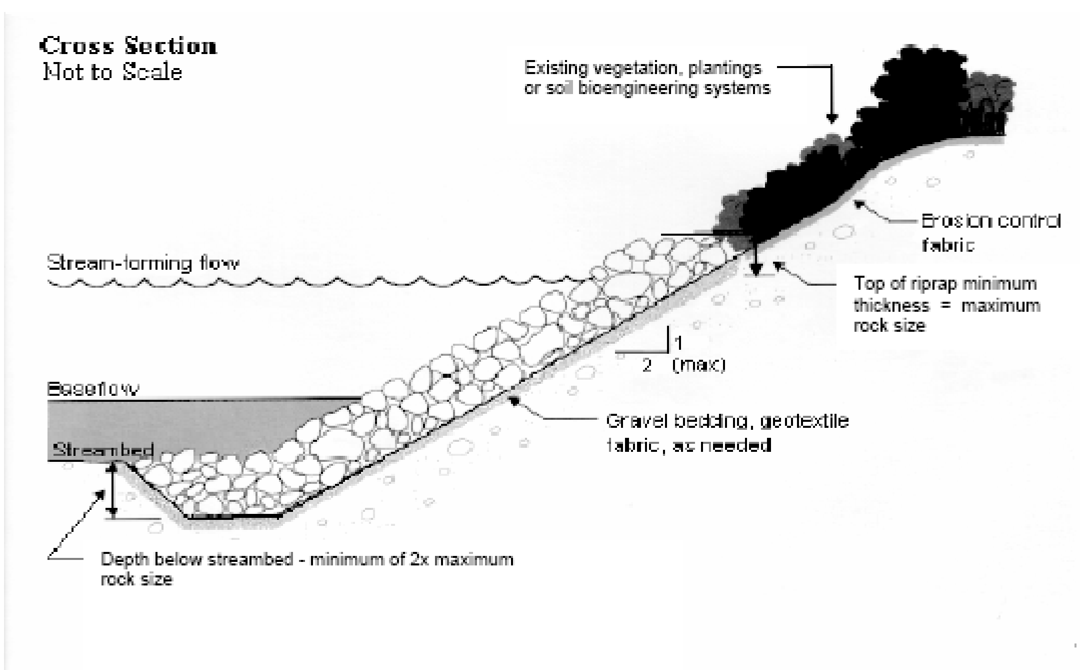


Figure 24. Cross-sectional view of riprap placement on the graded slope of a Streambank.

C3.2.2.7 Scour

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 Unit, 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 ($Q_{overbank2}$).

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:

$$\frac{y_2}{y_1} = \left[\frac{Q_2}{Q_1}\right]^{0.86} \left[\frac{W_1}{W_2}\right]^{k1}$$

and $y_s = y_2 - y_1 =$ average scour depth, ft

where y_1 = average depth in the upstream main channel, ft

y_2 = average depth in the contracted section (i.e., in the bridge opening), ft
W_1 = top width of water in the upstream main channel, ft
W_2 = top width of water in the main channel in the contracted section (i.e., in the bridge opening), ft
Q_1 = discharge in the upstream main channel transporting sediment, cfs.
(Q$_1$ does not include upstream overbank flow)
Q_2 = discharge in the contracted channel (i.e., bridge opening), cfs
(For Cases 1a and 1b, Q_2 may be the total flow going through the bridge opening. For Case 1c, Q_2 is not the total flow through the bridge since there is also some overbank Q adjacent to the channel under the bridge.)
k_1 = exponent. Assume $k_1 = 0.64$ to simplify the calculations since the range for k_1 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:

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

and $y_s = y_2 - y_1$ = average scour depth, feet

where y_2 = depth in the bridge opening, ft
Q = discharge through the bridge opening or on the overbank portion of the bridge opening, cfs
D_{50}= median diameter of material in overbank, feet (see attached sediment size table from HEC-20)
W_2= top width of water in bridge opening or overbank width in bridge opening (set-back distance), feet
y_1 = 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:

$$\left(\frac{y'_s}{w_p}\right) = 1.485\left(\frac{y_1}{w_p}\right)^{0.314} \qquad \text{Equation 1}$$

where

y'_s , unfactored depth of scour, ft

y_1 , unscoured depth of flow, ft

w_p , width of pier column, ft

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

$$y_s = (K)\,(y'_s) = (K)\,(w_p)\left(\frac{y'_s}{w_p}\right)$$

$$y_s = 1.485\,(K)\,(w_p)\left(\frac{y_1}{w_p}\right)^{0.314} \qquad \text{Equation 2}$$

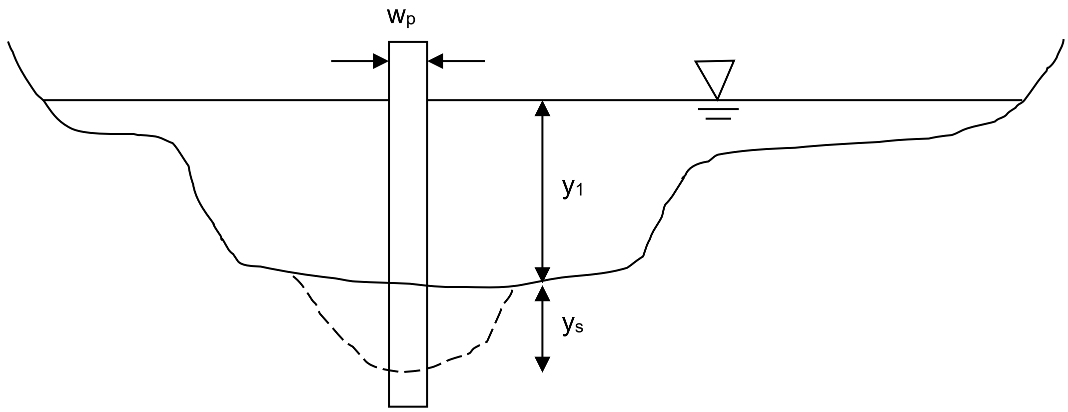
where y_s is depth of scour, ft

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.

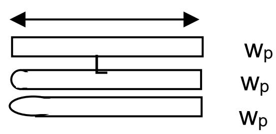
K_a, coefficient for angle of attack if angle is not zero (see table below).

Equation 2 should be used to calculate pier scour.



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

Rectangular	1.0
Semicircular	0.9
Elliptic	0.8



If angle of attack is not zero, use the following table adapted from Figure 39 in Bulletin No. 4 to determine K_a. In this table, L = length of pier, and w_p = width of pier. Use this K_a value in Equation 2 in place of K. 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 of 8, if the angle of attack changes from 0° to 15°, the factor K_a changes from 1.0 to 2.0, doubling the calculated pier scour.

Design Factors (K_a) for Piers Not Aligned With Flow

L/w_p / **Angle of Attack**	**4**	**6**	**8**	**10**	**12**	**14**
0°	1.0	1.0	1.0	1.0	1.0	1.0
5°	1.2	1.3	1.3	1.5	1.6	1.6
10°	1.4	1.5	1.7	1.9	2.1	2.3
15°	1.5	1.8	2.0	2.2	2.5	2.7
20°	1.7	2.0	2.3	2.5	2.8	3.0
25°	1.8	2.2	2.5	2.8	3.1	3.5
30°	1.9	2.4	2.7	3.1	3.4	3.8
35°	2.0	2.5	2.9	3.3	3.7	4.0
40°	2.1	2.7	3.1	3.6	4.0	4.3
45°	2.2	2.8	3.3	3.8	4.2	4.6

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 (D_{50} 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 (D_{50} 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):

$$D_{50} = \frac{(K\,V)^2}{153.6}$$

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 the 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

1. "Evaluating Scour at Bridges", Hydraulic Engineering Circular No. 18, Federal Highway Administration, Second Edition, April 1993.
2. "Evaluating Scour at Bridges", Hydraulic Engineering Circular No. 18, Federal Highway Administration, Third Edition, November 1995.
3. "Scour Around Bridge Piers and Abutments", Emmett M. Laursen and Arthur Toch, Iowa Highway Research Board, Bulletin No. 4, May 1956.
4. "Hydraulics of Bridge Waterways", Hydraulic Design Series No. 1, Federal Highway Administration, March 1978.
5. "Design of Riprap Revetment", Hydraulic Engineering Circular No. 11, Federal Highway Administration, 1989.
6. "Stream Stability at Highway Structures", Hydraulic Engineering Circular No. 20, Federal Highway Administration, February 1991.
7. "Stream Stability at Highway Structures", Hydraulic Engineering Circular No. 20, Federal Highway Administration, Second Edition, November 1995.
8. "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges", Federal Highway Administration, December 1995.
9. "Evaluating Scour at Bridges", Hydraulic Engineering Circular No. 18, Federal Highway Administration, Fifth Edition, April 2012.

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 (K_s)________
Angle of Attack _____ Coeff (K_{al})_______
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* ________ D_{50} ___________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. ____________

	Q_{200}	Q_{500} or $Q_{overtopping}$
Discharge (Q), cfs	____________	____________
Water surface elev.	____________	____________
y_1, depth in main channel, ft	____________	____________
Vel. in main channel, fps	____________	____________

CONTRACTION SCOUR

$V_c = 10.95\ y^{0.167}\ D_{50}^{0.33} =$ _______________ ft/sec. If V_c < average channel velocity, use live-bed scour equation. If V_c > 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_{200}	Q_{500} or $Q_{overtopping}$
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	_______________	_______________
$y_s = y_2 - y_1 =$ 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.

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

	Q_{200}	Q_{500} or $Q_{overtopping}$
______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	____________	_______________
$y_s = y_2 - y_1 =$ 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.

$$y_s = 1.485\,(K)\,(w_p)\left(\frac{y_1}{w_p}\right)^{0.314} \qquad \text{Equation 2}$$

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).

	Q_{200}	Q_{500} or $Q_{overtopping}$
y_1, ft		
w_p, ft		
K (either K_a or K_s)		
y_s, ft (from Equation 2)		

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

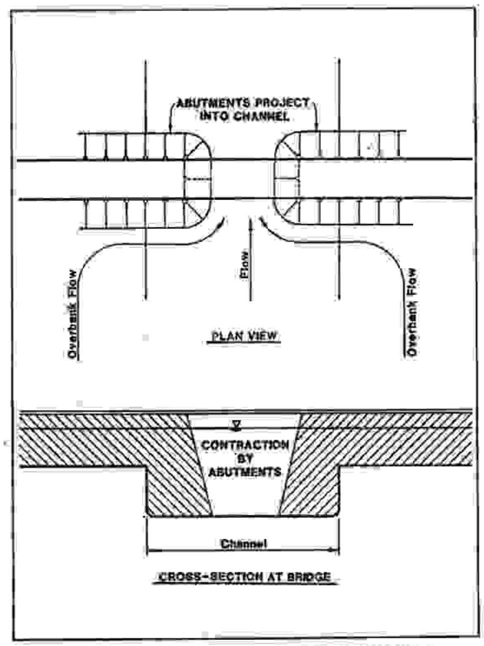
y_s, ft (pier)		
y_s, ft (contraction)		
Total scour, ft		
Normal streambed elev.		
Scour elevation		

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?

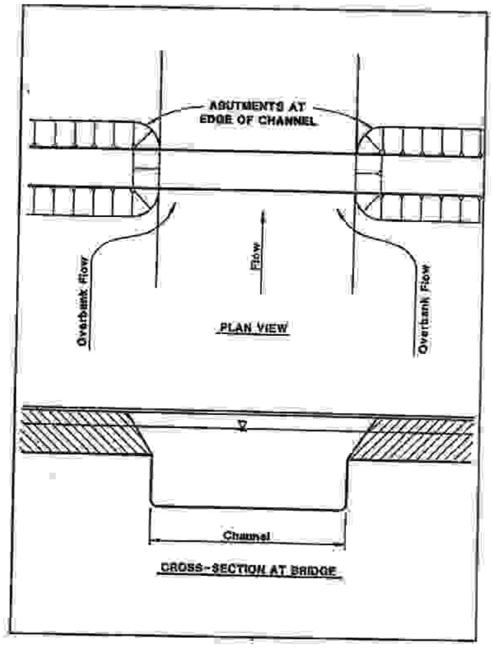
Sediment Grade Scale, from "Stream Stability at Highway Structures", Hydraulic Engineering Circular No. 20, Federal Highway Administration, Fourth Edition, April 2012.

