

Structure S051 03644 - Hydraulic Report

Decatur Bridge: NE 51 & IA 175 over Missouri River
Burt County, Nebraska & Monona County, Iowa
Project No.: STP-51-7(109)
Control No.: 32395

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Executive Summary

This report presents the results of the Phase 2 hydraulic study for replacement of the Decatur Bridge over the Missouri River at River Mile 691. The project site experiences a split flow condition between a main navigation channel and a secondary channel in the overbank. Damage from past flood events, particularly the 2011 flood, highlights the need for a replacement structure that demonstrates lower scour risk and no risk of roadway overtopping at the 100-year flood (Q100).

The Phase 2 study builds upon the Phase 1 study, which defined the site hydrology and recommended use of design flows from the 2023 USACE study, which are 50% higher than the effective FEMA flows from 2003. The Phase 1 study developed a project 2D hydraulic model, which was calibrated to the adjacent gage. Strategies to reduce flood risk were evaluated, including lengthening the bridge and adding an overflow structure. Recommendations included lengthening the bridge and raising the approach roadway.

Phase 2 refined the recommendations from Phase 1 into a proposed bridge layout and developed values for permitting and design. Eight bridge configurations were analyzed using the 2D model at various bridge length extensions (155 ft, 500 ft, 650 ft, and 800 ft) and bench elevations (1039, 1041, and 1043). These strategies were compared using their impact on water surface elevation, velocity, shear stress, scour, adjacent infrastructure, and cost.

The project team selected Strategy v_2B for the "Proposed Condition". It consists of lengthening the bridge by 650 feet and lowering the bench elevation to 1041, reducing the contraction in the secondary channel. It showed a reasonable balance between improved hydraulic performance and cost. Strategy v_3A was used as the "Alternate Condition". It consists of lengthening the bridge by 155 feet, raising the approach roadway, and maintaining the current flow pattern. It represents the minimum length of a replacement bridge and provides a reference for evaluating the benefits and impacts of the Proposed Condition.

The Proposed and Alternate Conditions were evaluated using the 2D model. The Proposed Condition demonstrates better scour performance than the Existing and Alternate Conditions. It results in lower velocity and shear stress around the bridge foundations. While both the Proposed and Alternate Conditions predict deep scour at the easternmost piers and abutment, the Proposed Condition is less severe and results in scour depths that can be accommodated in design.

Both the Proposed and Alternate Conditions eliminate the risk of roadway overtopping at the new Q100. Both result in a rise in flood elevations at the effective FEMA Q100 and would require a Conditional Letter of Map Revision (CLOMR). The rise is generally limited to the downstream secondary channel. The Proposed Condition reduces the existing backwater, and the Alternate increases it. The Proposed Condition increases the frequency of flood closure of the adjacent Iowa DNR access road.

The Proposed Condition is recommended to be used for final design. It demonstrates a lower scour risk, lower overtopping risk, and improved backwater. Though it will require a CLOMR, the 2D model shows the rise is limited to the downstream secondary channel and would not affect any insurable structures. Riprap apron countermeasures are recommended at the piers and abutment in the secondary channel. The piers should be founded on drilled shafts and the abutments should utilize piles with sheet pile walls.

The effective FEMA 1D model will be used for the CLOMR and floodplain permitting. The 1D model does not consider the split flow condition, and therefore shows a rise in the main channel that would affect insurable structures. To address this issue, the 1D model should be modified to incorporate a split flow condition by addition of a parallel reach for the secondary channel. Before additional modeling is performed, coordination with FEMA is required to confirm this approach.

While the Proposed Condition improves the backwater, it does not meet the Iowa DNR floodplain permit requirement. Further lengthening of the bridge to meet the backwater requirement is not feasible. A variance should be requested from Iowa DNR on the basis that the Proposed Condition significantly improves the Existing backwater. Further coordination with FEMA, USACE, Iowa DNR, and Nebraska DNR is recommended to finalize the design and confirm agency acceptance of the Proposed Condition.

Chapter 1

Introduction & Preliminary Design Data

1.1 Project Description

The Decatur Bridge carries Nebraska Highway 51 (N-51) and Iowa Highway 175 (IA-175) across the Missouri River at Decatur, Nebraska. This project, a joint undertaking by the Nebraska and Iowa Departments of Transportation (NDOT and Iowa DOT), aims to improve this critical river crossing. Phase 1, completed in January 2025, comprised a location and environmental study, resulting in a Feasibility Report (*Bridge Location and Environmental Study – Feasibility Report*) and a Bridge Report (*Decatur Bridge Location and Environmental Study - Bridge Report*). The *Bridge Report* detailed the existing bridge's condition, deficiencies, and flood resiliency concerns, identifying the need for a replacement structure. This Phase 2 Hydraulic Report focuses on the hydraulic analysis of a proposed replacement to enhance flood resiliency and mitigate scour risk, supporting Preliminary Design and a NEPA decision. Further environmental review will be conducted for the replacement alternative(s) carried forward from Phase 1 (Blue and Orange Alternatives). The Federal Highway Administration (FHWA) will serve as the lead federal agency for the NEPA process. Phase 3 will encompass Final Design and Permitting, followed by Construction in Phase 4.

The existing bridge crosses the Missouri River navigation channel at River Mile (RM) 691.0, between the Tieville Bend and Upper Decatur Bend. The Blue and Orange replacement Alternatives are offset from the existing alignment 77 feet north and south of the existing bridge, respectively. This report and associated hydraulic model use the Blue Alternative for results and recommendations. However, the results and recommendations would also apply to the Orange Alternative.

1.2 Phase 2 Hydraulic Study Objectives

The Phase 2 Hydraulic Study built upon the hydraulic study and model from Phase 1 as detailed in the *Bridge Report* and Attachment 4 to the *Bridge Report: Decatur Bridge Flood Resiliency Study* (Flood Resiliency Study). Reports can be made available upon request. Objectives for the Phase 2 Hydraulic Study include:

- 1) Refinement of the Phase 1 2D HEC-RAS Model for the Blue Alternative and the hydraulic improvement Strategies discussed in Chapter 7 of the *Flood Resiliency Study*,
- 2) Selection of a preferred hydraulic Strategy by NDOT and Iowa DOT from among the Strategies evaluated in Phase 2,
- 3) Development of a 1D HEC-RAS model from the effective FEMA model for Existing and Proposed Conditions,
- 4) Evaluation of floodplain impacts, and
- 5) Development of recommended design values, including water surface elevations, velocities, scour depths, and scour countermeasures.

HEC-RAS version 6.6 was used for all modeling.

1.3 Location and Reference Datum

The Project crosses the Nebraska/Iowa state line, connecting Burt County, NE to Monona County, IA. The west end of the Project is within the limits of the Village of Decatur, NE. The City of Onawa, IA, is approximately 5.8 miles east of the existing bridge. The existing bridge crosses the Missouri River at River Mile (RM) 691.0.

Other location information:

- USGS Quadrangle: Onawa SW
- Section 18, Range 46W, Township 83N
- Existing Bridge Latitude: 42° 00' 24.52" N
- Existing Bridge Longitude: 96° 14' 30.90" W
- NDOT District: 3
- Iowa DOT District: 3

Elevations in this Report are referenced to the North American Vertical Datum of 1988 (NAVD 88). The conversion factor from the National Geodetic Vertical Datum of 1929 (NGVD 29) to NAVD 88 at the Project location is approximately +0.47 feet.

Example: Elev. 100.00 (NGVD 29) = Elev. 100.47 (NAVD 88)

For Horizontal Datum, exhibits in this Report are referenced to the North American Datum of 1983 (NAD 83), Nebraska State Plane, Single Zone, with D.A.F. = 1.0003415

A topographic map of the site is shown in Figure 1-1.

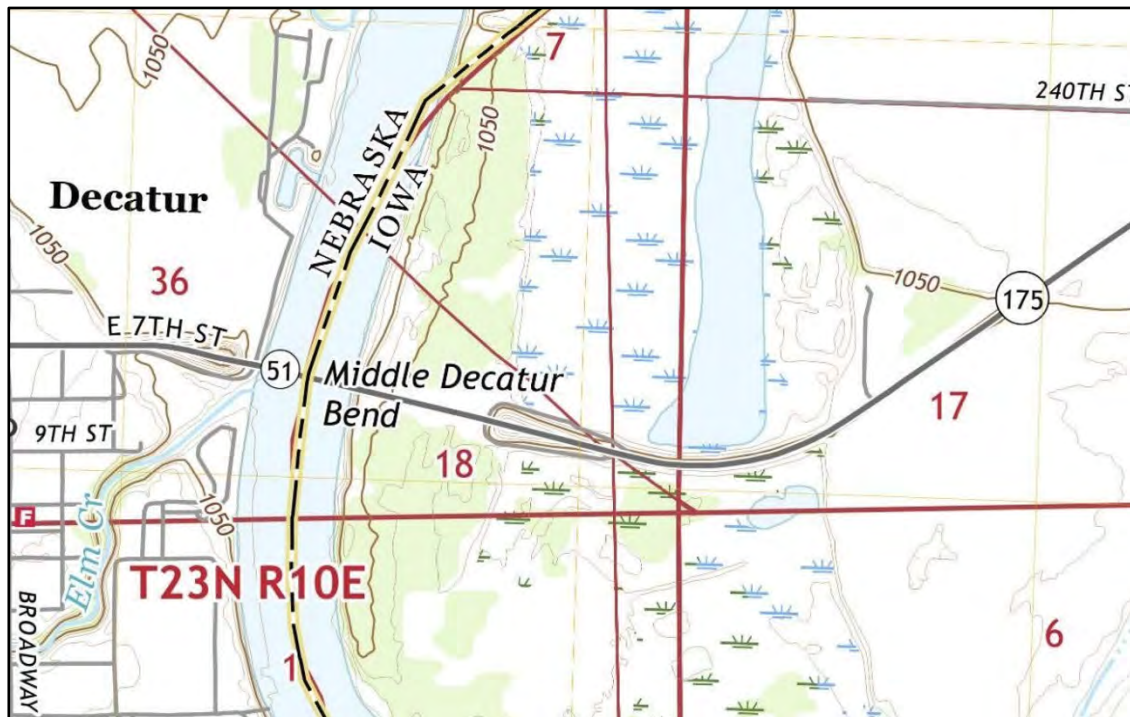


Figure 1-1. USGS Topographic Map.

1.4 Site Description

The Project site is located at River Mile 691 within the Decatur Bend of the Missouri River. See Figure 1-2 for a map of site features. The Nebraska/Iowa state line follows the centerline of the channel at the existing bridge location. The 100-Year Floodplain is approximately 4,300 feet wide at the existing bridge. The overbank on the Nebraska side is mostly level cropland, with a few neighborhoods along the bank. The overbank on the Iowa side is a mixture of wetlands, sandbars, and wooded areas.

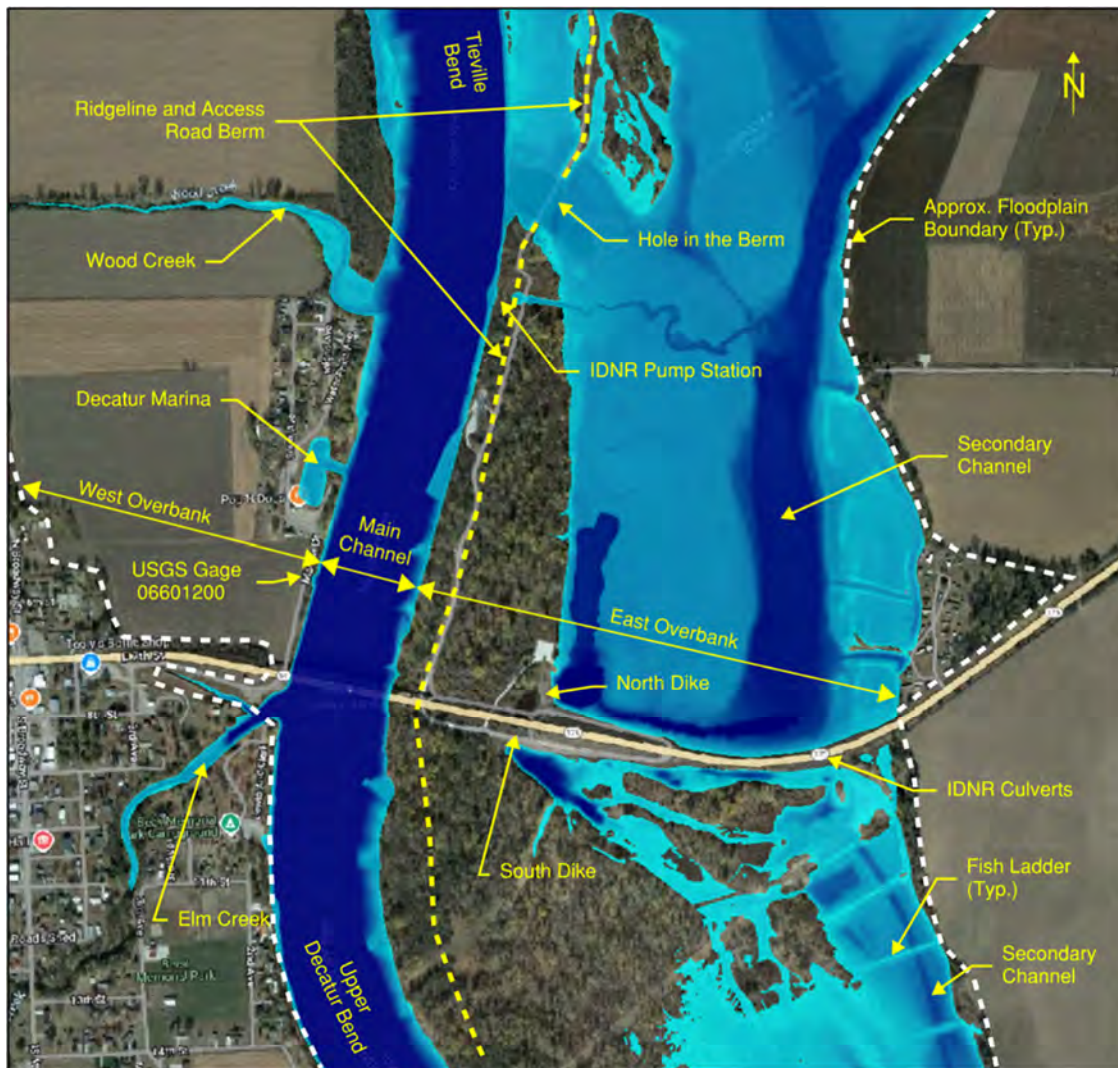


Figure 1-2. Hydraulic Site Features (Water level shown at Q10).

The Ordinary High Water elevation (OHW) is approximately 1035.0 at the existing bridge. See Section 3.7 of the *Bridge Report* for determination of OHW. The main channel is approximately 660 feet wide and 20 feet deep at OHW. USACE maintains a 300 foot wide by 9 foot deep navigation channel under the west span of the River Unit. The west bank near the existing bridge is sloped at approximately 1.75H:1V and is armored with riprap revetment. Channel training dikes are present along the east side of the river channel, on the inside of the bend. The east bank is sloped at approximately 2.5H:1V and is generally armored with riprap, except for an 800 foot long stretch right at the existing bridge that is sandy and sloughing. Wood Creek flows into the channel from the west, approximately 2,800 feet upstream of the existing bridge. Elm Creek flows into the channel from the southwest, approximately 150 feet downstream of the existing bridge. USGS Gage 06601200 (the Decatur Gage) is located on the west bank approximately 350 feet north of the bridge.

An old secondary channel is present on the east side of the floodplain. It was the main channel in the 1940s and 1950s, prior to construction of the USACE navigation channel. The secondary channel is separated from the main channel by a low wooded ridgeline extending north and south of the existing Decatur Bridge. Iowa DNR pumps water into the secondary channel to support wildlife management goals. The berm/access road was constructed to retain water within the secondary channel, but it is not fully functional because of holes in the berm from past flood events. During large flooding events, significant water volume is conveyed to the secondary channel.

The Iowa Approach embankment exists as an obstruction that restricts flow through the secondary channel. A pair of gated culverts, each 30 inches in diameter, run under the embankment and can be used by Iowa DNR to release water from the secondary channel. The culverts are used to maintain water levels for wildlife management goals and are not designed to serve as flood control. The upstream invert elevation of the culverts was surveyed at 1034.88, and the downstream invert elevation was surveyed at 1031.75. Their default condition is closed. Following the 2011 Flood, the Iowa DOT built a pair of riprap-armored dikes to protect Pier 8 and the East Abutment from scour. The top elevation of the North Dike is approximately 1048, and the top elevation of the South Dike is approximately 1049. In the secondary channel downstream of the roadway embankment are a series of "fish ladders" – weir dikes built by the USACE for biological purposes. The fish ladders were silted in by the 2011 Flood and are no longer functional.

1.5 FEMA Information

The effective flood insurance studies (FIS) are summarized in Table 1-1.

Table 1-1: Effective FEMA Studies

State	Flood Insurance Study (FIS)	FIS Title	Effective Date	FIRM Panel	Classification
Iowa	19133CV000B	Unincorporated Monona County	June 15, 2022	19133C0190D	Zone AE, with defined Floodway
Nebraska	31021CV000A	Village of Decatur and Unincorporated Burt County	June 5, 2020	31021C0090E	Zone A, no defined Floodway

Per NeDNR, FEMA has funded the development of a 2D model of the entire lower Missouri River using the flows from the 2023 MRFFS. The study is underway but will not be complete for several years. In a May 2025 meeting, both Iowa DNR and NeDNR stated that there are no imminent plans to update the FIS's at the Project location. Assuming a construction year of 2029, floodplain permitting for the Project will be based on the current FIS's.

FIRM 19133C0190D (Iowa) and 31021C0090E (Nebraska) are included as Attachment 3.

Chapter 2

Hydrology

See Chapter 3 of the *Bridge Report* for site hydrology, including information on dam regulation, drainage area, gage information, flood history, FEMA information, and hydrology studies. Key flows are summarized in Table 2-1. Annual Exceedance Probability (AEP) events are from the USACE 2023 Missouri River Flow Frequency Study (MRFFS). The 2023 MRFFS determined AEP events at Sioux City, IA and Omaha, NE. AEP events at Decatur were interpolated using drainage area ratios. Decatur has a drainage area of 316,130 square miles. Over 281,800 square miles of the drainage area is regulated by upstream dams.

The FEMA Q100 flow is from the 2003 Upper Missouri River Flow Frequency Study (UMRFFS). This is the currently effective flow for federal flood mapping. The effective FIRMs are based on the FEMA Q100. The FEMA Q100 is lower than the new Q100 from the 2023 MRFFS.

Per the Design Criteria developed in the Phase 1 *Bridge Report*, the design scour event is the Q200 and the check scour event is the Q500. NDOT Form BR14 *Hydrologic Reference Information* is included as Attachment 4.

Table 2-1: Key Flows and Flood Events at Decatur

Event/AEP/Elev	Q (cfs)	Notes
Max Dam Release Capacity	584,000	From USACE MRBWM
1952 Peak (Record Flood)	441,000	Q at Sioux City; before Gavins Point Dam built
Q500	297,200	2023 MRFFS, interpolated
Q200	271,000	2023 MRFFS, interpolated
Q100	220,600	2023 MRFFS, interpolated
2011 Peak	191,000	Record Post-Dam System Flood
Q50	161,700	2023 USACE FFS, transposed
2019 Peak	152,000	
2024 Peak	148,000	
FEMA Q100	141,800	Effective FEMA FIS (Q from 2003 UMRFFS)
Q25	127,100	2023 USACE FFS, interpolated
1997 Peak	100,000	
2% Flow Line	96,620	From gage history; Nav Season only (4/1 to 11/30)
Q10	94,400	2023 MRFFS, interpolated
Q5	76,200	2023 MRFFS, interpolated
Q2	49,800	2023 MRFFS, interpolated
OHW	47,560 ^A	Based on delineated OHW elevation = 1035.0
Q1.25	42,100	2023 MRFFS, interpolated
Min. Navigation Flow	25,000	From USACE MRBWM; minimum flow for nav service
Min. Winter Flow	12,000	From USACE MRBWM; minimum flow for water supply
Record Low Flow	7,070	Min. daily avg recorded at gage (3/12 & 3/13 of 2009)
^A Calculated from USGS Base Rating Curve at the Decatur Gage		

Chapter 3

Modeling

The HEC-RAS 2D model from Phase 1 was carried into Phase 2 for further refinement and investigation. For details of the Existing Condition model, see the *Flood Resiliency Study*. This includes information regarding Terrain, Boundary Conditions, Manning's n Values, Boundary Conditions, 2D Mesh, Existing Structures, Flow Hydrographs, Calibration, and Analysis Method. See page B.1 in Appendix B of this Report for the Existing Condition terrain.

In addition to the project specific 2D model, the effective 1D HEC-RAS model was obtained from FEMA. The 1D model was updated for use in floodplain permitting. Details about the 1D model are described in Section 3.4.

For the Phase 2 analysis additional flow hydrographs were added to the Phase 1 2D model, including the effective FEMA 100 year flow, 2023 MRFFS 200 year flow, and the peak flow from the 2024 flood. Several Manning's n-values were also modified downstream of the bridge in the secondary channel. Further evaluations of Phase 1 strategies were conducted (see Section 3.1). A Proposed solution was selected from these strategies, and the 2D model was updated to reflect final grading, bridge layout, refined Manning's n values, and mesh edits (see Section 3.2). This Proposed bridge configuration was then analyzed using the 1D model (see Section 3.4).

3.1 Hydraulic Strategy Evaluation – 2D Model

The Phase 2 analysis investigated several hydraulic strategies to mitigate flood-related risks, focusing on extending the bridge length and adjusting the bench elevation. These strategies were selected based on the findings of the Phase 1 *Flood Resiliency Study*. The primary variables considered were:

Bridge Length Extension: Four bridge length extensions were analyzed: 155 ft (no extension beyond the existing north dike), 500 ft, 650 ft, and 800 ft.

Bench Elevation: "bench elevation" is the term used for the new ground elevation below the additional spans of the extended bridge. Three bench elevations were considered: 1039 ft, 1041 ft, and 1043 ft. These bench elevations are higher than the existing ground upstream of the bridge. The bench elevation is therefore the elevation at which flow in the secondary channel begins.

The combination of these variables resulted in a total of 8 distinct strategies. Table 3-1 shows a summary of the strategies evaluated.

Table 3-1. Strategy Models

Model Name	Bridge Extension	Bench Elevation
v1A	800 ft	1039
v1B	800 ft	1041
v1C	800 ft	1043
v2A	650 ft	1039
v2B	650 ft	1041
v2C	650 ft	1043
v3A	155 ft	N/A
v5A	500 ft	1039

The strategy models were created by importing a new terrain file of the Blue Alternative created in OpenRoads into the Existing terrain model. This "base" terrain from OpenRoads was modeled to the longest bridge extension length at 800 ft. Once this "base" terrain was created, terrain modifications were made to

create the new “bench” area between the bridge abutment and existing north dike. Each strategy was then modified to match the bridge extension and bench elevation under investigation. Plots of the terrain for each strategy are shown in Appendix G.

Proposed piers were added to the Existing terrain based upon the preliminary bridge layout at the time of the investigation. The Existing pier terrain modifications were removed. The mesh and breaklines were adjusted to align with the new roadway alignment and pier locations. Mesh outside of the location of terrain and pier edits was not altered. Specific Manning’s edits were not made for each strategy, as differences would be minor. The Manning’s edits were adjusted for the final Proposed model.

Models for each strategy were run with the 10, 25, 100, 200, 500 year flows and the FEMA 100 year flow. Results are summarized in Appendix H, including a hydraulic strategy evaluation table. Pier and contraction scour were also calculated for each of the strategies; results are summarized at the end of Appendix E.

The project team, including both Nebraska and Iowa DOTs, met on February 26, 2025 and March 19, 2025 to compare the hydraulic performance of the strategies. **The project team selected strategy v_2B (650 ft bridge extension and 1041 bench elevation) to be used as the basis for the “Proposed Condition” and for preliminary design, based on hydraulic performance and relative costs.** This model moved forward for the final edits as described in Section 3.2.

3.2 Proposed Condition – 2D Model

Description

The proposed bridge was modeled on the Blue Alternative alignment, which is 77’-0” north of the existing bridge. The proposed bridge is a 2,730’-0” bridge with four units. The first unit is a single span NU 35 prestressed concrete girder unit (70’-0”). The second unit is a three span welded steel plate girder unit (270’-0” – 425’-0” – 325’-0”). The third and fourth units are both five span NU 78 prestressed concrete girder units (each at 155’-0” – 170’-0” – 170’-0” – 170’-0” – 155’-0”). The new clear roadway width will be 40’-0” clear and the piers will not be skewed. The Iowa abutment will be located approximately 650’-0” west of the existing abutment. The bench elevation for the grading under the bridge will peak at 1041 between the toe of the Iowa abutment embankment and the toe of the existing north dike. See Attachment 1 for Type, Size, and Location (TS&L) drawings of the proposed bridge used in the 2D model.

Terrain Modifications

The Proposed Condition model used the v_2B strategy as the base terrain. Several modifications were made for final refinement. See page B.2 in Appendix B for the Proposed terrain:

- At the Iowa abutment, the front slope was modified for a 10-foot berm and 3:1 slope.
- The north dike slope was adjusted to 2:1 at the bench area.
- A 1% slope was added to the bench area to allow positive drainage to the north during typical storm events.
- Minor slope adjustments were made where the Iowa DNR access road and north dike intersect.
- The profile grade was adjusted on the Iowa embankment to match the roadway Plan-in-Hand (PIH) plans.
- The Nebraska roadway embankment limits were adjusted to reflect the length of the first unit.
- Pier terrain edits were made to match the final span layout.
- The Iowa DNR boat channel upstream of the bridge was relocated approximately 100 feet north.

2D Mesh

The 2D mesh from the v_2B model was adjusted to refine the mesh around the revised span layout. No other adjustments to the mesh were made.

Manning's n-Values

The Existing Condition classification polygons were adjusted to fit the new alignment and the new opening of the bridge. See the *Flood Resilience Study* for specific details of the land cover and Manning's n-value selection. Figure 3-1 shows the adjusted classification polygons.

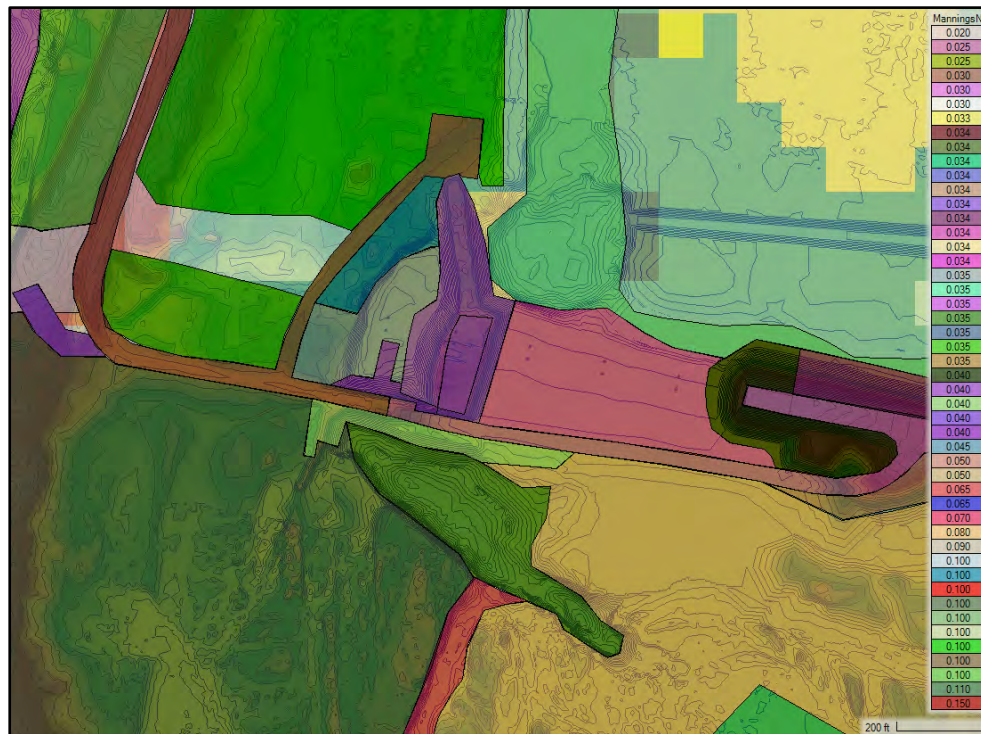


Figure 3-1. Proposed Manning's n-Value Classification Polygons.

3.3 Alternate Condition – 2D Model

For evaluation and comparison in this report, strategy v_3A will be used as the "Alternate Condition". This strategy consists of locating the new east abutment approximately 155 feet east of the existing abutment, raising the Iowa approach roadway above the Q100, and maintaining the current flow pattern of the secondary channel over the dikes. It represents the minimum tolerable length of a replacement bridge, and serves as a good reference for evaluating the benefits and impacts of the Proposed Condition.

As will be discussed in Chapter 4, the Alternate Condition results in extremely high velocities and shear stresses in the region around the dikes. Therefore, the Alternate Condition was evaluated for two cases. Alternate Case 1 assumes the dikes remain intact. Alternate Case 2 models the result if the dikes fail down to the existing ground elevation between them. See pages B.3 and B.4 in Appendix B for the terrain for each case.

3.4 Natural Condition

For this project, no data is available for the topography of the floodplain prior to construction of the existing bridge and roadway embankment. The Natural Condition was modeled by removing the existing bridge and removing the existing roadway embankment within the limits of the floodplain. The "natural" elevation of the embankment area was set to match the existing ground elevation immediately upstream and downstream of the embankment. The existing bridge pier terrain edits were also removed. See page B.5 in Appendix B for the terrain map of the Natural Condition.

The geometry and mesh of the Natural Condition model was kept the same as the Existing Condition. The Manning's values were only edited in the region around the existing embankment to match the upstream and downstream terrain.

3.5 Cross Sections

The cross-sections developed in the Phase 1 study were used to compare water surface elevations in Phase 2. See section 3.5.2 of the *Flood Resiliency Study* for details of the cross-section locations. Figure 3-2 shows the locations and names of these cross-sections.

Cross-section five (XS-5) is located at the USGS Gage. This is approximately 325 feet upstream of the existing bridge face and 240 feet upstream of the Proposed bridge face. This cross section also falls approximately 70 feet upstream of the edge of the Nebraska approach embankment and at the end of the north dike. This cross section is being used as the "upstream" bridge cross section for water surface elevation comparison for the 2D model results presented in Section 4.1 of the report.

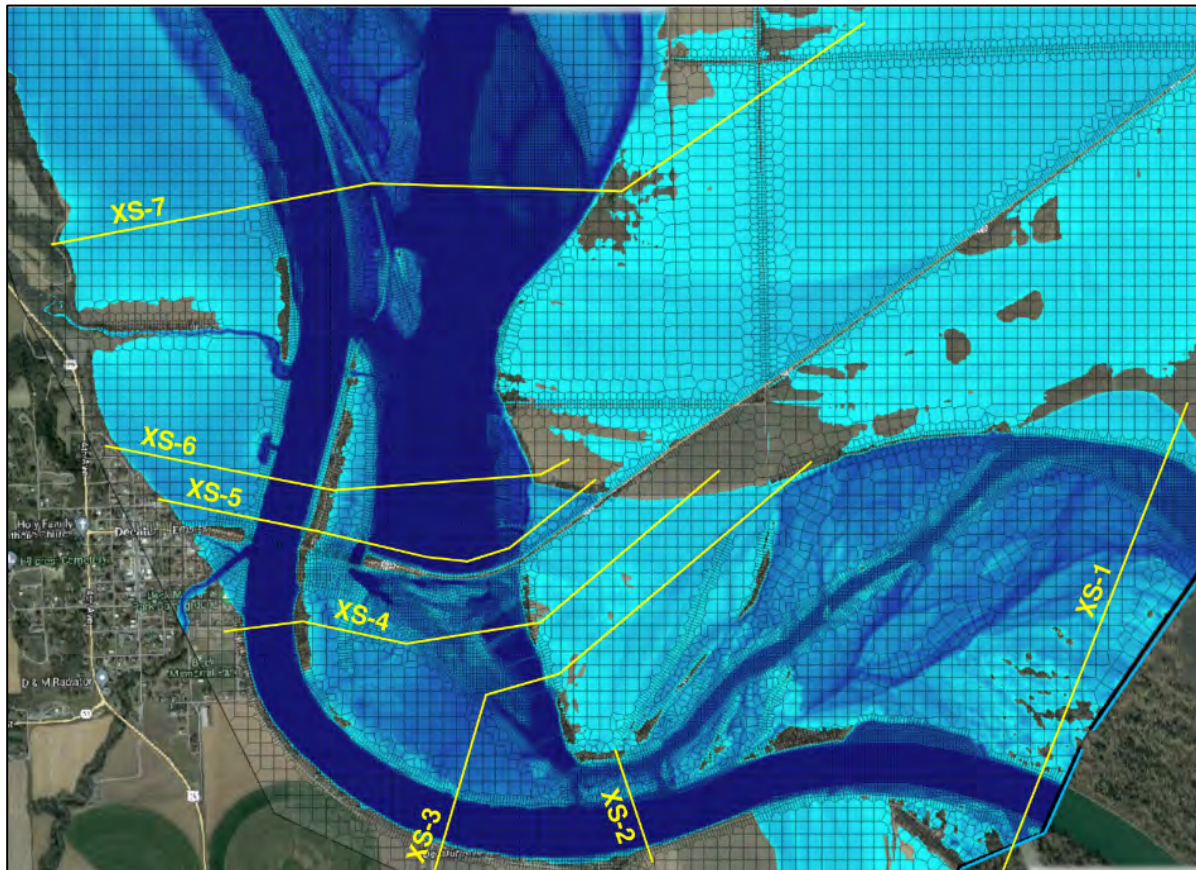


Figure 3-2. Cross Section Locations

3.6 1D HEC-RAS Model

Source of Data and Method of Analysis

The 1D analysis employed HEC-RAS version 6.6. The base HEC-RAS model used to develop the 1D model is the currently effective FEMA model, obtained from FEMA on April 29, 2025. This model was originally developed by the U.S. Army Corps of Engineers (USACE) Omaha District for FEMA in 2007 for Flood Insurance Studies (FIS) along the Missouri River. The model extends from Gavin's Point Dam to Rulo, Nebraska, and is a one-dimensional, steady-state model.

Three HEC-RAS models were developed for this project. The description and plan file names are below. The Alternate Condition was not evaluated with the 1D model.

- Duplicate Effective Condition
- Corrected Effective Condition (i.e. Existing Condition)
- Proposed Condition

Cross section locations are shown on pages F.1 and F.2 of Appendix F.

Hydrology

The hydrology for the 1D model utilized existing FEMA 100-year steady-flow data from the 2003 Upper Missouri River System Flow Frequency Study (UMRFFS) without modification. This data served as the primary basis for the model, which focuses on investigating the FEMA 100-year flow for permitting purposes.

Hydraulic Analysis

The Duplicate Effective plan was run without modification. The FEMA Q100 water surface elevations matched the published Base Flood Elevations (BFEs) in the effective FIS. The Duplicate Effective model does not consider the split flow condition at the FEMA Q100, or at any flood level. There is no parallel reach for the secondary channel. There is no multi-opening analysis at the bridge. The same water surface elevation is reported for flow in both the main channel and secondary channel.

The Corrected Effective Condition was developed from the Duplicate Effective Condition. A summary of the updates for the Corrected Effective Condition includes:

- Moved upstream and downstream bridge cross-sections (691.04 & 691.03) outside of the toe of the embankment for existing and proposed bridges (691.1 & 691)
- Updated terrain for cross-sections 691.75, 691.35, 690.95, and 690.52 from the 2D model terrain
- Added cross-section 691.2 to help better capture contraction in the east overbank
- Updated existing bridge information using existing plans and survey data
- Updated ineffective flow areas at cross-sections 691.75, 691.35, 690.95, 690.52, 690.13, and 689.72
- Updated reach lengths near bridge based upon additional and relocated cross-sections

Manning's roughness values were not modified beyond adjusting the limits to match the updated terrain. The Energy Method was utilized for all models.

The Corrected Effective was not modified to reflect the split flow condition. It is anticipated that this would require the addition of a parallel reach for the secondary channel, and determination of flow distribution between the channels. This analysis is outside the scope of the present study.

The Proposed Condition was developed from the Corrected Effective Condition. A summary of the changes for the Proposed Condition includes:

- Replaced existing bridge with proposed bridge
- Updated terrain in the Iowa overbank for cross-section 691 to account for grading changes in the Proposed Condition
- Adjusted ineffective flow areas at cross-sections 691.2, 691.1, 691, 690.95, 690.52, and 690.13

Chapter 4

Results

Results are reported in this chapter are for the Existing, Proposed, and Alternate Conditions. Results from other strategies that were not carried forward are included in Appendix H. All results refer to the 2D model except Section 4.5, which discusses the 1D model results.

4.1 Water Surface Elevation

Water surface elevations vary between the main and secondary channel due to separation of flow from the high ridgeline and the constriction of the secondary channel by the roadway embankment. Table 4-1 summarizes the Existing and Proposed water surfaces elevations at the bridge. Water surface elevations are taken at the face of the proposed bridge near the center of the main and secondary channel.

The Proposed Condition generally results in lower water surface elevations at the bridge for all events, as compared with the Existing Condition. The Alternate Condition results in higher water surface elevations than Existing for events at Q100 and above.

Table 4-1. Water Surface Elevation at Upstream Face of Bridge.

Event	Main Channel WSE (ft.)					Secondary Channel WSE (ft.)				
	Existing	Proposed		Alternate		Existing	Proposed		Alternate	
	WSE	WSE	Δ (ft.)	WSE	Δ (ft.)	WSE	WSE	Δ (ft.)	WSE	Δ (ft.)
Q10	1042.71	1042.58	-0.13	1042.71	0.00	1043.90	1043.56	-0.34	1043.90	0.00
Q25	1046.53	1046.05	-0.48	1046.53	0.00	1047.94	1046.96	-0.98	1047.94	0.00
FEMA Q100	1047.82	1047.20	-0.62	1047.79	-0.03	1049.28	1048.07	-1.21	1049.24	-0.04
2011 Flood (Q70)	1050.88	1050.25	-0.63	1050.80	-0.08	1052.24	1050.99	-1.25	1052.11	-0.13
Q100	1051.98	1051.60	-0.38	1052.09	0.11	1053.24	1052.29	-0.95	1053.34	0.1
Q200	1053.32	1053.28	-0.04	1053.57	0.25	1054.36	1053.88	-0.48	1054.71	0.35
Q500	1053.81	1053.87	0.06	1054.05	0.24	1054.74	1054.41	-0.33	1055.11	0.37

The cross sections from Section 3.4 were used to compare water surface elevations at the FEMA Q100 and Q100. Table 4-2 thru 4-5 shows the comparison of Existing, Proposed, and Alternate water surface elevations at these cross sections for both the main and secondary channels. Water surface elevations were also compared between Existing and Proposed Conditions in RAS Mapper for the entire 2D model boundary. Heat map exhibits showing the change in water surface elevation at these two events are included in Appendices C and D. Water surface profile plots and water surface elevation plots are also included in Appendices C and D.

Table 4-2. FEMA Q100 Average Water Surface Elevations – Proposed vs Existing.

XS	Description	Main Channel WSE (ft.)			Secondary Channel WSE (ft.)		
		Exist	Prop	Δ	Exist	Prop	Δ
7	~5000' Upstream	1049.24	1048.40	-0.84	1049.35	1048.35	-1.00
6	~1000' Upstream	1048.11	1047.44	-0.67	1049.28	1048.16	-1.12
5	240' Upstream Proposed Bridge Face	1047.90	1047.27	-0.63	1049.28	1048.11	-1.17
4	~1000' Downstream	1047.40	1046.88	-0.52	1045.18	1045.76	0.58
3	~5000' Downstream	1045.75	1045.61	-0.14	1045.18	1045.70	0.52
2	Confluence of Primary/Secondary Channel	1044.73	1044.72	-0.01	1044.73	1044.72	-0.01
1	Downstream Boundary Condition	1042.42	1042.41	-0.01	1040.60	1040.60	0.00

Table 4-3. FEMA Q100 Average Water Surface Elevations – Alternate vs Existing.

XS	Description	Main Channel WSE (ft.)			Secondary Channel WSE (ft.)		
		Exist	Alternate	Δ	Exist	Alternate	Δ
7	~5000' Upstream	1049.24	1049.21	-0.03	1049.35	1049.31	-0.04
6	~1000' Upstream	1048.11	1048.08	-0.03	1049.28	1049.24	-0.04
5	240' Upstream Proposed Bridge Face	1047.90	1047.87	-0.03	1049.28	1049.24	-0.04
4	~1000' Downstream	1047.40	1047.38	-0.02	1045.18	1045.20	0.02
3	~5000' Downstream	1045.75	1045.74	-0.01	1045.18	1045.20	0.02
2	Confluence of Primary/Secondary Channel	1044.73	1044.73	0.00	1044.73	1044.73	0.00
1	Downstream Boundary Condition	1042.42	1042.42	0.00	1040.60	1040.60	0.00

For the FEMA Q100 event, both the Proposed and Alternate Conditions result in lower water surface elevations in the main channel throughout the model. Both the Proposed and Alternate conditions result in a rise in the secondary channel immediately downstream of the bridge, but this rise dissipates by the confluence of the secondary and main channels (XS-2).

Table 4-4. Q100 Average Water Surface Elevations – Proposed vs Existing.

XS	Description	Main Channel WSE (ft.)			Secondary Channel WSE (ft.)		
		Exist	Prop	Δ	Exist	Prop	Δ
7	~5000' Upstream	1053.42	1052.80	-0.62	1053.46	1052.83	-0.63
6	~1000' Upstream	1052.30	1051.85	-0.45	1053.25	1052.48	-0.77
5	240' Upstream Proposed Bridge Face	1052.07	1051.67	-0.40	1053.24	1052.35	-0.89
4	~1000' Downstream	1051.53	1051.25	-0.28	1049.68	1050.02	0.34
3	~5000' Downstream	1049.79	1049.86	0.07	1049.64	1049.92	0.28
2	Confluence of Primary/Secondary Channel	1048.72	1048.80	0.08	1048.72	1048.80	0.08
1	Downstream Boundary Condition	1046.15	1046.16	0.01	1045.85	1045.77	-0.08

Table 4-5. Q100 Average Water Surface Elevations – Alternate vs Existing.

XS	Description	Main Channel WSE (ft.)			Secondary Channel WSE (ft.)		
		Exist	Alternate	Δ	Exist	Alternate	Δ
7	~5000' Upstream	1053.42	1053.52	0.10	1053.46	1053.56	0.10
6	~1000' Upstream	1052.30	1052.40	0.10	1053.25	1053.37	0.12
5	240' Upstream Proposed Bridge Face	1052.07	1052.18	0.11	1053.24	1053.34	0.10
4	~1000' Downstream	1051.53	1051.64	0.11	1049.68	1049.81	0.13
3	~5000' Downstream	1049.79	1049.90	0.11	1049.64	1049.76	0.12
2	Confluence of Primary/Secondary Channel	1048.72	1048.79	0.07	1048.72	1048.79	0.07
1	Downstream Boundary Condition	1046.15	1046.16	0.01	1045.85	1045.82	-0.03

For the Q100 event, the Proposed Condition results in lower water surface elevations at all locations upstream of the bridge. In the main channel, there is a rise at the confluence that generally dissipates by the downstream boundary of the model (XS-1). In the secondary channel, there is a rise immediately

downstream of the bridge. For the Q100 event, the Alternate Condition results in an upstream rise in both the main and secondary channels. This rise extends to the upstream and downstream limits of the model.

4.2 Backwater

Bridge backwater is defined as the difference in water surface elevation upstream of the bridge between Natural Condition and the Existing or Proposed Condition. For floodplain permitting, the Iowa DNR generally requires that the Proposed backwater at the Q100 event be less than or equal to 1 foot. The Iowa DNR defines the Natural Condition as the "pre-development" condition of the waterway, with no bridge or roadway embankment in place. Per input from the Iowa DNR in May 2025, the backwater requirement for Decatur should be assessed using the 2D model at the new Q100 event (2023 MRFFS)

Tables 4-6 and 4-7 summarize the backwater in the main channel and secondary channel. The Existing backwater exceeds the 1 foot requirement of the Iowa DNR. The Proposed Condition reduces the backwater. The Alternate Condition increases it.

Table 4-6. Main Channel Backwater (BW) at Q100 (2023 MRFFS).

XS	Description	Natural	Existing		Proposed		Alternate	
		WSE	WSE	BW (ft.)	WSE	BW (ft.)	WSE	BW (ft.)
7	~5000' Upstream	1051.70	1053.42	1.72	1052.80	1.1	1053.52	1.82
6	~1000' Upstream	1050.92	1052.30	1.38	1051.85	0.93	1052.40	1.48
5	240' Upstream Proposed Bridge Face	1050.80	1052.07	1.27	1051.67	0.87	1052.18	1.38

Table 4-7. Secondary Channel Backwater (BW) at Q100 (2023 MRFFS).

XS	Description	Natural	Existing		Proposed		Alternate	
		WSE	WSE	BW (ft.)	WSE	BW (ft.)	WSE	BW (ft.)
7	~5000' Upstream	1051.69	1053.46	1.77	1052.83	1.14	1053.56	1.87
6	~1000' Upstream	1050.93	1053.25	2.32	1052.48	1.55	1053.37	2.44
5	240' Upstream Proposed Bridge Face	1050.81	1053.24	2.43	1052.35	1.54	1053.34	2.53

4.3 Velocity

Table 4-8 shows peak velocities in the main and secondary channels. Figures 4-1 thru 4-3 show Existing, Proposed, and Alternate velocities for the Q200 flow. Contour lines in the plot represent 1 ft/s contours. Velocities are reduced in the main channel for both the Proposed and Alternate Condition, as compared to the Existing Condition. Proposed peak velocities in the secondary channel are much lower than the extreme velocities over the north and south dikes in the Existing Condition. Proposed velocities in the secondary channel immediately upstream and downstream of the bridge increased, but remained moderate. For the Alternate Condition, peak velocities in the vicinity of the dikes increased. Additional velocity plots for the 2011 Flood, Q200, and Q500 can be found in Appendix C for the Proposed Condition and Appendix D for the Alternate Condition.

Table 4-8. Peak Velocities for Large Floods.

Event	Main Channel WSE (ft/s)					Secondary Channel WSE (ft/s)				
	Existing	Proposed		Alternate		Existing	Proposed		Alternate	
	V	V	Δ	V	Δ	V	V	Δ	V	Δ
2011 Flood (Q70)	9.5	8.0	-1.5	9.0	-0.5	18.0	9.5	-8.5	21.5	3.5
Q200	10.0	9.5	-0.5	10.0	0.0	21.0	12.0	-9.0	21.0	0.0
Q500	10.0	9.5	-0.5	10.0	0.0	20.0	12.5	-7.5	20.5	0.5

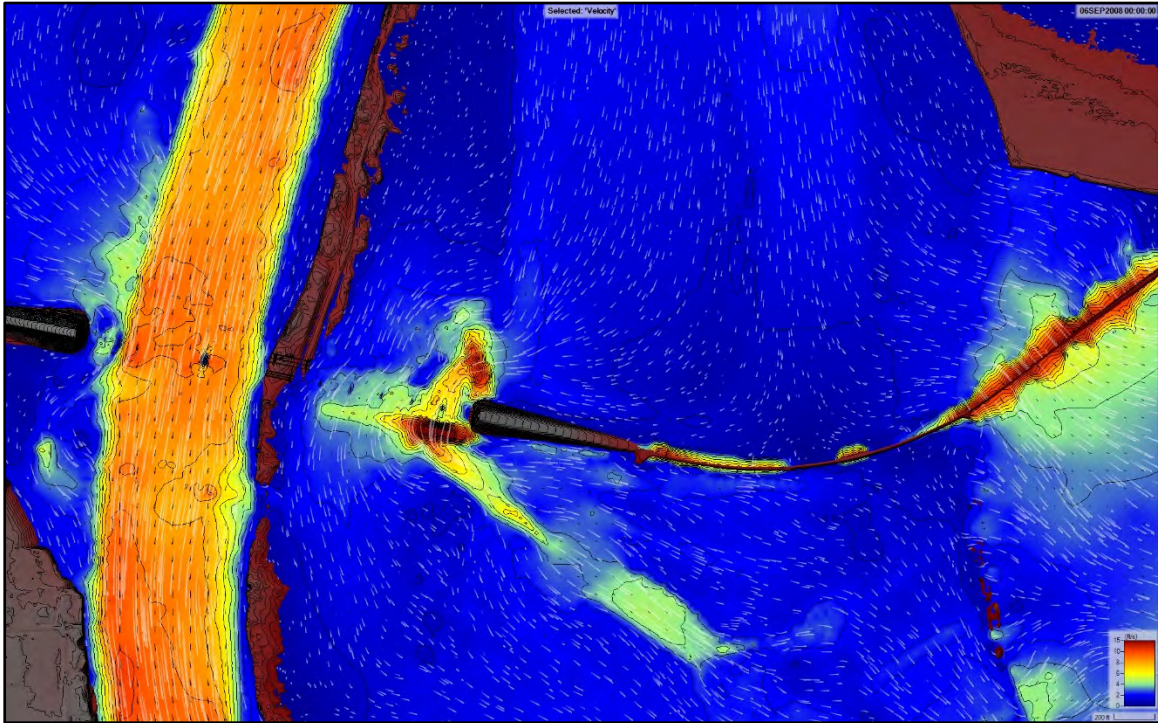


Figure 4-1. Existing Condition Velocity at Q200

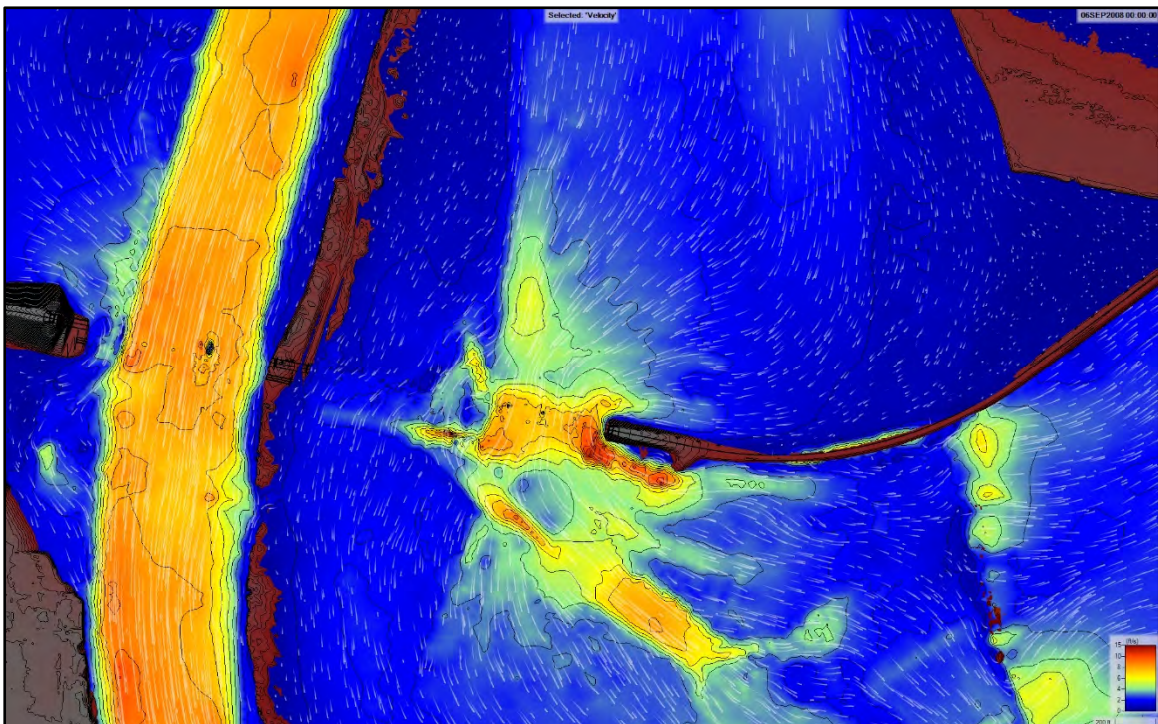


Figure 4-2. Proposed Condition Velocity at Q200

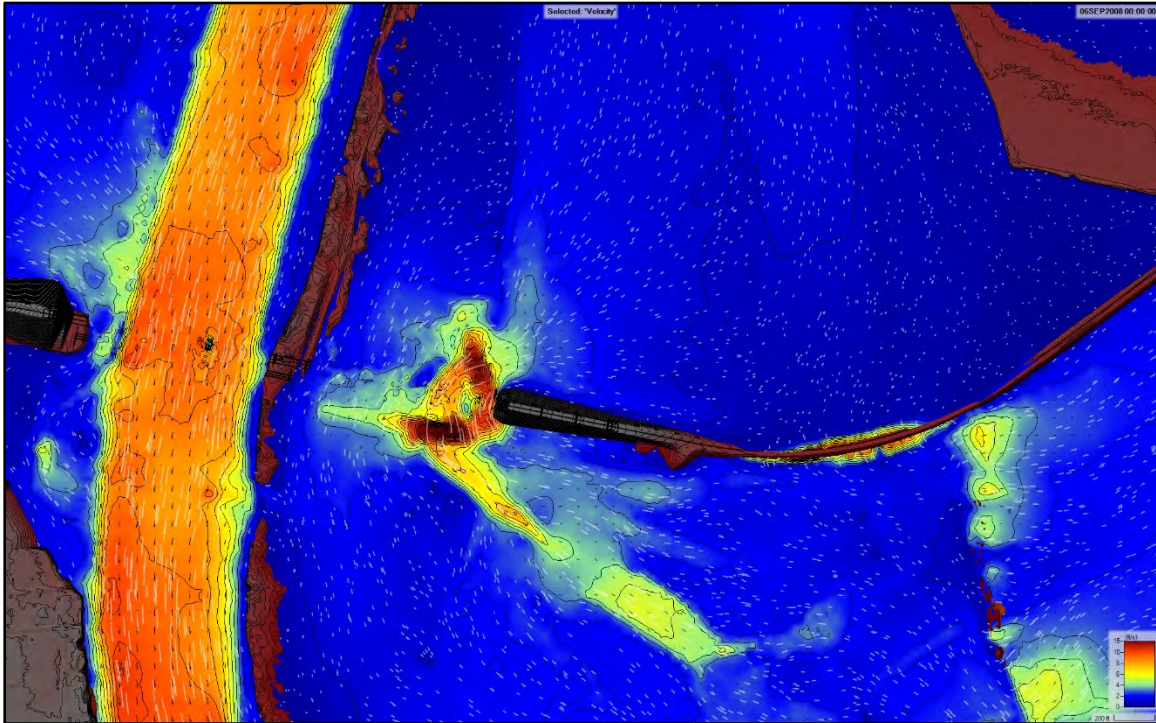


Figure 4-3. Alternate Condition Velocity at Q200

4.4 Shear Stress

Table 4-9 was developed to help compare shear stress results. It shows the permissible ground surface material for various shear stress ranges. The sources used in development of the table are listed below.

Table 4-9. Permissible Shear Stress/Material Ranges.

Lower Bound (psf)	Upper Bound (psf)	Permissible Material	Material Description
0	0.04	Bare Soil	Alluvial silt, sandy loam
0.041	1	Short Grass	Vegetation Class C (6" to 11"), short native/bunch grass
1.01	2.1	Long Grass	Vegetation Class B (12" to 20"), long native grasses
2.11	6	Riprap, $D_{50} \sim 18"$	Existing riprap (Iowa DOT Class B), MODOT Type 4
6.01	10	Riprap, $D_{50} \sim 24"$	Larger than existing riprap

Sources for "Permissible Shear Stress/Material":

- MODOT Engineering Policy Guide Section 750.6, "Erosion Control and Energy Dissipation", (2025)
- HEC-15, "Design of Roadside Channels with Flexible Linings", Section 2.2.3 (2005)
- Caltrans Highway Design Manual, Chapter 860 – Roadside Channels, (2020)
- USACE ERDC, "Stability Thresholds for Stream Restoration Materials", Fischenich (2001)

Table 4-10 shows a comparison of shear stress results for the Existing, Proposed, and Alternate Conditions. For the Proposed Condition, shear stresses at all events generally remain in the range where vegetative cover would be sufficient. Several hot spots would require riprap protection. For the Existing Condition and Alternate Condition, for the 2011 Flood (~ Q70) and larger, shear stresses over and around the dikes are in an extreme range where failure of the riprap protection is possible. This could lead to failure of the dikes, either through direct erosion of the riprap or undercutting around the edges.

Table 4-10. Shear Stress (γ) at Key Locations.

Event	Secondary Channel Average (psf)					Secondary Channel Peak (psf)				
	Existing	Proposed		Alternate		Existing	Proposed		Alternate	
	γ	γ	Δ	γ	Δ	γ	γ	Δ	γ	Δ
2011 Flood (Q70)	5.0	0.7	-4.3	5.0	0.0	7.0	2.5	-4.5	7.0	0.0
Q200	5.5	1.0	-4.5	5.0	-0.5	8.5	3.0	-5.5	7.0	-1.5
Q500	5.0	1.0	-4.0	5.0	0.0	8.0	3.0	-5.0	7.0	-1.0

Table 4-11 shows shear stress for the Proposed Condition through the bench area and on the IDNR Access Road. For the Proposed Condition, flow will be conveyed in the secondary channel and over the Iowa DNR access road at Q5 and above. However, for flows up to the FEMA Q100, maximum shear stresses in the bench area and on the access road remain very low.

Table 4-11. Proposed Condition Peak Shear Stresses.

Event	Peak Shear Stress (psf)	
	Bench Area	On IDNR Access Road
Q10	0.1	0.1
Q25	0.6	1.0
FEMA Q100	1.0	1.2
2011 Flood (Q70)	1.5	2.2
Q100	1.6	2.8
Q200	2.0	3.4
Q500	2.1	3.5

Figures 4-4 thru 4-6 show heat maps of shear stress at the Q200 event. The colors in the heat map correspond to the ranges in Table 4-9. Additional shear stress plots are included in Appendix C for the Proposed Condition, and Appendix D for the Alternate Condition.

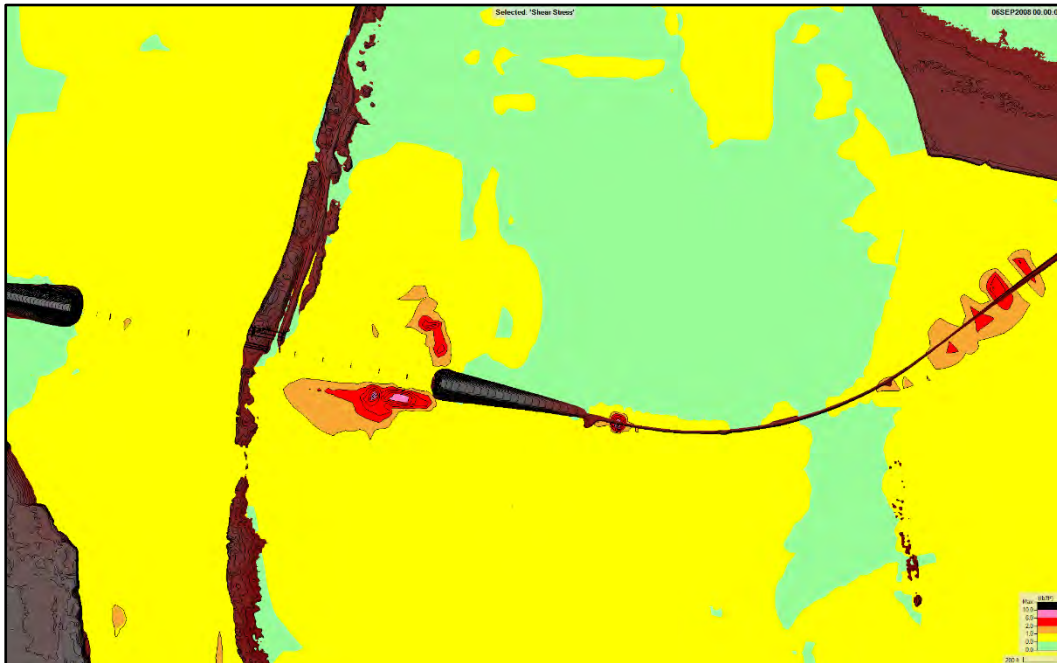


Figure 4-4. Existing Conditions Shear Stress at Q200

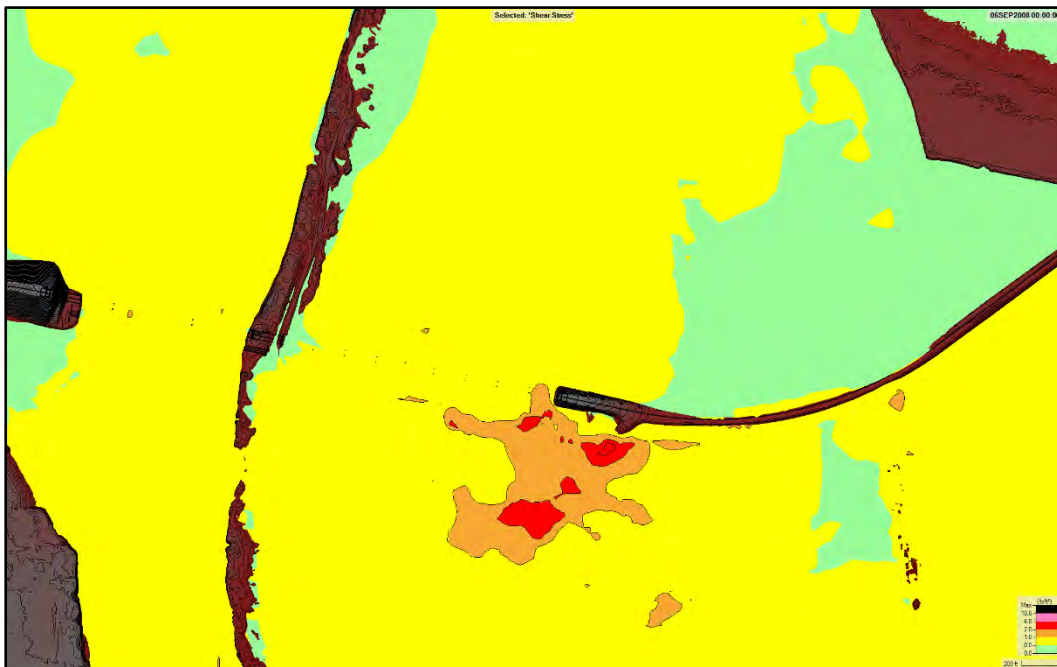


Figure 4-5. Proposed Conditions Shear Stress at Q200

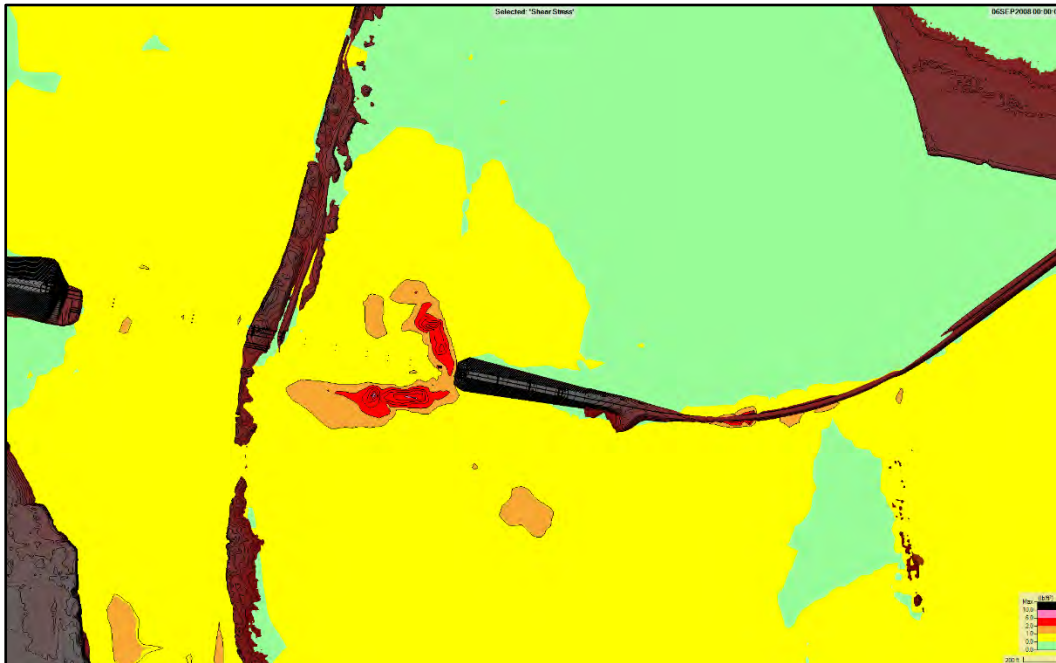


Figure 4-6. Alternate Conditions Shear Stress at Q200

4.5 Degradation and Scour

Long term degradation of the main channel over the 100-year service life of the bridge is estimated to be 7.5 feet. See section 3.12 of the *Bridge Report* for additional details on degradation. Long term degradation will be applied at Piers 2, 3, and 4.

Scour computations were performed in accordance with FHWA's Hydraulic Engineering Circular No. 18 (HEC 18) (Reference 13). Appendix E contains the scour calculations and diagrams showing the contraction, local pier scour, and abutment scour. Additional details on abutment front slope and scour countermeasures can be found in those calculations and are discussed in Section 5.4. Table 4-12 summarizes the scour results for the Proposed Condition for the Q200 and Q500 flows.

In the secondary channel, scour calculations were performed assuming "clear water" conditions. Due to vegetative cover and low shear stresses in the secondary channel upstream of the bridge, it was assumed that there would be insufficient supply of sediment for "live bed" conditions. For contraction and pier scour in the overbank, the calculation was split into three distinct "zones" based upon terrain breaks and areas of similar flow patterns observed in the 2D model. Figure 4-3 shows the three zones.

Abutment 1 and Pier 1 are located above the Q500 water surface elevation. Abutment 1 and Pier 1 will be designed for the minimum prescriptive scour depths in the *NDOT Hydraulic Guidelines*, assuming sand fill material.

Scour at Abutment 2 was calculated using the NCHRP method as described in HEC 18. The resultant scour is the maximum scour depth at the deepest point of the abutment scour hole. HEC 18 does not provide explicit guidance on a method for estimating scour at the face of the abutment. The scour at the face of Abutment 2 was determined by estimating the location of the deepest point of the scour hole at a distance "R" away from the abutment and applying an assumed failure slope back towards the abutment. The reported scour in Table 4-3 is the vertical distance from the initial "top of riprap" elevation at the berm to the intersection of the failure slope and the abutment face. See Appendix E for the scour diagram. This abutment scour design approach was presented to Iowa and Nebraska DOT during a meeting on June 11, 2025.

The effect of riprap countermeasures in reducing max scour depth was not included at either piers or abutments. The effect of a proposed riprap apron at Abutment 2 (see Section 5) was used in the estimation of the distance “R”.

Pier 13 is located near the location of the theoretical maximum abutment scour location (a distance “R” away from the abutment). The reported scour at Pier 13 is therefore conservatively assumed to be the maximum abutment scour from the NCHRP method.

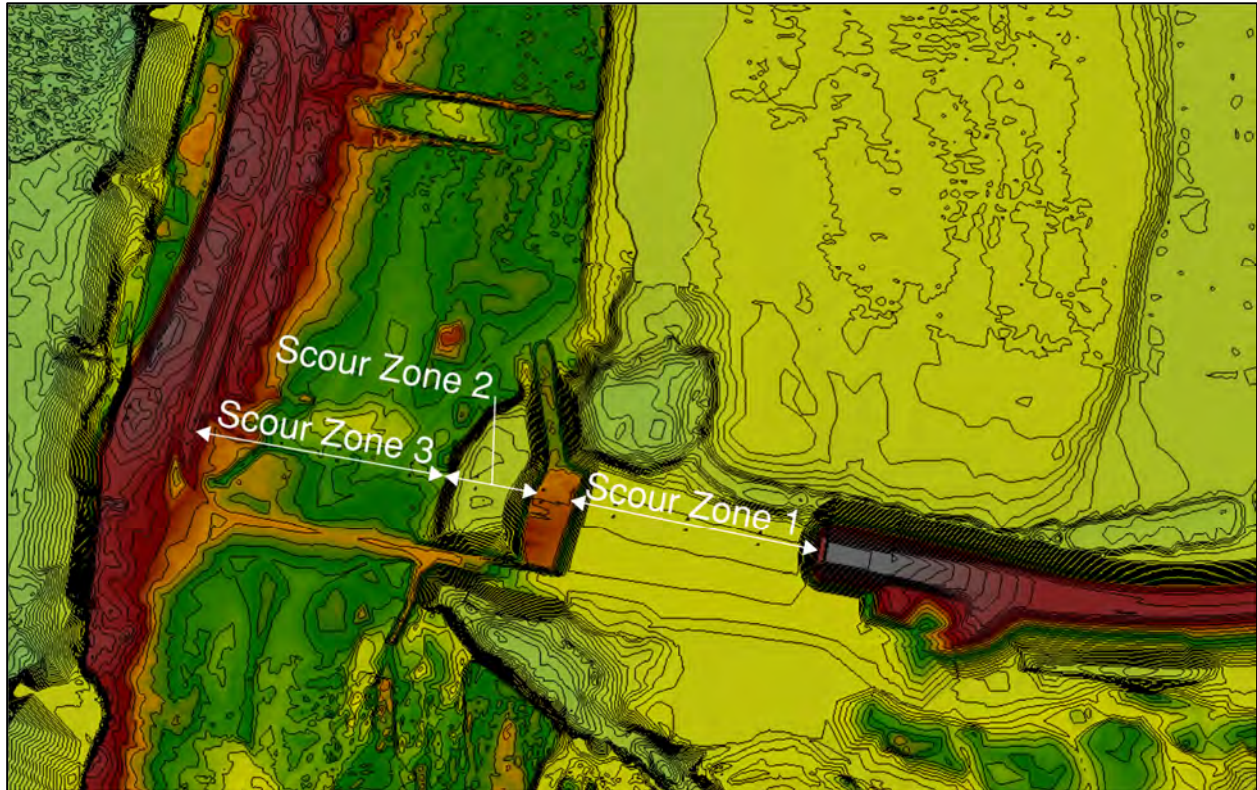


Figure 4-7. Iowa Overbank Scour “Zones”

Table 4-12. Scour Results for Proposed Condition.

Location	Proposed Ground Elevation (ft.)	Degrad. (ft.)	Q200 Calculated Design Scour				Q500 Calculated Check Scour			
			Contraction (ft.)	Local (ft.)	Total Scour + Degrad. (ft.)	Scour + Degrad. Elev.	Contraction (ft.)	Local (ft.)	Total Scour + Degrad. (ft.)	Scour + Degrad. Elev.
**Abut. 1	1094.0	0.0	0.0 / 6.0*	0.0	6.0	1088.0	0.0 / 12.0*	0.0	12.0	1082.0
Pier 1	1063.4	0.0	0.0 / 6.0*	0.0	6.0	1057.4	0.0 / 12.0*	0.0	12.0	1051.4
Pier 2	1035.1	7.5	3.2 / 6.0*	15.6	29.1	1006.0	3.6 / 12.0*	15.8	35.3	999.8
Pier 3	1018.9	7.5	3.2 / 6.0*	23.1	36.6	982.3	3.6 / 12.0*	23.2	42.7	976.2
Pier 4	1057.4	7.5	3.2 / 6.0*	7.2	20.7	1036.7	3.6 / 12.0*	7.8	27.3	1030.1
Pier 5	1055.2	0.0	0.0 / 6.0*	6.9	12.9	1042.3	0.0 / 12.0*	7.1	19.1	1036.1
Pier 6	1046.5	0.0	0.0 / 6.0*	6.9	12.9	1033.6	0.0 / 12.0*	7.1	19.1	1027.4
Pier 7	1045.2	0.0	0.0 / 6.0*	6.9	12.9	1032.3	0.0 / 12.0*	7.1	19.1	1026.1
Pier 8	1044.9	0.0	0.0 / 6.0*	6.9	12.9	1032.0	0.0 / 12.0*	7.1	19.1	1025.8
Pier 9	1031.0	0.0	4.7 / 6.0*	15.7	21.7	1009.3	8.0 / 12.0*	16.1	28.1	1002.9
Pier 10	1049.5	0.0	4.7 / 6.0*	15.7	21.7	1027.8	8.0 / 12.0*	16.1	28.1	1021.4
Pier 11	1039.8	0.0	30.1	17.2	47.2	992.5	31.8	17.4	49.3	990.6
Pier 12	1039.8	0.0	30.1	17.2	47.2	992.5	31.8	17.4	49.3	990.6
Pier 13	1039.8	0.0	30.1	17.2	70.9***	968.9	31.8	17.4	72.5***	967.3
**Abut. 2	1054.1	0.0	1.0 / 6.0*	0.0	6.0	1048.1	0.0 / 12.0*	0.0	12.0	1042.1

Note: NCHRP Abutment Scour is an amplification of Contraction Scour. Local = 0 feet.

*Minimum contraction scour per NDOT Hydraulic Analysis Guidelines for sand bed streams.

**Total Abutment Scour at Face of Abutment

*** Total scour at Pier 13 controlled by maximum Abutment Scour from NCHRP equation

Zone 1 Scour

Zone 2 Scour

Zone 3 Scour

Scour Comparison between Proposed and Alternate

For the Alternate Condition, contraction, pier, and abutment scour were calculated in Zone 2. Due to the extreme velocities and shear stresses, scour values were calculated for two cases. Case 1 assumes the dikes remain intact. Case 2 assumes the dikes fail down to the level of the existing ground between them.

Figure 4-5 compares the max total scour at a pier for the Proposed Condition and both Alternate Condition cases. The Proposed results are from Zone 1 and the Alternate results are from Zone 2. For events at Q70 and above, the max pier scour for the Proposed and Alternate Case 1 Conditions are similar. The max pier scour for Alternate Case 2 is much higher.

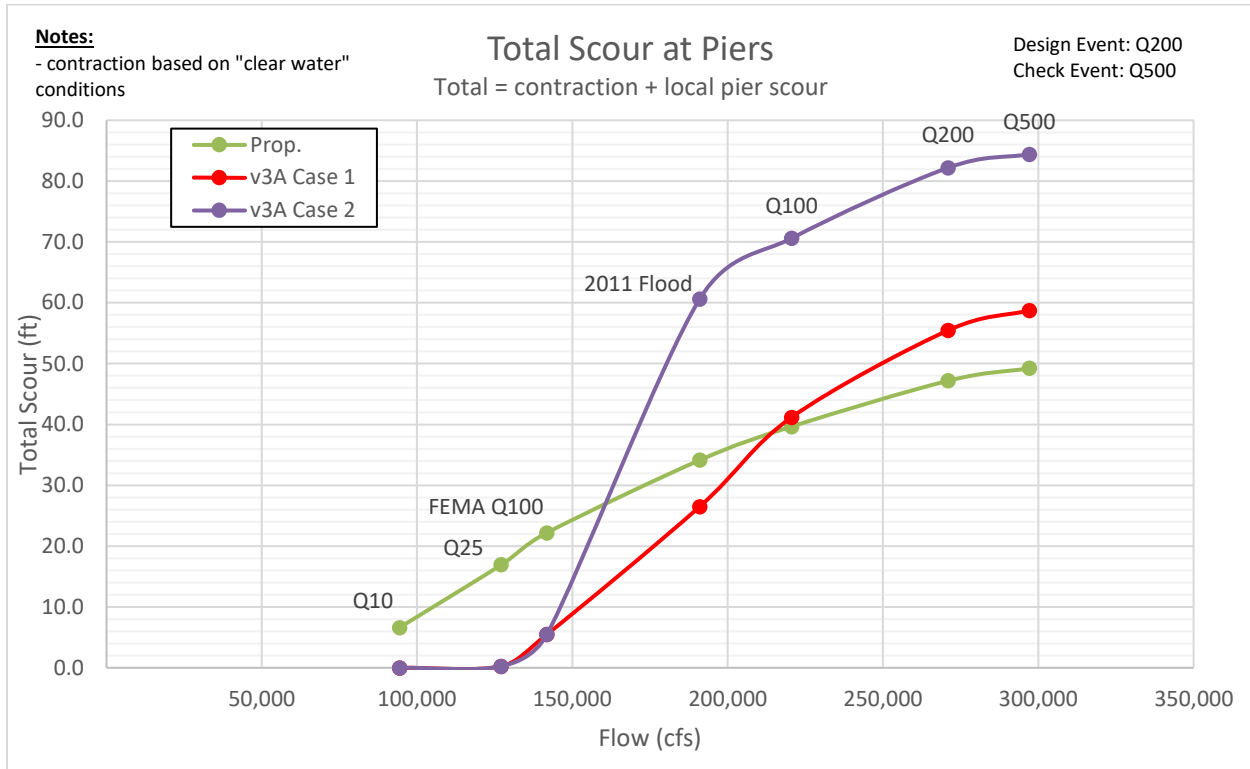


Figure 4-8. Proposed vs Alternate Total Scour Comparison.

Table 4-13. Max Abutment Scour at Q200 (per NCHRP Method).

Condition	Scour (ft.)
Proposed	70.9
Alternate Case 1 (dikes remain intact)	66.1
Alternate Case 2 (dikes fail)	102.9

Table 4-13 compares the maximum abutment scour at Abutment 2 for the Proposed and Alternate cases. Several conclusions can be drawn from Figure 4-5 and Table 4-13:

- 1) As long as the dikes remain intact (Alternate Case 1), the Alternate and Proposed Conditions result in similar predictions of max abutment scour. For pier scour, Alternate Case 1 would perform better than the Proposed at flows below Q100, and the Proposed Condition would perform better for flows above Q100.
- 2) However, if the dikes begin to fail (Alternate Case 2), the Proposed Condition performs significantly better than the Alternate Condition for both pier and abutment scour at all large flows. With the extreme shear stresses in the Alternate Condition, dike failure could be initiated by either undercutting around the riprap or erosion of the riprap itself.

4.6 Flow Distribution

As discussed in the *Flood Resiliency Study*, lengthening the bridge results in a change in the flow distribution between the main channel and the secondary channel. Additional flow is drawn to the secondary channel. Table 4-14 shows the flow distribution for the Existing, Proposed, and Alternate Conditions. This table also shows the removal of roadway overtopping at the Q100.

Table 4-14. Flow Distribution at Q100

Plan	Percentage of Flow in Conveyance Area			
	Main Channel	Secondary Channel	Roadway Overtopping	Spillage
Existing Condition	85.2%	10.6%	3.4%	0.8%
Proposed Condition	73.7%	26.2%	0.0%	0.1%
Alternate Condition	85.7%	13.3%	0.0%	1.0%

4.7 1D Results

Table 4-15 below shows the comparison of water surface elevation between the Existing and Proposed Conditions. The results show behavior similar to the 2D model: a decrease in water surface elevation upstream, followed by a rise within the vicinity of the bridge and continuing downstream until the main and secondary channels converge. The magnitude of the rise or fall in water surface elevation differs between the 2D and 1D models.

The effective 1D model does not incorporate a split flow condition. The same water surface elevation is reported for both the main and secondary channels. The downstream rise shows up in the main channel and would affect insurable structures along the Nebraska bank. Based on the 2D results, it is anticipated that revision of the 1D model to incorporate the split flow condition would limit the rise to the secondary channel. See Section 5.2 for recommended next steps for floodplain permitting.

For additional input and results such as flowline profile water surface elevation, flow output table, and cross-section plots see Appendix F.

Table 4-15. FEMA Q100 Water Surface Elevation Comparison: 1D Model

Location	River Mile – Model Station	Duplicate Effective Model WSE (ft.)	Corrected Effective Model WSE (ft.)	Proposed Conditions Model WSE (ft.)	Proposed Rise (ft.)
U/S	693.78	1049.11	1048.97	1048.95	-0.02
U/S	693.39	1048.79	1048.64	1048.62	-0.02
U/S	692.98	1048.62	1048.45	1048.43	-0.02
U/S	692.58	1048.55	1048.38	1048.37	-0.01
U/S	692.17	1048.3	1048.12	1048.10	-0.02
U/S	691.75	1047.8	1047.87	1047.85	-0.02
U/S	691.35	1047.37	1047.51	1047.49	-0.02
U/S	691.2	-	1047.50	1047.48	-0.02
U/S	691.1	-	1047.21	1047.28	0.07
U/S	691.04	1046.99	-	-	-
Prop/Exist Bridge	691.035	-	-	-	-
D/S	691.03	1046.96	-	-	-
D/S	691	-	1046.83	1046.88	0.05
D/S	690.95	1046.89	1046.85	1046.89	0.04
D/S	690.52	1046.33	1046.31	1046.31	0.00

D/S	690.13	1045.86	1045.94	1045.94	0.00
D/S	689.72	1045.52	1045.62	1045.62	0.00
D/S	689.31	1045.01	1045.01	1045.01	0.00
D/S	688.91	1044.66	1044.65	1044.65	0.00
D/S	688.49	1044.38	1044.38	1044.38	0.00
D/S	688.08	1043.98	1043.98	1043.98	0.00

4.8 Risk Assessment

Table 4-16 below summarizes the impacts of the Alternate and Proposed conditions on various hydraulic risks.

Table 4-16. Risk Assessment for Structure S051 03644.