SEDIMENT GRADE SCALE					
Size			*Approximate Sieve Mesh Openings (per inch)*		*Class*
Millimeters	Microns	Inches	Tyler	U.S. Standard	
4000-2000	---	180-160	---	---	Very Large Boulders
2000-1000	---	80-40	---	---	Large Boulders
1000-500	---	40-20	---	---	Medium Boulders
500-250	---	20-10	---	---	Small Boulders
250-130	---	10-5	---	---	Large Cobbles
130-64	---	5-2.5	---	---	Small Cobbles
64-32	---	2.5-1.3	---	---	Very Coarse Gravel
32-16	---	1.3-0.6	---	---	Coarse Gravel
16-8	---	0.6-0.3	2.5	---	Medium Gravel
8-4	---	0.3-0.16	5	5	Fine Gravel
4-2	---	0.16-0.08	9	10	Very Fine Gravel
2.00-1.00	2000-1000	---	16	18	Very Coarse Sand
1.00-0.50	1000-500	---	32	35	Coarse Sand
0.50-0.25	500-250	---	60	60	Medium Sand
0.25-0.125	250-125	---	115	120	Fine Sand
0.125-0.062	125-62	---	250	230	Very Fine sand
0.062-0.031	62-31	---			Coarse Silt
0.031-0.016	31-16	---			Medium Silt
0.016-0.008	16-8	---			Fine Silt
0.008-0.004	8-4	---			Very Fine Silt
0.004-0.0020	4-2	---			Coarse Clay
0.0020-0.0010	2-1	---			Medium Clay
0.0010-0.0005	1-0.5	---			Fine Clay
0.0005-0.0002	0.5-0.24	---			Very Fine Clay

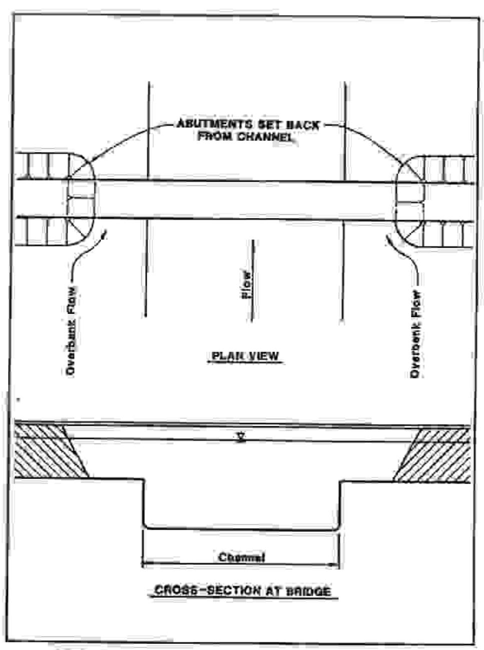
Case 1 Contraction Scour, from Appendix H, "Evaluating Scour at Bridges", Hydraulic Engineering Circular No. 18, Federal Highway Administration, Second Edition, April 1993.



Case 1A: Abutments project into channel

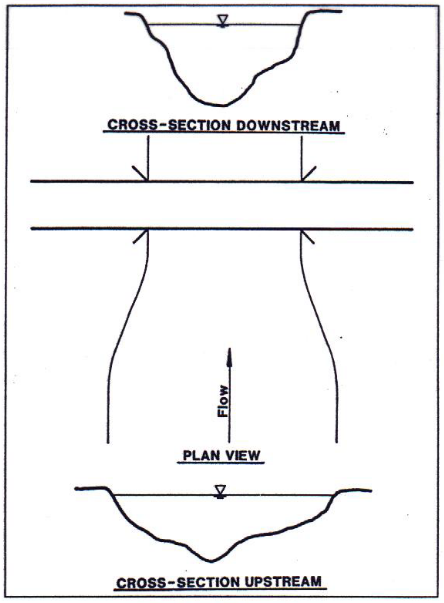


Case 1B: Abutments at edge of channel

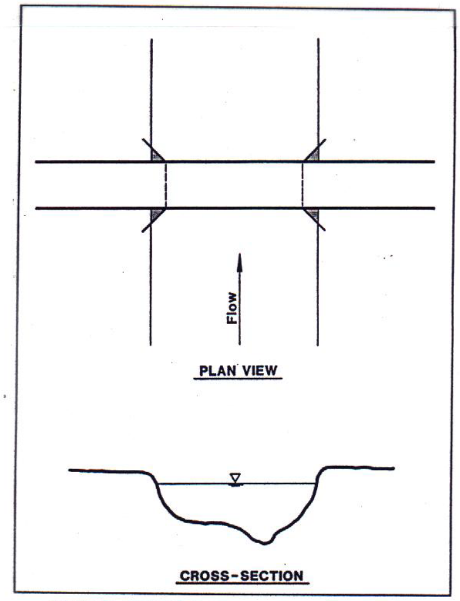


Case 1C: Abutments set back from channel

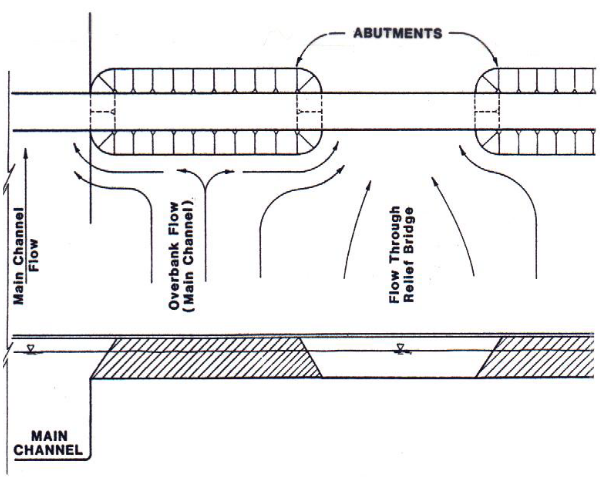
Cases 2, 3 and 4 Contraction Scour, from Appendix H, "Evaluating Scour at Bridges", Hydraulic Engineering Circular No. 18, Federal Highway Administration, Second Edition, April 1993.



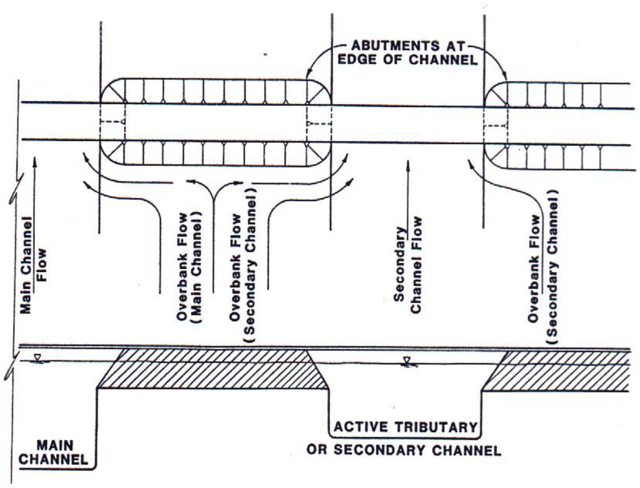
Case 2A: River narrows



Case 2B: Bridge abutments constrict flow



Case 3: Relief bridge over flood plain



Case 4: Relief bridge over secondary stream

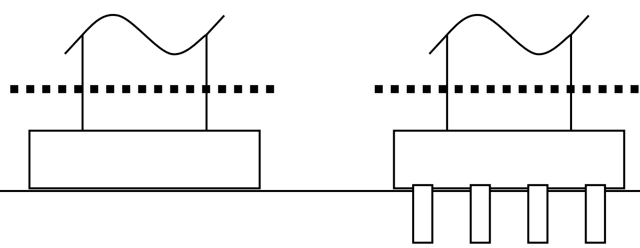
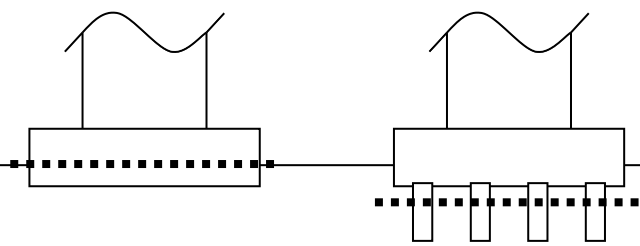
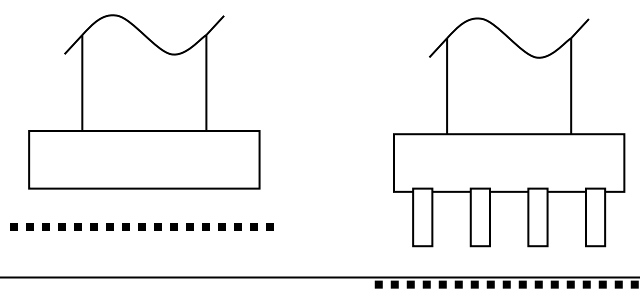
From "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges", Federal Highway Administration, December 1995.

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.

Code	Description
N	Bridge not over waterway.
U	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.
T	Bridge over "tidal" waters….
9	Bridge foundations (including piles) on dry land well above floodwater elevations.
8	Bridge foundations determined to be stable for assessed or calculated scour conditions; calculated scour is above top of footing. (Example A)
7	Countermeasures have been installed to correct a previously existing problem with scour. Bridge is no longer scour critical
6	Scour calculation/evaluation has not been made. <u>(Use only to describe cases where bridge has not yet been evaluated for scour potential.)</u>
5	Bridge foundations determined to be stable for calculated scour conditions; scour within limits of footing or piles. (Example B)
4	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.
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)

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

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

PB—C-8314

adapted
from Laursen,
Bulletin #4

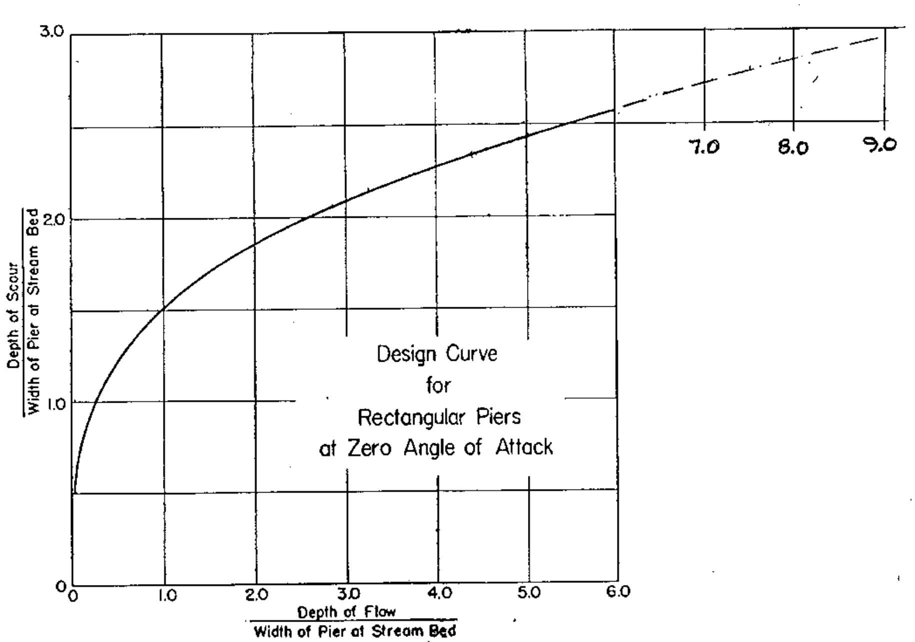


Fig. 38. Basic design curve for depth of scour.

From Laursen Bulletin #4

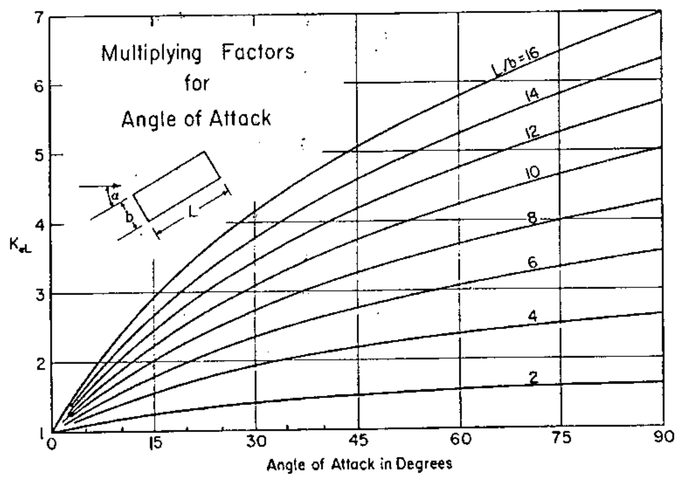


Fig. 39. Design factors for piers not aligned with flow.

TABLE V

Shape coefficients K_s for nose forms

(To be used *only* for piers aligned with flow)

Nose form	Length-width ratio	K_s
Rectangular		1.00
Semicircular		0.90
Elliptic	2:1	0.80
	3:1	0.75
Lenticular	2:1	0.80
	3:1	0.70

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 onEW-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)							
Bridge Length, feet (meters)	Bridge Skew						
	0 deg.	15 deg.		30 deg.		45 deg.	
	Equal	Long	Short	Long	Short	Long	Short
< 150 (45)	40 (12)	45 (14)	40 (12)	60 (18)	40 (12)	85 (26)	40 (12)
150-180 (45-55)	50 (16)	60 (19)	50 (16)	80 (24)	50 (16)	120 (36)	50 (16)
180-210 (55-65)	65 (20)	75 (23)	65 (20)	100 (30)	65 (20)	150 (45)	65 (20)
210-240 (65-75)	80 (24)	95 (28)	80 (24)	120 (36)	80 (24)	180 (54)	80 (24)
> 240 (75)	95 (28)	105 (32)	95 (28)	140 (42)	95 (28)	205 (63)	95 (28)

C3.2.2.7.6 Coding

C3.2.2.8 Riverine Infrastructure Database

C3.2.2.9 Datum Correlation

C3.2.2.10 Stream Stability

C3.2.2.10.1 Hydraulic Grade Line and Streambed Profile Determination

C3.2.2.10.2 Grade Control Structures

C3.2.2.11 State Water Trail and Paddling Routes

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 Unit 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 Units. Final Design Units 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-ofway).
* 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

Factors to be considered when determining whether the bridge side path will be raised or at-grade:

a) Ponding can occur at ramped transitions if approach sidewalk elevation is different from bridge sidewalk elevation. If the cross slope direction changes from on the approach to on the bridge the path must be warped and may be difficult to drain. Ponding can be especially troublesome during winter months.
b) Ramping between side path segments at different elevation must conform to accessibility requirements in accordance with ADA standards. SUDAS recommendations should also be checked for potential conflicts when ramped transitions are used.
c) Settlement, if it were to occur, could be potentially worsened by ramping and reversed cross slope, especially in terms of the effort necessary to properly repair.
d) Side path maintenance is by the local municipality, not the DOT. Typical equipment is a small mower type tractor with brush or plow blade attachment. Snow removal operation could be negatively affected by elevation tapers or ramps.
e) Imminent future conditions change can affect the design choice. Occasionally a bridge will be built in a rural area but with a raised side path due to impending development in the immediate surroundings that will eventually change the approach roadway section from rural to urban. There may be no current paved path in place, no path at all, or there may be an unpaved dirt path which is easier to ramp to the bridge trail without worry of some of the implications previously described, especially since it may not be a maintained path in its interim condition. Decisions on bridge side path elevation should be coordinated with external partners at the municipality by DOT District staff.
f) The TxDOT T411 or "Texas Classic" barrier (BDM 5.8.1.2.5), when used as a separator, is ideally employed without a raised sidewalk due to ADA noncompliance that occurs with the standard cross section. A design modification is necessary to make this barrier ADA compliant when the sidewalk is raised.
g) When the Modified B-25 Series barrier (BDM 5.8.1.2.5) is used as a side path separator, the path surface cannot be raised. This should not typically cause concern, since this barrier type is only used when paths are accommodated along high-speed (50 MPH and above) roadways which are typically rural in cross section.
h) If a side-mounted pedestrian and bicycle railing is proposed at the outer edge of deck along a side path, a raised sidewalk can provide the additional concrete section that is desirable for mounting such railings. It may also be desirable for the outermost segment of the side path to consist of a full-depth segment for the railing anchorage, in which case the raised side path slab would overlap the limit of the initial deck pour. An example of this detail can be provided by the Methods Engineer upon request.

Bridges that are stage-constructed will sometimes have a side path that is used for traffic during an early stage. Such side paths, if they will be raised in the ultimate condition, will not have continuity reinforcing integrated into deck construction. This reinforcing must be drilled into the deck concrete once the path surface is no longer used for staged traffic. The deck surface at the path must also be finished smooth to provide a proper riding surface during staged traffic operations, so this concrete must be mechanically roughened prior to raised side path slab placement. The preliminary designer shall ensure that adequate drainage is provided for each stage on the bridge.

On superelevated bridges with a rural or urban approach condition and an at-grade side path along the high side of the bridge, the path may be allowed to drain into the roadway through openings in the separation barrier. Cross slope must be sufficient to ensure that no accumulations of water in the roadway will enter the side path through the separator. On superelevated bridges with a side path along the low side, roadway and path drainage must be handled separately, potentially with drains on each side of the separation barrier. Drains within side paths must be trench-type drains with cover grates that are ADA compliant, i.e. with openings oriented transverse to the path and no wider than 0.5 inch. Drain grates must be set flush or below the path surface within 0.25 inch, with concrete tapered to the grate at a 2:1 maximum slope.

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.

$S := 111.5 \cdot ft$ Longest Span (feet)

$e := 0.03$ Superelevation (feet/feet)

$G_1 := -1.68$ Grade 1 vertical curve [+ increasing, - decreasing] (%)

$G_2 := 2.10$ Grade 2 vertical curve [+ increasing, - decreasing] (%)

$A := \frac{G_2 - G_1}{100}$ $A = 0.038$

$L := 984 \cdot ft$ Length vertical curve (feet)

$D_c := 1.75deg$ Degree of Horizontal Curvature (degree)

$C := 0.337 \cdot ft$ Final Beam Camber (feet) - From prestressed concrete beam standards

$D := 0.19 \cdot ft$ Dead load deflection - Elastic + 1/2 Plastic (feet) - From prestressed concrete beam standards

$T := 1.667 \cdot ft$ Top flange width (feet)

X = Haunch estimate along the centerline of the beam.

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

$X = 0.219 \cdot ft$ $X = 66.894 \cdot mm$

$T \cdot e = 0.6 \cdot in$

If T * e < 1 then X < 4 in. If T * e > 1 then X < 3 in.

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

C3.6.1.7 CWPG

The table below based on information 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>
<tr><td colspan="2">Superstructure</td><td colspan="2">Minimum Depth (Including Deck)

When variable depth members are used, values may be adjusted to account for changes in relative stiffness of positive and negative moment sections.</td></tr>
<tr><td>Material</td><td>Type</td><td>Simple Spans</td><td>Continuous Spans</td></tr>
<tr><td rowspan="3">Steel</td><td>Overall Depth of Composite I-Beam</td><td>0.040L</td><td>0.032L</td></tr>
<tr><td>Depth of I-Beam Portion of Composite I-Beam</td><td>0.033L</td><td>0.027L</td></tr>
<tr><td>Trusses</td><td>0.100L</td><td>0.100L</td></tr>
</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

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.

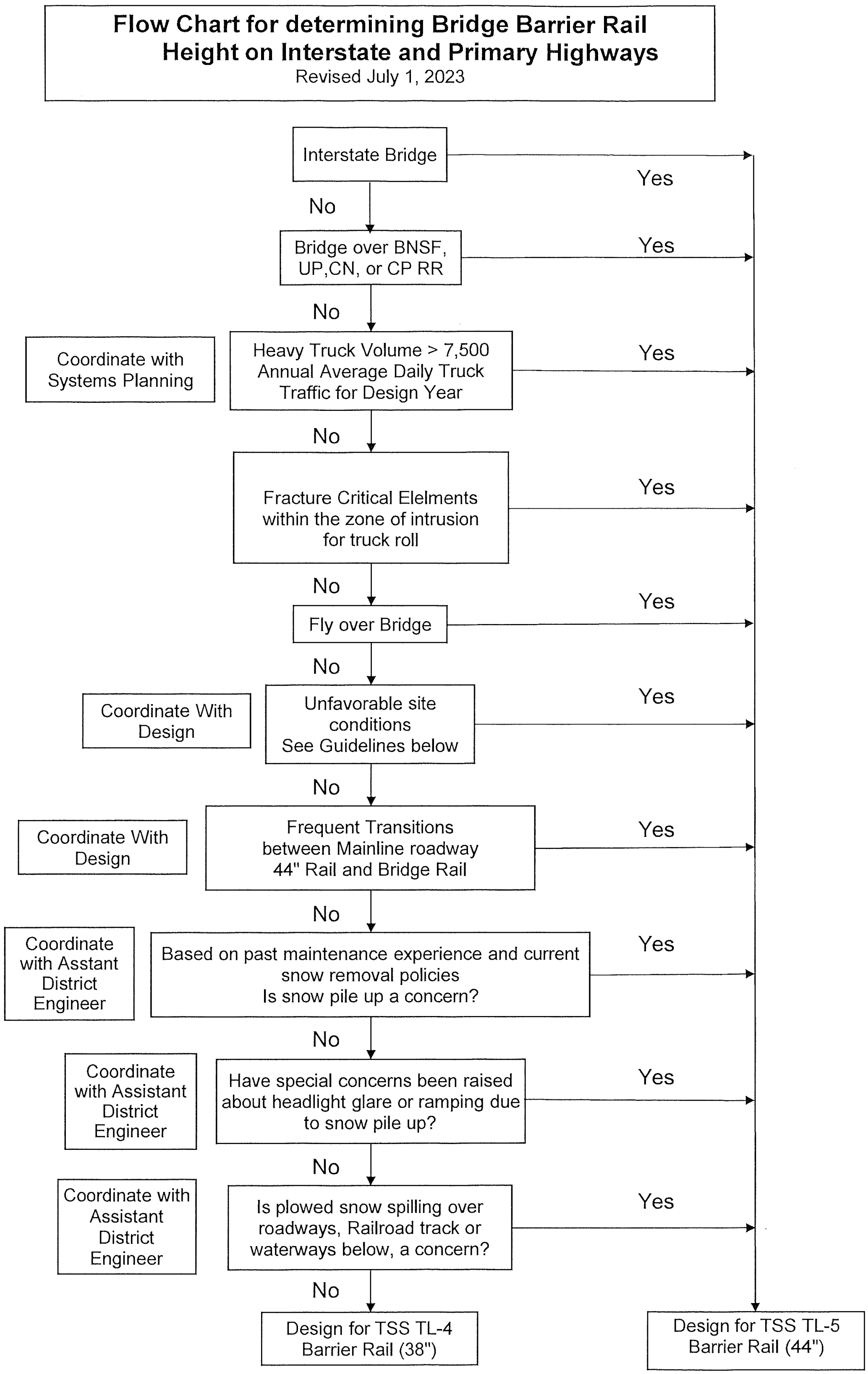
For situations where the profile grade line is at the centerline of approach roadway, elevations for the bridge deck will be established in accordance with BDM 1.7.1.

C3.6.5 Cross slope drainage

C3.6.6 Deck drainage

C3.6.7 Bridge inspection/maintenance accessibility

C3.6.8 Railings


Flow Chart for determining Bridge Barrier Rail Height on Interstate and Primary Highways
Revised July 1, 2023
Interstate Bridge
Yes
No
Bridge over BNSF, UP,CN, or CP RR
Yes
No
Coordinate with Systems Planning
Heavy Truck Volume > 7,500 Annual Average Daily Truck Traffic for Design Year
Yes
No
Fracture Critical Elelments within the zone of intrusion for truck roll
Yes
No
Fly over Bridge
Yes
No
Coordinate With Design
Unfavorable site conditions See Guidelines below
Yes
No
Coordinate With Design
Frequent Transitions between Mainline roadway 44" Rail and Bridge Rail
Yes
No
Coordinate with Assistant District Engineer
Based on past maintenance experience and current snow removal policies Is snow pile up a concern?
Yes
No
Coordinate with Assistant District Engineer
Have special concerns been raised about headlight glare or ramping due to snow pile up?
Yes
No
Coordinate with Assistant District Engineer
Is plowed snow spilling over roadways, Railroad track or waterways below, a concern?
Yes
No
Design for TSS TL-4 Barrier Rail (38")
Design for TSS 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

C3.6.8.1 Barrier Rail End Treatments

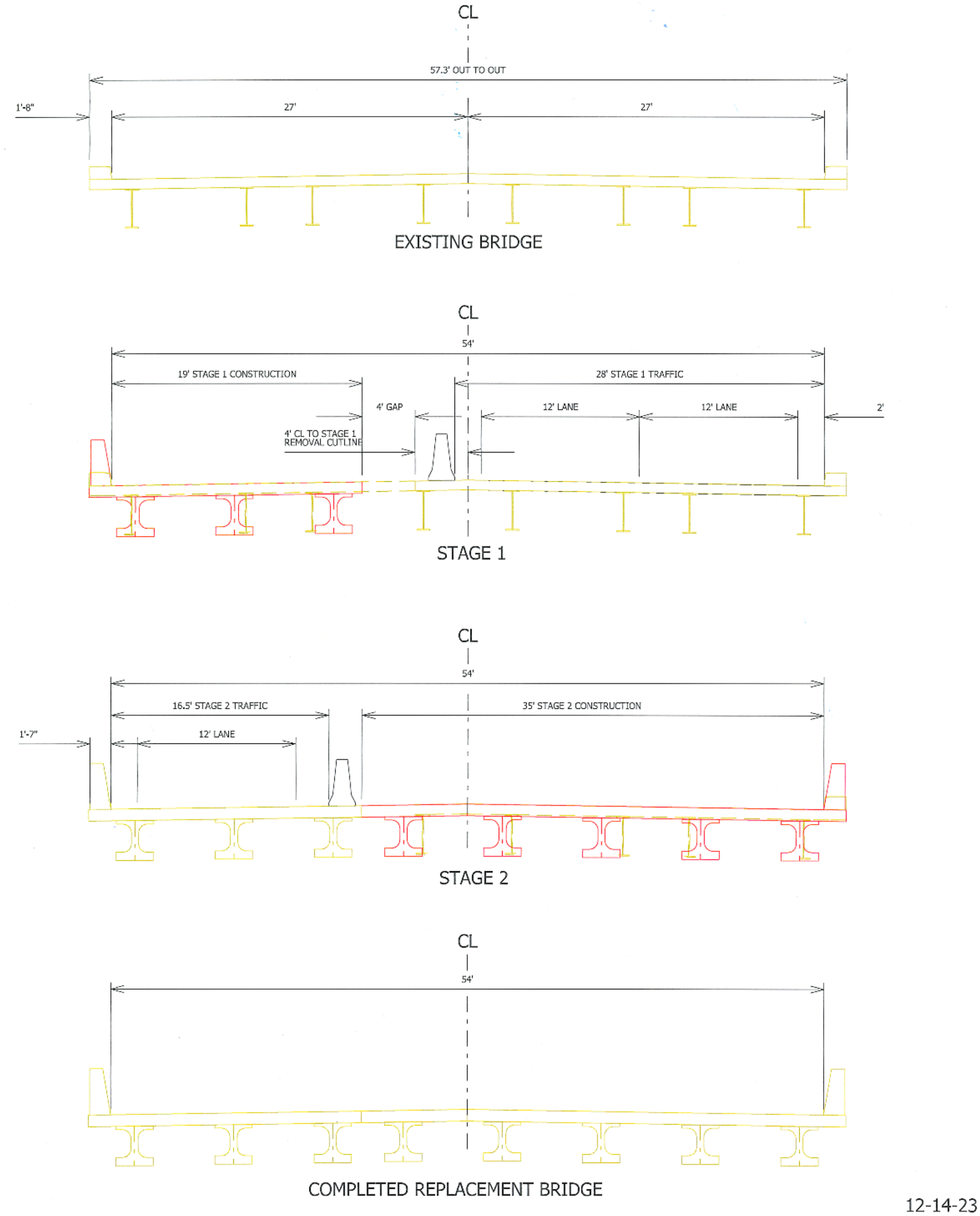
C3.6.8.2 Separation Rail

C3.6.9 Staging

Two example staging sketches are provided below.