Risk	Alternate	Proposed
Scour in Main Channel	Both the Proposed and Alternate Conditions result in reduced velocities and shear stresses in the main channel relative to the Existing Condition. The scour risk for piers in the main channel would be reduced by both.	
Scour in the Secondary Channel	The Alternate piers would be founded on drilled shafts, which are more scour resilient than the existing pile footings. In the secondary channel, the Alternate Condition is similar to the Existing Condition. Flow over the dikes is activated around Q40. The secondary channel is contracted to a very narrow area. For flows at Q70 and above, the shear stresses and velocities in the dike region (Zone 2) are extremely high. There is a meaningful risk of erosion of the riprap protection. There is also a risk of undermining around the edges of the dikes. Either could lead to failure of the dikes. Dike failure would allow a greater volume of flow through the same contracted area, significantly increasing the predicted scour depth. The Alternate Condition results in comparable scour risk to the Existing Condition.	The Proposed piers would be founded on drilled shafts, which are more scour resilient than the existing pile footings. For flows above Q70, the Proposed Condition results in lower velocities and shear stresses around the bridge, as compared with both the Existing and Alternate Condition. Lower shear stress decreases the likelihood of the initiation of scour. For the design and check events, the predicted max scour for the Proposed Condition is somewhat lower than the Alternate Case 1 condition, in which the dikes remain intact. However, if the dikes begin to fail (Alternate Case 2), the Proposed Condition predicts considerably less scour. For the design and check scour, the Proposed Condition results in lower scour risk than both the Existing and Alternate Conditions. For flows between Q5 and Q40, flow is activated for the Proposed Condition, but not the Existing or Alternate Condition. Therefore, for flows in this range, the Proposed Condition technically results in an increased scour risk in Zone 1 (the bench area). However, for events up to the effective FEMA Q100, shear stresses in Zone 1 are very low, so the probability of scour initiation is low. The Proposed piers would be designed for the design scour event (Q200), so the risk of structural damage from scour at lower flows is low. Riprap scour countermeasures are recommended at Piers 11-13 and at Abutment 2 (see Section 5). These countermeasures would reduce the risk of scour at all flow events.
Roadway Overtopping	In the Existing Condition, roadway overtopping occurs at Q70. This does not meet the design criteria of no overtopping at Q100 In the Alternate Condition, roadway overtopping occurs at approximately Q170, which meets the design criteria. At Q500, the Alternate Condition is overtopped by approximately 1.37 feet.	In the Proposed Condition, roadway overtopping occurs at approximately Q200, which meets the design criteria. At Q500, the Proposed Condition is overtopped by approximately 0.86 feet. The risk of roadway overtopping is lower than both the Existing and Alternate Conditions.

Table 4-16. Risk Assessment for Structure S051 03644 (Continued).

Risk	Alternate	Proposed
Rise in 100-Year Flood Elevations	At the Effective FEMA Q100, the Alternate Condition results in a downstream rise affecting approximately 200 acres. The area is primarily Iowa DNR wetland and farm fields. No insurable structures are located within the rise region. A CLOMR would be required. The effective 1D model would be used for both the CLOMR and for floodplain permitting. At the new Q100, the Alternate Condition results in a rise both up and downstream, extending to the limits of the 2D model. This is due to the elimination of roadway overtopping relief without a concurrent opening of the secondary channel.	At the Effective FEMA Q100, the Proposed Condition results in a downstream rise affecting approximately 260 acres. The area is primarily Iowa DNR wetland and farm fields. No insurable structures are located within the rise region. A CLOMR would be required. The effective 1D model would be used for both the CLOMR and for floodplain permitting. It would require modification to incorporate the split flow condition At the new Q100, the Proposed Condition results in a rise downstream of the bridge. The rise in the secondary channel decreases to 0.08 feet by the confluence with the main channel. A rise of 0.08 to 0.01 feet extends from the confluence to the downstream limit of the model. The Proposed Condition results in no practical increase in flood risk at the new Q100.
Backwater at Q100	At the new Q100, the Alternate Condition results in a higher backwater than the Existing Condition.	At the new Q100, the proposed condition results in a reduced backwater relative to the Existing Condition.
Pressure Flow / Stream Pressure on Superstructure	For both the Alternate and Proposed Conditions, the Q500 is below the low structure elevation. The Proposed Condition provides 2.21 feet of freeboard over the Q500. The risk of pressurized flow or stream pressure on the superstructure for flows of Q500 and below is negligible.	
Debris and Ice Build-up	For both the Alternate and Proposed Condition, the new piers would be comparable in type, width, and orientation to the existing piers. A strut would be included at all piers to help transfer ice and debris load between the upstream and downstream columns. No increase in the risk of debris or ice build-up is anticipated.	
Iowa DNR Access Road Overtopping	The Alternate Condition would maintain the elevation of the Iowa DNR access road. The access road would be overtopped and closed by approximately a Q40. There would be no increase in the frequency of this road being overtopped and closed.	The Proposed Condition would lower the Iowa DNR access road. The road would be overtopped and closed by approximately a Q5 event. Per the USGS gage record, a Q5 or higher has occurred approximately 12 times since 1988, or approximately once every 3 years. Flow has exceeded the Q5 on approximately 3.7% of days in the record. Increased flow over the access road may cause an increased frequency of washout damage to the road. However, for flows up to the effective FEMA Q100, which is comparable to the floods in 2019 and 2024, peak shear stress on the access road remains low. To protect the road and reduce the risk of washout, the "bench" portion of the access road would be paved and the edges of the road would be protected with a strip of riprap.

4.9 Recommendations

The Proposed Condition is recommended to be carried forward for final design. It demonstrates greater resiliency against flood-induced closure than the Existing or Alternate Conditions. The Proposed Condition demonstrates improved scour performance for large floods and a lower risk of scour-induced closure than the Existing or Alternate condition. The Proposed Condition meets freeboard requirements and presents a reduced risk of roadway overtopping relative to the Existing Condition. The Proposed Condition will require a CLOMR. However, the rise at the Effective FEMA Q100 is anticipated to be limited to the secondary channel. It is not anticipated to affect any insurable structures.

This analysis has been performed for the Blue Alternative. However, the same bridge length and bench elevation recommendation applies to the Orange Alternative as well. The hydraulic results for a downstream offset alignment would be comparable.

It is recommended in the NEPA and Final Design phases to meet with FEMA, the USACE, and Iowa DNR Wildlife Management to confirm the Proposed Condition with each stakeholder. Iowa DNR and Nebraska DNR floodplain management groups have not stated any concerns in preliminary design meetings regarding opening the secondary channel. Additional consultation with FEMA will be required to confirm the approach to 1D modeling for the CLOMR.

Chapter 5

Design and Permit Information

The *Preliminary Data Sheet* (NDOT Form BR 359A) is included as Attachment 5.

5.1 Water Surface Elevations and Freeboard

Table 5-1 shows the WSE and freeboard for each flood event for the Proposed Conditions. The WSE is taken at the maximum WSE value at the upstream edge of the bridge in the Iowa Overbank which is the controlling location for freeboard. Freeboard values meet or exceed the design requirements of 3 feet over the Q50, 1 foot over the Q100, and 0 feet over the Q500.

Table 5-1. Proposed Freeboard Values in Secondary Channel.

Year	Flow (cfs)	Prop. WSE (ft.)	Prop. Low Beam Elev. (ft.)	Freeboard (ft.)
Q10	94,400	1043.56	1056.62	13.06
Q25	127,100	1046.96		9.66
FEMA Q100	141,800	1048.07		8.55
2011 Flood (~Q70)	191,000	1050.99		5.63
Q100	220,600	1052.29		4.33
Q200	271,000	1053.88		2.74
Q500	297,200	1054.41		2.21

The low road elevation is 1053.78 from PIH Plans, at a location approximately 680 feet east of the new Abutment 2. The overtopping flood is approximately 271,000 cfs, which is a Q200 per the 2023 MRFFS. This meets the project design criteria of no roadway overtopping at the Q100 event. The waterway area below Q100 is approximately 36,790 square feet.

5.2 Floodplain Permitting

Rise in Base Flood Elevations

FEMA, NeDNR, and Iowa DNR all confirmed that the rise evaluation for floodplain permitting should be performed using the effective FEMA 1D model and the effective FEMA Q100. Table 4-2 in Chapter 4 shows the change in WSE from the Existing Condition to the Proposed Condition for the 2D model and Table 4-5 shows the change in WSE for the 1D effective FEMA model. Both 1D and 2D models show a decrease in water surface elevation upstream, a slight rise immediately downstream of the bridge, and a near no-rise condition at the confluence of the secondary and main channels. A CLOMR will be required to address the rise.

The 2D model shows that the downstream rise is limited to the secondary channel due to the split flow condition. No insurable structures are located within the affected area. However, the effective 1D model does not incorporate a split flow condition. The same water surface elevation is reported for both the main and secondary channels. The rise therefore shows up in the main channel and would affect insurable structures along the Nebraska bank.

To address this issue, the effective 1D model should be modified to show a split flow condition by addition of a parallel reach for the secondary channel. Before this additional modeling is performed, coordination with FEMA is required to confirm that this modeling approach would be acceptable for the CLOMR.

Iowa DNR Backwater Requirement

In a meeting on May 5, 2025, Iowa DNR confirmed that the backwater requirement should be evaluated at the new Q100 (2023 MRFFS). Tables 4-6 and 4-7 summarize the backwater for the Existing and Proposed Conditions. The Existing Condition greatly exceeds the Iowa DNR maximum backwater limit of 1 foot, in both the main channel and secondary channel. The Proposed Condition also does not meet the 1 foot requirement, but it significantly improves the backwater.

Further lengthening of the bridge to meet the backwater requirement is not feasible. The bridge length would have to increase by an additional 500 feet or more, resulting in a significant change in hydraulic impacts and increase in cost. A variance should be requested from Iowa DNR on the basis that the Proposed Condition significantly improves the Existing backwater.

5.3 Design Velocity for Stream Pressure Force

The Q200 and Q500 velocities for design forces are shown in Table 5-2. The three zones shown in the overbank are the same as the design scour zones are shown in Figure 4-3. The abutment and the superstructure are not submerged and would not experience stream pressure force.

Table 5-2. Stream Pressure Design Velocities.

Location	Design Velocity (ft/s)	
	Q200	Q500
Main Channel	8.0	8.1
Overbank "Zone 1"	10.4	10.7
Overbank "Zone 2"	4.1	4.3
Overbank "Zone 3"	2.1	2.2

5.4 Berm, Front Slopes, and Riprap

At Abutment 1, the proposed top of riprap elevation at the berm is 1094.0. This provides 2 feet of clearance below the low structure elevation. The berm length is 10 feet. The proposed front slope is 2:1 down to approximate elevation 1058. The bridge will have vertical drop deck drains. To protect the 2:1 slope from erosion, it will be armored with riprap 2 feet thick. The riprap will extend 5 feet outside the edges of the bridge deck. The riprap will be placed across the entire front slope, rather than just at the edges, due to the maintenance difficulty of mowing grass on the tall 2:1 slope. Riprap at the west abutment should meet the requirements of typical NDOT "Broken Concrete Riprap." See TS&L Plans.

Riprap revetment along the west bank of the main channel will be temporarily removed during construction of Pier 2, and will be replaced around the pier when complete, per USACE requirements.

After removal of existing Pier 1 on the west bank of the main channel, the bank will be regraded and armored with riprap revetment per USACE requirements.

At Abutment 2, the proposed top of riprap elevation at the berm is 1054.11. This is approximately 0.23 feet above the design flood of Q200, and provides approximately 2.74 feet of clearance below the low structure elevation. The berm length is 10 feet. The proposed front slope is 3:1 down to the "bench" area. The toe of the front slope will follow the 1% grade of the bench from south to north. The front slope will be armored with riprap 2 feet thick. A riprap apron will extend 65 feet from the toe of slope, and will wrap around the abutment. The apron will extend 80 feet east of the Abutment 2 centerline on both the upstream and downstream side slopes. Riprap for the front slope and apron should meet similar requirements to the

Iowa Class X Revetment used in the 2011 repair. A custom NDOT riprap Special Provision would be prepared.

Piers 11, 12, and 13 will each have a rectangular riprap apron. The apron will be 4.5 feet thick and will extend approximately 15 feet from the face of pier in all directions. At Pier 13, the apron will be connect to the abutment apron. Riprap at the piers should meet the same requirements as the riprap for Abutment 2.

A new riprap strip approximately 15 feet wide would be placed along each edge of the Iowa DNR access road. Typical NDOT "Broken Concrete Riprap" could be utilized in this location. Some riprap around the existing north and south dikes will need to be removed to complete grading. This riprap may be replaced and utilized in other locations.

5.5 Removals

The existing river pier (Pier 2) shall be removed to the lower of:

- 1) Elevation 1013.52, or
- 2) 2 feet below the lowest point in the channel at the existing pier location at the time of construction, including any scour holes at the pier.

It is recommended that an updated bathymetric survey be performed within 12 months before construction letting to provide bidding contractors with up-to-date bathymetry. The final Pier 2 elevation will be determined at that time. The existing bank pier (Pier 1) removal elevation will be determined during Section 408 coordination with the USACE. Removal limits of Nebraska and Iowa approach piers will be two feet below finished groundline in accordance with typical NDOT specifications.

The existing west embankment in front of the Abutment 1 front slope will be removed to approximately elevation 1057, as shown in the TS&L Plans. The existing east embankment and surrounding terrain shall be removed to an elevation of 1041 along the Iowa DNR roadway and slope north at 1% until it meets up with the existing terrain. The area around the existing east abutment will be removed to approximate elevation 1050 and will tie into the existing north dike. This embankment will slope at a 2:1 (H:V) on the east and south side of the embankment until it meets up with the proposed "bench" area. See TS&L Plans.

5.6 Pier Struts

Pier struts are anticipated for structural design due to scour risk, ice load on all piers, and vessel collision on Piers 2, 3, and 4. See TS&L Plans for preliminary strut locations and dimensions.

5.7 Temporary Structures in the Channel

It is anticipated that the Contractor will use a temporary structure in the non-navigation span of the main channel. During low flow seasons, the river in this span may be too low for barge operations. A temporary structure provides reliable access for drill rigs, crane operations, and girder and concrete delivery.

It is anticipated that the Contractor will prefer to utilize a causeway constructed of riprap and topped with gravel. The top of the causeway would be several feet above OHW. The causeway would extend from the east bank at OHW to Pier 3. If a causeway is not approved during the permitting process, the Contractor would alternatively may utilize an access bridge. The bridge would extend the same length as a causeway. The low structure of the access bridge would be a minimum of 3 feet above OHW, in accordance with typical NDOT requirements.

The Contractor may construct a small causeway at the new west bank pier (Pier 2) to facilitate access for drilled shaft installation.

The Contractor may construct a temporary dock for loading and unloading working spud barges.

Falsework bents are anticipated to be required for erection of girders in the River Unit. Several falsework bents may be constructed in Spans 2 and 4, including in the channel. At least one falsework bent is anticipated in Span 3 (the navigation channel).

The use of cofferdams is unlikely. If the water level exceeds OHW for an extended period during construction, there is a small possibility that the Contractor would utilize a cofferdam for construction of the struts at Pier 2 and 3, or for removal of the existing bank and river piers.

5.8 Ice

The bridge is subject to ice load on the River Unit and Iowa Approach Units. Criteria for ice load were established in the Phase 1 *Bridge Report*. Design crushing strength will be 24 kips per square foot (ksf). Design thickness will be 24 inches. Ice load will be applied concurrently with the Scour Design Flood (Q200) and all submerged components will be checked at this flood level.

Chapter 6

References

References reviewed in the preparation of this report are summarized as follows.

- 1) Record drawings of the Decatur Bridge and associated projects provided by NDOT to Burns & McDonnell
- 2) Record drawings of the Decatur Bridge and associated projects provided by Iowa DOT to Burns & McDonnell
- 3) Flood Insurance Study 31021CV000A, Burt County, Nebraska and Incorporated Areas, FEMA, June 5, 2020.
- 4) Flood Insurance Study 19133CV000B, Monona County, Iowa and Incorporated Areas, FEMA, June 15, 2022.
- 5) Effective HEC-RAS Model: Missouri River from Yankton, SD to Rulo, NE, FEMA Engineering Library, April, 2025
- 6) Missouri River Flow Frequency Study: Yankton, South Dakota to Hermann, Missouri, USACE, June 2023.
- 7) NDOT Hydraulic Analysis Guidelines
- 8) Iowa DOT Bridge Design Manual
- 9) Decatur Bridge Location and Environmental Study Bridge Report, Rev. 1, Burns & McDonnell Jan. 2025
- 10) Decatur Bridge Flood Resiliency Study, Burns & McDonnell, Rev. 1, Jan. 2025
- 11) Decatur Bridge Vessel Collision Study, Burns & McDonnell, Rev. 1, Jan. 2025
- 12) NDOT Hydrologic Analysis for Structure S008 14151
- 13) Hydraulic Engineering Circular (HEC) No. 18, Scour at Bridges, 2012
- 14) Hydraulic Engineering Circular (HEC) No. 20, Stream Stability at Highway Structures, 2012
- 15) Hydraulic Engineering Circular (HEC) No. 23, Bridge Scour Countermeasures, 2009
- 16) NCHRP 24-19a (587), Bridge Abutment Scour Countermeasures, 2007
- 17) NCHRP 24-20, Abutment Scour, 2010
- 18) Countermeasures & Toe Protection for Spill-Through Abutments, Melville, 2006
- 19) FHWA Hydraulic Toolbox Version 5.4

Appendix A – Site Photographs



On Bank at Pier 1, Looking Upstream.



On Bank at Pier 1, Looking Downstream.



Main Channel Span, Looking Northeast

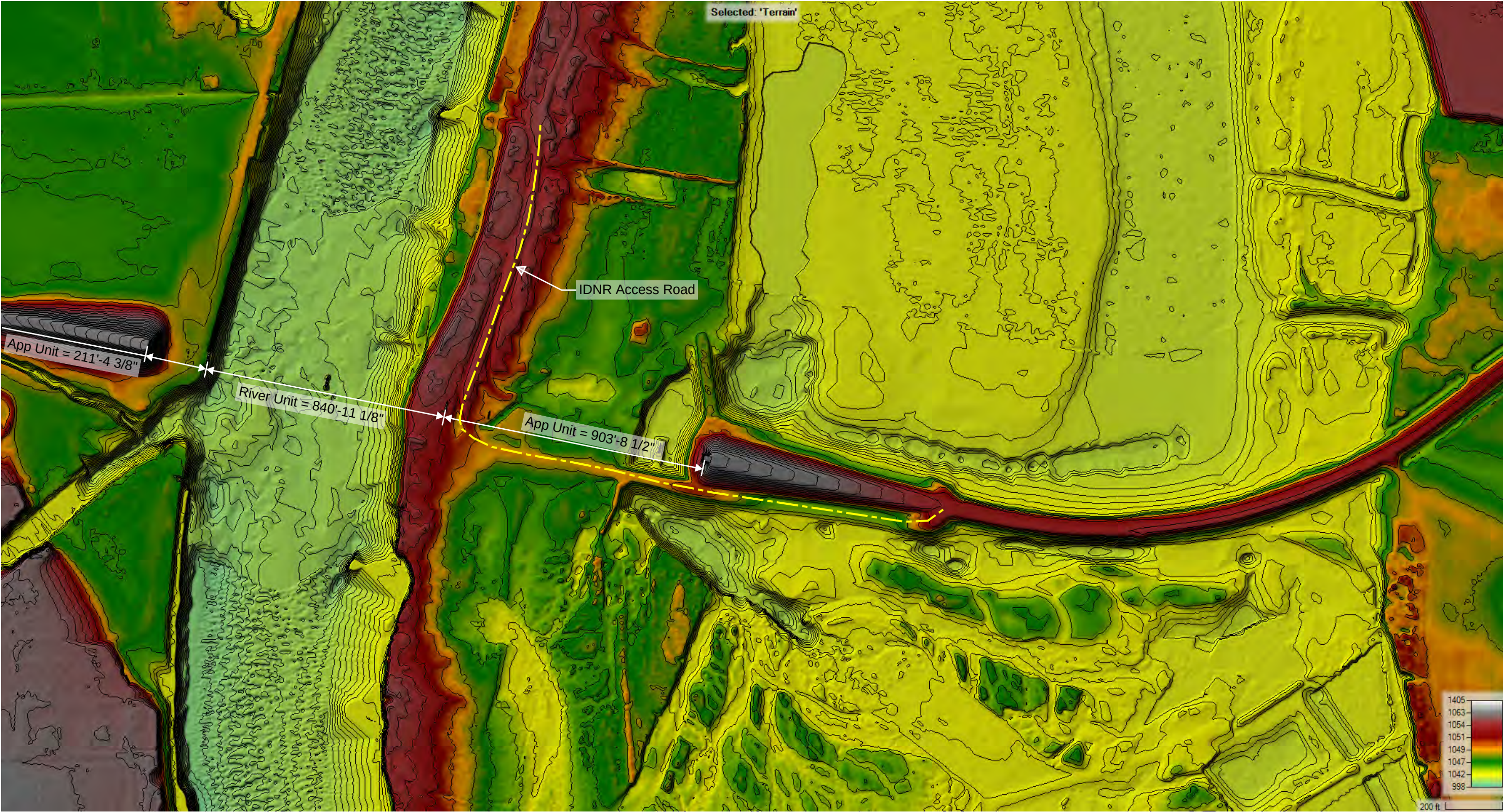


Iowa Approach Span, Looking Southwest

Appendix B – Terrain Plots

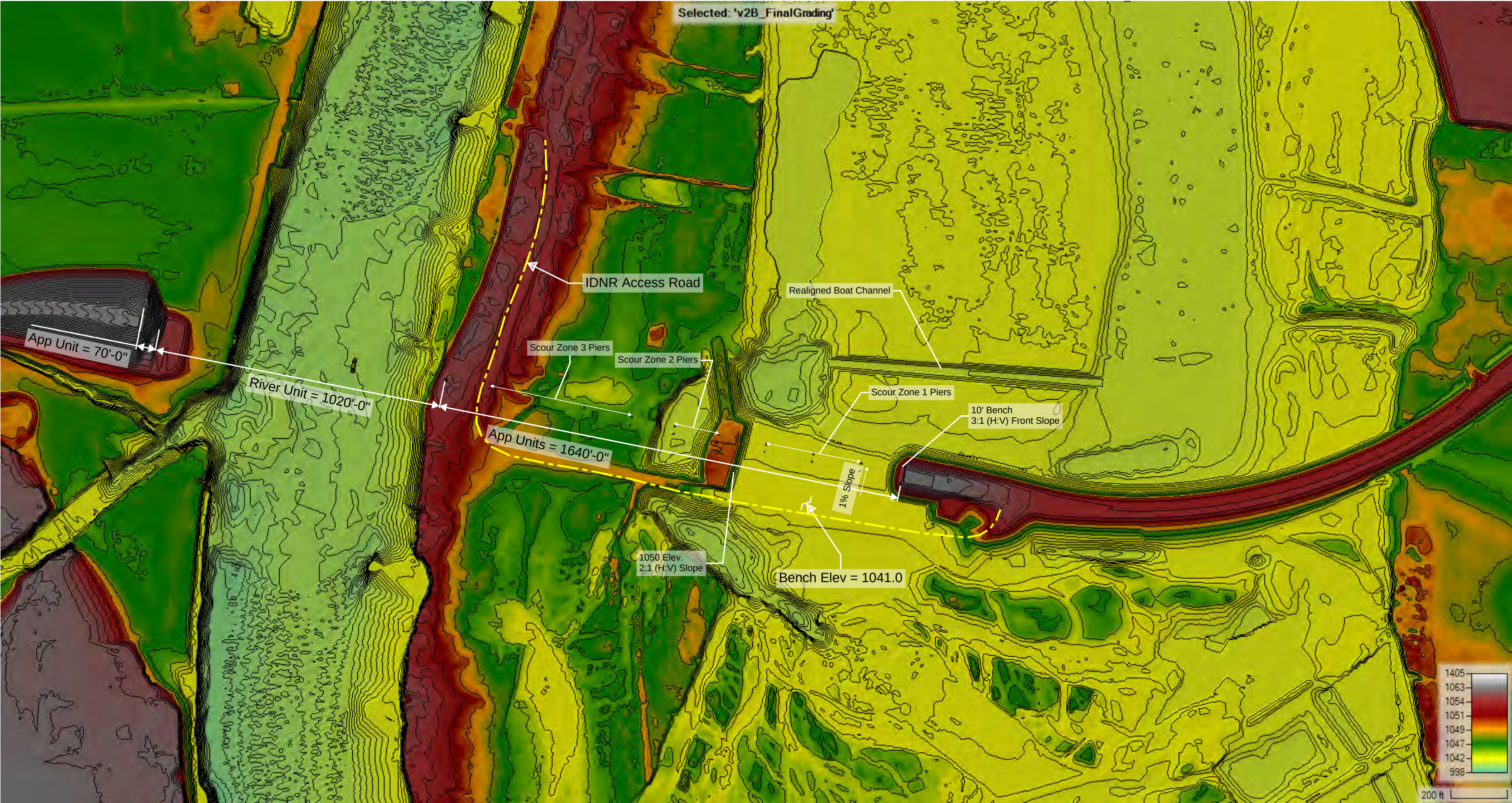
Existing Condition

Total length of Iowa Approach Units = 903'-8 1/2"
New abutment re-location east of existing abutment = n/a
Bench Elevation = n/a



Proposed Condition (refinement of Strategy v_2B)

Total length of Iowa Approach Units = 1,640'-0"
New abutment re-location east of existing abutment = 650'-0"
Bench Elevation = 1041.0



Alternate Condition - Case 1 (Strategy v_3A w/ dikes intact)

Total length of Iowa Approach Units = 1,145'-0"

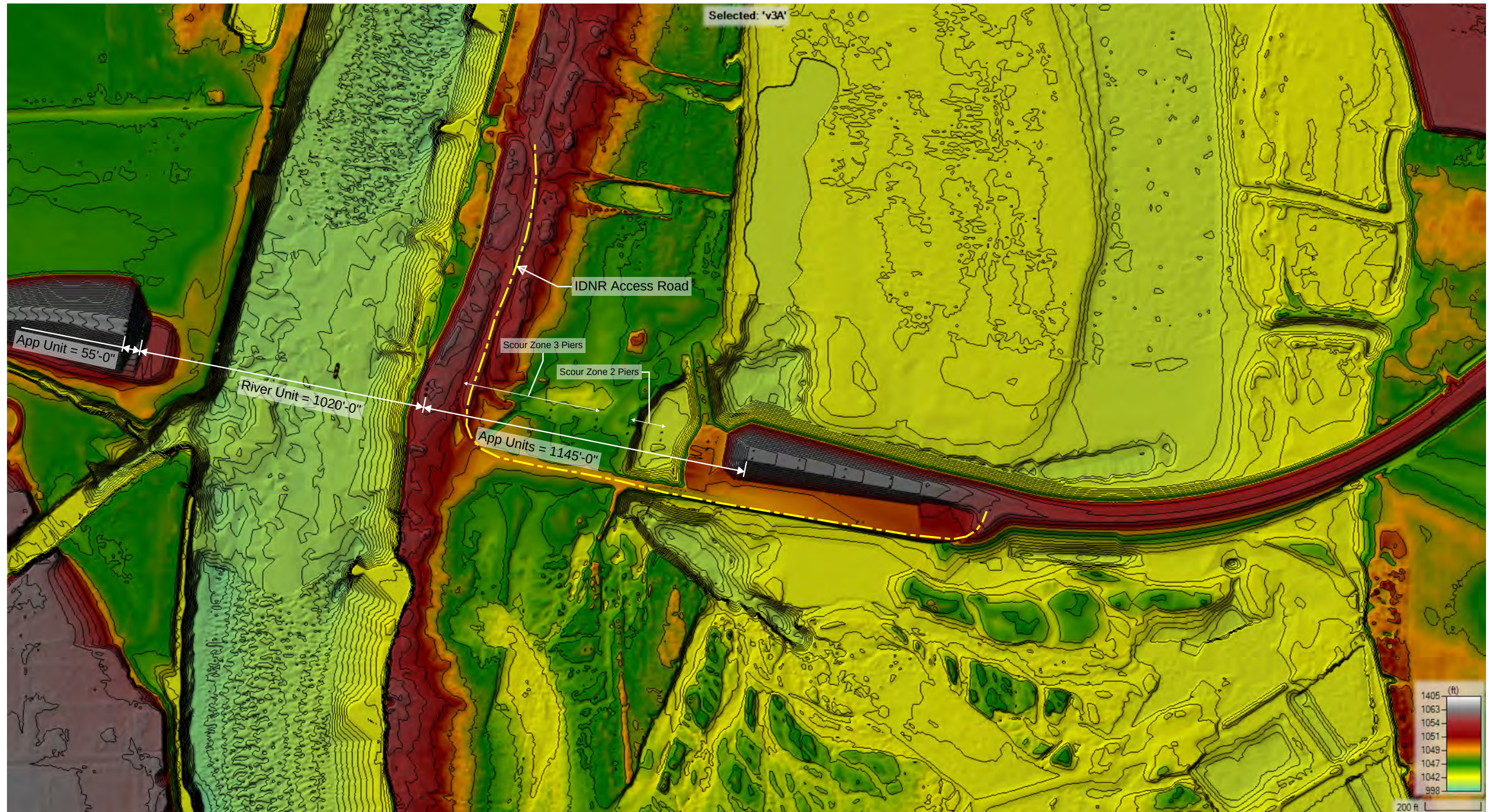
New abutment re-location east of existing abutment = 155'-0"

Bench Elevation = n/a

Notes:

- This case represents the shortest bridge that would maintain the existing hydraulic opening over the North Dike
- **This case is not recommended due to the upstream rise at Q100, which would impact several residences and other structures.**
- No improvement in hydraulic performance relative to Existing Conditions
- Included in analysis only as a alternate condition

B.3



Alternate Condition - Case 2 (Strategy v_3A w/ failed dikes)

Notes:

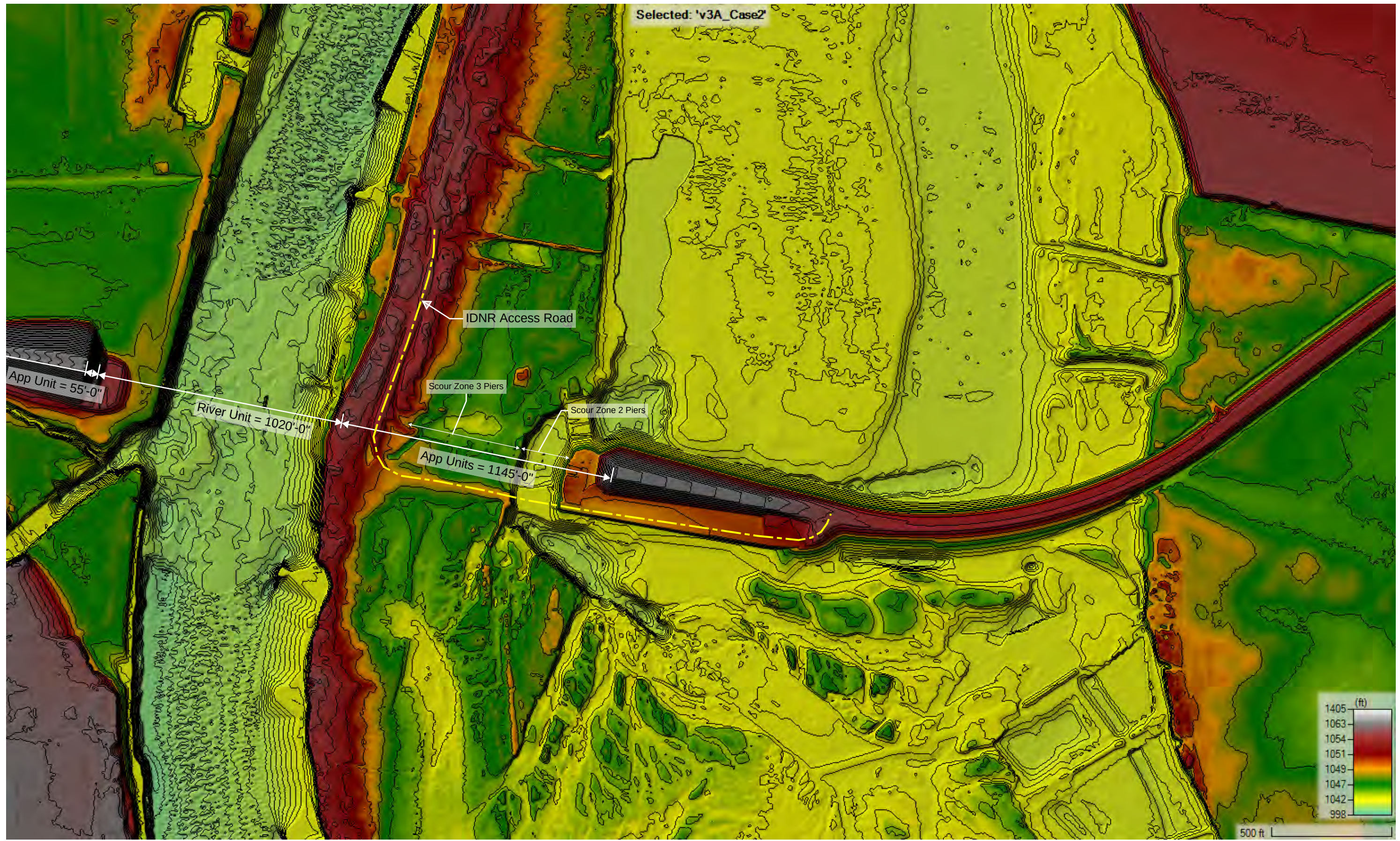
- Due to high velocity and shear stress over dikes, this case is added as a comparison to see what would happen for the Alternate Condition if the dikes fail.

B.4

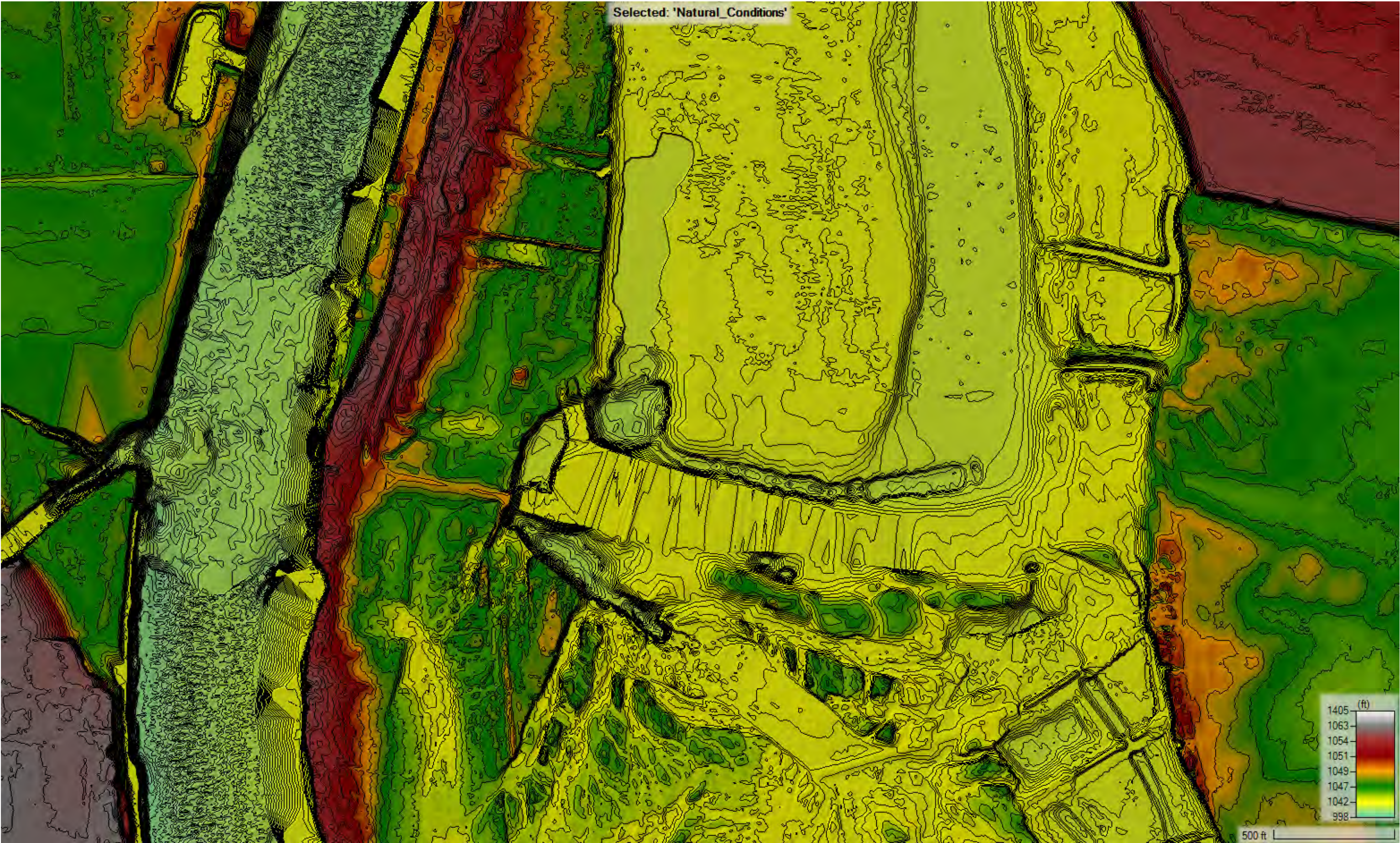
Total length of Iowa Approach Units = 1,145'-0"

New abutment re-location east of existing abutment = 155'-0"

Bench Elevation = n/a



Natural Condition



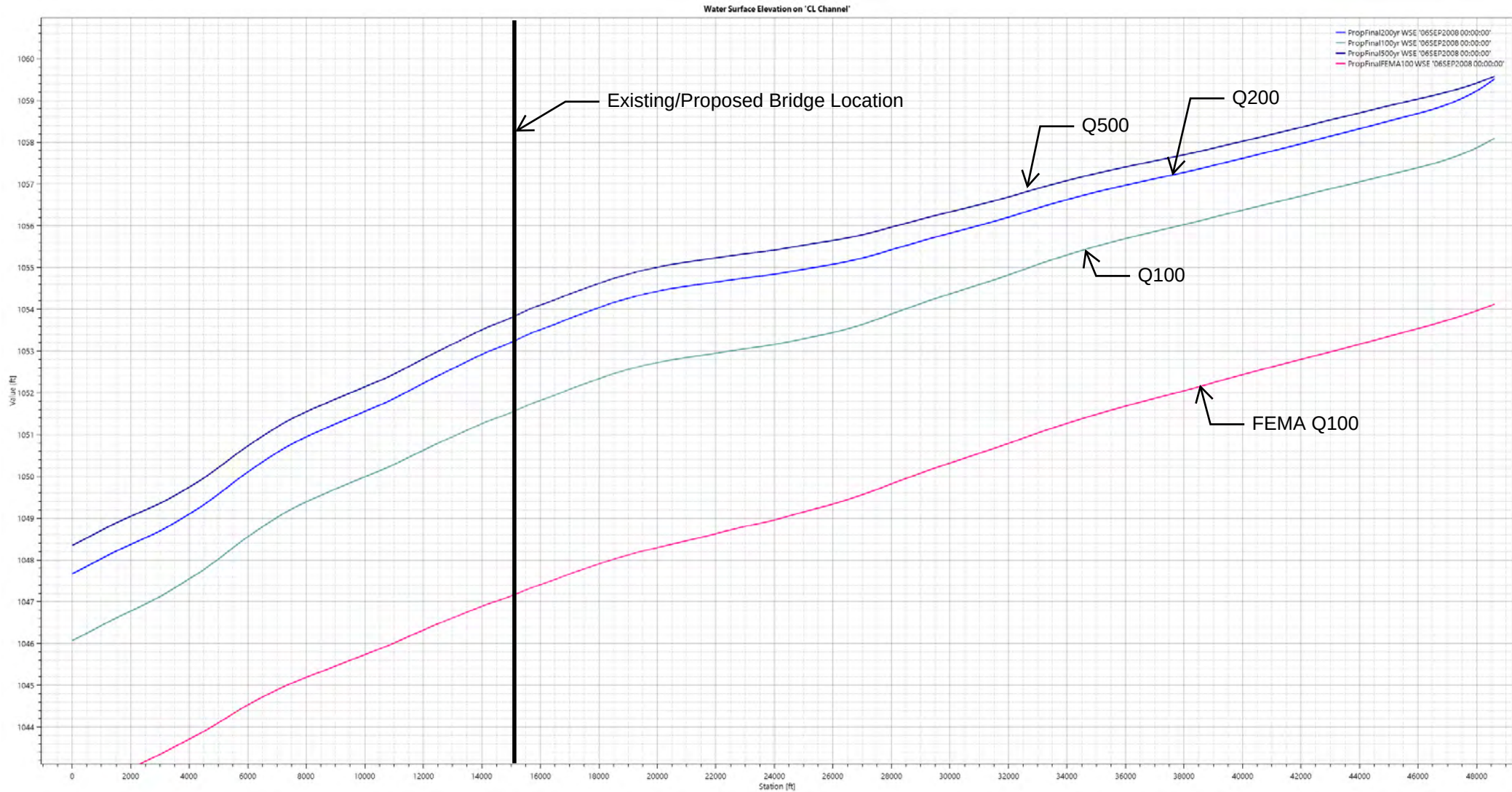
Appendix C – Proposed 2D Results

Proposed Conditions - WSE Profile Along CL Main Channel

C.1

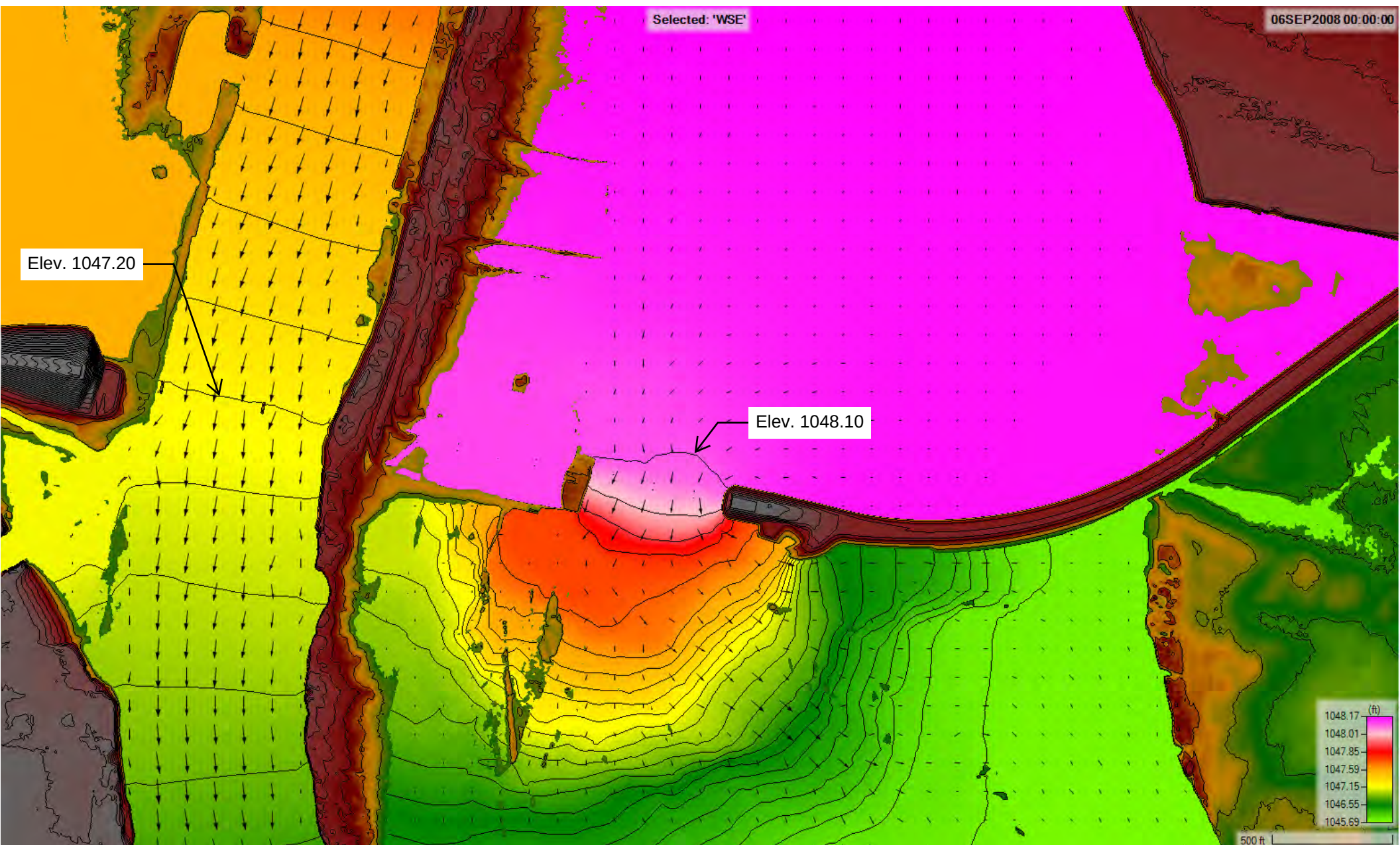
Notes:

Q100 thru Q500 are interpolated from the 2023 MRFFS.
FEMA Q100 is the effective flow in the FIS.