EXAMPLE - PROPOSED CONCEPT STAGING SKETCH

NOTE: ANCHOR TBR TO THE DECK IF OFFSET TO DROPOFF IS LESS THAN 45 INCHES.



Complex Staging Example:

C3.7 Substructures

C3.7.1 Skew

C3.7.2 Abutments

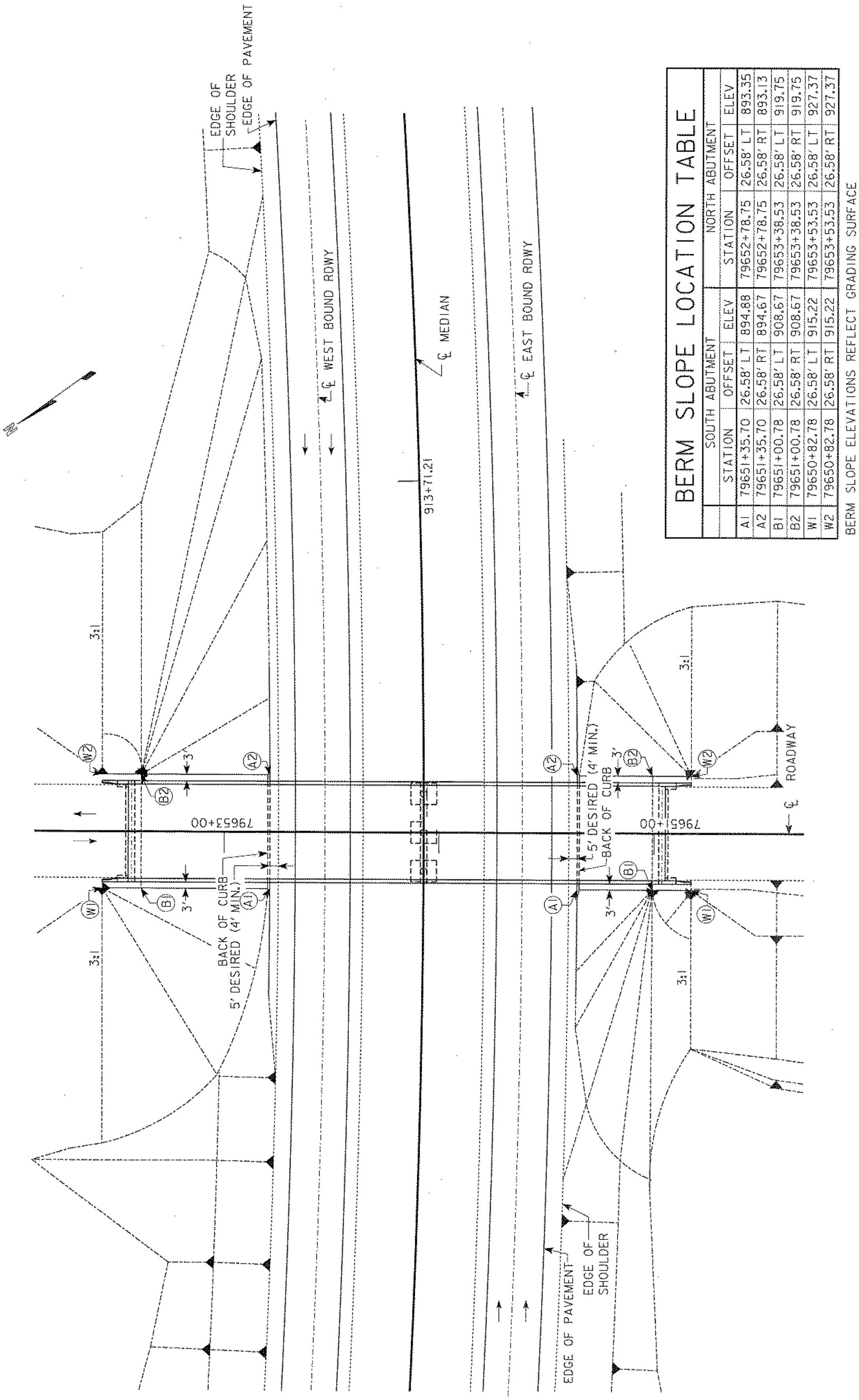
C3.7.3 Berms

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.



BERM SLOPE LOCATION TABLE

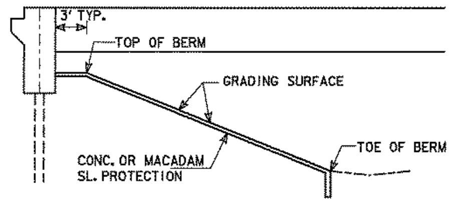
	SOUTH ABUTMENT			NORTH ABUTMENT		
	STATION	OFFSET	ELEV	STATION	OFFSET	ELEV
A1	79651+35.70	26.58' LT	894.88	79652+78.75	26.58' LT	893.35
A2	79651+35.70	26.58' RT	894.67	79652+78.75	26.58' RT	893.13
B1	79651+00.78	26.58' LT	908.67	79653+38.53	26.58' LT	919.75
B2	79651+00.78	26.58' RT	908.67	79653+38.53	26.58' RT	919.75
W1	79650+82.78	26.58' LT	915.22	79653+53.53	26.58' LT	927.37
W2	79650+82.78	26.58' RT	915.22	79653+53.53	26.58' RT	927.37

BERM SLOPE ELEVATIONS REFLECT GRADING SURFACE

REVISED JULY 2013

EXAMPLE BERM SLOPE LOCATION TABLE (BSLT)

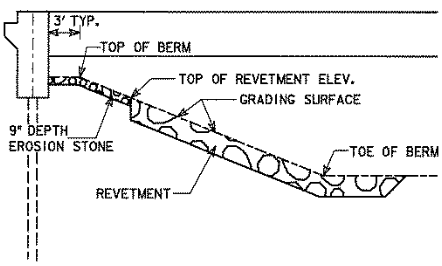
SLOPE PROTECTION LOCATION FOR BSLT GRADING SURFACES



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 QUANITIES WILL BE CALCULATED IN FINAL DESIGN.

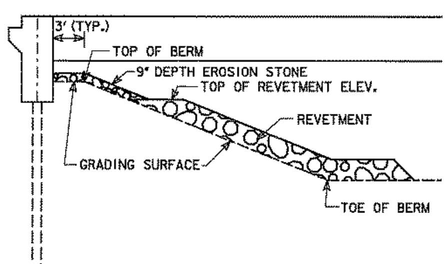
CONCETE OR MACADAM SLOPE PROTECTION



NOTES:

1. BSLT POINTS GIVEN AT GRADING SURFACE = TOP OF EROSION STONE AND TOP OF EMBEDDED REVETMENT.
2. THE GRADING SURFACE SHALL BE LABELED ON THE TSL 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. REVETMENT AND EROSION STONE BERM ARMORING ARE PLACED BELOW THE GRADING SURFACE AND WILL REQUIRE "CORE OUT". DEFINE LIMITS OF THE CORE OUT IN THE PLANS. THE BERM ARMORING QUANTITIES TABLE SHALL INCLUDE (AS APPLICABLE) CLASS 10 EXCAVATION, ENGINEERING FABRIC, EROSION STONE AND REVETMENT. BERM ARMORING GENERALLY INCLUDES QUANTITIES TO THE FACE OF THE ABUTMENT.
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

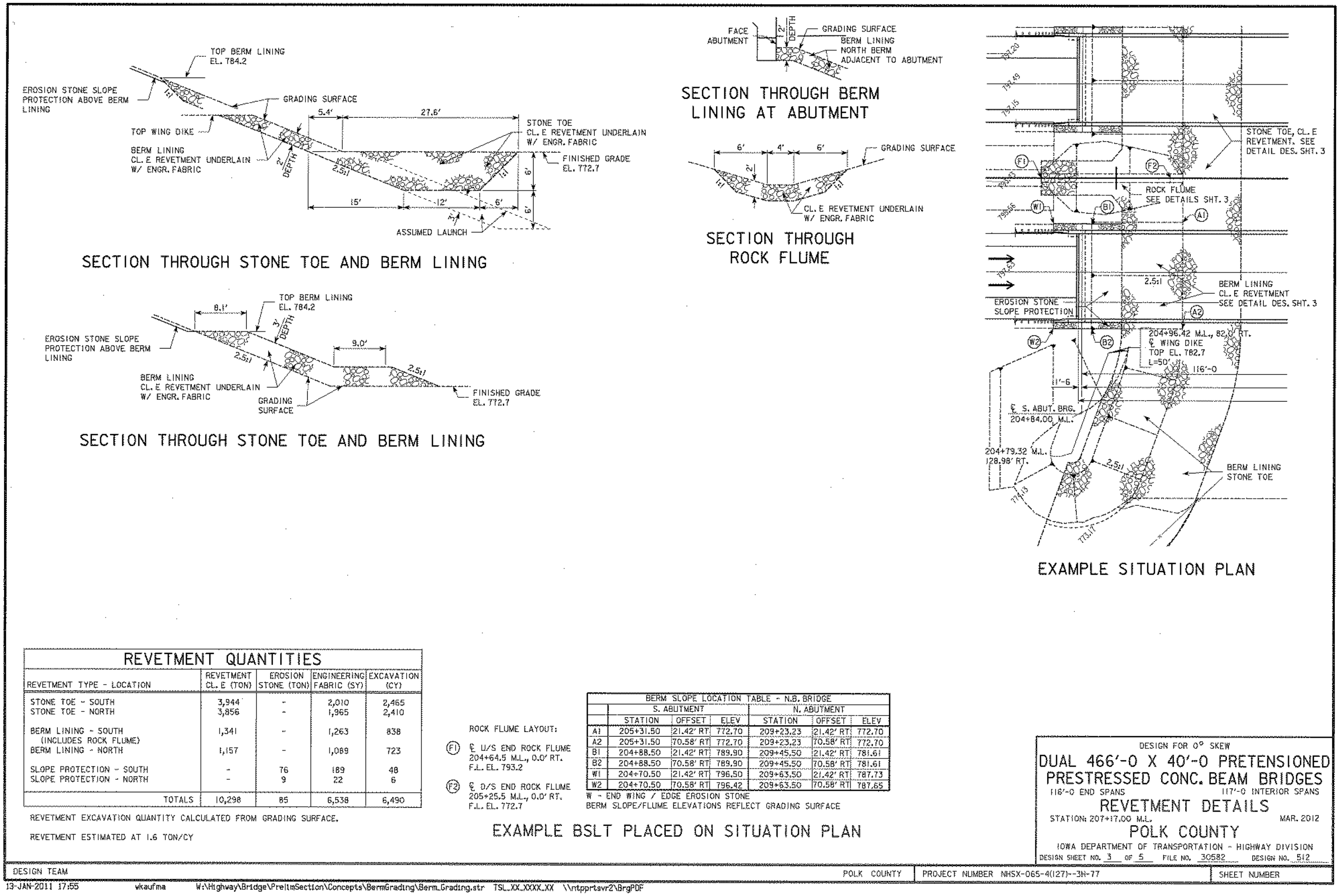


NOTES:

1. BSLT POINTS GIVEN AT GRADING SURFACE = BASE OF EROSION STONE AND BASE OF NON-EMBEDDED REVETMENT.
2. THE GRADING SURFACE SHALL BE LABELED ON THE TSL 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)

4-24-12



REVETMENT QUANTITIES

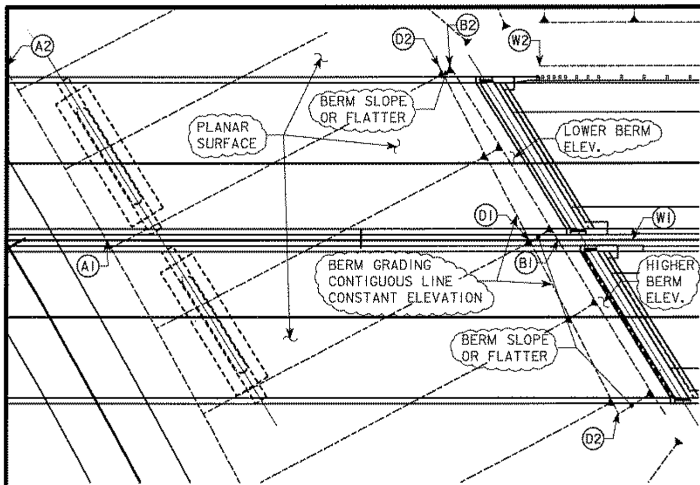
REVETMENT TYPE - LOCATION	REVETMENT CL. E (TON)	EROSION STONE (TON)	ENGINEERING FABRIC (SY)	EXCAVATION (CY)
STONE TOE - SOUTH	3,944	-	2,010	2,465
STONE TOE - NORTH	3,856	-	1,965	2,410
BERM LINING - SOUTH (INCLUDES ROCK FLUME)	1,341	-	1,263	838
BERM LINING - NORTH	1,157	-	1,089	723
SLOPE PROTECTION - SOUTH	-	76	189	48
SLOPE PROTECTION - NORTH	-	9	22	6
TOTALS	10,298	85	6,538	6,490

REVETMENT EXCAVATION QUANTITY CALCULATED FROM GRADING SURFACE.

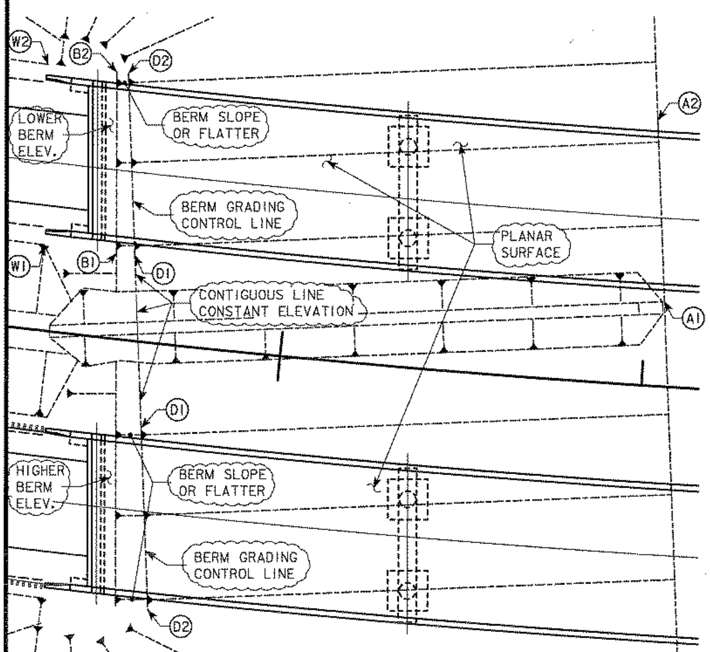
REVETMENT ESTIMATED AT 1.6 TON/CY

BERM SLOPE LOCATION TABLE - N.B. BRIDGE

	S. ABUTMENT			N. ABUTMENT		
	STATION	OFFSET	ELEV	STATION	OFFSET	ELEV
A1	205+31.50	21.42' RT	772.70	209+23.23	21.42' RT	772.70
A2	205+31.50	70.58' RT	772.70	209+23.23	70.58' RT	772.70
B1	204+88.50	21.42' RT	789.90	209+45.50	21.42' RT	781.61
B2	204+88.50	70.58' RT	789.90	209+45.50	70.58' RT	781.61
W1	204+70.50	21.42' RT	796.50	209+63.50	21.42' RT	787.73
W2	204+70.50	70.58' RT	796.42	209+63.50	70.58' RT	787.65



DUAL BRIDGES - MINIMAL SEPARATION



DUAL BRIDGES - TYPICAL SEPARATION

NOTES:

FOR DUAL BRIDGES A BERM GRADING CONTROL LINE WILL BE PROVIDED.

THE BERM GRADING CONTROL LINE IS A CONTIGUOUS 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 CONTIGUOUS LINE OUT-OUT THE 'B' 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 BTM. FTG.):

INTEGRAL - 0.5 FT.
STUB - 0.75 FT.

THE PROVISIONS OF ARTICLE 3.2.7.2 (SLOPING OF ABUT. FOOTING/BERM) SHOULD BE REVIEWED FOR APPLICABILITY. THE PROVISIONS OF THE ABOVE ARTICLE SHALL GOVERN.

FOR SITES WHERE THE 'B' POINTS CANNOT BE ADJUSTED TO FORM A CONTIGUOUS LINE AT A CONSTANT ELEVATION, 'D' 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 'B' AND 'D' POINTS MATCHES OR IS FLATTER THAN THE BERM SLOPE BELOW THE GRADING CONTROL LINE.

BERM SLOPE LOCATION TABLE

POINTS	WEST ABUTMENT			EAST ABUTMENT		
	STATION	OFFSET	ELEV.	STATION	OFFSET	ELEV.
A1	891+04.80	23.40' LT	1200.80	895+34.10	23.40' LT	1200.80
A2	890+99.60	72.50' LT	1200.80	895+39.50	72.50' LT	1200.80
D1	889+57.40	23.40' LT	1249.28	896+70.50	23.40' LT	1245.70
D2	889+50.00	72.50' LT	1249.28	896+78.00	72.50' LT	1245.70
B1	889+52.25	23.40' LT	1250.28	896+76.00	23.40' LT	1246.70
B2	889+46.67	72.50' LT	1250.28	896+81.58	72.50' LT	1246.70
W1	889+32.20	23.40' LT	1257.74	896+96.05	23.40' LT	1254.17
W2	889+27.25	72.50' LT	1257.84	897+01.00	72.50' LT	1254.27

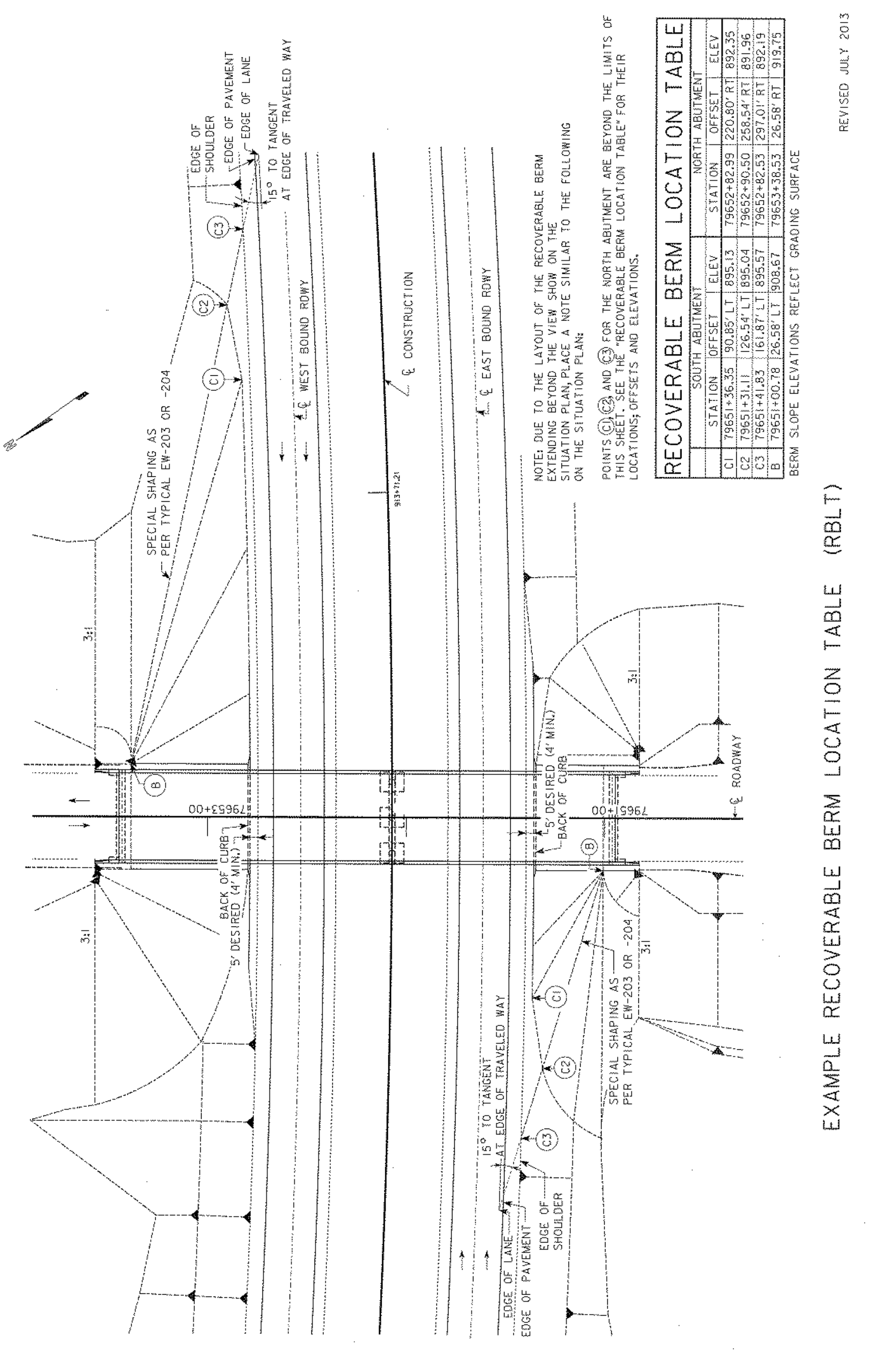
BERM SLOPE ELEVATIONS REFLECT THE GRADING SURFACE. BERM GRADING BELOW BERM GRADING CONTROL LINE DEFINED BY CONTROL LINE.
BERM GRADING CONTROL LINE IS DEFINED BY 'D' POINTS IN ABOVE TABLE.
[ALTERNATE NOTE FOR ABOVE WHEN 'D' POINTS NOT REQUIRED - SEE NOTES]
BERM GRADING CONTROL LINE IS DEFINED BY 'B' POINTS IN ABOVE TABLE.

DUAL BRIDGES
BERM SLOPE DEFINITION

REV. DATE: 5-01-13

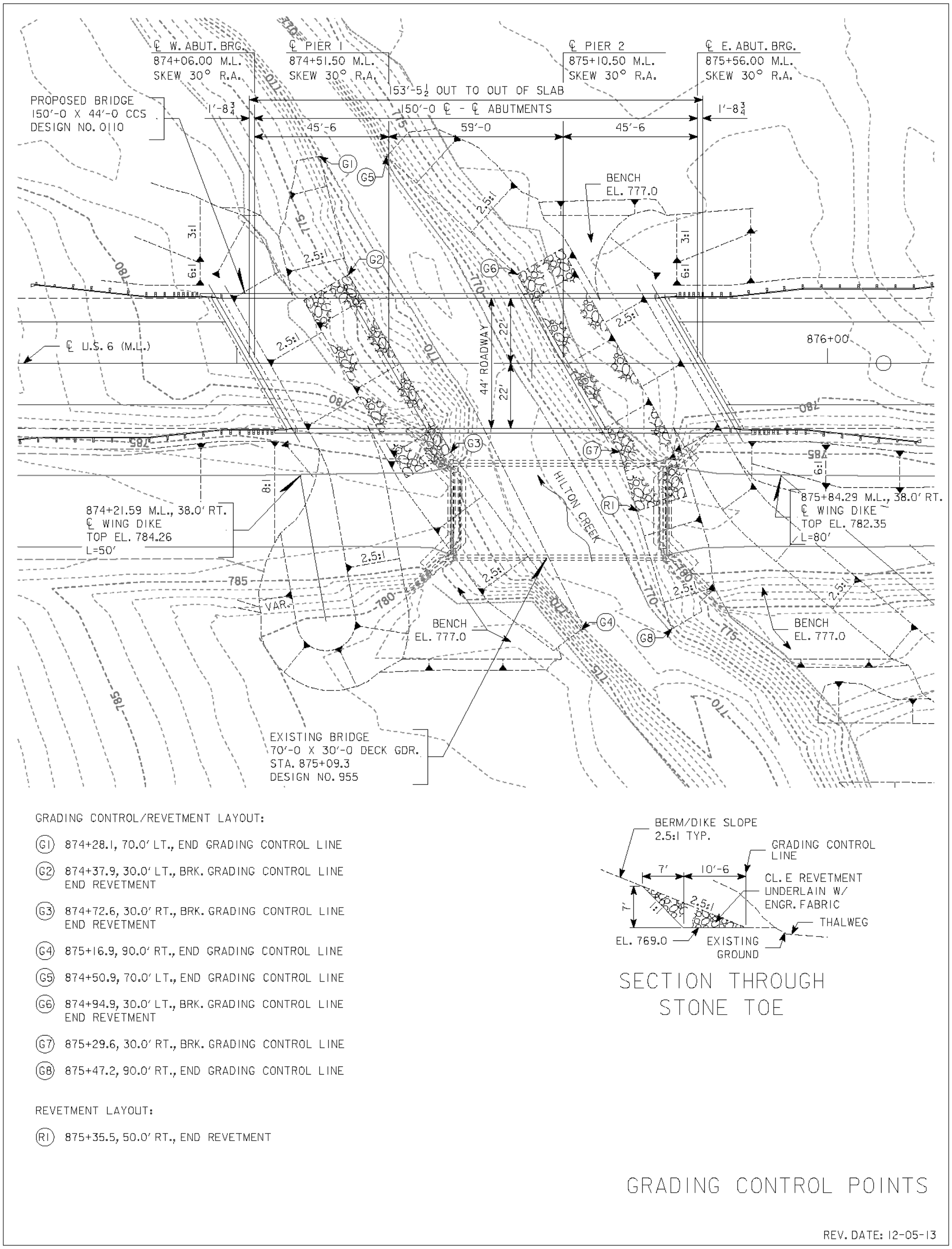
C3.7.3.4 Recoverable berm location table

See also the BSLT example in C3.2.7.3.3.