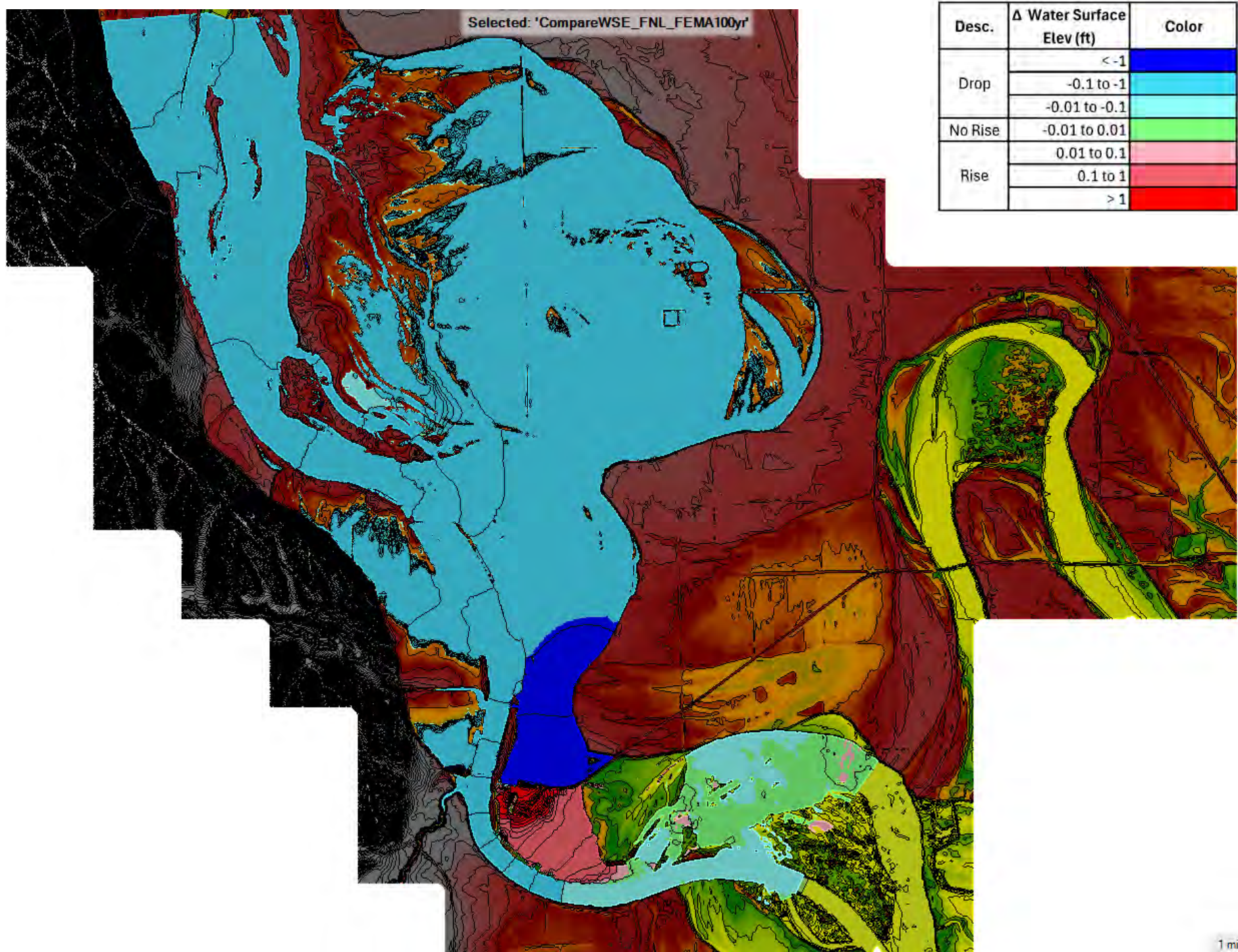


Proposed Conditions FEMA Q100 WSE with 0.1' Contours

C.2

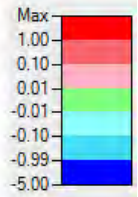


Proposed - Change in WSE vs. Existing Condition
Effective FEMA Q100



Desc.	Δ Water Surface Elev (ft)	Color
Drop	< -1	Dark Blue
	-0.1 to -1	Light Blue
	-0.01 to -0.1	Very Light Blue
No Rise	-0.01 to 0.01	Green
Rise	0.01 to 0.1	Pink
	0.1 to 1	Red
	> 1	Dark Red

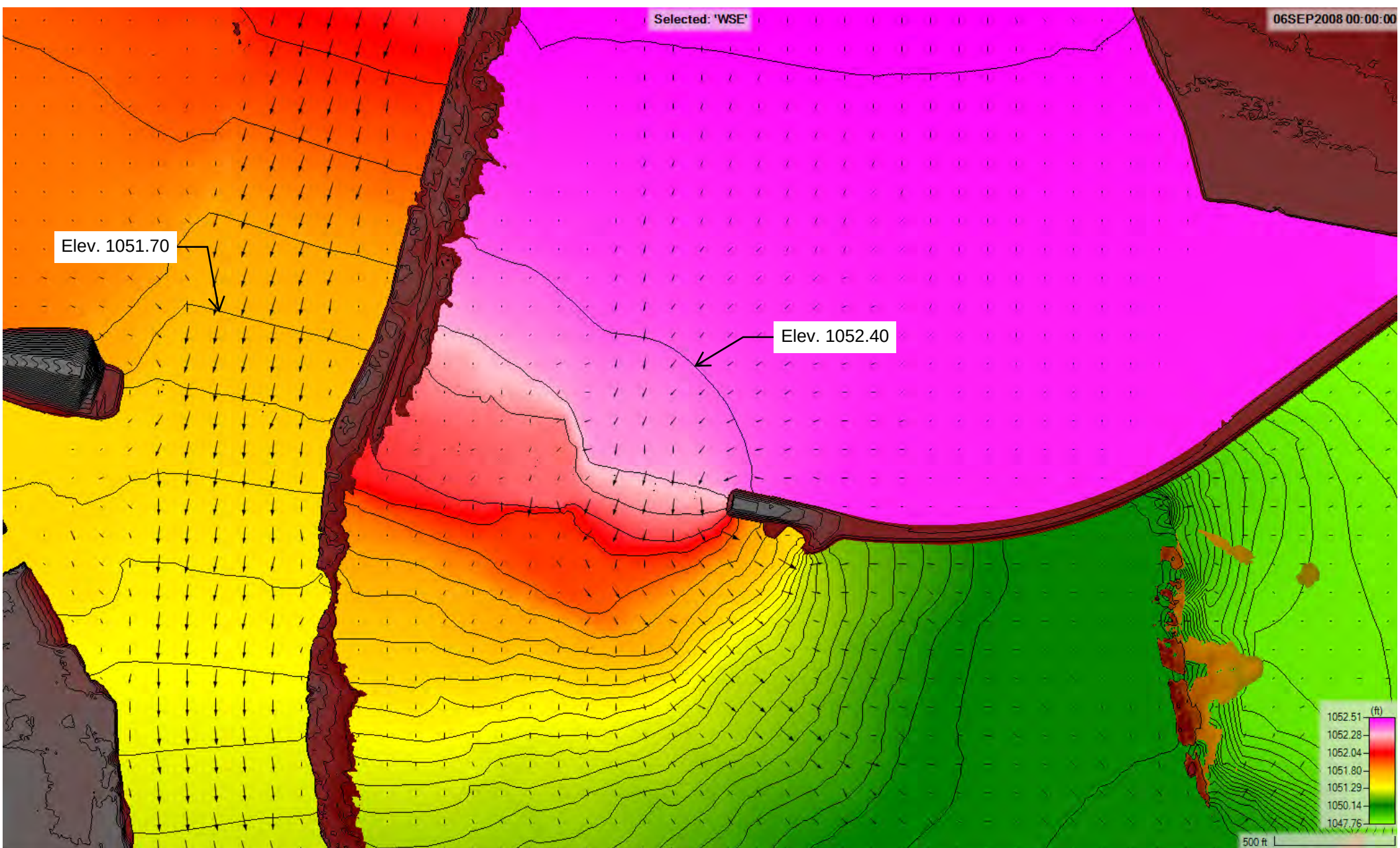
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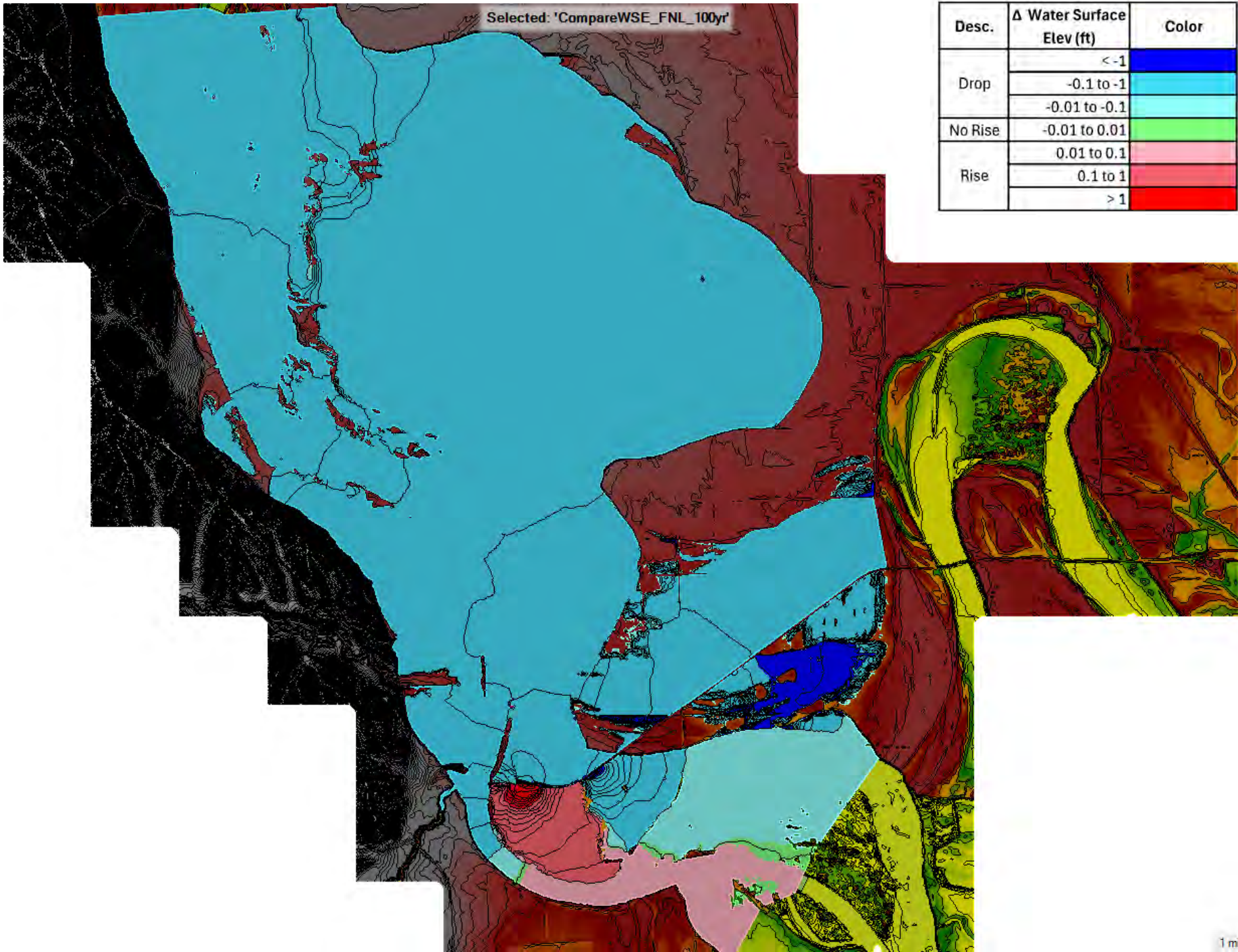
1 mi

Proposed Conditions Q100 WSE with 0.1' Contours

C.4

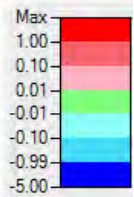


Proposed - Change in WSE vs. Existing Condition
Q100



Desc.	Δ Water Surface Elev (ft)	Color
Drop	< -1	Dark Blue
	-0.1 to -1	Light Blue
	-0.01 to -0.1	Very Light Blue
No Rise	-0.01 to 0.01	Green
Rise	0.01 to 0.1	Light Pink
	0.1 to 1	Red
	> 1	Dark Red

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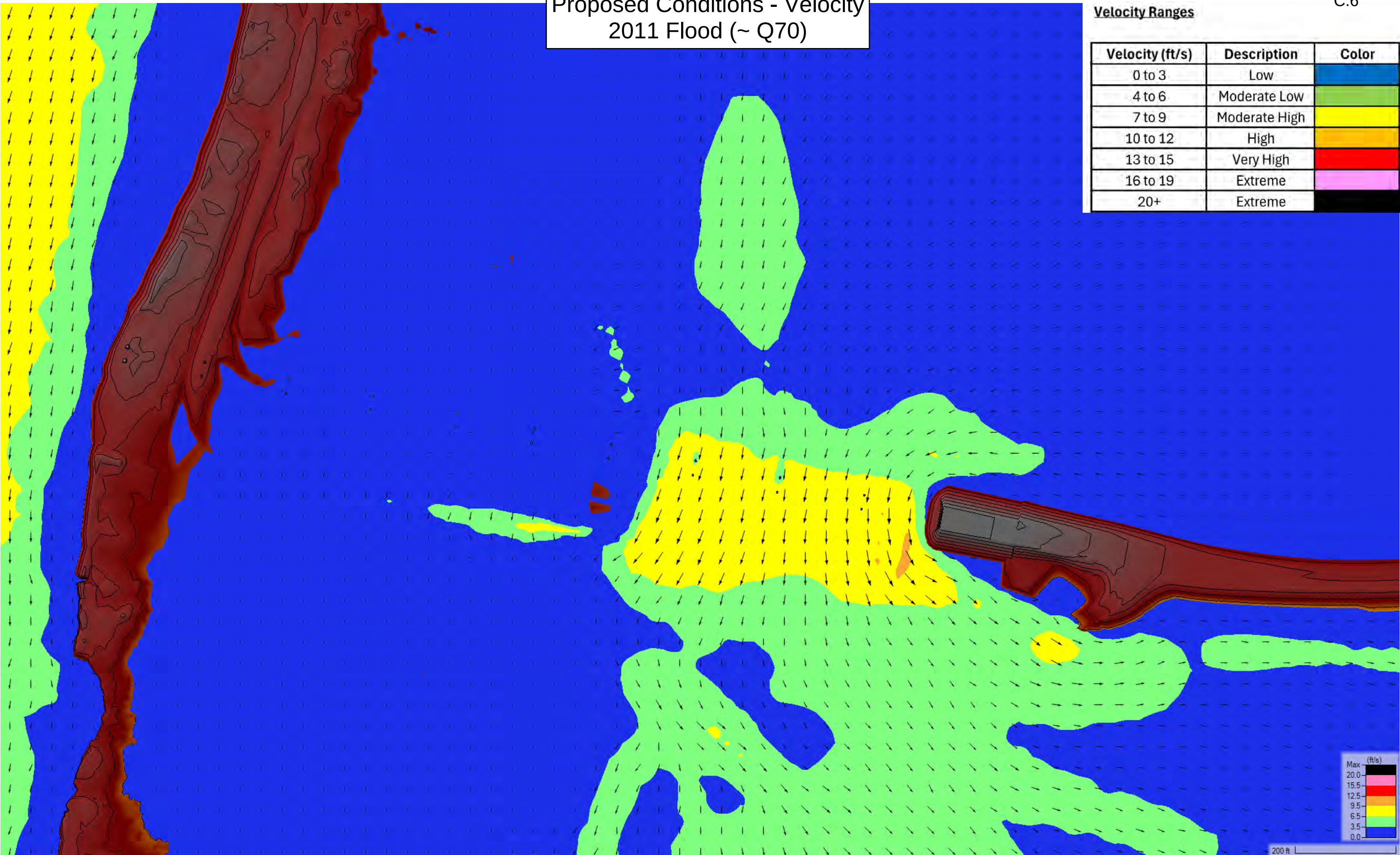


1 mi

Proposed Conditions - Velocity
2011 Flood (~ Q70)

Velocity Ranges

Velocity (ft/s)	Description	Color
0 to 3	Low	Blue
4 to 6	Moderate Low	Light Green
7 to 9	Moderate High	Yellow
10 to 12	High	Orange
13 to 15	Very High	Red
16 to 19	Extreme	Pink
20+	Extreme	Black

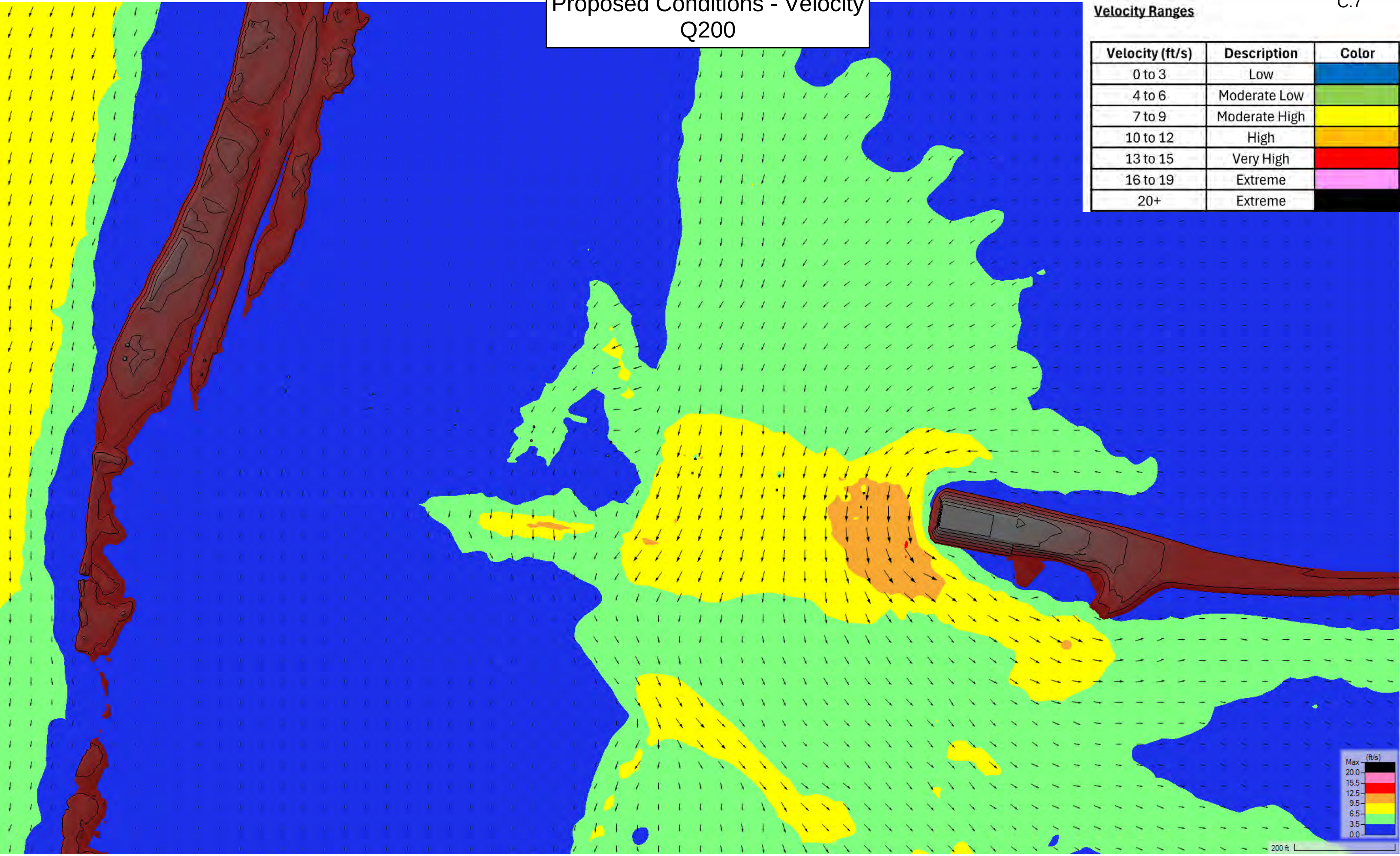


Proposed Conditions - Velocity
Q200

C.7

Velocity Ranges

Velocity (ft/s)	Description	Color
0 to 3	Low	Blue
4 to 6	Moderate Low	Light Green
7 to 9	Moderate High	Yellow
10 to 12	High	Orange
13 to 15	Very High	Red
16 to 19	Extreme	Pink
20+	Extreme	Black

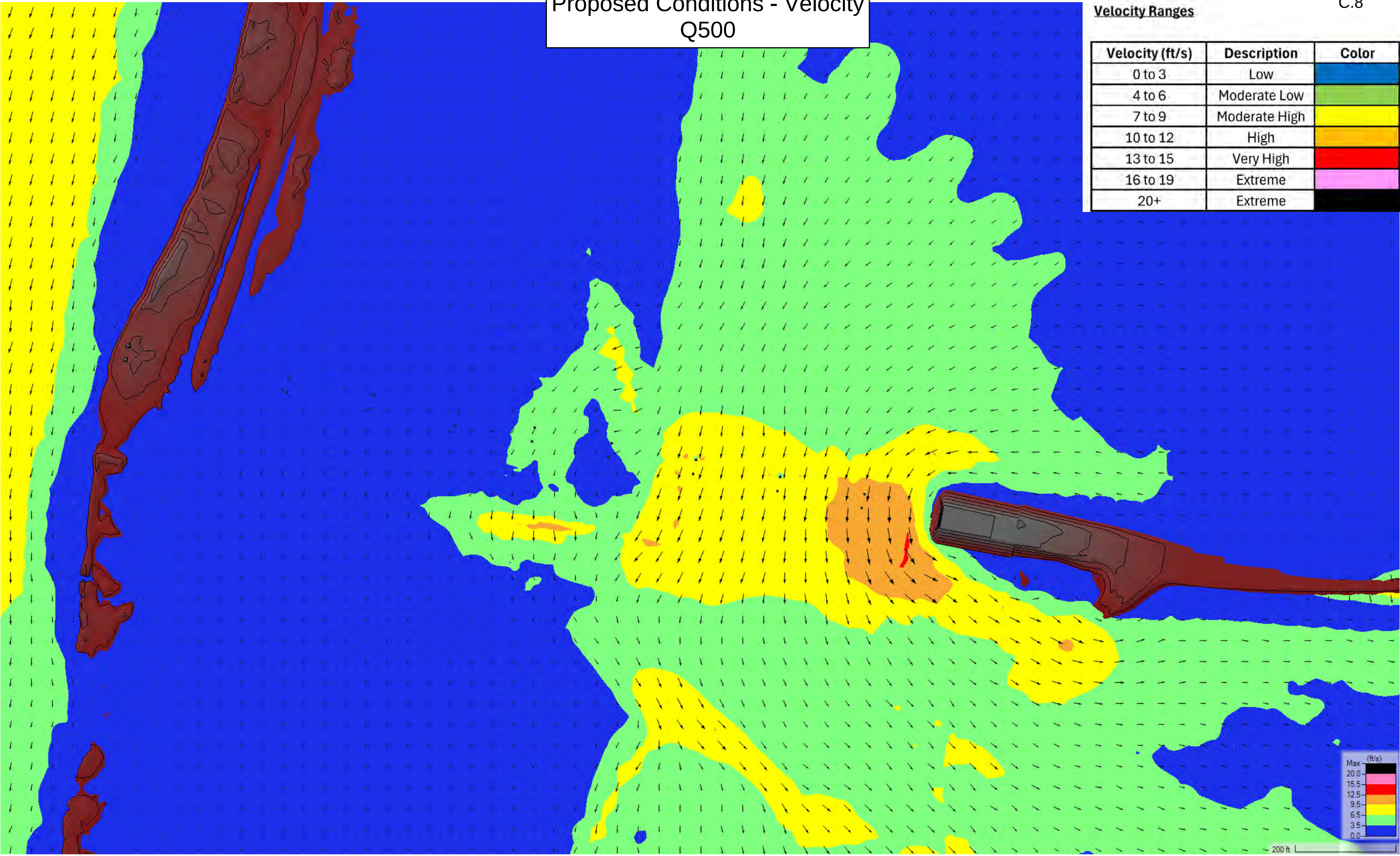


Proposed Conditions - Velocity
Q500

C.8

Velocity Ranges

Velocity (ft/s)	Description	Color
0 to 3	Low	Blue
4 to 6	Moderate Low	Light Green
7 to 9	Moderate High	Yellow
10 to 12	High	Orange
13 to 15	Very High	Red
16 to 19	Extreme	Pink
20+	Extreme	Black





STATE OF NEBRASKA
DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

PROJ. LOC.: Decatur Bridge PROJ. NO.: STP-51-7(109) BM PROJ. #: 157887
COUNTY: Burt STATION: ##### MADE BY: ASF
CN: 32395 STRUCTURE: S051 03644 DATE: 02/26/25
DESC: Phase 2 Hydro Strategy Evaluation CHECK BY: ---

Shear Stress Heat Map

Shear Stress Ranges

Lower Bound (psf)	Upper Bound (psf)	Permissible Material	Material Description
0	0.04	Bare Soil	Alluvial silt, sandy loam
0.041	1	Short Grass	Vegetation Class C (6" to 11"), short native/bunch grass
1.01	2.1	Long Grass	Vegetation Class B (12" to 20"), long native grasses
2.11	6	Riprap, $D_{50} \sim 18"$	Existing riprap (Iowa DOT Class B), MODOT Type 4
6.01	10	Riprap, $D_{50} \sim 24"$	Larger than existing riprap

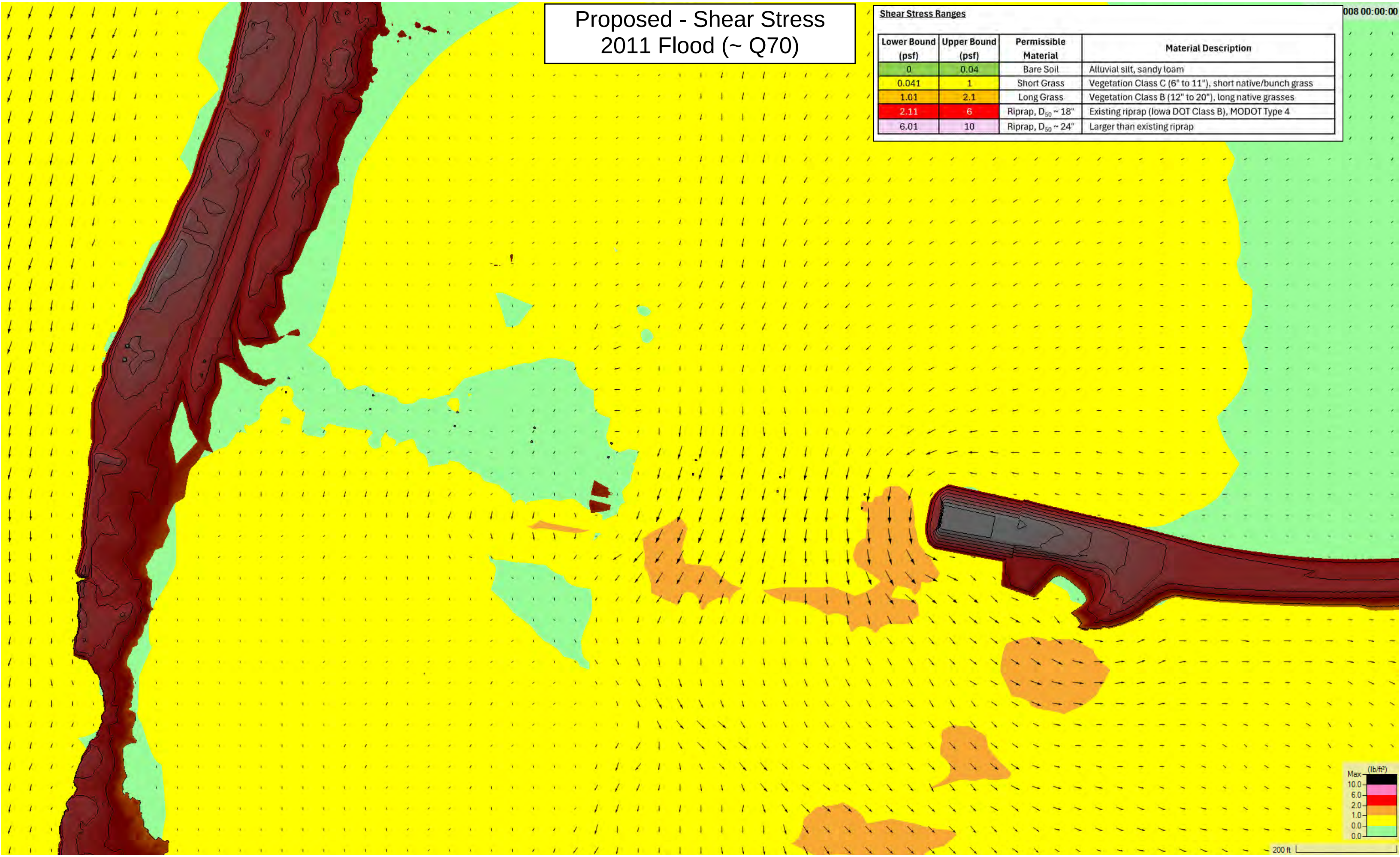
Sources for "Permissible Shear Stress/Material"

The table was developed using a combination of values from the following sources:

- MODOT Engineering Policy Guide Section 750.6, "Erosion Control and Energy Dissipation", (2025)
- HEC-15, "Design of Roadside Channels with Flexible Linings", Section 2.2.3 (2005)
- Caltrans Highway Design Manual, Chapter 860 - Roadside Channels, (2020)
- USACE ERDC, "Stability Thresholds for Stream Restoration Materials", Fischenich 2001

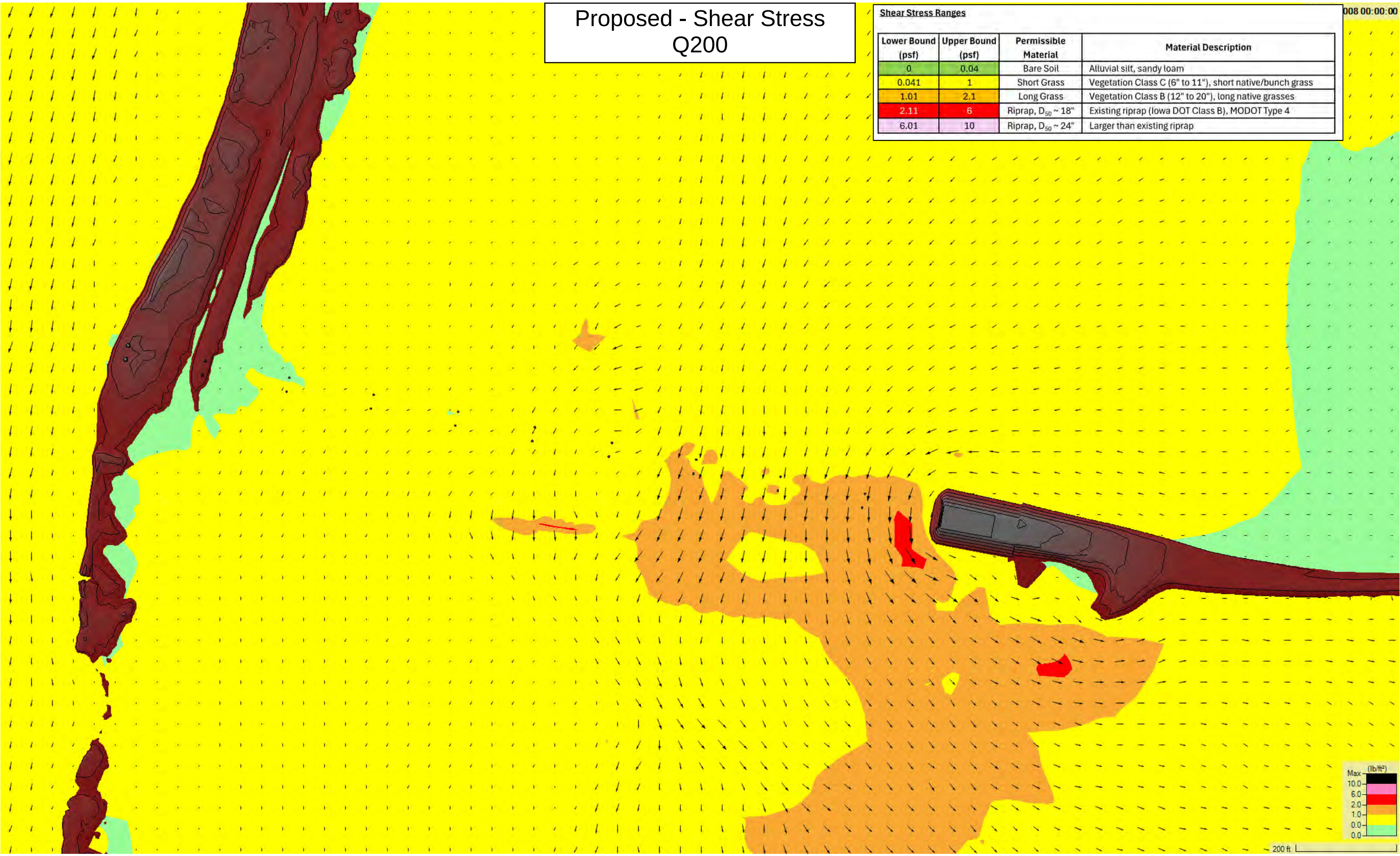
Proposed - Shear Stress
2011 Flood (~ Q70)

Shear Stress Ranges			
Lower Bound (psf)	Upper Bound (psf)	Permissible Material	Material Description
0	0.04	Bare Soil	Alluvial silt, sandy loam
0.041	1	Short Grass	Vegetation Class C (6" to 11"), short native/bunch grass
1.01	2.1	Long Grass	Vegetation Class B (12" to 20"), long native grasses
2.11	6	Riprap, D ₅₀ ~ 18"	Existing riprap (Iowa DOT Class B), MODOT Type 4
6.01	10	Riprap, D ₅₀ ~ 24"	Larger than existing riprap



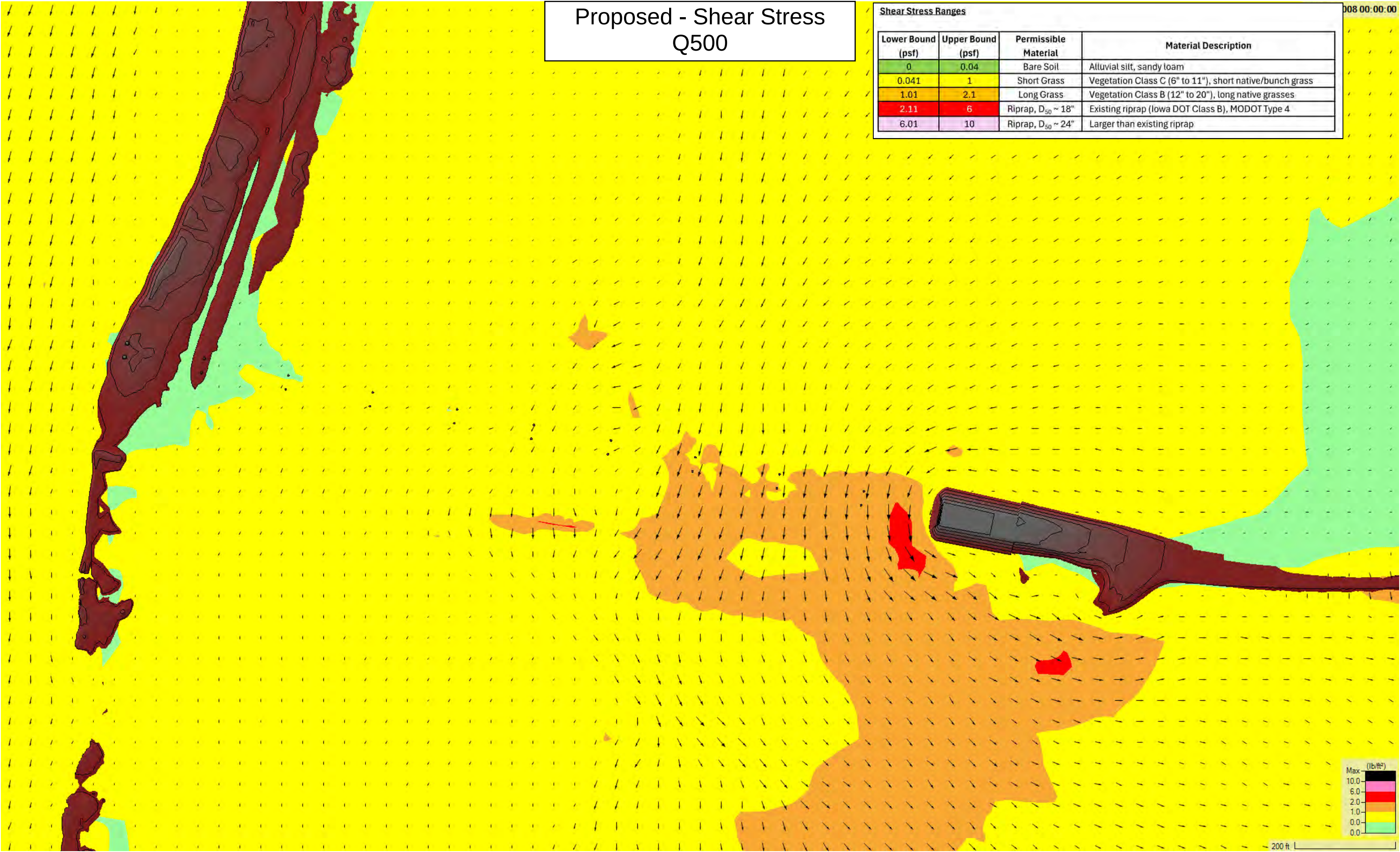
Proposed - Shear Stress
Q200

Shear Stress Ranges			
Lower Bound (psf)	Upper Bound (psf)	Permissible Material	Material Description
0	0.04	Bare Soil	Alluvial silt, sandy loam
0.041	1	Short Grass	Vegetation Class C (6" to 11"), short native/bunch grass
1.01	2.1	Long Grass	Vegetation Class B (12" to 20"), long native grasses
2.11	6	Riprap, D ₅₀ ~ 18"	Existing riprap (Iowa DOT Class B), MODOT Type 4
6.01	10	Riprap, D ₅₀ ~ 24"	Larger than existing riprap