C3.7.3.5 Slope protection

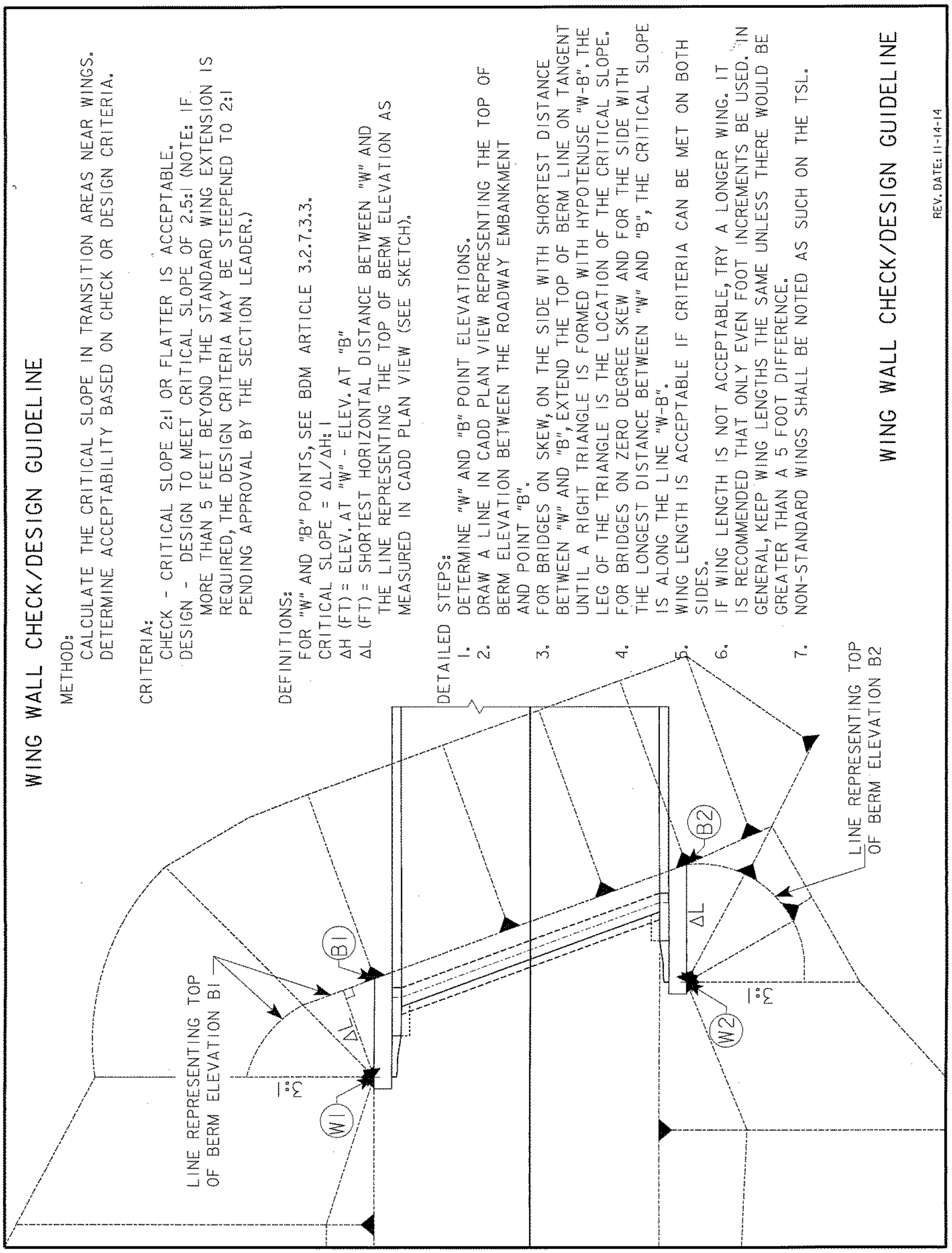
C3.7.3.6 Grading control points



C3.7.3.7 Mechanically Stabilized Earth (MSE) Walls adjacent to abutments

C3.7.4 Piers and pier footings

C3.7.5 Wing walls



C3.7.6 Foundation Conflicts

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 Bar) 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).

<u>Mandatory Items for the Situation Plan layout sheet(s)</u>

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)
    - Revetment (A)
    - Slope Protection Note (A)

- Guardrail Indicated
- Topography (A)
- Minimum Vertical Clearance Location (overhead bridges)
- Scale Bar
- Horizontal Clearance to Piers (overhead bridges)
- Existing Contours, supplemented with ground shots (A)
- Proposed Contours (A) (may supplement BSB terrain with proposed grading slope lines if desired to provide clarity of proposed berm grading)

2. Longitudinal Section
3. Typical Approach Section
4. Location Data (for consistency, place above the title block)
5. Survey Control Point

(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.

<u>Preferred Items for the Situation Plan layout sheet(s) (In order of preference)</u>

1. Proposed Grade
2. Hydraulic Data
3. Traffic Estimate
4. General Utilities Cell and Notes
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.

<u>Mandatory Items for the Site Plan layout sheet</u>

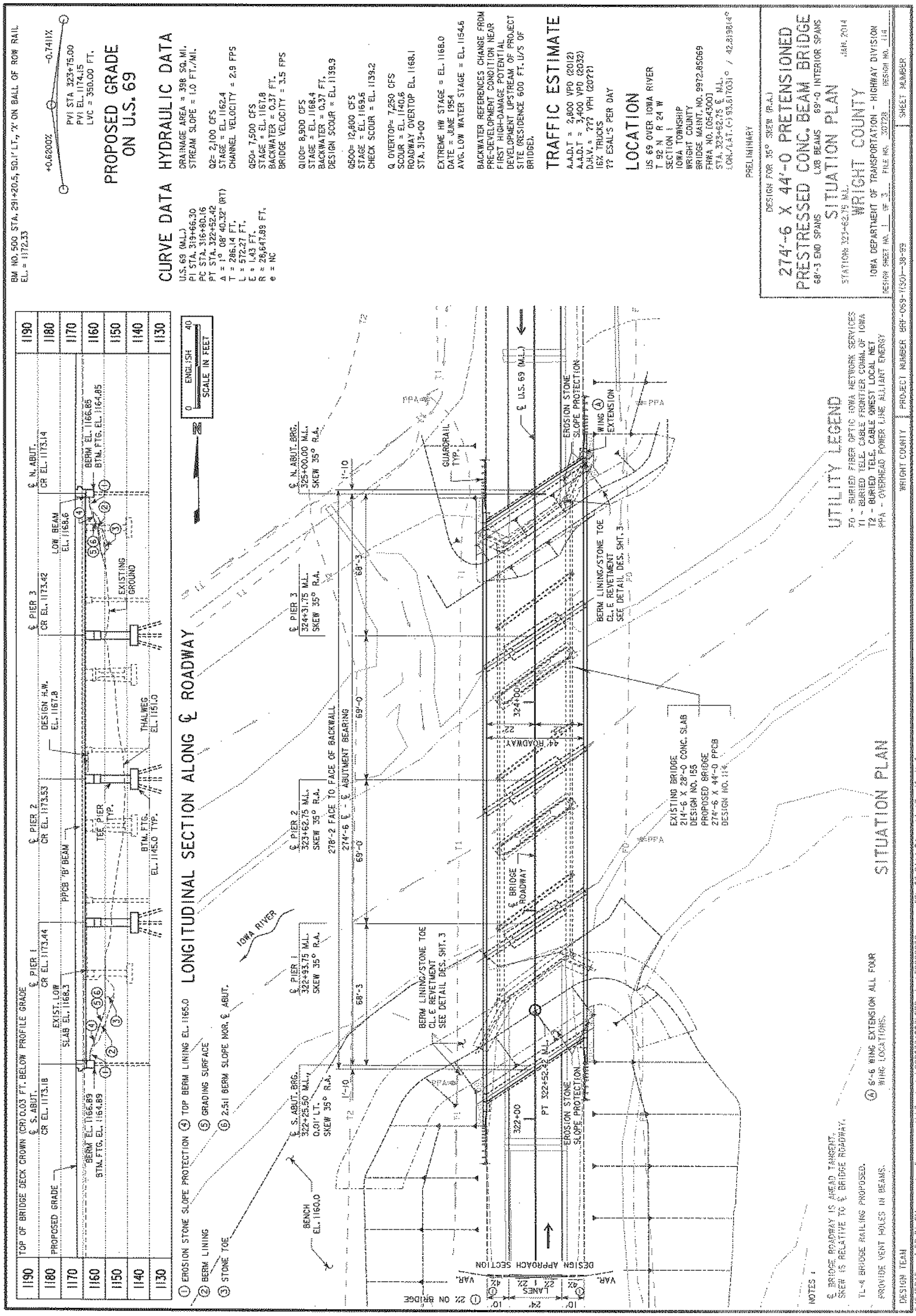
1. Site Plan
    - SITE PLAN heading under plan view layout
    - North Arrow
    - Centerline Roadway Alignments and labels
    - Centerline Stationing labels
    - Proposed Structure(s) (B)
    - Existing Structure(s) (B)
    - Existing Contours (B), supplemented with ground shots
    - Proposed Contours (B)
    - Revetment (B)
    - Guardrail Indicated
    - Topography (B)
    - Scale Bar
    - Beginning & End Bridge Stations at Centerline Abutment Bearings

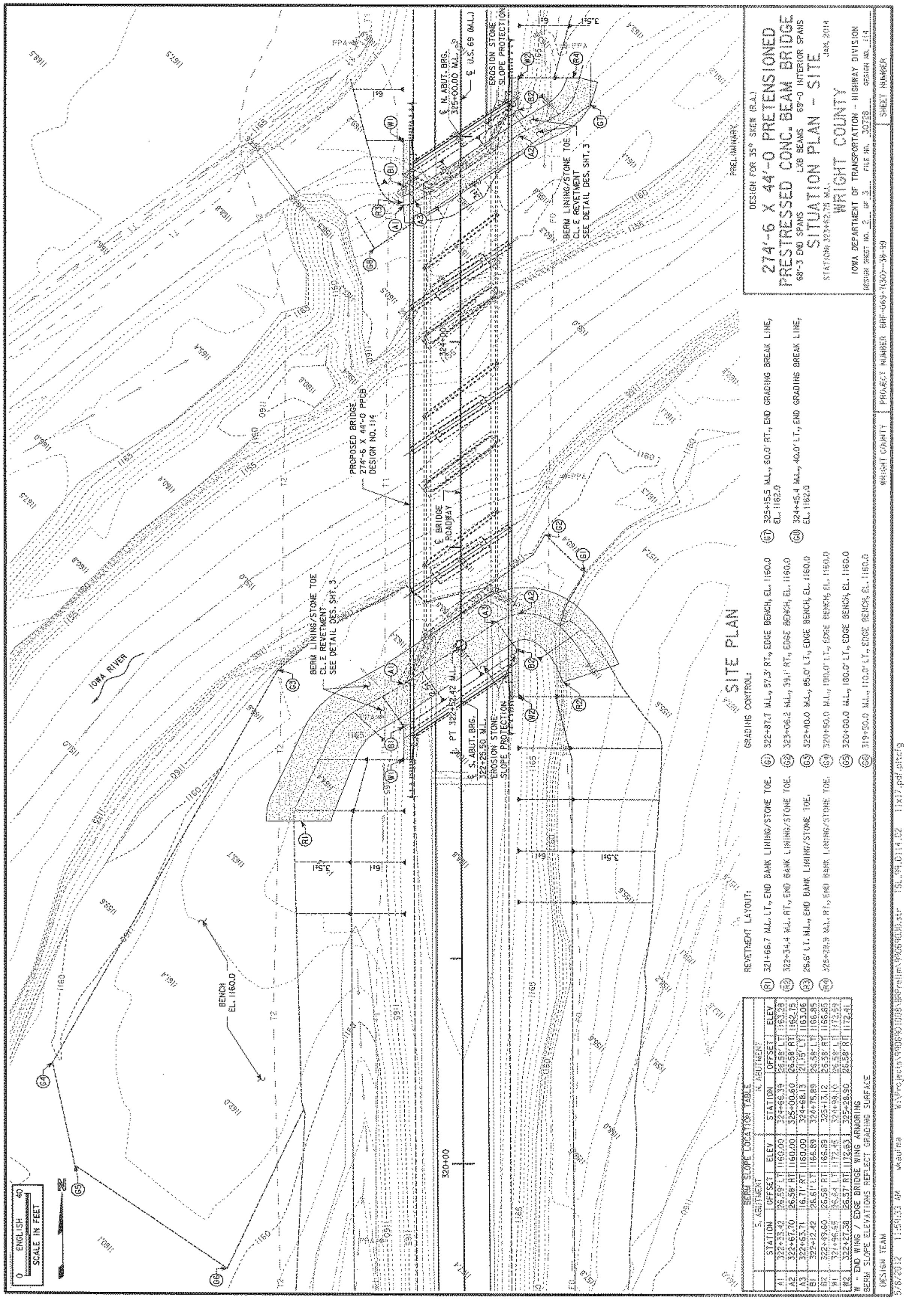
(B) These items should not be edited extensively on the site plan layout sheet and a more comprehensive treatment of these items should be shown on this sheet where extensive editing may have been necessary on the situation plan layout sheet(s).

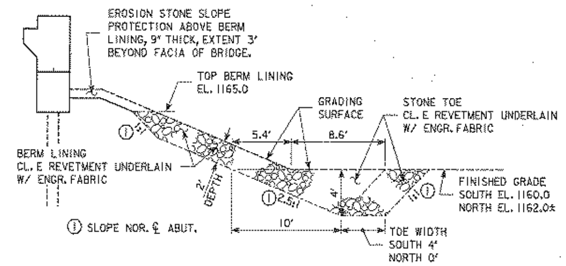
The preferred items listed are expected to be shown on the site plan layout sheet but due to space restrictions may be shown on the situation plan layout sheet(s).

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


LONGITUDINAL SECTION ALONG ℄ ROADWAY
SITUATION PLAN
PROPOSED GRADE ON U.S. 69
CURVE DATA
HYDRAULIC DATA
TRAFFIC ESTIMATE
LOCATION
UTILITY LEGEND
274'-6 X 44'-0 PRETENSIONED PRESTRESSED CONC. BEAM BRIDGE
SITUATION PLAN
WRIGHT COUNTY
IOWA DEPARTMENT OF TRANSPORTATION - HIGHWAY DIVISION


SITE PLAN
IOWA RIVER
PROPOSED BRIDGE
274'-6 X 44'-0 PPCB
DESIGN NO. 114
274'-6 X 44'-0 PRETENSIONED
PRESTRESSED CONC. BEAM BRIDGE
SITUATION PLAN - SITE
WRIGHT COUNTY
IOWA DEPARTMENT OF TRANSPORTATION - HIGHWAY DIVISION
PRELIMINARY
SCALE IN FEET



SECTION THROUGH STONE TOE AND BERM LINING

REVETMENT QUANTITIES

REVETMENT TYPE - LOCATION	REVETMENT CL. E (TON)	EROSION STONE (TON)	ENGINEERING FABRIC (SY)	EXCAVATION (CY)
BERM LINING\STONE TOE - SOUTH	837	-	781	523
BERM LINING\STONE TOE - NORTH	234	-	269	146
SLOPE PROTECTION - SOUTH	-	24.5	61	15.3
SLOPE PROTECTION - NORTH	-	24.2	60	15.1
TOTALS	1,071	48.7	1,171	699

REVETMENT EXCAVATION QUANTITY CALCULATED FROM GRADING SURFACE.
REVETMENT AND EROSION STONE ESTIMATED AT 1.6 TON/CY

PRELIMINARY

DESIGN FOR 35° SKEW (R.A.)