Proposed - Shear Stress
Q500

Shear Stress Ranges			
Lower Bound (psf)	Upper Bound (psf)	Permissible Material	Material Description
0	0.04	Bare Soil	Alluvial silt, sandy loam
0.041	1	Short Grass	Vegetation Class C (6" to 11"), short native/bunch grass
1.01	2.1	Long Grass	Vegetation Class B (12" to 20"), long native grasses
2.11	6	Riprap, D ₅₀ ~ 18"	Existing riprap (Iowa DOT Class B), MODOT Type 4
6.01	10	Riprap, D ₅₀ ~ 24"	Larger than existing riprap

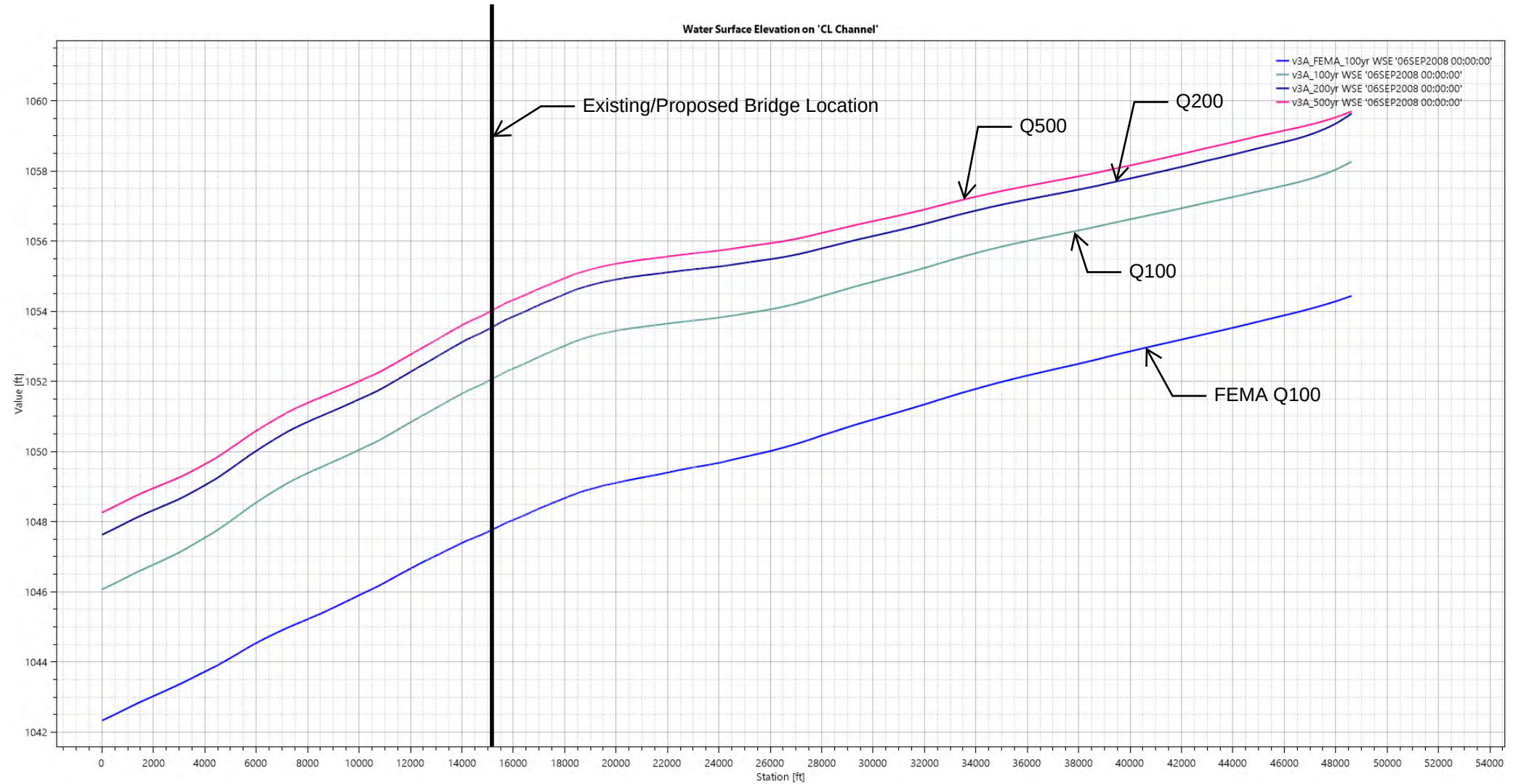


Appendix D – Alternate 2D Results

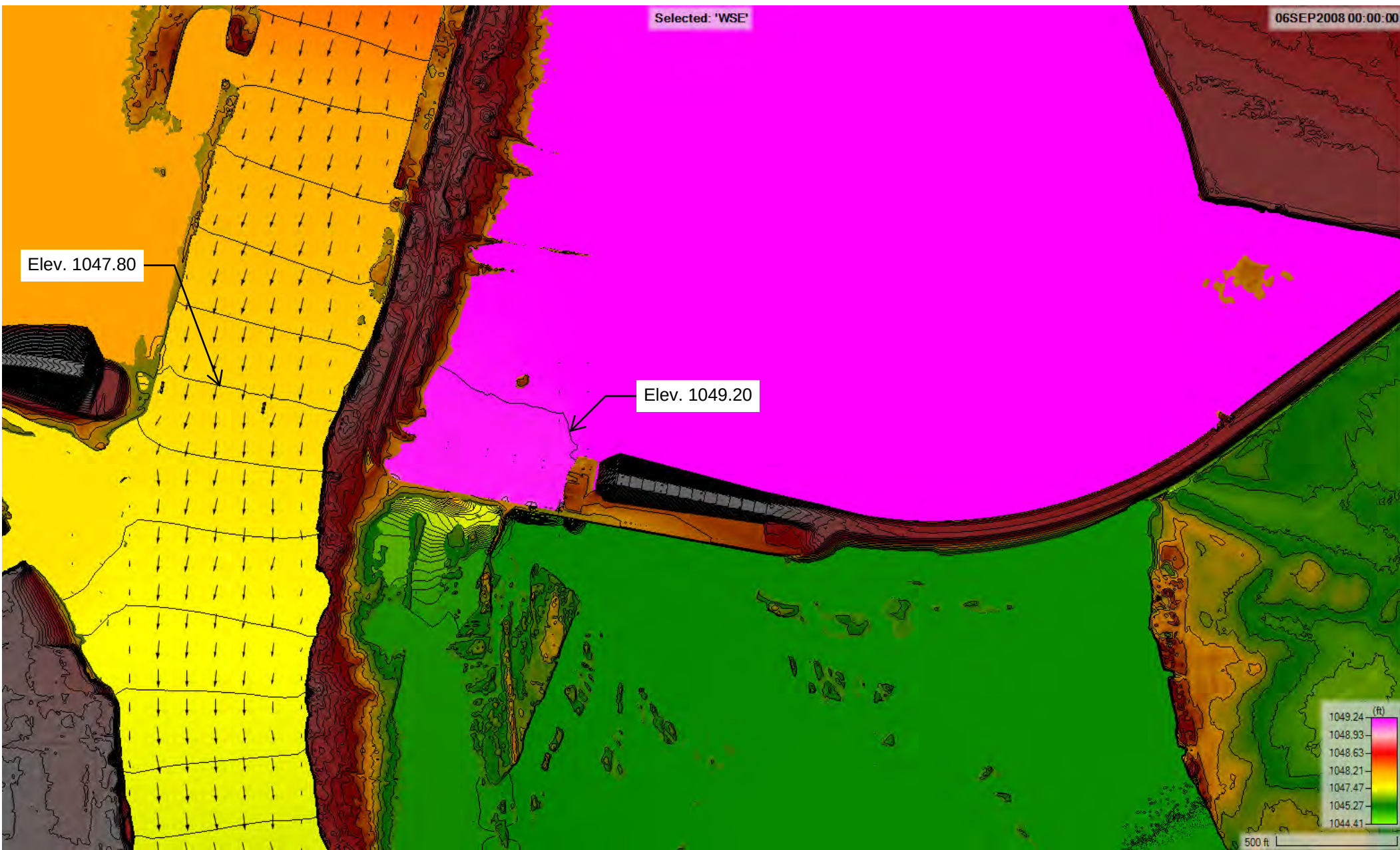
Alternate Conditions - WSE Profile Along CL Main Channel

Notes:

Q100 thru Q500 are interpolated from the 2023 MRFFS.
FEMA Q100 is the effective flow in the FIS.

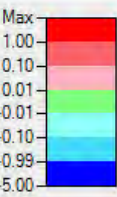


Alternate Conditions FEMA Q100 WSE with 0.1' Contours



Strategy v_3a - Change in WSE vs. Existing Condition
Effective FEMA Q100

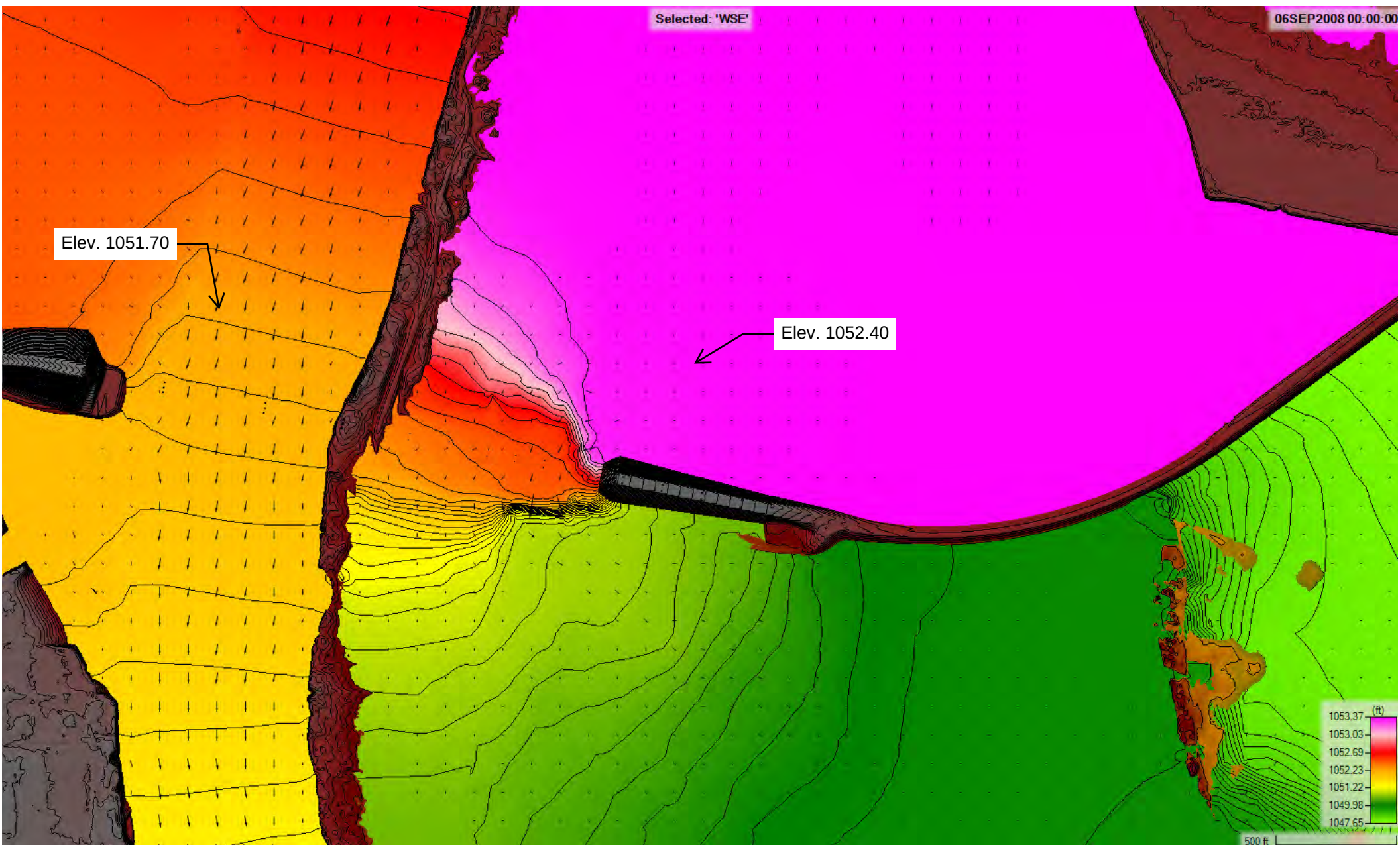
Desc.	Δ Water Surface Elev (ft)	Color
Drop	< -1	Blue
	-0.1 to -1	Light Blue
	-0.01 to -0.1	Very Light Blue
No Rise	-0.01 to 0.01	Green
Rise	0.01 to 0.1	Light Pink
	0.1 to 1	Red
	> 1	Dark Red



1 mi

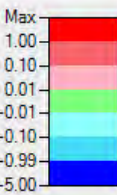
Alternate Conditions Q100 WSE with 0.1' Contours

D.4



Strategy v_3a - Change in WSE vs. Existing Condition
Q100 (new)

Desc.	Δ Water Surface Elev (ft)	Color
Drop	< -1	Blue
	-0.1 to -1	Light Blue
	-0.01 to -0.1	Very Light Blue
No Rise	-0.01 to 0.01	Green
Rise	0.01 to 0.1	Light Pink
	0.1 to 1	Red
	> 1	Dark Red

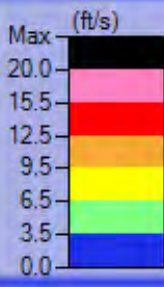
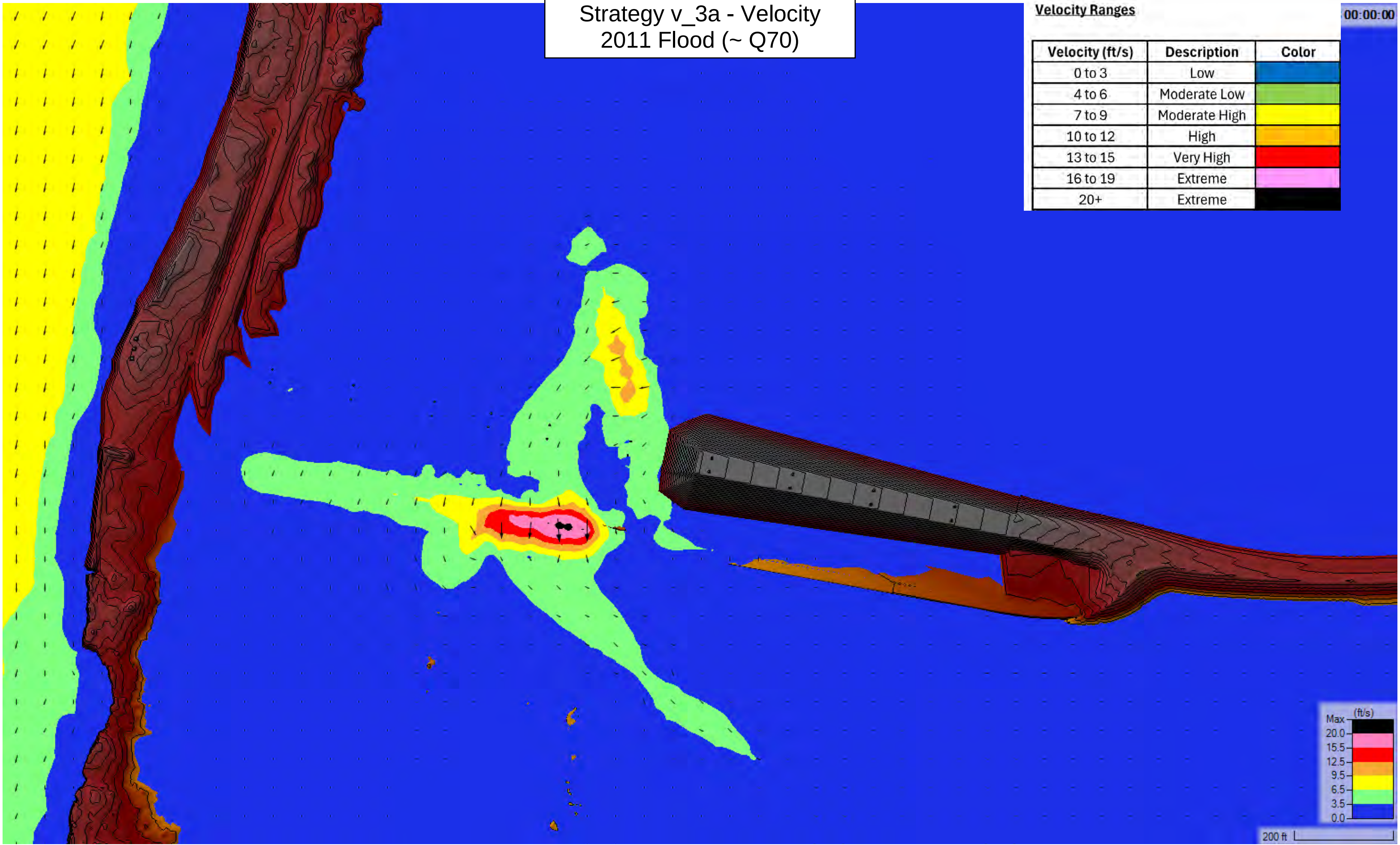


1 mi

Strategy v_3a - Velocity
2011 Flood (~ Q70)

Velocity Ranges

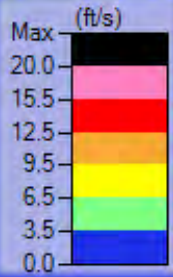
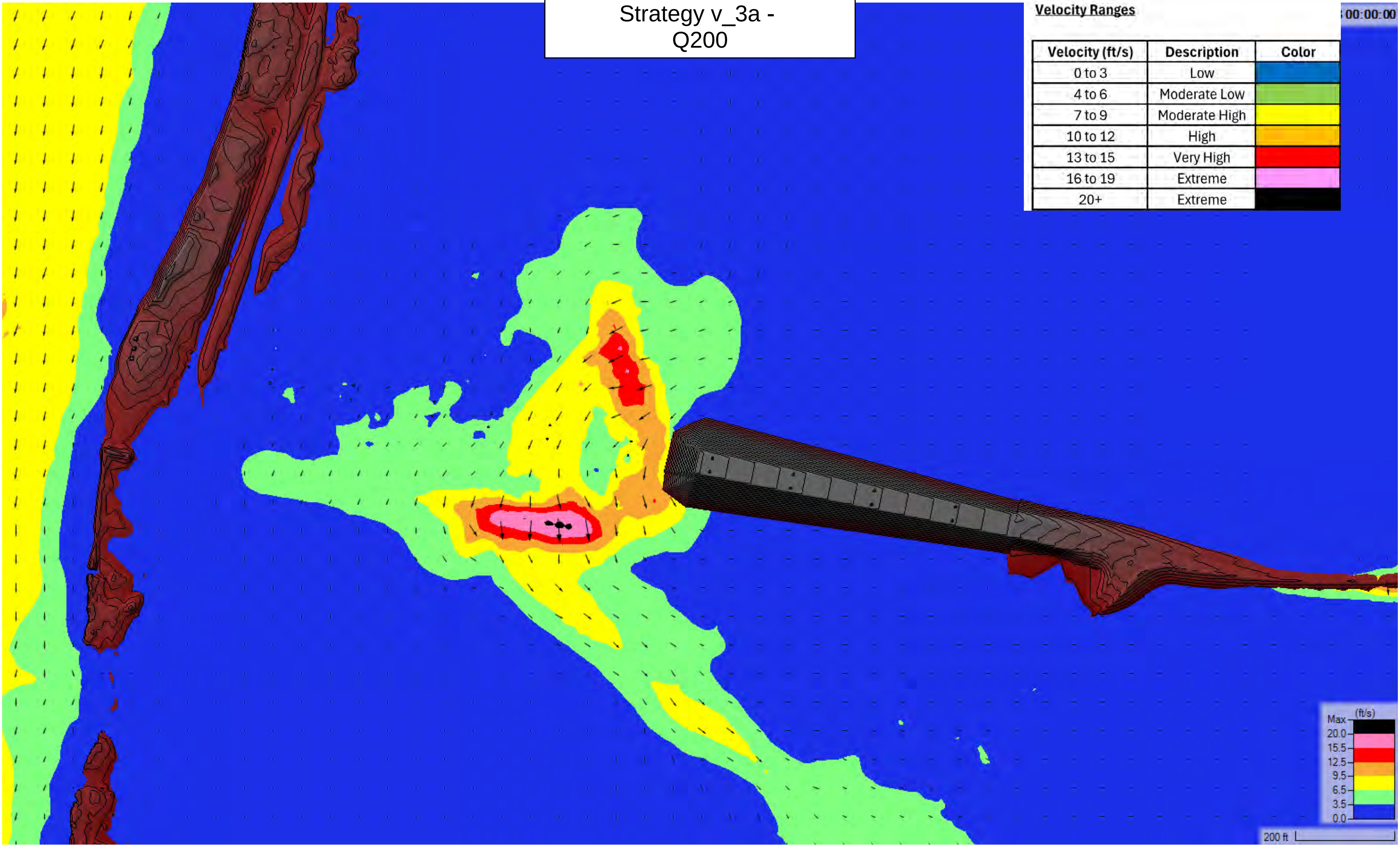
Velocity (ft/s)	Description	Color
0 to 3	Low	Blue
4 to 6	Moderate Low	Green
7 to 9	Moderate High	Yellow
10 to 12	High	Orange
13 to 15	Very High	Red
16 to 19	Extreme	Pink
20+	Extreme	Black



Strategy v_3a -
Q200

Velocity Ranges

Velocity (ft/s)	Description	Color
0 to 3	Low	Blue
4 to 6	Moderate Low	Green
7 to 9	Moderate High	Yellow
10 to 12	High	Orange
13 to 15	Very High	Red
16 to 19	Extreme	Pink
20+	Extreme	Black

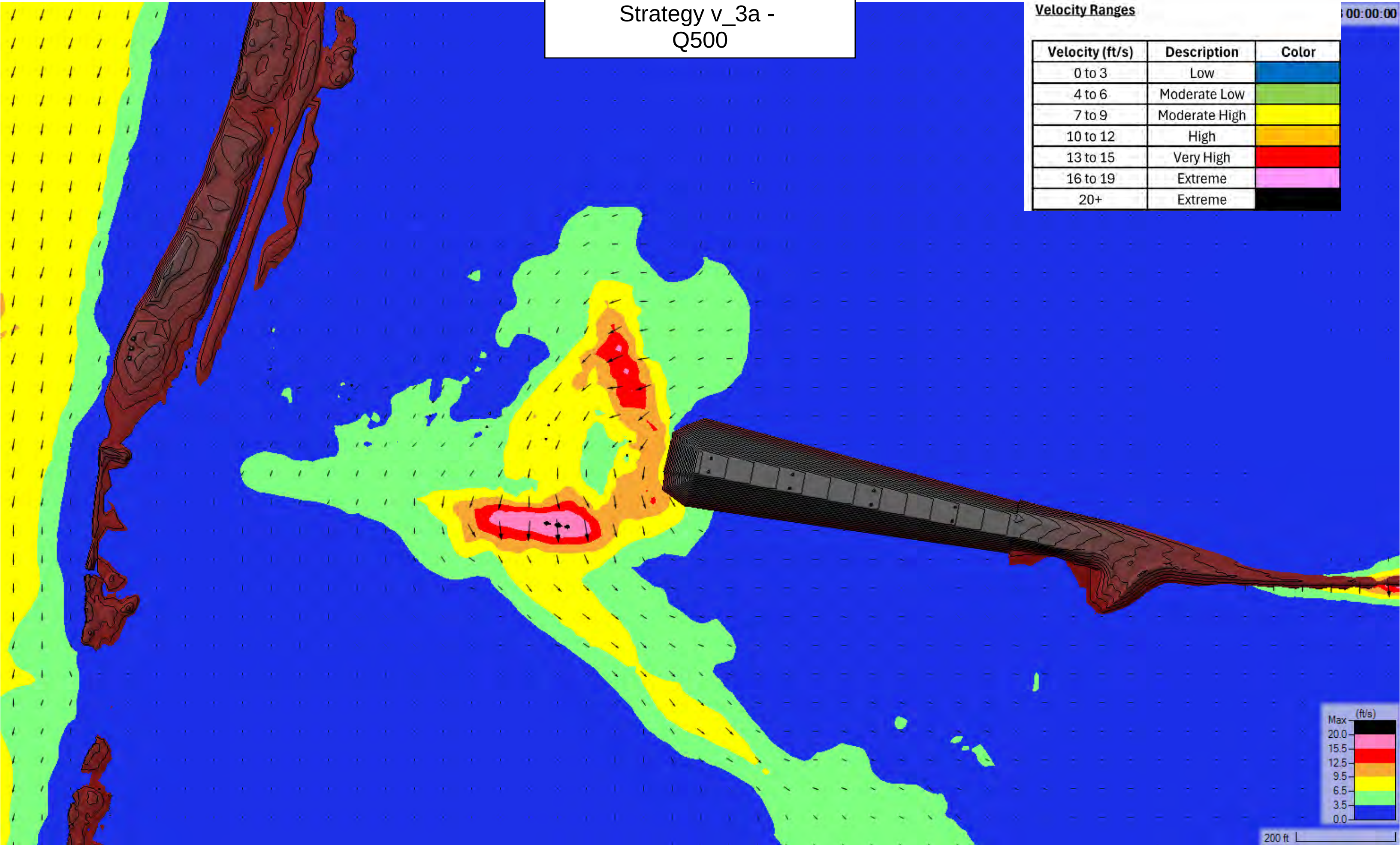


200 ft

Strategy v_3a -
Q500

Velocity Ranges

Velocity (ft/s)	Description	Color
0 to 3	Low	Blue
4 to 6	Moderate Low	Green
7 to 9	Moderate High	Yellow
10 to 12	High	Orange
13 to 15	Very High	Red
16 to 19	Extreme	Pink
20+	Extreme	Black





STATE OF NEBRASKA
DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

PROJ. LOC.: Decatur Bridge PROJ. NO.: STP-51-7(109) BM PROJ. #: 157887
COUNTY: Burt STATION: ##### MADE BY: ASF
CN: 32395 STRUCTURE: S051 03644 DATE: 02/26/25
DESC: Phase 2 Hydro Strategy Evaluation CHECK BY: ---

Shear Stress Heat Map

Shear Stress Ranges

Lower Bound (psf)	Upper Bound (psf)	Permissible Material	Material Description
0	0.04	Bare Soil	Alluvial silt, sandy loam
0.041	1	Short Grass	Vegetation Class C (6" to 11"), short native/bunch grass
1.01	2.1	Long Grass	Vegetation Class B (12" to 20"), long native grasses
2.11	6	Riprap, $D_{50} \sim 18"$	Existing riprap (Iowa DOT Class B), MODOT Type 4
6.01	10	Riprap, $D_{50} \sim 24"$	Larger than existing riprap

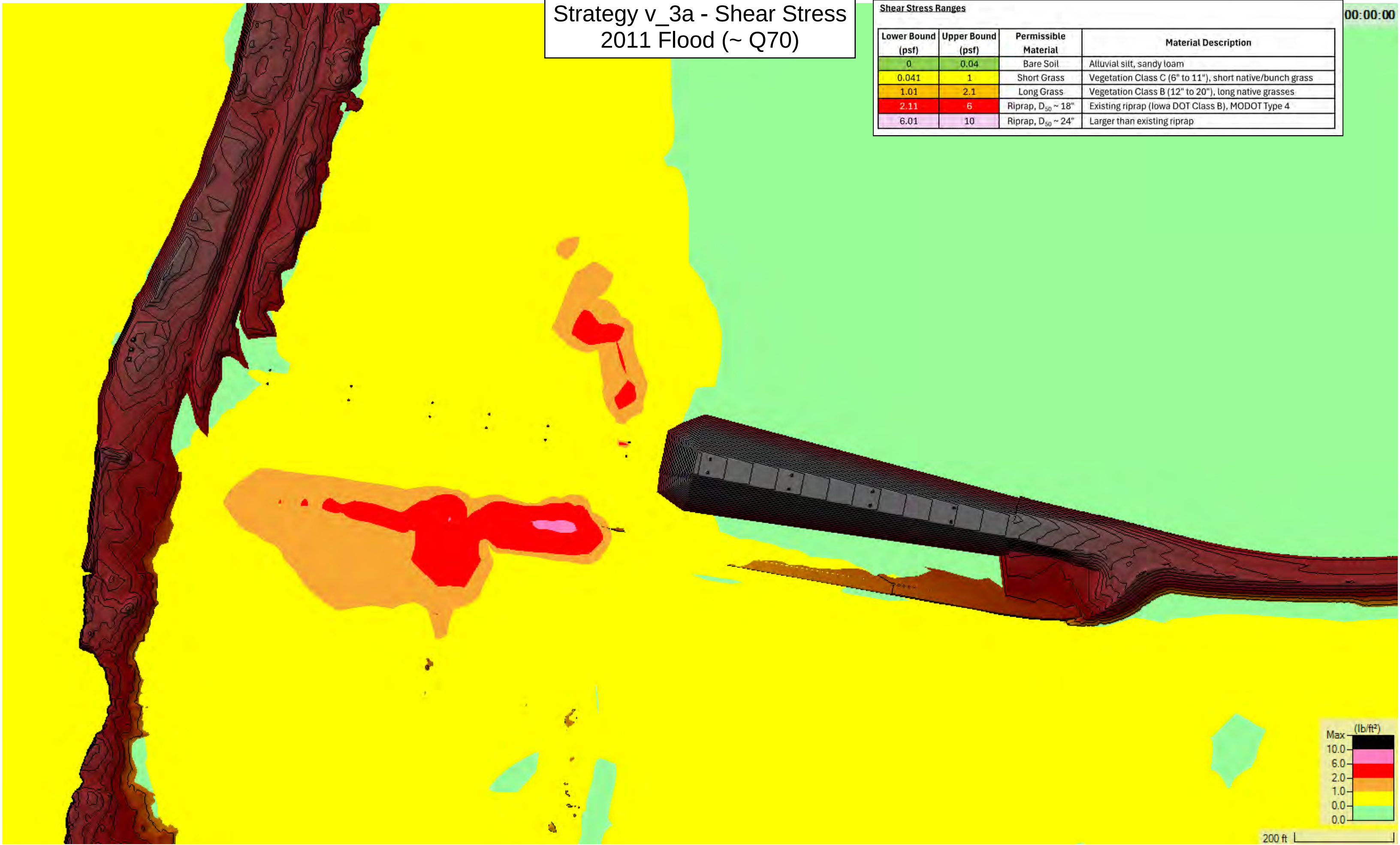
Sources for "Permissible Shear Stress/Material"

The table was developed using a combination of values from the following sources:

- MODOT Engineering Policy Guide Section 750.6, "Erosion Control and Energy Dissipation", (2025)
- HEC-15, "Design of Roadside Channels with Flexible Linings", Section 2.2.3 (2005)
- Caltrans Highway Design Manual, Chapter 860 - Roadside Channels, (2020)
- USACE ERDC, "Stability Thresholds for Stream Restoration Materials", Fischenich 2001

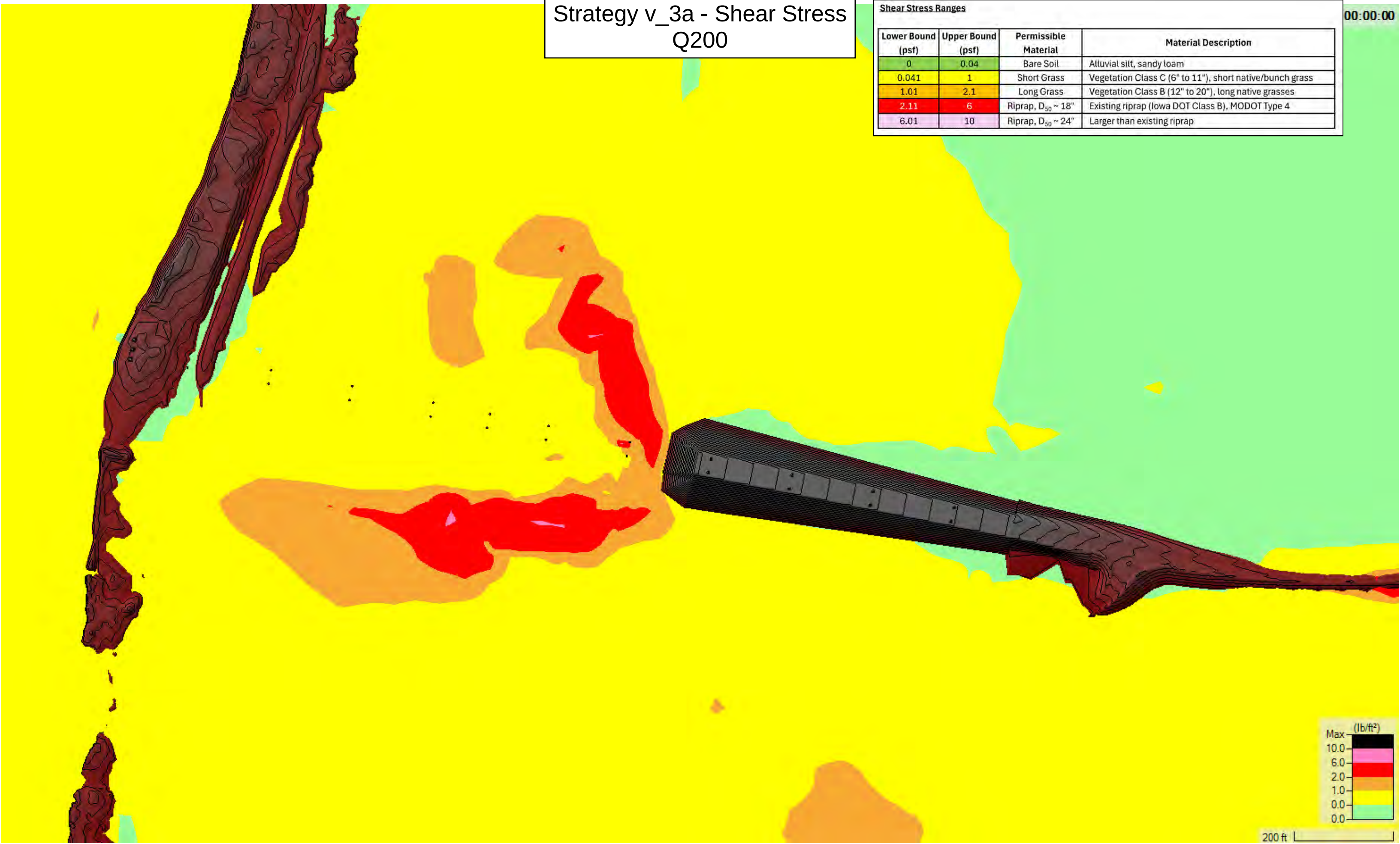
Strategy v_3a - Shear Stress
2011 Flood (~ Q70)

Shear Stress Ranges			
Lower Bound (psf)	Upper Bound (psf)	Permissible Material	Material Description
0	0.04	Bare Soil	Alluvial silt, sandy loam
0.041	1	Short Grass	Vegetation Class C (6" to 11"), short native/bunch grass
1.01	2.1	Long Grass	Vegetation Class B (12" to 20"), long native grasses
2.11	6	Riprap, D ₅₀ ~ 18"	Existing riprap (Iowa DOT Class B), MODOT Type 4
6.01	10	Riprap, D ₅₀ ~ 24"	Larger than existing riprap



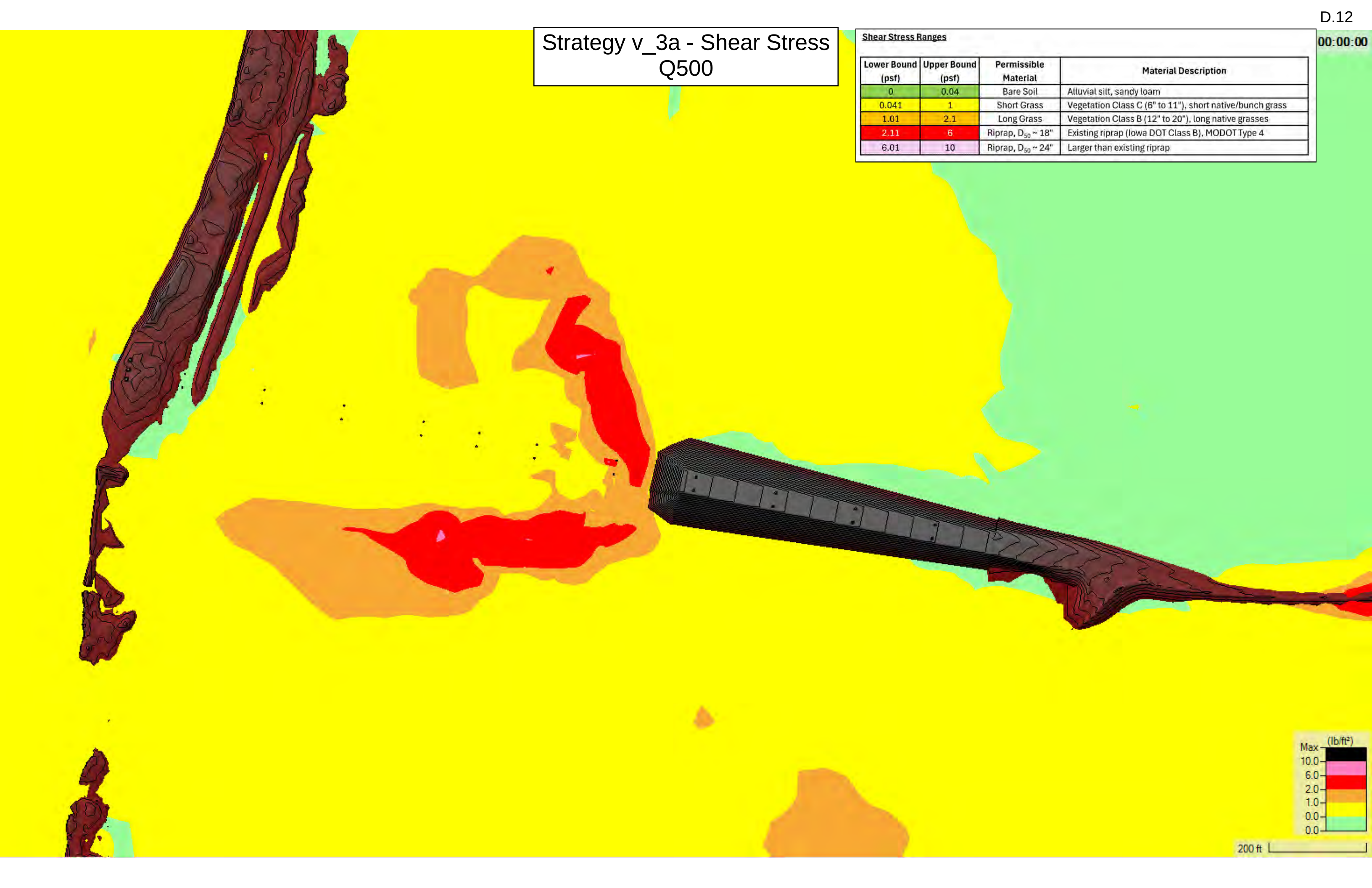
Strategy v_3a - Shear Stress
Q200

Shear Stress Ranges			
Lower Bound (psf)	Upper Bound (psf)	Permissible Material	Material Description
0	0.04	Bare Soil	Alluvial silt, sandy loam
0.041	1	Short Grass	Vegetation Class C (6" to 11"), short native/bunch grass
1.01	2.1	Long Grass	Vegetation Class B (12" to 20"), long native grasses
2.11	6	Riprap, D ₅₀ ~ 18"	Existing riprap (Iowa DOT Class B), MODOT Type 4
6.01	10	Riprap, D ₅₀ ~ 24"	Larger than existing riprap



Strategy v_3a - Shear Stress
Q500

Shear Stress Ranges			
Lower Bound (psf)	Upper Bound (psf)	Permissible Material	Material Description
0	0.04	Bare Soil	Alluvial silt, sandy loam
0.041	1	Short Grass	Vegetation Class C (6" to 11"), short native/bunch grass
1.01	2.1	Long Grass	Vegetation Class B (12" to 20"), long native grasses
2.11	6	Riprap, D ₅₀ ~ 18"	Existing riprap (Iowa DOT Class B), MODOT Type 4
6.01	10	Riprap, D ₅₀ ~ 24"	Larger than existing riprap



Appendix E – Scour Calculations

Nebraska DOT
Control Number (CN): 32395
Structure: S051 03644

Decatur Bridge

Scour Design

By:
Jon Kuchem, PE

Check By:
Andrew Ferdon, PE
Tom Greer, PE, CFM
Govinda Karki, PE

BMCD Project No. 157887

August 11, 2025



Scour Design

Table of Contents

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- B.2 2D HEC-RAS Model Exhibits

C. Contraction Scour

- C.1 Summary & Iowa Overbank Calculations
- C.2 Main Channel FHWA Toolbox Calculation

D. Pier Scour

- D.1 Summary and Calculation
- D.2 “Pier Scour Multiplier”
- D.3 Rip-Rap Size

E. Abutment Scour

- E.1 Design Approach
- E.2 NCHRP Abutment Scour
- E.3 Rip Rap Apron
- E.4 Scour at Abutment Front Face
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F. Alternate Conditions

- F.1 Pier & Contraction Scour
- F.2 Abutment Scour (Q200)

G. References

Appendix A – Other Alternatives Not Carried Forward Scour Comparison

A. Scour Design Summary, Notes & Assumptions



STATE OF NEBRASKA
DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

PROJ. LOC.: Decatur Bridge	PROJ. NO.: STP-51-7(109)	BM PROJ. #: 157887
COUNTY: Burt	STATION: ###+###.##	MADE BY: JTK
CN: 32395	STRUCTURE: S051 03644	DATE: 03/04/25
DESC: Phase 2 Hydro Strategy Evaluation	CHECK BY: ---	

Scour Calculation Notes and Assumptions

- Design Scour = Q200. Check Scour = Q500.
- Secondary Channel broken up into three zones to calculate scour. Zone 1 is the newly opened channel from moving the abutment east. Zone 2 is between the two dikes in the existing scour hole. Zone 3 is from the scour hole to the high ground between the main and secondary channels.
- Secondary Channel contraction scour values assumed to be clear-water contraction scour. Based upon upstream terrain, flow vectors, and review of 2011 flood it seems a lower likelihood that there will be enough sediment to brought from upstream to have a live bed condition.
- Main Channel contraction scour calculated similar to 1D. Contraction width & flow calculated for channel only, overbanks would be calculated separately.
- Secondary Channel pier scour assumed two column bent with 5' diameter columns and 7.5' diameter shafts per preliminary design. Use 7.5' diameter in pier scour equation.
- Scour calculated per HEC-18 5th edition methodology for contraction and pier scour.
See Abutment Scour section for methodology and scour countermeasure design.
- The Nebraska Abutment and Pier 1 on the approach embankment is higher than the Q500 WSE.
Scour not applicable.

<u>Event</u>	<u>Flow (cfs)</u>	
Q10	94,400	
Q25	127,100	
FEMA Q100	141,800	
2011 Flood (Q70)	191,000	
Q100	220,600	
Q200	271,000	<i>Design Scour</i>
Q500	297,200	<i>Check Scour</i>



STATE OF NEBRASKA
DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

PROJ. LOC.: Decatur Bridge PROJ. NO.: STP-51-7(109) BM PROJ. #: 157887
COUNTY: Burt STATION: ###+###.## MADE BY: JTK
CN: 32395 STRUCTURE: S051 03644 DATE: 08/11/25
DESC: Scour Depth Summary CHECK BY:

Scour Depth Summary

AASHTO LRFD Section:

	= User inputs
Q200	= Scour Design Flood
Q500	= Scour Check Flood

Notes:

- % of Scour from AASHTO 10th Edition, T2.6.4.4.4-1
- EE 2: CV B case modified to be more conservative, per Project Design Criteria
- Scour components from Prelim Hydraulic Model

% of Scour Depth concurrent w/ each Limit State

Limit State	Degrad	Contr	Local	Flood
SER	100%	100%	100%	Scour Design Flood
STR	100%	100%	100%	Scour Design Flood
EE 1	50%	50%	50%	Scour Design Flood
EE 2: CV A, IC	50%	50%	50%	Scour Design Flood
EE 2: CV B	50%	50%	50%	Scour Design Flood
EE 2: Check Flood	100%	100%	100%	Scour Check Flood

		Scour Components							
Zone	Sub	Ground Elev.	Degrad.	Q200			*Q500		
				Calc. Contr.	Control Contr.	Local	Calc. Contr.	Control Contr.	Local
n/a	*A1	1094.0	0	0	6	0	0	12	0
n/a	*P1	1063.4	0	0	6	0	0	12	0
River	P2	1035.1	7.5	3.2	6.0	15.6	3.6	12.0	15.8
	P3	1018.9	7.5	3.2	6.0	23.1	3.6	12.0	23.2
Bank	P4	1057.4	7.5	3.2	6	7.2	3.6	12	7.8
3	P5	1055.2	0	0	6	6.9	0	12	7.1
	P6	1046.5	0	0	6	6.9	0	12	7.1
	P7	1045.2	0	0	6	6.9	0	12	7.1
	P8	1044.9	0	0	6	6.9	0	12.0	7.1
2	P9	1031.0	0	4.7	6	15.7	6.8	12.0	16.1
	P10	1049.5	0	4.7	6	15.7	6.8	12.0	16.1
1	P11	1039.8	0	30.1	30.1	17.2	31.8	31.8	17.4
	P12	1039.8	0	30.1	30.1	17.2	31.8	31.8	17.4
	**P13	1039.8	0	30.1	70.9	0	31.8	72.5	0
n/a	***A2	1054.1	0	0	6	0	0	12	0

STR		EE 2: CV A, IC		EE 2: CV B		EE 2: Check Flood	
Total	Scour	Total	Scour	Total	Scour	Total	Scour
Scour	Elev.	Scour	Elev.	Scour	Elev.	Scour	Elev.
6.0	1088.0	3.0	1091.0	3.0	1091.0	12.0	1082.0
6.0	1057.4	3.0	1060.4	3.0	1060.4	12.0	1051.4
29.1	1006.0	14.6	1020.6	14.6	1020.6	35.3	999.8
36.6	982.3	18.3	1000.6	18.3	1000.6	42.7	976.2
20.7	1036.7	10.4	1047.1	10.4	1047.1	27.3	1030.1
12.9	1042.3	6.5	1048.8	6.5	1048.8	19.1	1036.1
12.9	1033.6	6.5	1040.1	6.5	1040.1	19.1	1027.4
12.9	1032.3	6.5	1038.8	6.5	1038.8	19.1	1026.1
12.9	1032.0	6.5	1038.5	6.5	1038.5	19.1	1025.8
21.7	1009.3	10.9	1020.2	10.9	1020.2	28.1	1002.9
21.7	1027.8	10.9	1038.7	10.9	1038.7	28.1	1021.4
47.3	992.5	23.7	1016.2	23.7	1016.2	49.2	990.6
47.3	992.5	23.7	1016.2	23.7	1016.2	49.2	990.6
70.9	968.9	35.5	1004.4	35.5	1004.4	72.5	967.3
6.0	1048.1	3.0	1051.1	3.0	1051.1	12.0	1042.1

*At Abut 1 and Pier 1, use prescriptive minimum scour from NDOT Hydraulic Analysis Guidelines

**At Pier 13, use max abutment scour depth from NCHRP Method. NCHRP method reports an amplified contraction scour (no separate local component)

***Scour at face of Abut 2 is projected back from NCHRP max scour depth at a failure slope

Zone 3 (Piers West of Existing Scour Hole)

Model/Location	Bridge Extension (ft)	Bench Elevation	Flow	Contraction Scour	Pier Scour	Long Term Degredation	Total Scour
PropFinal	650	1041	Q10	0.0	0.0	0	0.0
			Q25	0.0	0.7	0	0.7
			FEMA Q100	0.0	1.6	0	1.6
			2011 Flood	0.0	5.1	0	5.1
			Q100	0.0	6.0	0	6.0
			Q200	0.0	6.9	0	6.9
			Q500	0.0	7.1	0	7.1

Zone 2 (Pier in Existing Scour Hole)

Model/Location	Bridge Extension (ft)	Bench Elevation	Flow	Contraction Scour	Pier Scour	Long Term Degredation	Total Scour
PropFinal	650	1041	Q10	0.0	0.0	0	0.0
			Q25	0.0	0.4	0	0.4
			FEMA Q100	0.0	1.3	0	1.3
			2011 Flood	0.0	10.5	0	10.5
			Q100	0.0	13.3	0	13.3
			Q200	4.7	15.7	0	20.5
			Q500	6.8	16.3	0	23.1

Zone 1 (Piers in Newly Opened Channel)

Model/Location	Bridge Extension (ft)	Bench Elevation	Flow	Contraction Scour	Pier Scour	Long Term Degredation	Total Scour
PropFinal	650	1041	Q10	0.0	6.6	0	6.6
			Q25	5.6	11.3	0	17.0
			FEMA Q100	9.5	12.7	0	22.2
			2011 Flood	19.0	15.2	0	34.2
			Q100	23.6	16.1	0	39.7
			Q200	30.1	17.2	0	47.2
			Q500	31.8	17.4	0	49.3

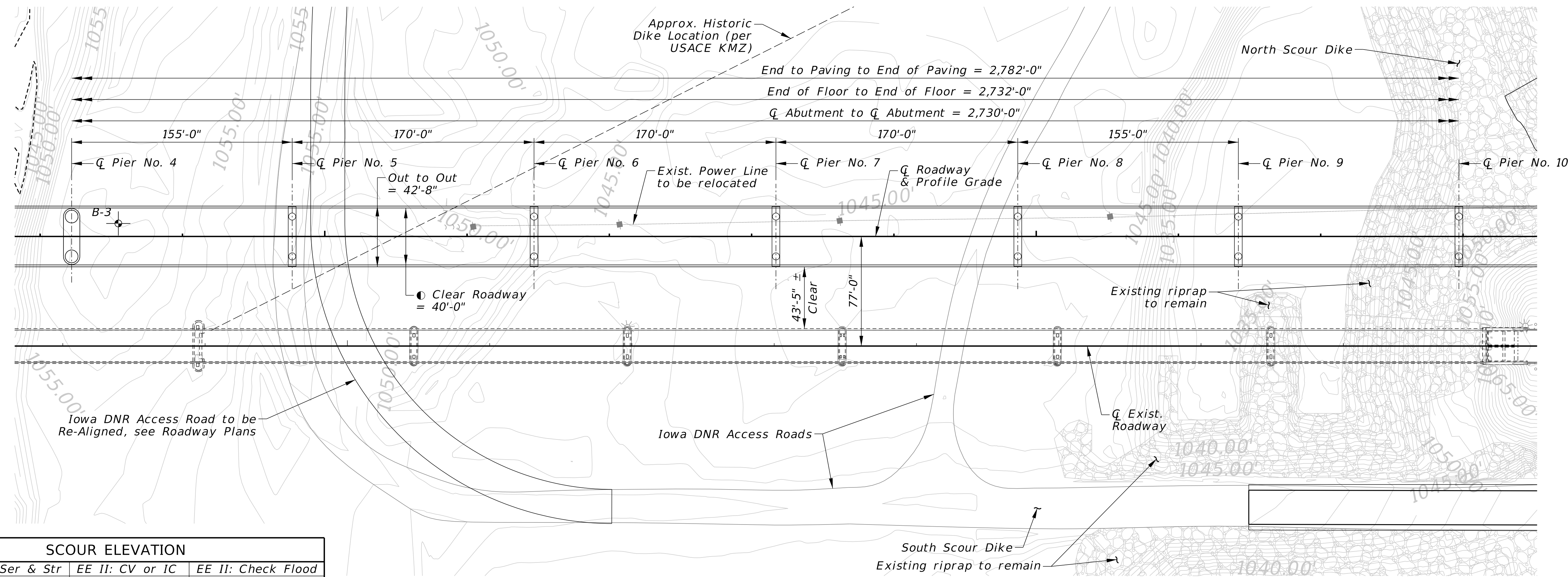
Main Channel Pier

Model/Location	Bridge Extension (ft)	Bench Elevation	Flow	Contraction Scour	Pier Scour	Long Term Degredation	Total Scour
PropFinal/Pier 2	N/A	N/A	Q200	3.2	15.6	7.5	26.3
			Q500	3.6	15.8	7.5	26.9
PropFinal/Pier 3	N/A	N/A	Q200	3.2	23.1	7.5	33.8
			Q500	3.6	23.2	7.5	34.3
PropFinal/Pier 4	N/A	N/A	Q200	3.2	7.2	7.5	17.9
			Q500	3.6	7.8	7.5	18.9

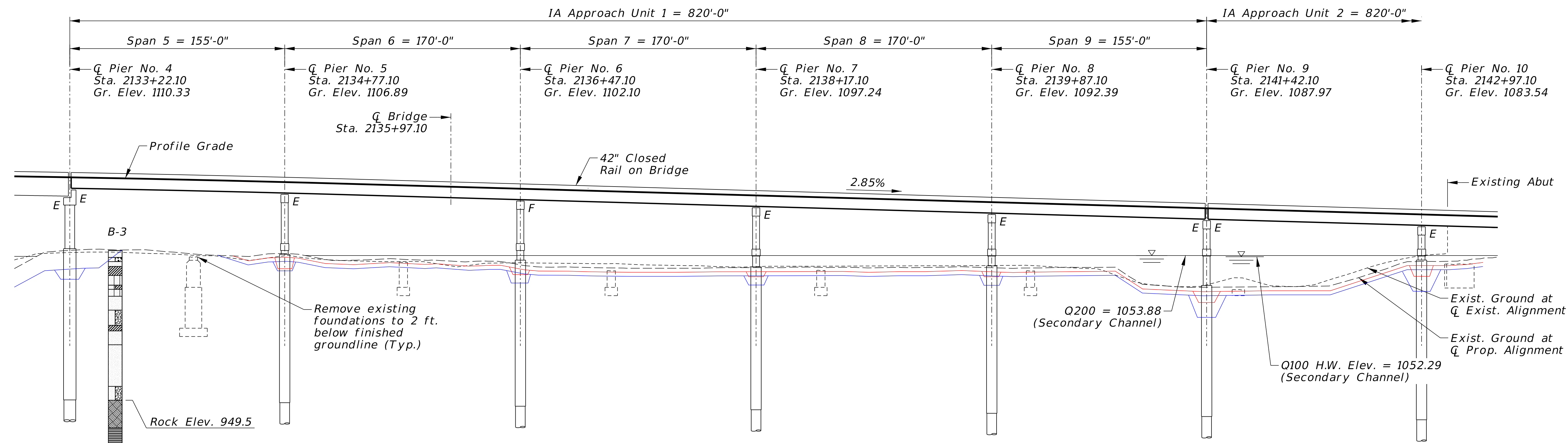
Iowa Abutment

Model/Location	Bridge Extension (ft)	Bench Elevation	Flow	NCHRP Abutment Scour	Failure Slope back to Abut. Face	Long Term Degredation	Total Scour at Abutment Face
PropFinal/Pier 2	650	1041	Q200	70.9	-78.4	0	-7.5
			Q500	72.5	-80.5	0	-8.0

B. Exhibits



SCOUR ELEVATION			
Load Case	Ser & Str	EE II: CV or IC	EE II: Check Flood
Event	Q200	Q200	Q500
Pier 5	1042.3	1048.8	1036.1
Pier 6	1033.6	1040.1	1027.4
Pier 7	1032.3	1038.8	1026.1
Pier 8	1032.0	1038.5	1025.8
Pier 9	1009.3	1020.2	1002.9



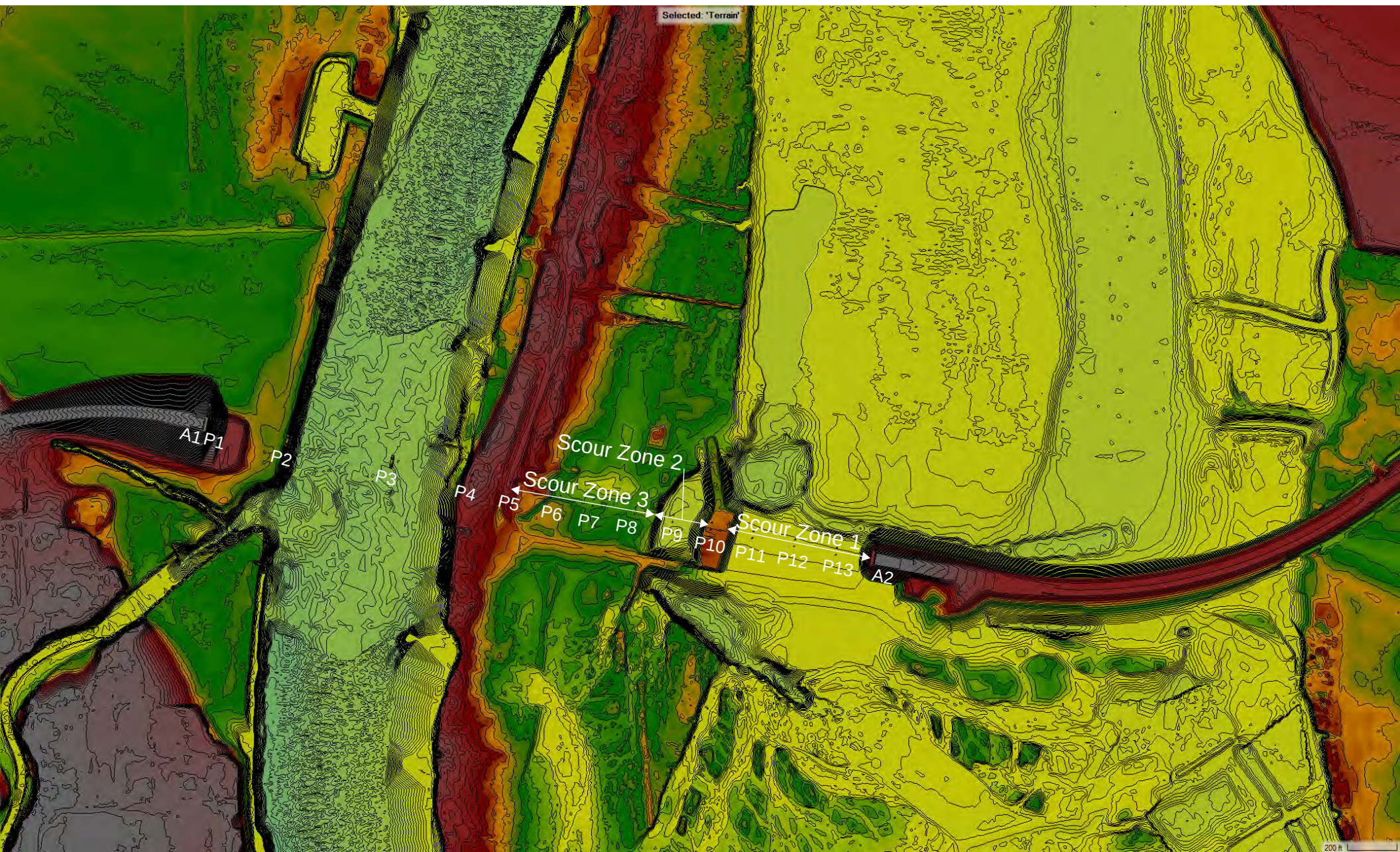
NOTES:
Contours on this sheet show proposed conditions.

Blue = Scour for STR 1
Red = Scour for EE 2: Ice

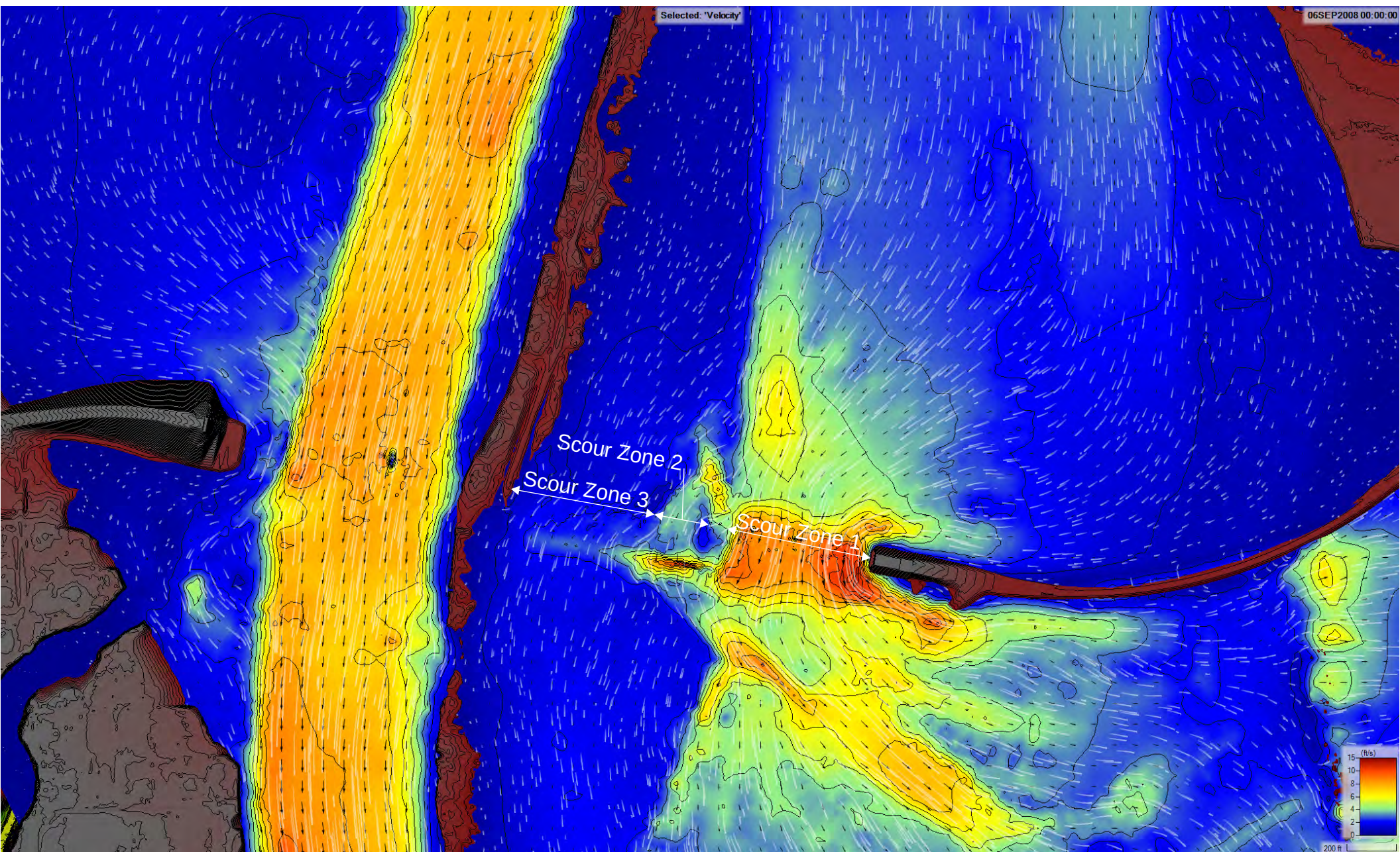
BURNS & MCDONNELL
ENGINEERING COMPANY, INC.
1010 N. 102ND COURT, SUITE 200
OMAHA, NE 68114
Office: 402-408-3013
Nebraska C.O.A. #CA-0621

XXX
PROJECT NUMBER STP-51-7(109)
C.N. 32395
LOCATION Decatur Bridge
STRUCTURE NUMBER S051 03644
2730'-0" 14-SPAN MISSOURI RIVER BRIDGE PLAN & ELEVATION - SHEET 2 OF 3 DATE AUG 2025 Q.C. ENGINEER: J. Sarson
SKW 0° ROADWAY 40'-0" FHWA CONST. TYPE X441 DESIGN LIVE LOAD HL-93 COUNTY Burt HWY. NO. N-51 REF. POST. XXX.XX STA. 2125+97.10 DESIGN CODE LRFD 10th Edition DESIGNED BY: A. Ferdon DETAILED BY: T. McDowell
NEBRASKA Good Life. Great Journey. DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION 1500 Nebraska Parkway Lincoln, NE 68502 Office: 402-479-4701 STATE BRIDGE ENGINEER Ross D. Barron, P.E. State of Nebraska E-14018
PRELIMINARY PLAN SUBJECT TO CHANGE
SPECIAL PLAN NO. X
3 9

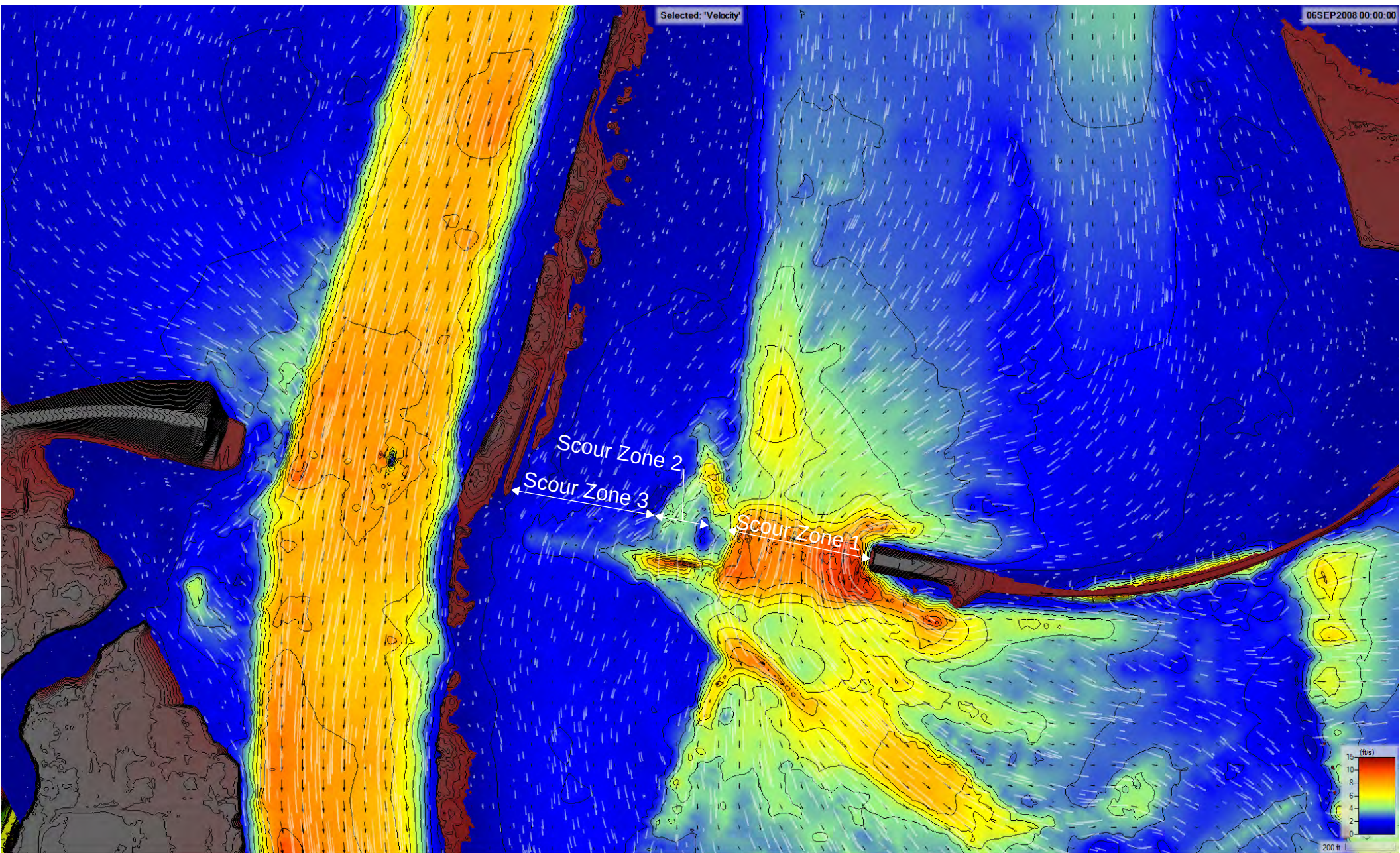
Terrain: Proposed Conditions (1' Contours)



Q200 Velocity (1ft/s Contours)



Q500 Velocity (1ft/s Contours)



C. Contraction Scour

Contraction Scour Notes:

- Secondary Channel contraction scour values assumed to be clear-water contraction scour. Based upon upstream terrain, flow vectors, and review of 2011 flood it seems a lower likelihood that there will be enough sediment to brought from upstream to have a live bed condition.
- D50 values take from preliminary geotechnical borings. 0.3 mm is typical value over first several layers in Zones 1-3. Use 0.2 mm in Main Channel.
- See attached exhibit for width of each zone and profile line the discharge is measured over. For contraction scour discharge through the bridge opening is the only location needed. Profile lines in model labeled "Zone 1", "Zone 2", and "Zone 3".
- Contraction Scour for main channel calculated using FHWA Hydraulic Toolbox Version 5.4. See attached input/output. Main channel allowed to calculate either live bed or clear-water equations.
- Depth for secondary channel taken from RAS Mapper depth layer at centerline bridge at the pier used for pier scour calculations. In main channel, depth is taken as a weighted average from the "RAS Mapper (depth layer) profile line table." Profile lines are "Scour-Main-Channel-Bridge" and "Scour - Upstream Main Channel". Weighted average used for main channel instead of RAS Mapper value at pier due to varying depth along bridge centerline in the main channel compared to more consistent values in the secondary channel.

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3} W^2} \right]^{3/7} \quad (6.4)$$

$$y_s = y_2 - y_o = (\text{average contraction scour depth}) \quad (6.5)$$

where:

- y_2 = Average equilibrium depth in the contracted section after contraction scour, ft (m)
- Q = Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W , ft³/s (m³/s)
- D_m = Diameter of the smallest nontransportable particle in the bed material (1.25 D_{50}) in the contracted section, ft (m)
- D_{50} = Median diameter of bed material, ft (m)
- W = Bottom width of the contracted section less pier widths, ft (m)
- y_o = Average existing depth in the contracted section, ft (m)
- K_u = 0.0077 English units
- K_u = 0.025 SI units

Equation 6.4 is a rearranged version of Equation 6.1. As discussed in Section 6.7, a reasonable lower limit of D_{50} equal to 0.2 mm can be applied to this equation. Using a size smaller than 0.2 mm will over-estimate clear-water contraction scour.

Because D_{50} is not the largest particle in the bed material, the scoured section can be slightly armored. Therefore, the D_m is assumed to be 1.25 D_{50} . For stratified bed material the depth of scour can be determined by using the clear-water scour equation sequentially with successive D_m of the bed material layers.

Zone 3

Model/Location	Flow	D50 (mm)	Discharge Bridge (cfs)	Width Bridge (ft)	Depth Bridge (ft)	Contraction Scour
PropFinal	Q10	0.3	0	583	0.0	0.0
	Q25	0.3	2	583	1.8	0.0
	FEMA Q100	0.3	25	583	2.9	0.0
	2011 Flood	0.3	1647	583	5.7	0.0
	Q100	0.3	2878	583	7.0	0.0
	Q200	0.3	4801	583	8.6	0.0
	Q500	0.3	5483	583	9.1	0.0

Zone 2

Model/Location	Flow	D50 (mm)	Discharge Bridge (cfs)	Width Bridge (ft)	Depth Bridge (ft)	Contraction Scour
PropFinal	Q10	0.3	0	212	0.0	0.0
	Q25	0.3	2	212	15.6	0.0
	FEMA Q100	0.3	26	212	16.8	0.0
	2011 Flood	0.3	4292	212	19.6	0.0
	Q100	0.3	7733	212	20.8	0.0
	Q200	0.3	12180	212	22.4	4.7
	Q500	0.3	13520	212	22.9	6.8

Zone 1

Model/Location	Flow	D50 (mm)	Discharge Bridge (cfs)	Width Bridge (ft)	Depth Bridge (ft)	Contraction Scour
PropFinal	Q10	0.3	2784	576	4.4	0.0
	Q25	0.3	14548	576	7.8	5.6
	FEMA Q100	0.3	21041	576	8.9	9.5
	2011 Flood	0.3	38365	576	11.8	19.0
	Q100	0.3	47064	576	13.1	23.6
	Q200	0.3	59333	576	14.7	30.1
	Q500	0.3	62861	576	15.2	31.8

Main Channel

Model/Location	Flow	D50 (mm)	Discharge Bridge (cfs)	Width Bridge (ft)	Depth Bridge (ft)	Contraction Scour*
PropFinal	Q200					3.2
	Q500					3.6

*Contraction Scour from FHWA Hydraulic Toolbox

Main Channel Q200 Contraction Scour:

Contraction Scour

Computation Method:
Clear-Water and Live-Bed Scour

Parameter	Value	Units	Notes
Pressure Scour Method			
Compute Pressure Scour (Vertical Contraction Scour)	☐		
Input Parameters			
Input Parameters for Scour Condition			
Average Depth Upstream of Contraction (y1)	31.03	ft	(hu) in pressure scour
D50 (D50)	0.199949	mm	0.2 mm is the lower limit for noncohesive material
Average Velocity Upstream (V)	6.15	ft/s	
Computed Contraction Scour Condition	Live Bed		Vc < V
Input Parameters for Live Bed			
Temperature of Water	60.00	°F	Used to determine the fall velocity
Slope of Energy Grade Line at Approach Section (S1)	0.000280	ft/ft	
Discharge in Contracted Section (Q2)	182313.00	cfs	
Discharge Upstream that is Transporting Sediment (Q1)	171225.00	cfs	
Bottom Width in Contracted Section (W2)	708.00	ft	Remove widths occupied by piers
Width Upstream that is Transporting Sediment (W1)	780.00	ft	
Depth Prior to Scour in Contracted Section (y0)	31.85	ft	
Unit Weight of Water (gamma w)	62.40	lb/ft^3	
Unit Weight of Sediment (gamma s)	165.00	lb/ft^3	
Results			
Results of Scour Condition			
Critical velocity above which bed material of size D and smaller will be transported (Vc)	1.72	ft/s	
Results of Clear Water Method			
Diameter of the smallest nontransportable particle in the bed material (Dm)	0.249936	mm	
Average Depth in Contracted Section after Scour (y2)	110.25	ft	
Scour Depth (ys)	78.40	ft	Negative values imply 'zero' scour depth
Results of Live Bed Method			
k1 (k1)	0.690000		
Shear Velocity (V*)	0.53	ft/s	
Fall Velocity (T)	0.08	ft/s	
Average Depth in Contracted Section after Scour (y2)	35.01	ft	
Scour Depth (ys)	3.16	ft	Negative values imply 'zero' scour depth
Shear Applied to Bed by Live-Bed Scour (theta 0)	0.5024	lb/ft^2	
Shear Required for Movement of D50 Particle (Tau c)	0.0026	lb/ft^2	
Recommendations			
Recommended Scour Condition	Live Bed		Determined by comparing scour depths (including long-term degradation)
Recommended Scour Depth	3.16	ft	Negative values imply 'zero' scour depth
Override Recommended Scour Condition	☐		

OK
Cancel

Main Channel Q500 Contraction Scour:

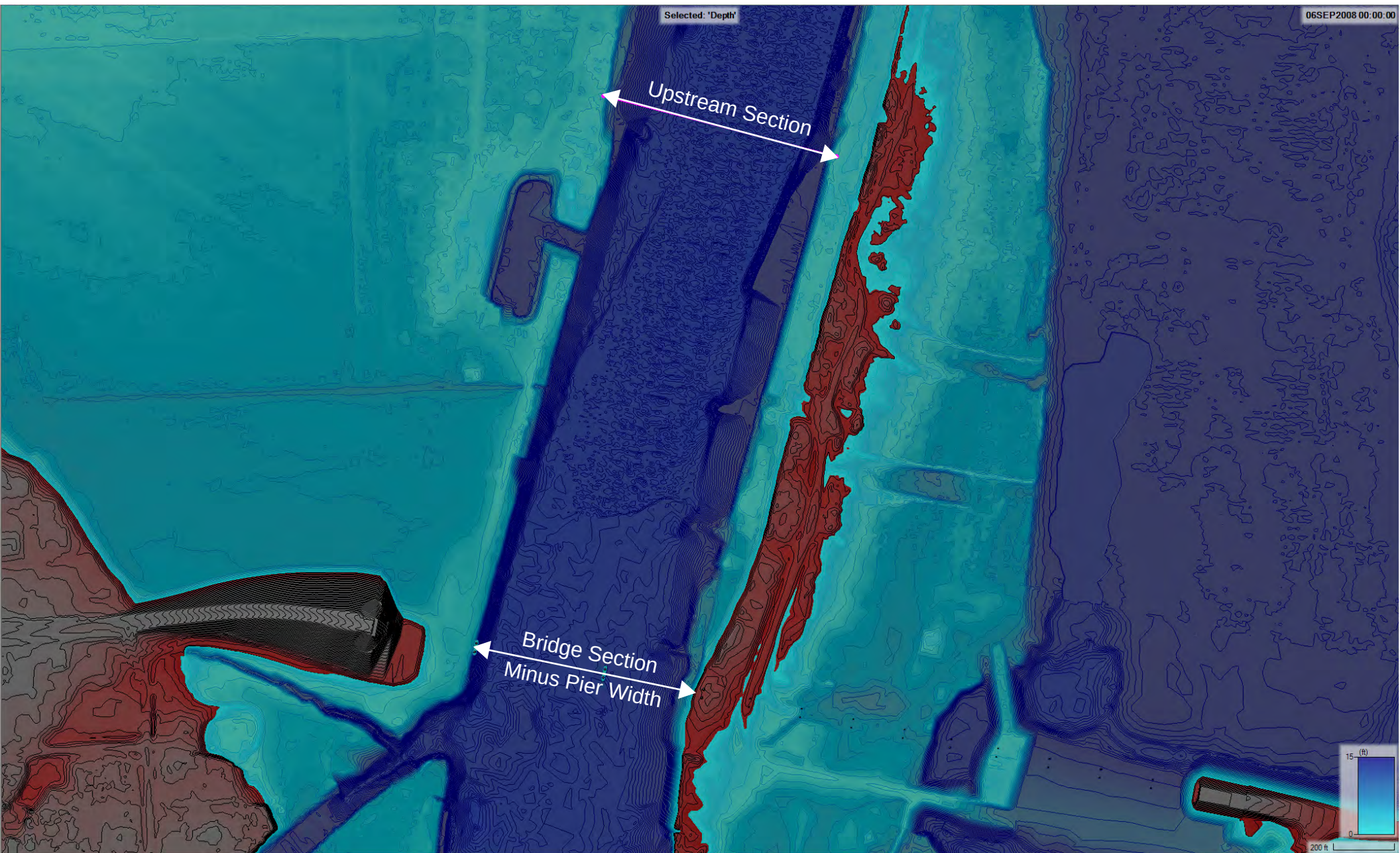
Contraction Scour

Computation Method:
Clear-Water and Live-Bed Scour

Parameter	Value	Units	Notes
Pressure Scour Method			
Compute Pressure Scour (Vertical Contraction Scour)	☐		
Input Parameters			
Input Parameters for Scour Condition			
Average Depth Upstream of Contraction (y1)	31.73	ft	(hu) in pressure scour
D50 (D50)	0.200000	mm	0.2 mm is the lower limit for noncohesive material
Average Velocity Upstream (V)	6.20	ft/s	
Computed Contraction Scour Condition	Live Bed		Vc < V
Input Parameters for Live Bed			
Temperature of Water	60.00	°F	Used to determine the fall velocity
Slope of Energy Grade Line at Approach Section (S1)	0.000280	ft/ft	
Discharge in Contracted Section (Q2)	187957.00	cfs	
Discharge Upstream that is Transporting Sediment (Q1)	174317.00	cfs	
Bottom Width in Contracted Section (W2)	708.00	ft	Remove widths occupied by piers
Width Upstream that is Transporting Sediment (W1)	780.00	ft	
Depth Prior to Scour in Contracted Section (y0)	32.57	ft	
Unit Weight of Water (gamma w)	62.40	lb/ft^3	
Unit Weight of Sediment (gamma s)	165.00	lb/ft^3	
Results			
Results of Scour Condition			
Critical velocity above which bed material of size D and smaller will be transported (Vc)	1.73	ft/s	
Results of Clear Water Method			
Diameter of the smallest nontransportable particle in the bed material (Dm)	0.250000	mm	
Average Depth in Contracted Section after Scour (y2)	113.16	ft	
Scour Depth (ys)	80.59	ft	Negative values imply 'zero' scour depth
Results of Live Bed Method			
k1 (k1)	0.690000		
Shear Velocity (V*)	0.53	ft/s	
Fall Velocity (T)	0.08	ft/s	
Average Depth in Contracted Section after Scour (y2)	36.19	ft	
Scour Depth (ys)	3.62	ft	Negative values imply 'zero' scour depth
Shear Applied to Bed by Live-Bed Scour (theta 0)	0.5054	lb/ft^2	
Shear Required for Movement of D50 Particle (Tau c)	0.0026	lb/ft^2	
Recommendations			
Recommended Scour Condition	Live Bed		Determined by comparing scour depths (including long-term degradation)
Recommended Scour Depth	3.62	ft	Negative values imply 'zero' scour depth
Override Recommended Scour Condition	☐		

OK
Cancel

Main Channel Contraction Locations



D. Pier Scour



STATE OF NEBRASKA
DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

PROJ. LOC.: Decatur Bridge PROJ. NO.: STP-51-7(109) BM PROJ. #: 157887
COUNTY: Burt STATION: ###+###.## MADE BY: JTK
CN: 32395 STRUCTURE: S051 03644 DATE: 06/09/25
DESC: Phase 2 Hydro Strategy Evaluation CHECK BY: ---

Pier Scour Notes:

- RAS Mapper layer created to create multiplier so that pier diameter/spacing and angle of attack are the only spreadsheet inputs.
See separate attached hand calc of RAS Mapper layer equation.
- Value selected from RAS Mapper multiplier max around columns in direction of flow or upstream of columns if pier was causing local reduction in Froude number due to obstruction (i.e. Main River Channel Pier)
- For secondary channel, just using worst pier in each "zone" for pier scour design value.
- Drilled shafts sticking just above ground in overbank and above streambed in main stream. Use drilled shaft diameter not column diameter in calculation
- Pier 4 on high ground does not experience flow. For scour design, using flow in channel upstream near the bank for the scenario that in future the channel bank is cut back further and pier would see flow.
- See attached pier scour angle of attack exhibits for measured values.