274'-6 X 44'-0 PRETENSIONED PRESTRESSED CONC. BEAM BRIDGE

68'-3 END SPANS LXB BEAMS 69'-0 INTERIOR SPANS

SITUATION PLAN - MISC.

STATION: 323+62.75 M.L. JAN. 2014

WRIGHT COUNTY

IOWA DEPARTMENT OF TRANSPORTATION - HIGHWAY DIVISION

DESIGN SHEET NO. 3 OF 3 FILE NO. 30728 DESIGN NO. 114

DESIGN TEAM | WRIGHT COUNTY | PROJECT NUMBER BRF-069-7(30)--38-99 | SHEET NUMBER

5/8/2012 11:34:48 AM wkaufma W:\Projects\9906901008\BRPrelim\99069030.str TSL_99_0114_03 11x17_pdf.pltcfg

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.

9. Nishnabotna River. From Missouri River to north line T67N, R42W, Fremont County, northeast of Hamburg.

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	Proj. No.: BRF-014-2(34)--38-59
Des. No.: 1054	Pin No.: 09-59-014-010
Maint. No.: 5927.3S014	FHWA No.: 34460
On IA 14 over English Creek	Sta.: 502+19.1
Section 13,T73N,R21W	
Functional Class:	ADT: 2580 vpd
By: D. Claman	Date: 5/17/2010

Existing Bridge:

Type: I-Beam	Length x Width: 60' x 30'
Pier Type: N/A	Abut. Type: Stub
Spans: 60	Approach Pavement Width: 30
Skew: 0	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 false-work.

Option A - Stage 110' x 46' CCS Bridge

Type: CCS	Length x Width: 110' x 46'
Pier Type: Pile Bent	Abutment Type: Integral
Spans: 1 @ 35', 2@27.5'	Skew: 0.0

Stage Traffic: Yes, One 15' Lane - Stage 1, One 12' Lane - Stage 2
Costs:

Bridge - 110' x 46' @ $75/sf	= $ 379,500
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.

Page 1 of 2

Bridge Concept Statement **4/12/2011**

Lucas County
BRF-014-2(34)—38-59

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

Type: CCS	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 (10%)	= $ 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

Bridges and Structures Bureau Attachment for Concept Statement

Date: August 1, 2023
By: John Q. Engineer
Location: U.S. 65 over East Branch Beaverdam Creek

County: Cerro Gordo
Project No.: BRFN-065-8(68)—39-17
Pin No.: 17-17-065-010

1. Regulatory/Coordination
    a. Iowa DNR Flood Plain permit = No
    b. Iowa DNR Sovereign Lands permit = No
    c. Local Record of Coordination = Yes
    d. Flood Insurance Study = Yes. Zone A Panel 19033C0275C, May 16, 2012
    e. Drainage District = No (March 2012 D.D. Map prepared by Cerro Gordo County Auditor’s Office)
    f. Corps of Engineers Section 408 = No
    g. Iowa State Water Trail or Paddling Route = No
    h. Historic Structure = No
    i. Federally owned land in vicinity = No
    j. USGS or Iowa Flood Center (IFC) gage or sensor impacted = No

2. Hydrologic/Hydraulic Analysis/RIDB Dataset
    a. Design discharges determined = Yes (USGS 13-5086)
    b. Hydraulic analysis done = No (2D model recommended)
    c. Riverine Infrastructure Database (RIDB) = Yes, an RIDB dataset will be developed as part of this project. The RIDB network location is BeaverdamC_EB_Cer_9.9.
    d. Project development hydraulic analysis will comply with the RIDB Guidelines at a minimum.

3. Structure/Roadway Layout Considerations
    a. A grade raise of 0.3-0.6’ will keep low beam at the same level as existing. Recommend the maximum possible roadway profile grade raise that can be obtained within the approach roadway.
    b. A slight channel shift is considered to center the channel within the bridge.

4. Special construction issues
    a. Shallow bedrock may require consideration of wall piers with spread footing on rock in lieu of pile bent piers.
    b. It is desirable for new structure foundations to avoid existing foundations when possible.

5. Special survey = Yes. See below.

6. Aesthetic enhancements = No.

7. Other
    a. The roadway will be closed during construction with traffic placed on an off-site detour.

b. Use of wing dikes on the north side was reviewed and not carried forward due to ground geometry upstream of the bridge.

Special Survey:

We request the following in addition to the routine survey data-

A. Lowest ground and floor elevations for the 3 agricultural structures located on the north side of 170th Street and west of U.S. 65 (upstream of the project). A description of the contents within the buildings is also requested to determine level of damage potential.
   Link to KMZ =
   http://dotnet/pw:/projectwise.dot.int.lan:PWMain/Documents/Projects/1706501017/BRPrelim/DOT/Support/Survey 3 Ag Buildings Upstream of U.S. 65 MP86.3 Bridge Replacement.kmz
   Survey 3 Ag Buildings Upstream of U.S. 65 MP86.3 Bridge Replacement.kmz

B. Survey of the quad culvert downstream of the bridge on Pheasant Ave. (For each barrel:
   a. rise and span
   b. structure headwall inlet and outlet flowlines
   c. obvert
   d. if silted record silted thalweg in addition to structure flowline.
   e. Observation top of parapet at facia.

   Link to KMZ =
   Survey County Quad RCB.kmz

C. Roadway centerline profile on U.S. 65 between B55 (170th Street) and the project location capturing the low roadway overtopping elevation at the low point.

D. Roadway centerline profile on B55 (170th Street) between the 3 Agricultural buildings and proceeding to the intersection with U.S. 65.

E. For the purpose of determining any needed LiDAR bias correction to the project datum, follow RIDB data guidelines, Part 6B.3). The recommended procedure includes collection of XYZ observations for 20+ points divided between at least 2 discrete locations.

F. Project development data collection will comply with the RIDB Guidelines at a minimum.

Form 621004wd
06-95

Iowa Department of Transportation

EXAMPLE

FIELD NOTES FOR BRIDGES AND LARGE CULVERTS (20' SPAN)
PRIMARY ROAD SYSTEM

LOCATION

1. County Boone Civil Twp. Worth Sec. 21 Twp. 83N Range 26W
2. Over (☐River, ☒Cr., ☐Dr. Ditch) Peese Creek Highway No. Oriole Road
3. Proj. No. ER-624-0(8)--28-08 Sta. Pres. Struct. 8+28.00 Aerial Map No.
   Sta. Prop. Struct. 8+28.00

GENERAL DATA (FIELD)

4. Drainage Area 8.75 sq-mi Character Hilly to flat Approx. length and width 4.8 mi. x 2.8 mi
5. Extreme highwater: Date of occurrence 1993 Information from Ledges State Park Flood Pole
   (Elev. near site 892.5 Location STA 6+47.21, RT 152.27') (Elev. Upstream
   Location) (Elev. downstream Location)
6. Typical highwater: Elev. 863.5 Occurs every 2 Years. Date of last occurrence Unknown
7. Average low water: (Elev. at site 862.47 Average streambed 862.27) (Water elev. 862.47 on date of survey 12/10/2010)
   (Water elev. 865.52 upstream 582 Ft.) (Water elev. 858.31 downstream 494 Ft.) Fall in stream 35.38 Ft./mi.
8. List buildings in flood plain None Location Floor Elev.
9. Upstream Land Use State Park Anticipate any Change? No
10. Is stream deepening or filling? Filling Approx. amount per year Unknown
11. Is stream widening? No Show direction, rate and amount)
12. Does stream carry appreciable amount of ice? No Elev. Of high ice
13. Does stream carry appreciable amount of large driftwood? Yes
14. Bench Mark No. BM503 RR Spike in West Face of Flood Pole Northwest of G001 STA 6+47.21, RT. 152.27'

PRESENT OR OLD STRUCTURE

15. Superstructure: Type Dual 20.5' x 7.25' Aluminum Box Culvert Skew angle 27.42° L.A.
16. Substructure: Type N/A
17. Span lengths N/A Roadway width 22' Type of floor N/A
18. Culvert: Span 20.5' Ht. 7.25' Length B-B Ppts. 59' Flowline Lt. 859.0 Rt. 859.0
19. Grade elev. 868.0 Date built 2000 IDOT Design No. SP-624-0(5)--7C-06
20. Condition of superstructure Damaged beyond repair
21. Condition of substructure
22. Remarks: Existing dual culverts damaged beyond repair from August 2010 flood.

PROPOSED STRUCTURE (OFFICE)

23. Superstructure: Type 120' x 30' Continuous Concrete Slab Bridge Skew angle 30° L.A.
24. Substructure: Type P10L, Integral Abutments
25. Span lengths (Bridge): 36.5', 47.0',36.5' Culvert B-B Ppts.
26. Culvert: Span Ht. Flowline Lt. Rt. Length Lt. Rt.
27. Roadway width 30' Type of floor Concrete Class of loading HL-93
28. Type of railing TL-4, Open Rail Option Type of curb
29. Grade elev. 871.96 Abut. Footing elev. 865.66 Pier footing elev. 858.25
30. Length and type of pilings: Abuts. HP10x42 - 45' Piers HP10x42 - 50' (P1), 55' (P2)
31. Design highwater: Elev. 867.00 Frequency 50 Year Area 8.75 sq-mi Discharge 2,272 cfs
32. What provision is made for overflow? None
33. Can channel be cleared to provide more waterway? No Are wing dikes to be provided? No
34. Is excessive local scour probable? No Probable max. depth of scour below streambed 4.40 ft.
35. Disposition of existing structure Remove
36. 2007 ADT = 530 VPD
37. Remarks:

County Boone
Project. No. ER-624-0(8)--28-08
File No. 30586 PIN 11-08-624-010
Design No. 211 Maint. No. 0800.3S624

Field Notes by Adam Bullerman, P.E. Date 2-25-11
Title Project Engineer

(over)

EXAMPLE

VALLEY 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 it is an average section or a control section. Enough ground shots will be taken to outline the valley to an elevation well above extreme highwater. Special care will be taken to accurately outline the main channel. Each shot should be identified; that is (FP) flood plain, (TB) top of bank, (ES) edge of stream, etc. Mannings equation roughness factors will be assigned each shot. Include site photos with this information.

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

Distance	Elevation	(N) Roughness	Remarks

Distance	Elevation	(N) Roughness	Remarks

PLAT OF DRAINAGE AREA

The drainage area is to be platted as completely and accurately as possible and to the largest practicable scale on a separate sheet. Use a definite scale, as 1" equals ¼, ½, 1 or 2 miles, and indicate what scale has been used. In addition to the outlines of the watershed, indicate the positions of the streams and, roughly, the character of the soil and the relative locations of the steep and flat portions. Whenever practicable, the above information should be secured by going over the area either on foot or in a car. For most watersheds the information may be secured from the best existing data, soil maps, U.S.G.S. maps and Bulletin No. 7-1.H.R.B. No plat is necessary if the area is listed in Bulletin Number 7.

Remarks:

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

Marginal No.	
5	Extreme highwater due to backwater from Saylorville Lake
10	Excessive silt deposition at this site is due to backwater from Saylorville Lake
18	Culvert flowline data based on construction plans since flow-line data could not be obtained due to culvert damage

IMPORTANT NOTE

The information given on this form must in all cases be supplemented by complete plat 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.

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:

15.8.3—Earth Load

The provisions of Article 3.11 shall apply.

The possibility of difference between the actual finished grade and that shown on the contract documents should be considered in the design.

C15.8.3

Article 3.11.5.10 contains specific requirements for the determination of earth pressure on sound barrier foundation components.

Soil build-up against sound barriers has been observed in some locations. Owners may determine the earth loads for the worst load case assuming an allowance in the finished grade elevation.

15.8.4—Vehicular Collision Forces

Sound barrier systems consisting of a traffic railing and a sound barrier that have been successfully crash-tested may be used with no further analysis.

The depth of aesthetic treatments into the traffic face of sound barrier that may be subjected to vehicular collision shall be kept to a minimum.

Sound barrier materials shall be selected to limit shattering of the sound barrier during vehicular collision.

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 13 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 4.0 ft: vehicular collision force of 4.0 kips shall be

C15.8.4

Minimizing the depth of aesthetic treatment into the traffic face of sound barriers that may be in contact with a vehicle during a collision reduces the possibility of vehicle snagging.

Sound barrier systems may contain sacrificial components or components that could need repair after vehicular collision. Limiting shattering of sound barriers is particularly important for sound barriers mounted on bridges crossing over other traffic. When reinforced concrete panels are utilized for structure-mounted sound barriers, it is recommended that two mats of reinforcement are used to reduce the possibility of the concrete shattering during vehicular collision. Restraint cables placed in the middle of concrete panels may be used to reduce shattering while avoiding the increased panel thickness required to accommodate two layers of reinforcement.

The bridge overhang or moment slabs need not to be designed for more force effects than the resistance of the base connection of the sound barrier.

The design strategy involving a controlled failure scenario is similar in concept to the use of capacity protected design to resist seismic forces. Some damage to the soundwall, traffic barrier, or connections is often preferable to designing an overhang or moment slab for force effects due to vehicular collision. The bridge overhang or moment slabs need not be designed for more force effects than the resistance of the base connection of the sound barriers.

Some guidance on desirable structural performance of sound barriers can be found in European Standard EN1794-2 (2003).