Zone 3 (Pier West of Scour Hole)

Model/Location	Flow	Pier Diameter	Pier Spacing (Total)*	Angle of Attack	K2 Factor	RAS Mapper Multiplier	Pier Scour
PropFinal	Q10	7.5	23	0	1.000	0.00	0.0
	Q25	7.5	23	0	1.000	0.20	0.7
	FEMA Q100	7.5	23	0	1.000	0.42	1.6
	2011 Flood	7.5	23	0	1.000	1.38	5.1
	Q100	7.5	23	0	1.000	1.63	6.0
	Q200	7.5	23	0	1.000	1.86	6.9
	Q500	7.5	23	0	1.000	1.92	7.1

Zone 2 (Pier in Scour Hole)

Model/Location	Flow	Pier Diameter	Pier Spacing (Total)*	Angle of Attack	K2 Factor	RAS Mapper Multiplier	Pier Scour
PropFinal	Q10	7.5	23	18	1.517	0.00	0.0
	Q25	7.5	23	18	1.517	0.08	0.4
	FEMA Q100	7.5	23	18	1.517	0.23	1.3
	2011 Flood	7.5	23	18	1.517	1.87	10.5
	Q100	7.5	23	18	1.517	2.37	13.3
	Q200	7.5	23	18	1.517	2.80	15.7
	Q500	7.5	23	18	1.517	2.90	16.3

Zone 1 (Pier Closest to Abutment)

Model/Location	Flow	Pier Diameter	Pier Spacing (Total)*	Angle of Attack	K2 Factor	RAS Mapper Multiplier	Pier Scour
PropFinal	Q10	7.5	23	7	1.225	1.46	6.6
	Q25	7.5	23	7	1.225	2.50	11.3
	FEMA Q100	7.5	23	7	1.225	2.80	12.7
	2011 Flood	7.5	23	7	1.225	3.34	15.2
	Q100	7.5	23	7	1.225	3.54	16.1
	Q200	7.5	23	7	1.225	3.78	17.2
	Q500	7.5	23	7	1.225	3.84	17.4

*Pier Spacing is from face to face of column per HEC-18 Section 7.6.
Confirmed K2 calculation with FHWA Hydraulic Toolbox calculation.



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CN: 32395 STRUCTURE: S051 03644 DATE: 06/09/25
DESC: Phase 2 Hydro Strategy Evaluation CHECK BY: ---

Pier Scour Notes:

- RAS Mapper layer created to create multiplier so that pier diameter/spacing and angle of attack are the only spreadsheet inputs. See separate attached hand calc of RAS Mapper layer equation.
- Value selected from RAS Mapper multiplier max around columns in direction of flow or upstream of columns if pier was causing local reduction in Froude number due to obstruction (i.e. Main River Channel Pier)
- For secondary channel, just using worst pier in each "zone" for pier scour design value.
- Drilled shafts sticking just above ground in overbank and above streambed in main stream. Use drilled shaft diameter not column diameter in calculation
- Pier 4 on high ground does not experience flow. For scour design, using flow in channel upstream near the bank for the scenario that in future the channel bank is cut back further and pier would see flow.

Main Channel Pier (Pier 3)

Model/Location	Flow	Pier Diameter	Pier Spacing (Total)*	Angle of Attack	K2 Factor	RAS Mapper Multiplier	Pier Scour
PropFinal	Q200	12	24	5	1.108	4.14	23.1
	Q500	12	24	5	1.108	4.17	23.2

Pier 2

Model/Location	Flow	Pier Diameter	Pier Spacing (Total)*	Angle of Attack	K2 Factor	RAS Mapper Multiplier	Pier Scour
PropFinal	Q200	12	24	5	1.108	2.80	15.6
	Q500	12	24	5	1.108	2.83	15.8

Pier 4

Model/Location	Flow	Pier Diameter	Pier Spacing (Total)*	Angle of Attack	K2 Factor	RAS Mapper Multiplier	Pier Scour
PropFinal	Q200	12	24	5	1.108	1.30	7.2
	Q500	12	24	5	1.108	1.40	7.8

*Pier Spacing is from face to face of column per HEC-18 Section 7.6. Total pier spacing is number of spaces times the space.
Confirmed K2 calculation with FHWA Hydraulic Toolbox calculation.

Pier Scour Multiplier

- Pier Scour per HEC-18 5th edition
- Calculate "multiplier" from constants in the equation to plot within 2D HEC-RAS

$$\frac{y_s}{y_1} = 2.0 K_1 K_2 K_3 \left(\frac{q}{y_1} \right)^{0.65} F_r^{0.43} \quad (7.1)$$

Let $K_1 = 1.0$ for cylinder or group of cylinders

$K_3 = 1.1$ Clear Water Scour + Plane Bed

y_s = Scour Depth (ft)

y_1 = Flow Depth (ft) ← RAS Mapper Variable

K_1 = Correction Factor, Pier Nose

K_2 = Correction Factor, Angle of Attack

K_3 = Correction Factor, Bed Condition

q = pier width (ft)

F_r = Froude Number ← RAS Mapper Variable

$$y_s = 2(1) K_2 (1.1) y_1 \left(\frac{q}{y_1} \right)^{0.65} F_r^{0.43}$$

$$y_s = 2.2 y_1 \frac{q^{0.65}}{y_1^{0.65}} F_r^{0.43} K_2$$

$$\star \quad y_s = K_2 q^{0.65} \left[2.2 \frac{q^{0.35}}{y_1^{0.35}} F_r^{0.43} \right]$$

Pier Scour Multiplier in RAS Mapper (Calculated Layer)

→ Use depth map layer for y_1

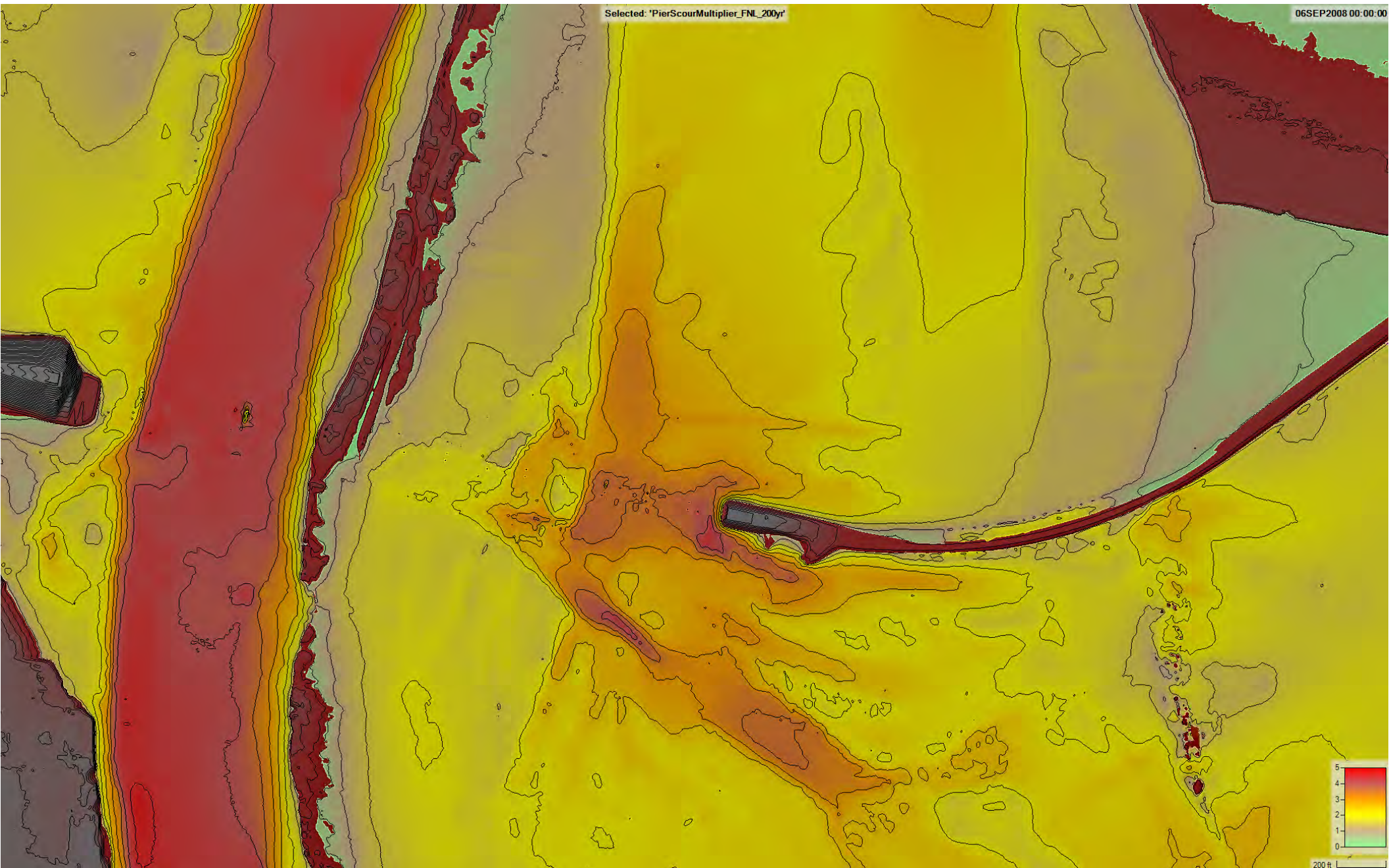
→ Use Froude Number map layer for F_r

In Excel:

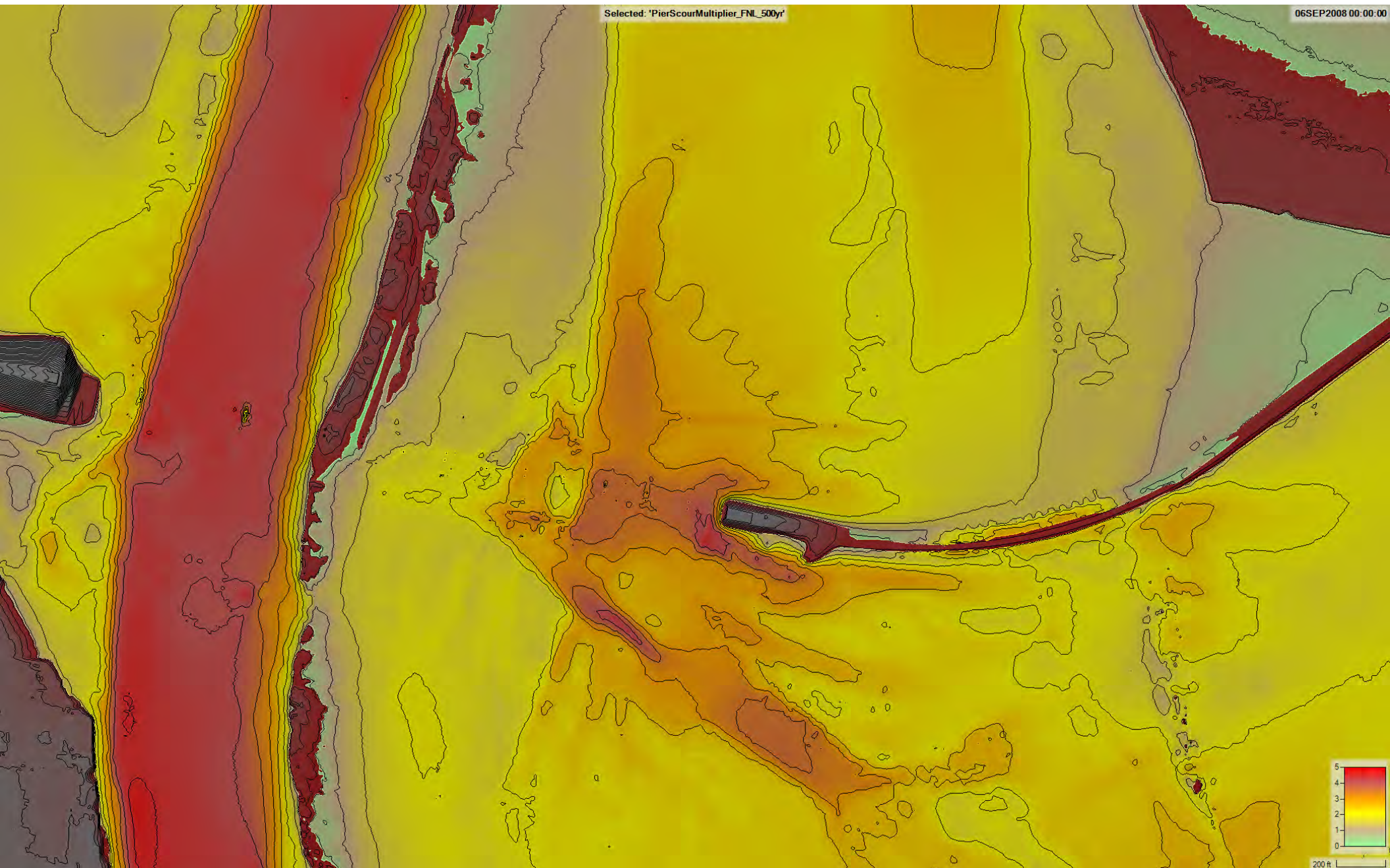
$$\star \quad y_s = q^{0.65} \times K_2 \times \text{Pier Scour Multiplier}$$



Q200 Pier Scour Multiplier (0.5 Contours)



Q500 Pier Scour Multiplier (0.5 Contours)



Revised Isbash Method (2001)

Median Riprap Size, where 50% is finer by weight

Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design
Guidance-Third Edition Volume 2 (HEC 23)

Isbash Method (Revised 2001)

HEC 23 Vol 2, Section 11.3, Equation 11.1
p. 216 of pdf

Notes:

$K_1 =$	1.5		1.5 for round-nose piers, 1.7 for square-face piers
$K_2 =$	0.9		for pier in straight reach
$V_{avg} =$	N/A	ft/s	Average Velocity
$g =$	32.2	ft/s ²	Acceleration due to gravity
$S_g =$	2.4		Specific Gravity of Riprap
$V_{max} =$	9.1	ft/s	Maximum Velocity (in secondary channel at bridge)
$V_{des} =$	13.65	ft/s	Design Velocity (local value known from 2D Model)
$D_{50} =$	1.43 ft 17.16 in		Median Riprap Size

Use same riprap as at Abutment 2. Define a custom NDOT Type similar to the Iowa Class X used in the 2011 repair.

$$d_{50} = \frac{0.692(V_{des})^2}{(S_g - 1)2g} \quad (11.1)$$

where:

- d_{50} = Particle size for which 50% is finer by weight, ft (m)
- V_{des} = Design velocity for local conditions at the pier, ft/s (m/s)
- S_g = Specific gravity of riprap (usually taken as 2.65)
- g = Acceleration due to gravity, 32.2 ft/s² (9.81 m/s²)

$$V_{des} = K_1 K_2 V_{avg} \quad (11.2)$$

$$V_{des} = K_1 V_{max} \quad (11.3)$$

where:

- V_{des} = Local velocity at pier, ft/s (m/s)
- K_1 = Shape factor equal to 1.5 for round-nose piers or 1.7 for square-faced piers
- K_2 = Velocity adjustment factor for location in the channel (ranges from 0.9 for a pier near the bank in a straight reach, to 1.7 for a pier located in the main current of flow around a sharp bend)
- V_{avg} = Channel average velocity at the bridge, ft/s (m/s)
- V_{max} = Maximum velocity in the active channel, ft/s (m/s)

Revised Isbash Method (2001)

Median Riprap Size, where 50% is finer by weight

*Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design
Guidance-Third Edition Volume 2 (HEC 23)*

Pier Riprap Extents:

a =	7.5	ft	Pier (shaft) Diameter
2a =	15.0	ft	
t =	4.3	ft	thickness of riprap layer

Extend pier a minimum of 15' around each shaft and a thickness of 4.5 feet.

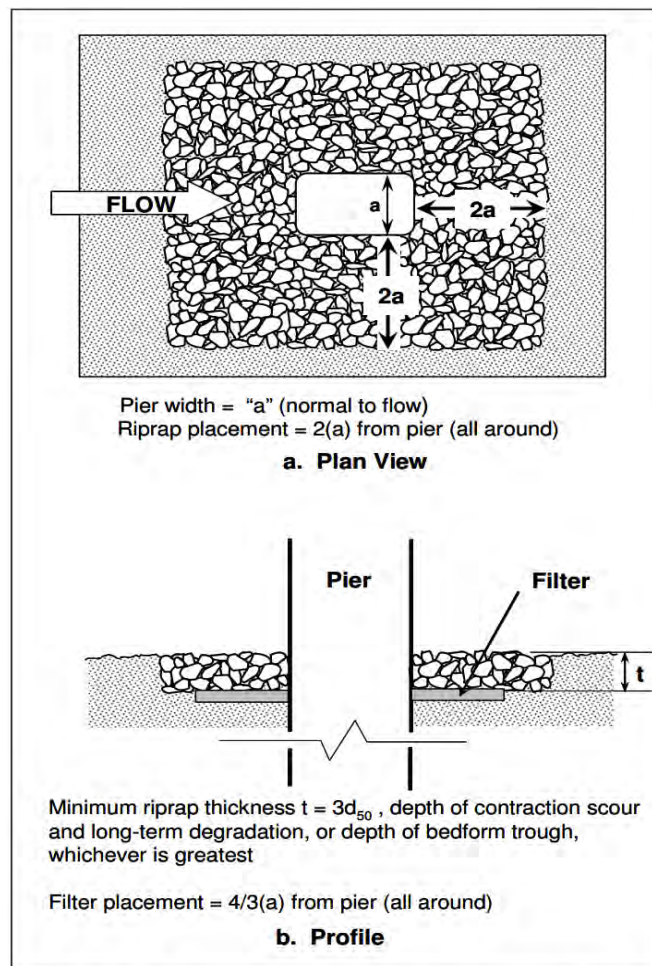


Figure 11.15. Riprap layout diagram for pier scour protection.

E. Abutment Scour



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CN: 32395	STRUCTURE: S051 03644	DATE: 08/11/25
DESC: Abutment Scour	CHECK BY:	

East Abutment Scour - Design Approach

AASHTO LRFD Section:



(Gray cells are User Inputs)

Steps in Design Process

Calculated for Q200 and Q500. Controlling output provided.

1) Calculate scour at East Abutment (y_s)

- a) Method: NCRHP 24-20; total abutment scour = amplification of contraction scour
- b) Case B: Abutment "far" from main channel bank; clear water scour
- c) Spill-through abutment

Primary Countermeasure: Place riprap apron to "push" location of max scour further from abutment face

2) Determine minimum apron width (W) to avoid undermining of front slope toe

- a) Method: Countermeasures for Toe Protection of Spill-through Abutments (Melville et. al., 2006)
- b) Note: same method is referenced in NCHRP Report 587 and HEC-23

3) Determine distance from face of abutment to calculated max scour depth (R)

- a) Method: Countermeasures for Toe Protection of Spill-through Abutments (Melville et. al., 2006)
- b) Include effect of riprap apron, W
- c) Define "R" from front face of abutment (NCHRP 24-20), rather than toe of front slope (Melville 2006)

4) Calculate "geotechnical limit to scour" ($d_{s,max}$)

- a) $d_{s,max}$ = theoretical max scour depth before front slope fails
- b) Method: NCHRP 24-20
- c) Use results from Boring B-4 to estimate friction angle of front slope material and subgrade
- d) If $d_{s,max} > y_s$
 - front slope is theoretically stable
 - theoretically, scour at front face of abutment would not occur
- e) If $d_{s,max} < y_s$
 - front slope would theoretically fail before y_s is reached
 - failure of front slope would "open up" the conveyance area
 - scour process would change to a different model
- d) Goal: increase $d_{s,max}$ to be above y_s ; revise W to increase R

5) Extend riprap apron along upstream & downstream sides of embankment

- a) Method: HEC-23

6) Check riprap layout against shear stress plot from HEC-RAS

- a) Increase footprint if needed to address high shear stress areas



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CN:	32395	STRUCTURE:	S051 03644	DATE:	08/11/25
DESC:	Abutment Scour	CHECK BY:			

East Abutment Scour - Design Approach

AASHTO LRFD Section:

-
- 7) Determine minimum riprap size (D50)
 - a) Method: HEC-23

Additional Secondary Countermeasure:

Design abutment to accommodate scour; Use sheet pile wall. Use tie rods & anchor block if required.

- 8) Assume that front slope failure may occur. Calculate scour at face of abutment.
 - a) Assume an X:1 failed slope from deepest scour at R back to abutment
 - b) Determine "X" using a conservative assumption of the angle of repose
 - c) Assume angle of repose ~ friction slope
 - d) Compare scour at face of abutment with NDOT minimum scour in NDOT "Hydraulic Analysis Guidelines". Use controlling scour.
- 9) Design sheet pile abutment
 - a) Determine friction angle and soil weight from Boring B-4
 - b) Use tie rods and anchor block if required

East Abutment Scour - NCHRP Method Q200

AASHTO LRFD Section:



(Gray cells are User Inputs)

Case B - Spill-through Abutment

Description

- abutment "far" from main channel bank
- clear water scour

$q_f = 40 \text{ ft}^2/\text{s}$
 $q_{2f} = 150 \text{ ft}^2/\text{s}$
 $K_u = 11.17$
 $D_{50,mm} = 0.3 \text{ mm}$
 $D_{50} = 0.00098 \text{ ft}$

unit discharge of floodplain flow upstream of bridge

unit discharge in constricted opening, accounting for non-uniform flow distribution

dimensionless constant, English units

particle size with 50 percent finer

convert to feet

$$D_{50} = D_{50,mm} \times (1 \text{ ft}/304.8 \text{ mm})$$

$y_c = 67.0 \text{ ft}$

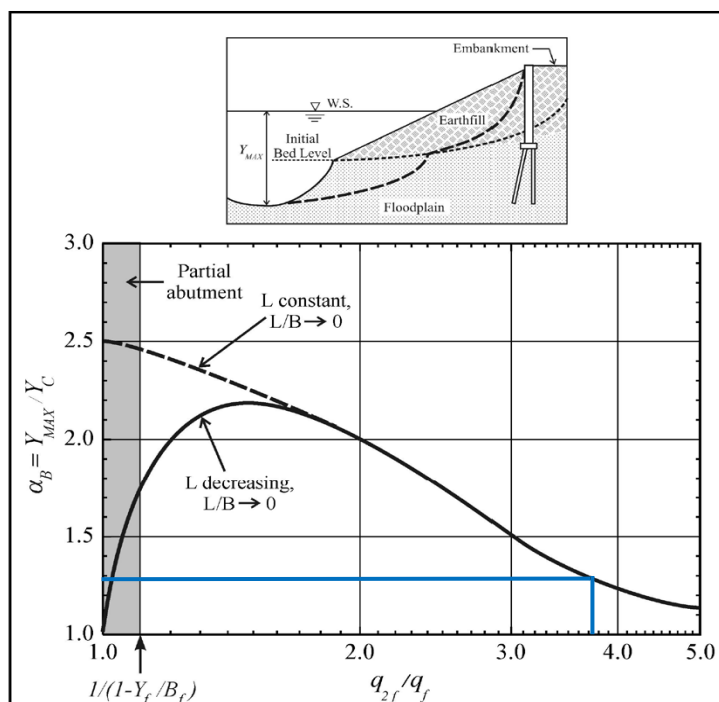
flow depth including clear water contraction scour

$$y_c = \left(\frac{q_{2f}}{K_u (D_{50})^{1/3}} \right)^{6/7}$$

$q_{2f}/q_f = 3.75$

$\alpha_B = 1.28$

amplification factor for spill through abutments & clear water (from graph)





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COUNTY:	Burt	STATION:	###+###.##	MADE BY:	JTK
CN:	32395	STRUCTURE:	S051 03644	DATE:	08/11/25
DESC:	Abutment Scour	CHECK BY:			

East Abutment Scour - NCHRP Method Q200

AASHTO LRFD Section:

$y_0 =$	14.8	ft
$y_{max} =$	85.7	ft
$y_s =$	70.9	ft

flow depth prior to scour

max flow depth from abutment scour

$$y_{max} = \alpha_B y_c$$

abutment scour depth

$$y_s = y_{max} - y_0$$

(Note: this is the max scour depth at a distance "R" from the abutment face)

East Abutment Scour - NCHRP Method Q500

AASHTO LRFD Section:



(Gray cells are User Inputs)

Case B - Spill-through Abutment

Description

- abutment "far" from main channel bank
- clear water scour

$q_f = 40 \text{ ft}^2/\text{s}$
 $q_{2f} = 160 \text{ ft}^2/\text{s}$
 $K_u = 11.17$
 $D_{50,mm} = 0.3 \text{ mm}$
 $D_{50} = 0.00098 \text{ ft}$

unit discharge of floodplain flow upstream of bridge

unit discharge in constricted opening, accounting for non-uniform flow distribution

dimensionless constant, English units

particle size with 50 percent finer

convert to feet

$$D_{50} = D_{50,mm} \times (1 \text{ ft}/304.8 \text{ mm})$$

$y_c = 70.8 \text{ ft}$

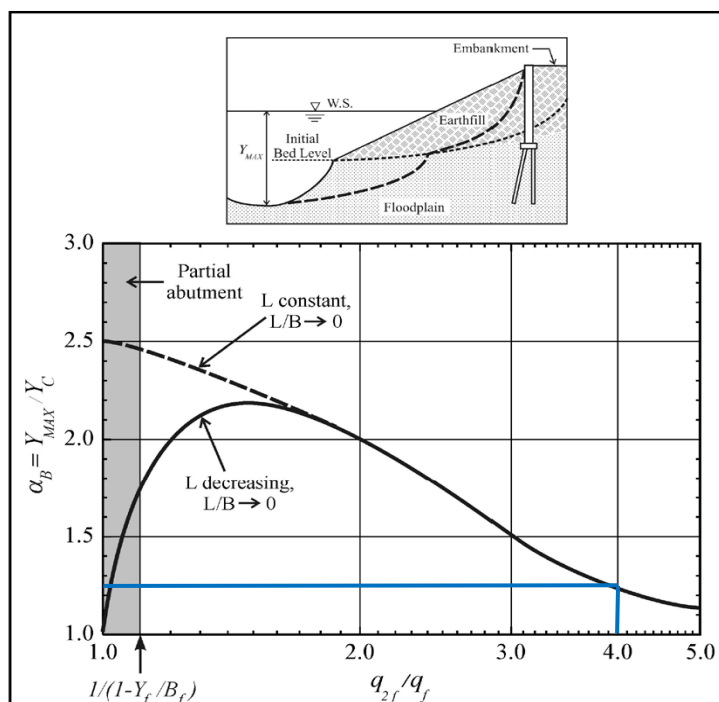
flow depth including clear water contraction scour

$$y_c = \left(\frac{q_{2f}}{K_u (D_{50})^{1/3}} \right)^{6/7}$$

$q_{2f}/q_f = 4$

$\alpha_B = 1.24$

amplification factor for spill through abutments & clear water (from graph)





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COUNTY:	Burt	STATION:	####.##	MADE BY:	JTK
CN:	32395	STRUCTURE:	S051 03644	DATE:	08/11/25
DESC:	Abutment Scour	CHECK BY:			

East Abutment Scour - NCHRP Method Q500

AASHTO LRFD Section:

$y_0 =$	15.3	ft
$y_{max} =$	87.8	ft
$y_s =$	72.5	ft

flow depth prior to scour

max flow depth from abutment scour

$$y_{max} = \alpha_B y_c$$

abutment scour depth

$$y_s = y_{max} - y_0$$

(Note: this is the max scour depth at a distance "R" from the abutment face)



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CN: 32395	STRUCTURE: S051 03644	DATE: 08/11/25
DESC: Abutment Riprap	CHECK BY:	

East Abutment - Riprap Apron Extent Q200

AASHTO LRFD Section:



(Gray cells are User Inputs)

Assumptions

- Spill-through abutment
- NCHRP Scour Case B: abutment is set back from main channel, clear-water scour
- use Method 2 --> Method 1 shown for comparison

Apron Width, W

Method 1

- Source: HEC-23 Section 14.2

$y_0 =$	14.8	ft	flow depth in overbank area near the embankment
$W_1 =$	29.6	ft	$W_1 = 2 \times y_0$
$W_{max} =$	25	ft	max recommended width (per Atayee et al. 1993)
$W =$	25	ft	apron width $W = Min(W_1, W_{max})$

Method 2

- Source 1: Countermeasure Toe Protection at Spill-Through Abutments (Melville et al. 2006)
- Source 2: NCHRP 587 - Countermeasures to Protect Bridge Abutments from Scour

Step 1: determine minimum apron extent (W_o) to prevent undermining of front slope toe

$d_{sf} =$	70.9	ft	local scour depth measured below flood channel bed level
$y_f =$	14.8	ft	flow depth in flood channel (i.e. in front of abutment); same as y_0 in Method 1
$C_3 =$	0.5		Constant; 0.5 for riprap, 1.4 for cable-tied block (CTB)
$\lambda =$	1.35		Constant; 1.35 for riprap, 1.0 for CTB

$$W_o = \text{input } 61.3 \text{ ft} \quad W_o = y_f C_3 \left(\frac{d_{sf}}{y_f} \right)^\lambda$$

$$W = \text{input } 65 \text{ ft}$$

apron width to use

Note: this is the minimum apron width to theoretically avoid undermining of the toe of the front slope

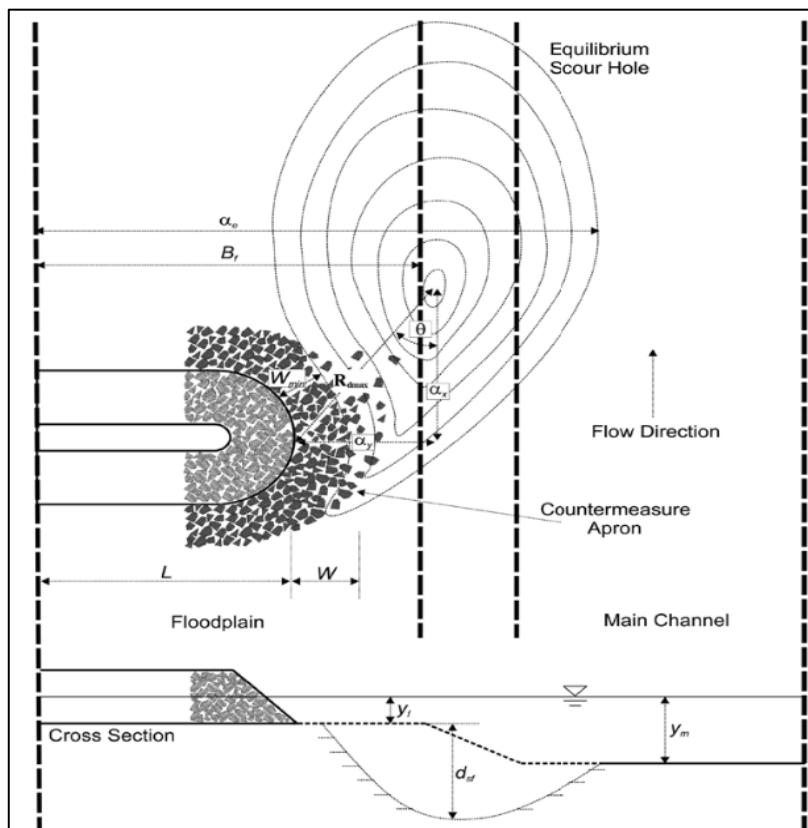
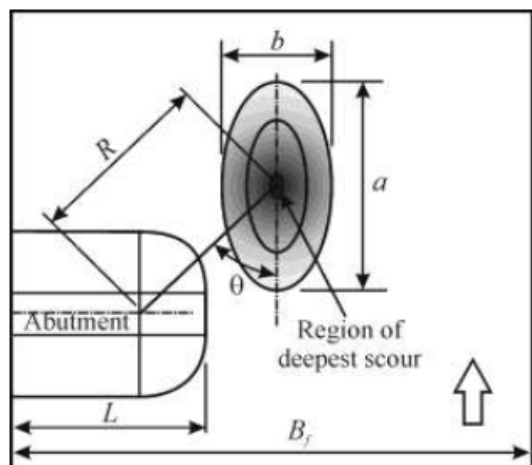
East Abutment - Riprap Apron Extent Q200

AASHTO LRFD Section:

**Figure 8-12 from NCHRP 587*

**Note: Figure 1 from Melville et. al. 2006 is similar*

NCHPR 24-20 Figure 9.4(a)



Location of Max Scour Depth, R

- Source: Countermeasure Toe Protection at Spill-Through Abutments (Melville et al. 2006)
- Define "R" from front face of abutment (NCHRP 24-20), rather than toe of front slope (Melville 2006)

L =	1100	ft
$(L/y_f)_1 =$	74.3	
$(L/y_f)_m =$	10	
$(L/y_f) =$	10.0	

length of roadway embankment causing a contraction

max ratio for which the equations in Melville (2006) were developed

Minimum of previous two values

$C_1 =$	4
$\phi =$	0.4

Constant; 4 for riprap

Constant; 0.4 for riprap

R =	184.1	ft
-----	-------	----

$$R = y_f \times C_1 \left(\frac{L}{y_f} \right)^{0.2} \times \left(1 + \frac{W}{y_f} \right)^{\phi}$$

East Abutment - Riprap Apron Extent Q200

AASHTO LRFD Section:

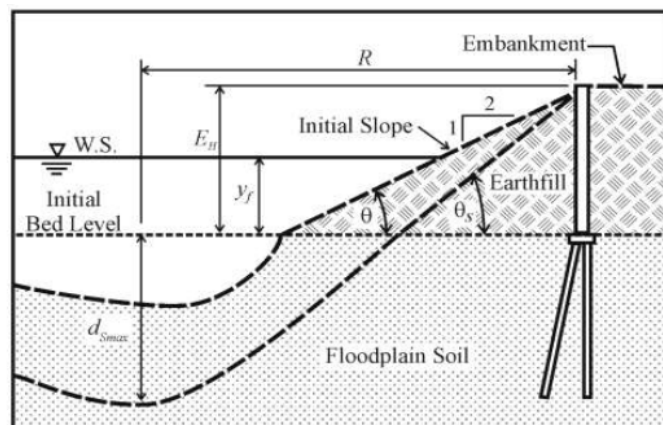
Calculate "geotechnical limit to scour" ($d_{s,max}$)

- Source: NCHRP 24-20

E_H =	24	ft	embankment height
θ_s =	32	deg	friction slope of earthfill and floodplain material (from Boring B-4)
$d_{s,max}$ =	91.0	ft	geotechnical limit to scour
y_s =	73.4	ft	calculated max scour depth
	Yes		$d_{s,max} > y_s$?

$$d_{s,max} = R \tan(\theta_s) - E_H$$

NCHRP 24-20 Figure 9.3(a)



if Yes:

- front slope is theoretically stable at y_s
- theoretical scour at front face of abutment = zero

if No:

- front slope would theoretically fail before y_s is reached
- failure of front slope would "open up" conveyance area
- scour process would change to different model

Goal: increase R by increasing W such that $d_{s,max} > y_s$

Apron on sides of embankment

- Source: HEC-23

- wrap around upstream and downstream side of embankment

L_{r1} =	25	ft	length of apron, 1
L_{r2} =	29.6	ft	length of apron, 2
L_r =	29.6	ft	$L_r = \text{Max}(L_{r1}, L_{r2})$
L_r =	80	ft	Value to use

$$L_{r2} = 2 \times y_f$$

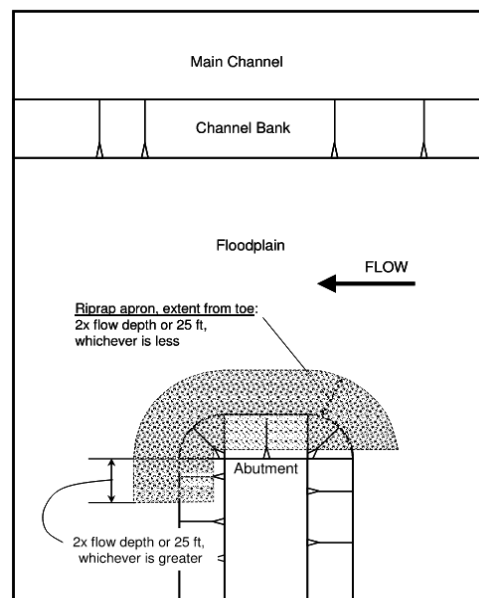
$$L_r = \text{Max}(L_{r1}, L_{r2})$$

Check: Plot riprap layout

Compare with shear stress plot heat map from HEC-RAS.