Very limited information is available on crash-testing of sound barrier systems. The requirements of this Article, including the magnitude of collision forces, are mostly based on engineering judgment and observations made during crash-testing of traffic railings without sound barriers.

In the absence of crash test results for sound barrier systems, sound barriers that have not been crash-tested are often used in conjunction with vehicular railings that have been crash-tested as stand-alone railings, i.e. without sound barriers. The collision forces specified 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.

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

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.

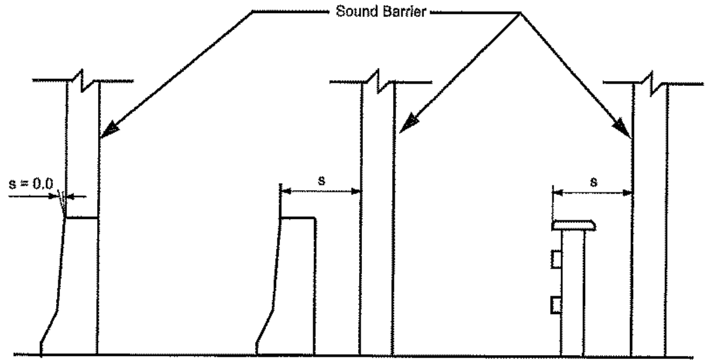


Figure 15.8.4-1—Sound Barrier Setback Distance

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_n, specified in Appendix A13.

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.

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.

The vehicular railing part of the sound barrier/railing 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.

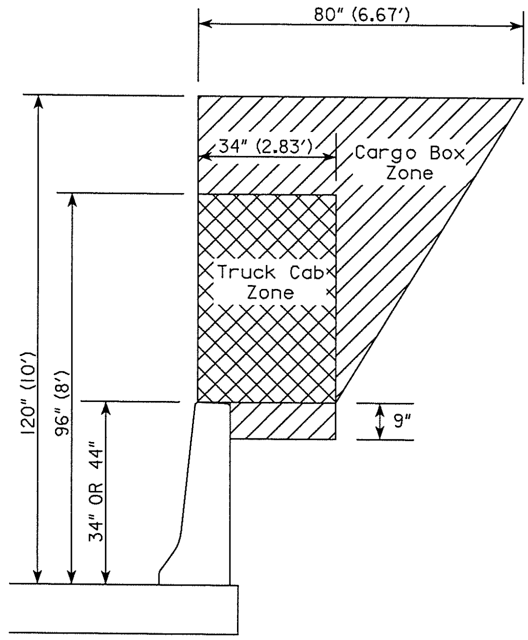
Unless otherwise specified by the Owner, vehicular collision forces shall be considered in the design of sound barriers.

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.

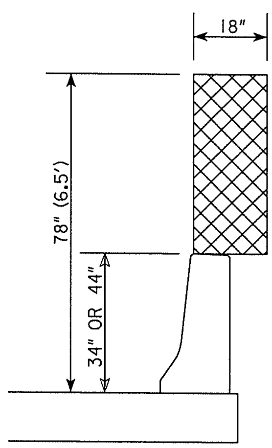
C3.13 Submittals

C3.14 Zone of Intrusion

Figures adapted from AASHTO Roadside Design Guide,4th Edition.



3.14C, 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)



3.14C, 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 COORPERATIVE 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.

6-2-17

C3.15 Temporary Bridges

Monitoring Plan (a.k.a. Plan of Action or POA) Example

Needs to be finalized following the submittal of the shop drawings for temporary bridge and revetment design. See ??'s below

Bridglet No.: 5934.8B034
County: Lucas
Route: US 34
Stream: Wolf Hollow (Detour)
District: 5 - Chariton Garage
Location: US 034 Over Wolf Hollow (Detour), 2.1 miles E of E Jct US 65
Type: Minimum 40'-0 x 28' Single span. Type TBD - Contractor choice meeting minimum size

Interim Instructions:

Site is Project BRFN-034-6(95)--39-59, US 34 Detour over Wolf Hollow, 2.1 mi E of E Jct US 65. Bridge is a temporary detour bridge and is not in the NBIS.

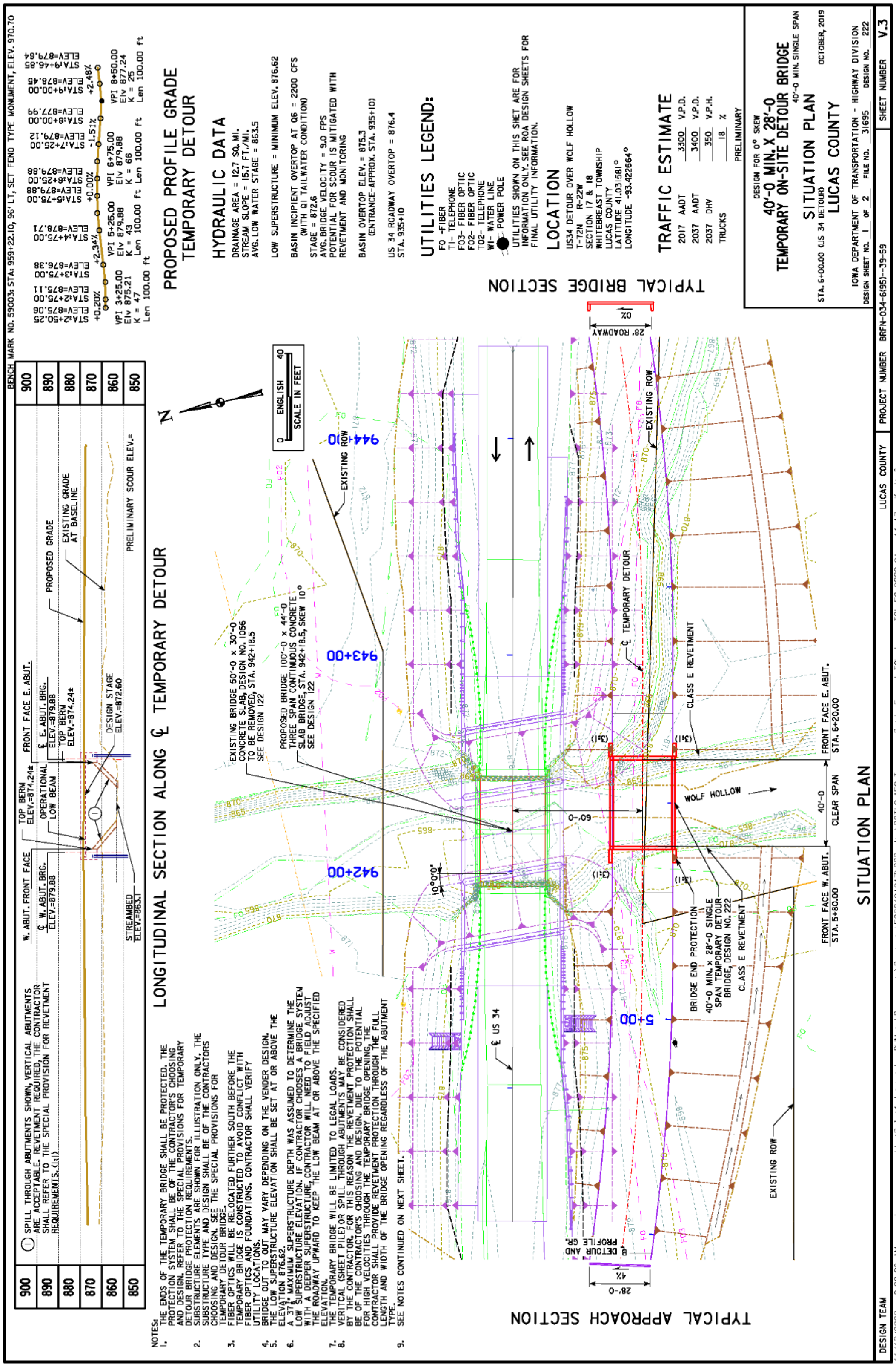
Excessive scour could occur for floods approaching the incipient overtop discharge, which is approximately a 6-yr. event in the Wolf Hollow basin. The bridge shall be checked for scour for events that meet or exceed the 5-year event. The Bridge Watch rainfall trigger should be set to the 5-yr. rainfall event. Upon alert, the site should be monitored to determine if the monitor water surface has been exceeded. If the monitor water surface elevation of 872.6 measured directly downstream (north) of the bridge is exceeded, a scour inspection shall be performed.

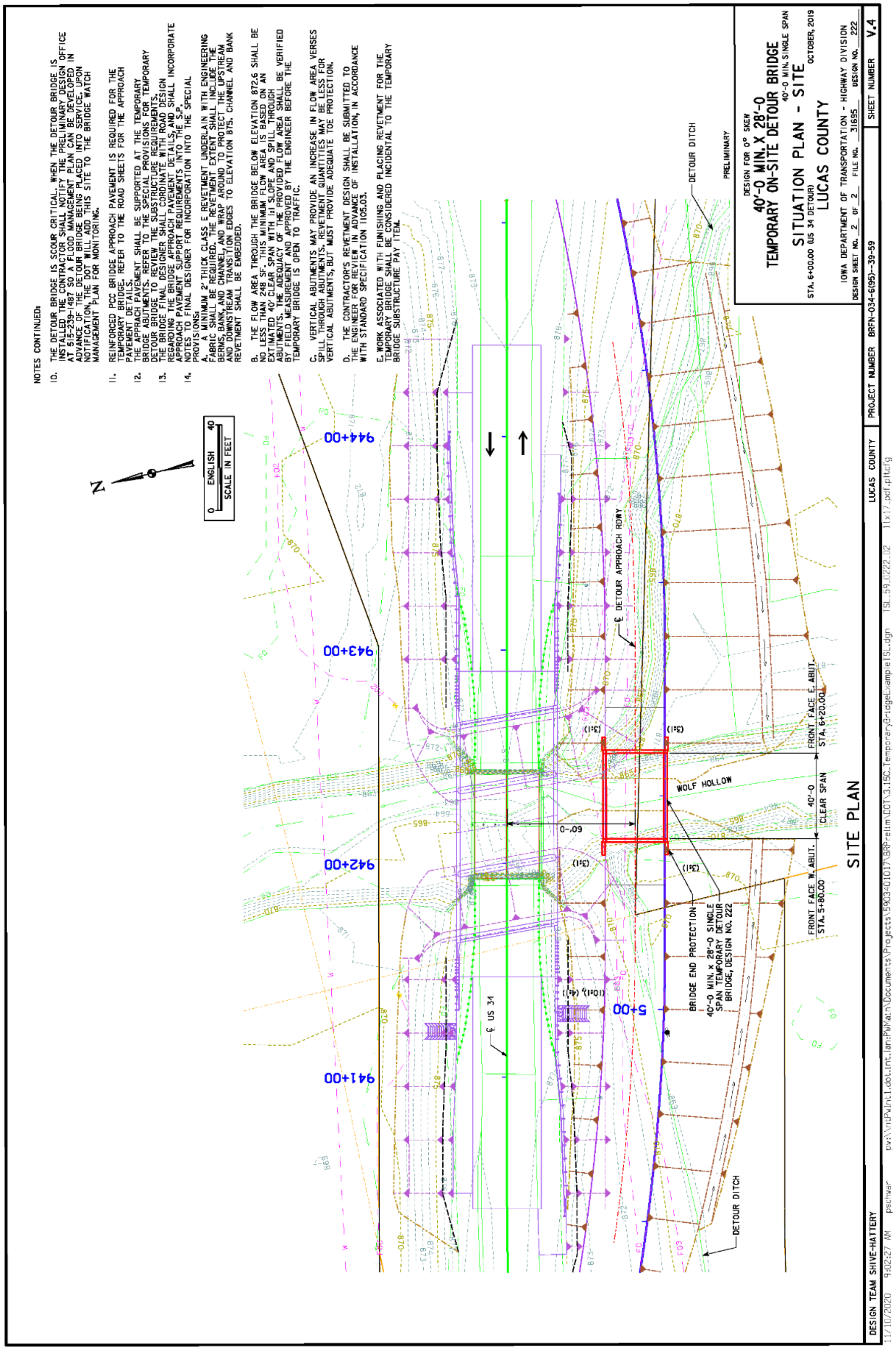
The bridge is classified as Critical. The bridge shall be inspected for integrity at the abutments once the critical water surface has been reached. The critical water surface elevation is El. 872.6 measured directly downstream (north) of the bridge. This elevation corresponds to the incipient overtop discharge of 2200 cfs. This elevation is ?? ft. below the minimum low beam. Reference Elevation - C.L. Detour Roadway C.L. W. Abutment, El. 879.88.

The abutment type is of the contractor's choosing and design. The primary scour concern at this bridge is the scour depth at the face of abutments. Undermining of the abutments could result in loss of road approach material. The bridge shall be closed to traffic if the ground surface in front of the abutments becomes lower than elevation ?? (??' below low beam). The bridge should remain closed until the integrity of the abutments can be evaluated for safety or the channel erosion is repaired.

Attachments:
A Bridge Design Sht. 1, Design No. 222
B Detour Roadway Plan Sht. F1

LATITUDE 41.031744 N
LONGITUDE 93.422639 W


PROPOSED PROFILE GRADE
TEMPORARY DETOUR
HYDRAULIC DATA
UTILITIES LEGEND:
LOCATION
TRAFFIC ESTIMATE
LONGITUDINAL SECTION ALONG ℄ TEMPORARY DETOUR
TYPICAL BRIDGE SECTION
TYPICAL APPROACH SECTION
SITUATION PLAN
WOLF HOLLOW
LUCAS COUNTY


SITE PLAN
WOLF HOLLOW
DETOUR DITCH
LUCAS COUNTY

C3.16 Resiliency/Climate Change