Comments: Length of apron on sides set at 80 feet to provide protection on wraparound regions with high shear stress. Extend apron on side slopes up to Elev. 1045

HEC-23 Figure 14.7





STATE OF NEBRASKA
DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

PROJ. LOC.:	Decatur Bridge	PROJ. NO.:	STP-51-7(109)	BM PROJ. #:	157887
COUNTY:	Burt	STATION:	###+###.##	MADE BY:	JTK
CN:	32395	STRUCTURE:	S051 03644	DATE:	08/11/25
DESC:	Abutment Riprap	CHECK BY:			

East Abutment - Riprap Apron Extent Q500 Check

AASHTO LRFD Section:



(Gray cells are User Inputs)

Assumptions

- Spill-through abutment
- NCHRP Scour Case B: abutment is set back from main channel, clear-water scour
- use Method 2 --> Method 1 shown for comparison

Apron Width, W

Method 1

- Source: HEC-23 Section 14.2

$y_0 =$	15.3	ft	flow depth in overbank area near the embankment
$W_1 =$	30.6	ft	$W_1 = 2 \times y_0$
$W_{\max} =$	25	ft	max recommended width (per Atayee et al. 1993)
$W =$	25	ft	apron width $W = \text{Min}(W_1, W_{\max})$

Method 2

- Source 1: Countermeasure Toe Protection at Spill-Through Abutments (Melville et al. 2006)
- Source 2: NCHRP 587 - Countermeasures to Protect Bridge Abutments from Scour

Step 1: determine minimum apron extent (W_o) to prevent undermining of front slope toe

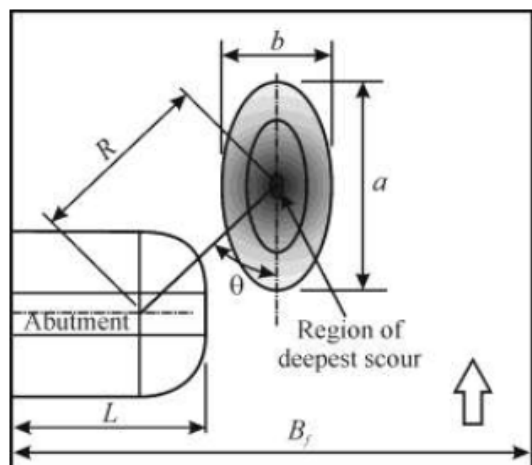
$d_{sf} =$	72.5	ft	local scour depth measured below flood channel bed level
$y_f =$	15.3	ft	flow depth in flood channel (i.e. in front of abutment); same as y_0 in Method 1
$C_3 =$	0.5		Constant; 0.5 for riprap, 1.4 for cable-tied block (CTB)
$\lambda =$	1.35		Constant; 1.35 for riprap, 1.0 for CTB
$W_o =$	62.5	ft	$W_o = y_f C_3 \left(\frac{d_{sf}}{y_f} \right)^\lambda$
$W =$	65	ft	apron width to use

East Abutment - Riprap Apron Extent Q500 Check

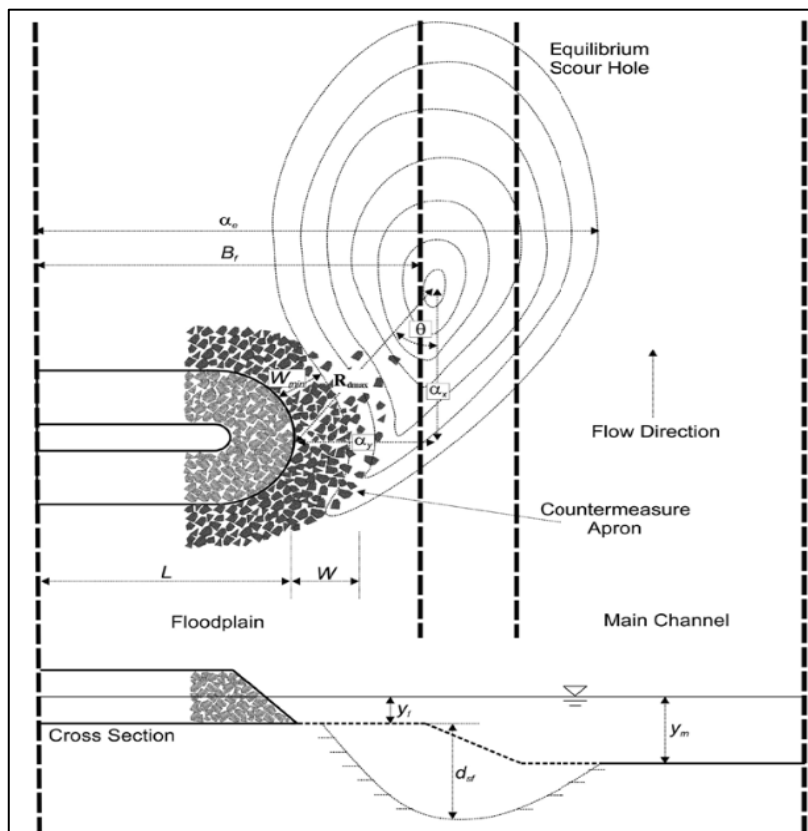
AASHTO LRFD Section:

*Note: Figure 1 from Melville et. al. 2006 is similar

NCHRP 24-20 Figure 9.4(a)



Location of Max Scour Depth, R



- Source: Countermeasure Toe Protection at Spill-Through Abutments (Melville et al. 2006)
- Define "R" from front face of abutment (NCHRP 24-20), rather than toe of front slope (Melville 2006)

$L =$	1100	ft
$(L/y_f)_1 =$	71.9	
$(L/y_f)_m =$	10	
$(L/y_f) =$	10.0	

length of roadway embankment causing a contraction

max ratio for which the equations in Melville (2006) were developed

Minimum of previous two values

$C_1 =$	4
$\phi =$	0.4

Constant; 4 for riprap

Constant; 0.4 for riprap

$R =$	188.3	ft
-------	-------	----

$$R = y_f \times C_1 \left(\frac{L}{y_f} \right)^{0.2} \times \left(1 + \frac{W}{y_f} \right)^{\phi}$$

East Abutment - Riprap Apron Extent Q500 Check

AASHTO LRFD Section:

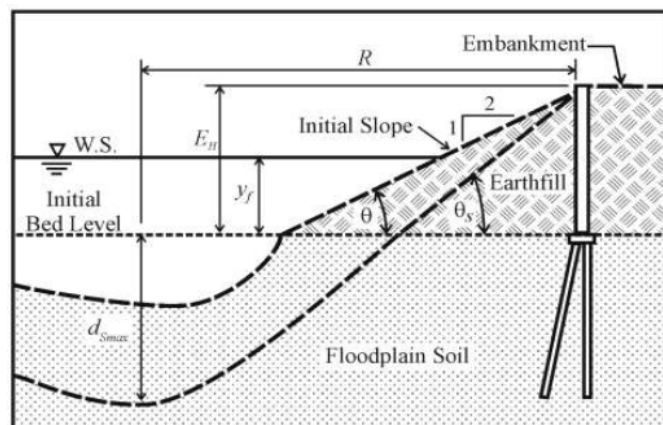
Calculate "geotechnical limit to scour" ($d_{s,max}$)

- Source: NCHRP 24-20

E_H =	24	ft	embankment height
θ_s =	32	deg	friction slope of earthfill and floodplain material (from Boring B-4)
$d_{s,max}$ =	93.6	ft	geotechnical limit to scour
y_s =	73.4	ft	calculated max scour depth
	Yes		$d_{s,max} > y_s$?

$$d_{s,max} = R \tan(\theta_s) - E_H$$

NCHRP 24-20 Figure 9.3(a)



if Yes:

- front slope is theoretically stable at y_s
- theoretical scour at front face of abutment = zero

if No:

- front slope would theoretically fail before y_s is reached
- failure of front slope would "open up" conveyance area
- scour process would change to different model

Goal: increase R by increasing W such that $d_{s,max} > y_s$

Apron on sides of embankment

- Source: HEC-23

- wrap around upstream and downstream side of embankment

L_{r1} =	25	ft	length of apron, 1
L_{r2} =	30.6	ft	length of apron, 2
L_r =	30.6	ft	$L_r = \text{Max}(L_{r1}, L_{r2})$
L_r =	80	ft	Value to use

$$L_{r2} = 2 \times y_f$$

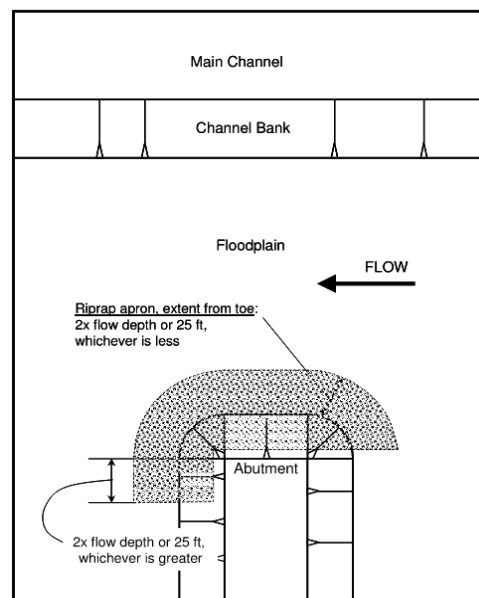
$$L_r = \text{Max}(L_{r1}, L_{r2})$$

Check: Plot riprap layout

Compare with shear stress plot heat map from HEC-RAS.

Comments: Length of apron on sides set at 80 feet to provide protection on wraparound regions with high shear stress. Extend apron on side slopes up to Elev. 1045

HEC-23 Figure 14.7



East Abutment - Scour at Abutment Face Q200

AASHTO LRFD Section:



(Gray cells are User Inputs)

Notes

- Goal: estimate scour at face of abutment
- Assume front slope undergoes slope stability failure at max scour depth, y_s
- Assume an 2:1 failed slope from y_s at distance R back to abutment
- Estimate X using a conservative assumption of the friction slope

Scour at Abutment Face

R =	184.1	ft
$\theta_s =$	32	deg
$\theta_{s1} =$	26.6	deg
X =	2.00	

input from Riprap Apron calcs
friction slope estimated from Boring B-4
conservative assumption for friction slope
 $X = 1/\tan(\theta_{s1})$

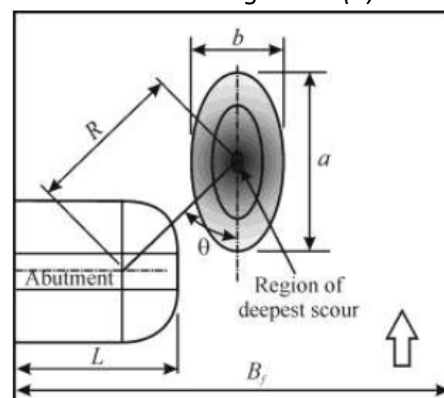
$y_s =$	70.9	ft
$Y_{R1} =$	1040.5	
$Y_{R2} =$	969.6	

max scour depth at location R
initial ground elevation at location R (downstream of bridge but not all the way to top of 1041 bench)
ground elevation at R after scour
 $Y_{R2} = Y_{R1} - y_s$

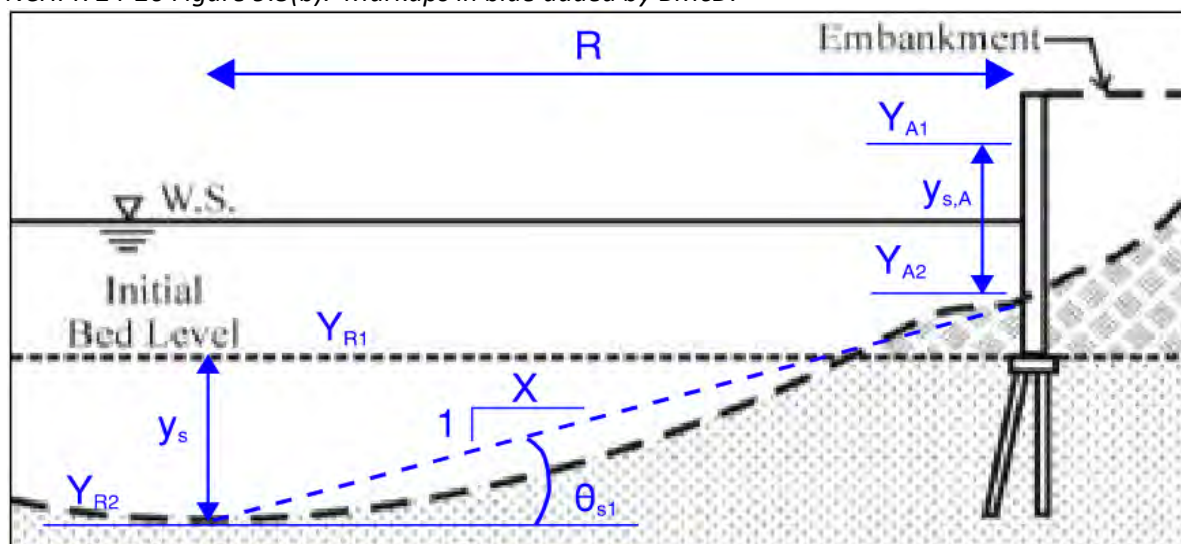
$Y_{A2} =$	1061.7	ft
$Y_{A1} =$	1054.11	ft
$y_{s,A} =$	-7.5	ft

ground elevation at abutment face after scour
initial top of riprap elevation at abutment face before scour
scour at face of abutment
 $y_{s,A} = Y_{A1} - Y_{A2}$

NCHPR 24-20 Figure 9.4(a)



NCHPR 24-20 Figure 9.3(b). Markups in blue added by BMcD.



East Abutment - Scour at Abutment Face Q500

AASHTO LRFD Section:



(Gray cells are User Inputs)

Notes

- Goal: estimate scour at face of abutment
- Assume front slope undergoes slope stability failure at max scour depth, y_s
- Assume an 2:1 failed slope from y_s at distance R back to abutment
- Estimate X using a conservative assumption of the friction slope

Scour at Abutment Face

R =	188.3	ft
$\theta_s =$	32	deg
$\theta_{s1} =$	26.6	deg
X =	2.00	

input from Riprap Apron calcs
friction slope estimated from Boring B-4
conservative assumption for friction slope
 $X = 1/\tan(\theta_{s1})$

$y_s =$	72.5	ft
$Y_{R1} =$	1040.5	
$Y_{R2} =$	968.0	

max scour depth at location R
initial ground elevation at location R
ground elevation at R after scour

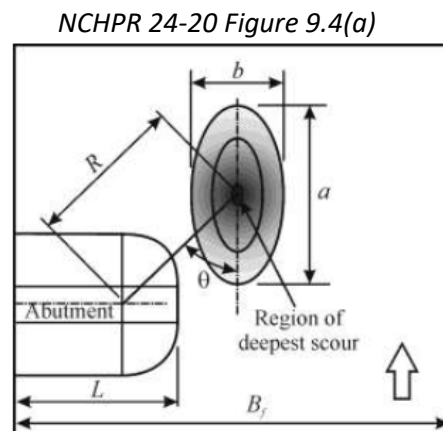
$$Y_{R2} = Y_{R1} - y_s$$

$Y_{A2} =$	1062.2	ft
$Y_{A1} =$	1054.11	ft
$y_{s,A} =$	-8.0	ft

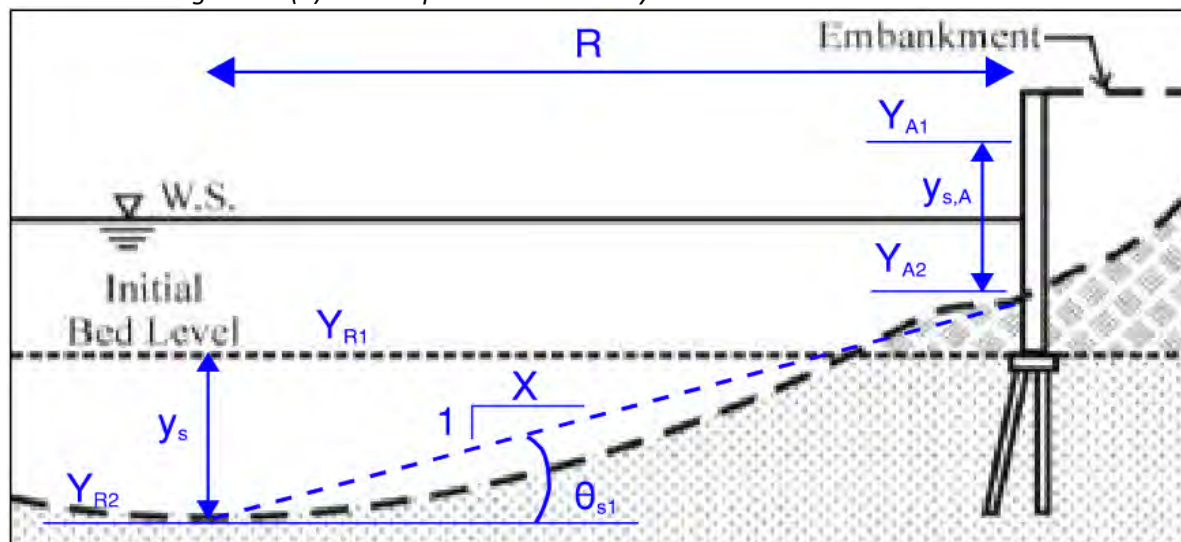
ground elevation at abutment face after scour
initial top of riprap elevation at abutment face before scour
scour at face of abutment

$$Y_{A2} = Y_{R2} + R \tan(\theta_{s1})$$

$$y_{s,A} = Y_{A1} - Y_{A2}$$



NCHPR 24-20 Figure 9.3(b). Markups in blue added by BMcD.



East Abutment - Riprap Size

AASHTO LRFD Section:

(Gray cells are User Inputs)

Material

$\gamma_r = 165$ pcf unit weight of riprap --> use 165 pcf, corresponding to an S_s of 2.65.
 $\gamma_w = 62.4$ pcf unit weight of water
 $S_s = 2.64$ specific gravity of rock riprap $S_s = \gamma_r / \gamma_w$

Note this is greater than NDOT
Section 905 minimum of 140 pcf.

Method 1 - Based on Isbash approach & critical velocity

Source: Method in HEC-23 Design Guideline 14 (from Lagasse, 2001)
Velocity/Froude number similar through opening at Q200/Q500

$V = 9$ ft/s characteristic average velocity in the contracted section
 $g = 32.2$ ft/s² gravitational acceleration
 $y_f = 14.8$ ft depth of flow in the contracted opening
 $Fr = 0.412$ Froude number $FR = V / ((gy_f)^{0.5})$

Case A

$K_A = 0.89$ Constant for Case A Spill-through abut: $K_A = 0.89$
 Vertical wall abut: $K_A = 1.02$

$D_{50,A} = 1.36$ ft median stone diameter
 $D_{50,A} = y_f \times \frac{K_A}{S_s - 1} \times \left[\frac{V^2}{gy_f} \right]$

Case B

$K_B = 0.61$ Constant for Case B Spill-through abut: $K_B = 0.61$
 Vertical wall abut: $K_B = 0.69$

$D_{50,B} = 4.28$ ft median stone diameter
 $D_{50,B} = y_f \times \frac{K_A}{S_s - 1} \times \left[\frac{V^2}{gy_f} \right]^{0.14}$

Selection

Case = A $Fr \leq 0.80$, Case A $Fr > 0.80$, Case B
 $D_{50,c} = 1.36$ ft calculated median stone diameter

Method 2 - Based on Shield's approach & critical shear stress

Source: HEC-20 (4th Edition)

$D_{50} = 1.36$ ft
 $K_s = 0.03$ Shield's parameter: per HEC-20, use 0.03 as a traditional lower bound for riprap
 $\tau_c = 4.19$ psf critical shear stress $\tau_c = F(\gamma_r - \gamma_w)D_{50}$
 $\tau_{des} = 2.05$ psf peak shear stress around abutment from HEC-RAS results
 OK $\tau_{des} < \tau_c?$

East Abutment - Riprap Size

AASHTO LRFD Section:

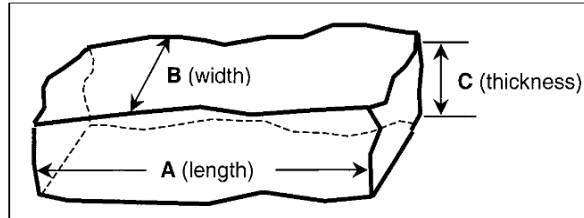
Specification

Source: HEC-23

$D_{50, \min} = 1.36$ ft Minimum from calcs

Assume "D" corresponds to the intermediate particle dimension, "B"

Figure 5.1 from HEC-23



$W_{50, \min} = 352.8$ lb $W_{50, \min} = 0.85(\gamma_s D_{50, \min}^3)$

$D_{50, \min} = 16.32$ in convert to inches

$D_{50} = 18$ in D50 for specifications

Class = V

Recommended Gradation Tables from HEC-23

Table 5.1. Minimum and Maximum Allowable Particle Size in Inches.								
Nominal Riprap Class by Median Particle Diameter		d_{15}		d_{50}		d_{85}		d_{100}
Class	Size	Min	Max	Min	Max	Min	Max	Max
I	6 in	3.7	5.2	5.7	6.9	7.8	9.2	12.0
II	9 in	5.5	7.8	8.5	10.5	11.5	14.0	18.0
III	12 in	7.3	10.5	11.5	14.0	15.5	18.5	24.0
IV	15 in	9.2	13.0	14.5	17.5	19.5	23.0	30.0
V	18 in	11.0	15.5	17.0	20.5	23.5	27.5	36.0
VI	21 in	13.0	18.5	20.0	24.0	27.5	32.5	42.0
VII	24 in	14.5	21.0	23.0	27.5	31.0	37.0	48.0
VIII	30 in	18.5	26.0	28.5	34.5	39.0	46.0	60.0
IX	36 in	22.0	31.5	34.0	41.5	47.0	55.5	72.0
X	42 in	25.5	36.5	40.0	48.5	54.5	64.5	84.0

Note: Particle size d corresponds to the intermediate ("B") axis of the particle.

NDOT largest class is Type C:

W100 = 700 lbs

W50 = 150 lbs

W10 = 10 lbs

Iowa DOT largest class is Class B:

W100 = 650 lbs

W80 = 500 lbs

W50 = 275 lbs

W10 = 25 lbs

Iowa DOT used Class X in 2011 repair:

W100 = 1200 lbs

W80 = 900 lbs

W50 = 500 lbs

W10 = 50 lbs

Table 5.2. Minimum and Maximum Allowable Particle Weight in Pounds.								
Nominal Riprap Class by Median Particle Weight		W_{15}		W_{50}		W_{85}		W_{100}
Class	Weight	Min	Max	Min	Max	Min	Max	Max
I	20 lb	4	12	15	27	39	64	140
II	60 lb	13	39	51	90	130	220	470
III	150 lb	32	93	120	210	310	510	1100
IV	300 lb	62	180	240	420	600	1000	2200
V	1/4 ton	110	310	410	720	1050	1750	3800
VI	3/8 ton	170	500	650	1150	1650	2800	6000
VII	1/2 ton	260	740	950	1700	2500	4100	9000
VIII	1 ton	500	1450	1900	3300	4800	8000	17600
IX	2 ton	860	2500	3300	5800	8300	13900	30400
X	3 ton	1350	4000	5200	9200	13200	22000	48200

Note: Weight limits for each class are estimated from particle size by: $W = 0.85(\gamma_s d^3)$ where d corresponds to the intermediate ("B") axis of the particle, and particle specific gravity is taken as 2.65.

Selection:

Abut 2: Define a custom NDOT Type similar to the Iowa Class X used in the 2011 repair. Use on front slope and apron.

Abut 1: Use NDOT "Broken Concrete Riprap" on front slope.

F. Alternate Conditions



STATE OF NEBRASKA
DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

PROJ. LOC.:	Decatur Bridge	PROJ. NO.:	STP-51-7(109)	BM PROJ. #:	157887
COUNTY:	Burt	STATION:	###+###.##	MADE BY:	JTK
CN:	32395	STRUCTURE:	S051 03644	DATE:	03/04/25
DESC:	Phase 2 Hydro Strategy Evaluation	CHECK BY:	---		

Scour Calculation Notes and Assumptions

- Design Scour = Q200. Check Scour = Q500.
- Secondary Channel broken up into three zones to calculate scour. Zone 1 is the newly opened channel from moving the abutment east. Zone 2 is between the two dikes in the existing scour hole. Zone 3 is from the scour hole to the high ground between the main and secondary channels. Zone 2 is only being calculated for this comparison as the worst zone for the alternate conditions (v3A).
- Case 1 is the base alternate conditions (v3A). Case 2 is modeled if the extreme velocities and shear stresses caused the north and south dikes to fail.
- Secondary Channel contraction scour values assumed to be clear-water contraction scour. Based upon upstream terrain, flow vectors, and review of 2011 flood it seems a lower likelihood that there will be enough sediment to brought from upstream to have a live bed condition.
- Secondary Channel pier scour assumed two column bent with 5' diameter columns and 7.5' diameter shafts per preliminary design. Use 7.5' diameter in pier scour equation.
- Scour calculated per HEC-18 5th edition methodology for contraction and pier scour.

<u>Event</u>	<u>Flow (cfs)</u>	
Q10	94,400	
Q25	127,100	
FEMA Q100	141,800	
2011 Flood (Q70)	191,000	
Q100	220,600	
Q200	271,000	<i>Design Scour</i>
Q500	297,200	<i>Check Scour</i>



STATE OF NEBRASKA
DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

PROJ. LOC.: Decatur Bridge PROJ. NO.: STP-51-7(109) BM PROJ. #: 157887
COUNTY: Burt STATION: ##### MADE BY: JTK
CN: 32395 STRUCTURE: S051 03644 DATE: 03/04/25
DESC: Phase 2 Hydro Strategy Evaluation CHECK BY: ---

v3A Case 1 & Case 2 Comparison

Zone 2 (Pier in Existing Scour Hole) - Case 1

Model/Location	Bridge Extension (ft)	Bench Elevation	Flow	Contraction Scour	Pier Scour	Long Term Degredation	Total Scour
PropFinal	650	1041	Q10	0.0	0.0	0	0.0
			Q25	0.0	0.3	0	0.3
			FEMA Q100	0.0	5.5	0	5.5
			2011 Flood	9.4	17.1	0	26.5
			Q100	21.0	20.2	0	41.2
			Q200	33.0	22.5	0	55.5
			Q500	35.7	23.0	0	58.8

Zone 2 (Pier in Existing Scour Hole) - Case 2

Model/Location	Bridge Extension (ft)	Bench Elevation	Flow	Contraction Scour	Pier Scour	Long Term Degredation	Total Scour
PropFinal	650	1041	Q10	0.0	0.0	0	0.0
			Q25	0.0	0.3	0	0.3
			FEMA Q100	0.0	5.5	0	5.5
			2011 Flood	37.0	23.7	0	60.7
			Q100	45.7	25.0	0	70.6
			Q200	55.4	26.8	0	82.2
			Q500	57.4	27.0	0	84.4



STATE OF NEBRASKA
DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

PROJ. LOC.: Decatur Bridge PROJ. NO.: STP-51-7(109) BM PROJ. #: 157887
COUNTY: Burt STATION: ###+###.## MADE BY: JTK
CN: 32395 STRUCTURE: S051 03644 DATE: 06/09/25
DESC: Phase 2 Hydro Strategy Evaluation CHECK BY: ---

Pier Scour Notes:

- RAS Mapper layer created to create multiplier so that pier diameter/spacing and angle of attack are the only spreadsheet inputs.
See separate attached hand calc of RAS Mapper layer equation.
- Value selected from RAS Mapper multiplier max around columns in direction of flow or upstream of columns if pier was causing local reduction in Froude number due to obstruction (i.e. Main River Channel Pier)
- For secondary channel, just using worst pier in each "zone" for pier scour design value.
- Drilled shafts sticking just above ground in overbank and above streambed in main stream. Use drilled shaft diameter not column diameter in calculation
- Pier 4 on high ground does not experience flow. For scour design, using flow in channel upstream near the bank for the scenario that in future the channel bank is cut back further and pier would see flow.
- See attached pier scour angle of attack exhibits for measured values.

v3A Case 1 & Case 2 Comparison

Zone 2 (Pier in Scour Hole) - Case 1

Model/Location	Flow	Pier Diameter	Pier Spacing (Total)*	Angle of Attack	K2 Factor	RAS Mapper Multiplier	Pier Scour
PropFinal	Q10	7.5	23	18	1.517	0.00	0.0
	Q25	7.5	23	18	1.517	0.05	0.3
	FEMA Q100	7.5	23	18	1.517	0.98	5.5
	2011 Flood	7.5	23	18	1.517	3.05	17.1
	Q100	7.5	23	18	1.517	3.60	20.2
	Q200	7.5	23	18	1.517	4.00	22.5
	Q500	7.5	23	18	1.517	4.10	23.0

Zone 2 (Pier in Scour Hole) - Case 2

Model/Location	Flow	Pier Diameter	Pier Spacing (Total)*	Angle of Attack	K2 Factor	RAS Mapper Multiplier	Pier Scour
PropFinal	Q10	7.5	23	18	1.517	0.00	0.0
	Q25	7.5	23	18	1.517	0.05	0.3
	FEMA Q100	7.5	23	18	1.517	0.98	5.5
	2011 Flood	7.5	23	18	1.517	4.21	23.7
	Q100	7.5	23	18	1.517	4.44	25.0
	Q200	7.5	23	18	1.517	4.77	26.8
	Q500	7.5	23	18	1.517	4.81	27.0

*Pier Spacing is from face to face of column per HEC-18 Section 7.6.
Confirmed K2 calculation with FHWA Hydraulic Toolbox calculation.

Contraction Scour Notes:

- Secondary Channel contraction scour values assumed to be clear-water contraction scour. Based upon upstream terrain, flow vectors, and review of 2011 flood it seems a lower likelihood that there will be enough sediment to brought from upstream to have a live bed condition.
- D50 values take from preliminary geotechnical borings. 0.3 mm is typical value over first several layers in Zones 1-3. Use 0.2 mm in Main Channel.
- See attached exhibit for width of each zone and profile line the discharge is measured over. For contraction scour discharge through the bridge opening is the only location needed. Profile lines in modeled labeled "Zone 1", "Zone 2", and "Zone 3".
- Contraction Scour for main channel calculated using FHWA Hydraulic Toolbox Version 5.4. See attached input/output. Main channel allowed to calculate either live bed or clear-water equations.
- Depth for secondary channel taken from RAS Mapper depth layer at centerline bridge at the pier used for pier scour calculations. In main channel, depth is taken as a weighted average from the "RAS Mapper (depth layer) profile line table." Profile lines are "Scour-Main-Channel-Bridge" and "Scour - Upstream Main Channel". Weighted average used for main channel instead of RAS Mapper value at pier due to varying depth along bridge centerline in the main channel compared to more consistent values in the secondary channel.

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3} W^2} \right]^{3/7} \quad (6.4)$$

$$y_s = y_2 - y_o = (\text{average contraction scour depth}) \quad (6.5)$$

where:

- y_2 = Average equilibrium depth in the contracted section after contraction scour, ft (m)
- Q = Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W , ft³/s (m³/s)
- D_m = Diameter of the smallest nontransportable particle in the bed material (1.25 D_{50}) in the contracted section, ft (m)
- D_{50} = Median diameter of bed material, ft (m)
- W = Bottom width of the contracted section less pier widths, ft (m)
- y_o = Average existing depth in the contracted section, ft (m)
- K_u = 0.0077 English units
- K_u = 0.025 SI units

Equation 6.4 is a rearranged version of Equation 6.1. As discussed in Section 6.7, a reasonable lower limit of D_{50} equal to 0.2 mm can be applied to this equation. Using a size smaller than 0.2 mm will over-estimate clear-water contraction scour.

Because D_{50} is not the largest particle in the bed material, the scoured section can be slightly armored. Therefore, the D_m is assumed to be 1.25 D_{50} . For stratified bed material the depth of scour can be determined by using the clear-water scour equation sequentially with successive D_m of the bed material layers.

v3A Case 1 & Case 2 Comparison

Zone 2 (Case 1)

Model/Location	Flow	D50 (mm)	Discharge Bridge (cfs)	Width Bridge (cfs)	Depth Bridge (ft)	Contraction Scour
PropFinal	Q10	0.3	0	212	0.0	0.0
	Q25	0.3	1	212	16.6	0.0
	FEMA Q100	0.3	896	212	17.8	0.0
	2011 Flood	0.3	13365	212	20.0	9.4
	Q100	0.3	20314	212	21.1	21.0
	Q200	0.3	28016	212	22.4	33.0
	Q500	0.3	29906	212	22.9	35.7

Zone 2 (Case 2)

Model/Location	Flow	D50 (mm)	Discharge Bridge (cfs)	Width Bridge (cfs)	Depth Bridge (ft)	Contraction Scour
PropFinal	Q10	0.3	0	212	0.0	0.0
	Q25	0.3	1	212	16.6	0.0
	FEMA Q100	0.3	896	212	17.8	0.0
	2011 Flood	0.3	28712	212	19.6	37.0
	Q100	0.3	34697	212	20.9	45.7
	Q200	0.3	41631	212	22.4	55.4
	Q500	0.3	43178	212	22.9	57.4



STATE OF NEBRASKA
DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

PROJ. LOC.: Decatur Bridge	PROJ. NO.: STP-51-7(109)	BM PROJ. #: 157887
COUNTY: Burt	STATION: ###+###.##	MADE BY: ASF
CN: 32395	STRUCTURE: S051 03644	DATE: 05/23/25
DESC: Abutment Scour	CHECK BY:	

East Abutment Scour - Design Approach

AASHTO LRFD Section:



(Gray cells are User Inputs)

Steps in Design Process

Calculated for Q200 and Q500. Controlling output provided.

1) Calculate scour at East Abutment (y_s)

- a) Method: NCRHP 24-20; total abutment scour = amplification of contraction scour
- b) Case B: Abutment "far" from main channel bank; clear water scour
- c) Spill-through abutment

Primary Countermeasure: Place riprap apron to "push" location of max scour further from abutment face

2) Determine minimum apron width (W) to avoid undermining of front slope toe

- a) Method: Countermeasures for Toe Protection of Spill-through Abutments (Melville et. al., 2006)
- b) Note: same method is referenced in NCHRP Report 587 and HEC-23

3) Determine distance from face of abutment to calculated max scour depth (R)

- a) Method: Countermeasures for Toe Protection of Spill-through Abutments (Melville et. al., 2006)
- b) Include effect of riprap apron, W
- c) Define "R" from front face of abutment (NCHRP 24-20), rather than toe of front slope (Melville 2006)

4) Calculate "geotechnical limit to scour" ($d_{s,max}$)

- a) $d_{s,max}$ = theoretical max scour depth before front slope fails
- b) Method: NCHRP 24-20
- c) Use results from Boring B-4 to estimate friction angle of front slope material and subgrade
- d) If $d_{s,max} > y_s$
 - front slope is theoretically stable
 - theoretically, scour at front face of abutment would not occur
- e) If $d_{s,max} < y_s$
 - front slope would theoretically fail before y_s is reached
 - failure of front slope would "open up" the conveyance area
 - scour process would change to a different model
- d) Goal: increase $d_{s,max}$ to be above y_s ; revise W to increase R

5) Extend riprap apron along upstream & downstream sides of embankment

- a) Method: HEC-23

6) Check riprap layout against shear stress plot from HEC-RAS

- a) Increase footprint if needed to address high shear stress areas



STATE OF NEBRASKA
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BRIDGE DIVISION

PROJ. LOC.:	Decatur Bridge	PROJ. NO.:	STP-51-7(109)	BM PROJ. #:	157887
COUNTY:	Burt	STATION:	###+###.##	MADE BY:	ASF
CN:	32395	STRUCTURE:	S051 03644	DATE:	05/23/25
DESC:	Abutment Scour	CHECK BY:			

East Abutment Scour - Design Approach

AASHTO LRFD Section:

-
- 7) Determine minimum riprap size (D50)
 - a) Method: HEC-23

Additional Conservative Theoretical Protection Countermeasure:

Design abutment to accommodate scour; Use sheet pile wall w/ tie rods & anchor block

- 8) Assume that front slope failure may occur. Calculate scour at face of abutment.
 - a) Assume an 2:1 failed slope from deepest scour at R back to abutment
 - b) Determine "X" using a conservative assumption of the angle of repose
 - c) Assume angle of repose ~ friction slope
- 9) Design sheet pile abutment
 - a) Determine friction angle and soil weight from Boring B-4
 - b) Use tie rods and anchor block if required

East Abutment Scour - NCHRP Method Q200 v3A Case 1

AASHTO LRFD Section:



(Gray cells are User Inputs)

Case B - Spill-through Abutment

Description

- abutment "far" from main channel bank
- clear water scour

$q_f = 25 \text{ ft}^2/\text{s}$
 $q_{2f} = 180 \text{ ft}^2/\text{s}$
 $K_u = 11.17$
 $D_{50,mm} = 0.3 \text{ mm}$
 $D_{50} = 0.00098 \text{ ft}$

unit discharge of floodplain flow upstream of bridge

unit discharge in constricted opening, accounting for non-uniform flow distribution

dimensionless constant, English units

particle size with 50 percent finer

convert to feet

$$D_{50} = D_{50,mm} \times (1 \text{ ft}/304.8 \text{ mm})$$

$y_c = 78.3 \text{ ft}$

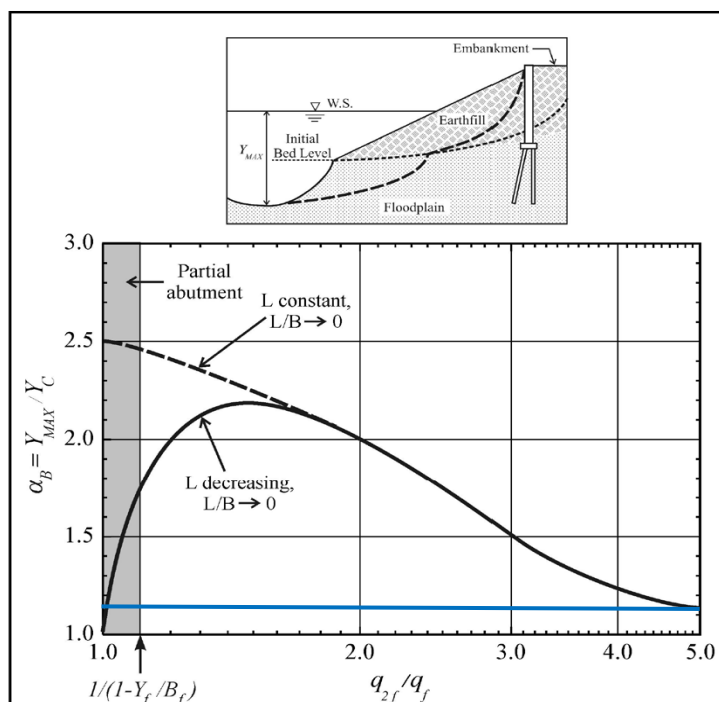
flow depth including clear water contraction scour

$$y_c = \left(\frac{q_{2f}}{K_u (D_{50})^{1/3}} \right)^{6/7}$$

$q_{2f}/q_f = 7.2$

$\alpha_B = 1.13$

amplification factor for spill through abutments & clear water (from graph)





STATE OF NEBRASKA
DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

PROJ. LOC.:	Decatur Bridge	PROJ. NO.:	STP-51-7(109)	BM PROJ. #:	157887
COUNTY:	Burt	STATION:	####.###	MADE BY:	ASF
CN:	32395	STRUCTURE:	S051 03644	DATE:	05/23/25
DESC:	Abutment Scour	CHECK BY:			

East Abutment Scour - NCHRP Method Q200 v3A Case 1

AASHTO LRFD Section:

$y_0 =$	22.4	ft	flow depth prior to scour	
$y_{max} =$	88.5	ft	max flow depth from abutment scour	$y_{max} = \alpha_B y_c$
$y_s =$	66.1	ft	abutment scour depth	$y_s = y_{max} - y_0$

East Abutment Scour - NCHRP Method Q200 v3A Case 2

AASHTO LRFD Section:



(Gray cells are User Inputs)

Case B - Spill-through Abutment

Description

- abutment "far" from main channel bank
- clear water scour

$q_f = 30 \text{ ft}^2/\text{s}$
 $q_{2f} = 270 \text{ ft}^2/\text{s}$
 $K_u = 11.17$
 $D_{50,mm} = 0.3 \text{ mm}$
 $D_{50} = 0.00098 \text{ ft}$

unit discharge of floodplain flow upstream of bridge

unit discharge in constricted opening, accounting for non-uniform flow distribution

dimensionless constant, English units

particle size with 50 percent finer

convert to feet

$$D_{50} = D_{50,mm} \times (1 \text{ ft}/304.8 \text{ mm})$$

$y_c = 110.9 \text{ ft}$

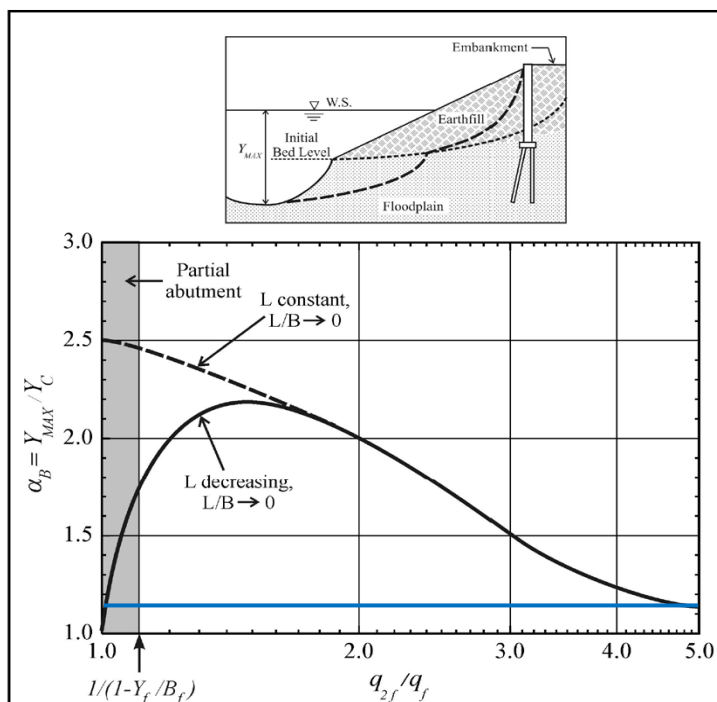
flow depth including clear water contraction scour

$$y_c = \left(\frac{q_{2f}}{K_u (D_{50})^{1/3}} \right)^{6/7}$$

$q_{2f}/q_f = 9$

$\alpha_B = 1.13$

amplification factor for spill through abutments & clear water (from graph)





STATE OF NEBRASKA
DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

PROJ. LOC.:	Decatur Bridge	PROJ. NO.:	STP-51-7(109)	BM PROJ. #:	157887
COUNTY:	Burt	STATION:	###+###.##	MADE BY:	ASF
CN:	32395	STRUCTURE:	S051 03644	DATE:	05/23/25
DESC:	Abutment Scour	CHECK BY:			

East Abutment Scour - NCHRP Method Q200 v3A Case 2

AASHTO LRFD Section:

$y_0 =$	22.4	ft
$y_{max} =$	125.3	ft
$y_s =$	102.9	ft

flow depth prior to scour

max flow depth from abutment scour

abutment scour depth

$$y_{max} = \alpha_B y_c$$

$$y_s = y_{max} - y_0$$

G. References

References

1. HEC-18, Scour at Bridges (2012)
2. NCHRP 24-20, Abutment Scour (2010)
3. HEC-23, Bridge Scour Countermeasures (2009)
4. NCHRP 24-19a (587) Bridge Abutment Scour Countermeasures (2007)
5. Countermeasures & Toe Protection for Spill-Through Abutments (Melville 2006)
6. HEC 20, Stream Stability at Highway Structures (2012)
7. FHWA Hydraulic Toolbox 5.4

Appendix A:

Other Alternatives Not Carried Forward - Scour Comparison



STATE OF NEBRASKA
DEPARTMENT OF TRANSPORTATION
BRIDGE DIVISION

PROJ. LOC.: Decatur Bridge PROJ. NO.: STP-51-7(109) BM PROJ. #: 157887
COUNTY: Burt STATION: ##### MADE BY: JTK
CN: 32395 STRUCTURE: S051 03644 DATE: 03/04/25
DESC: Phase 2 Hydro Strategy Evaluation CHECK BY: ---

Main Channel Pier

Model/Location			Flow	Contraction Scour	Pier Scour	Long Term Degredation	Total Scour
v5a	500	1039	Q200	10.64	18.5	7.5	36.6
			Q500	11.35	18.6	7.5	37.5

Notes:

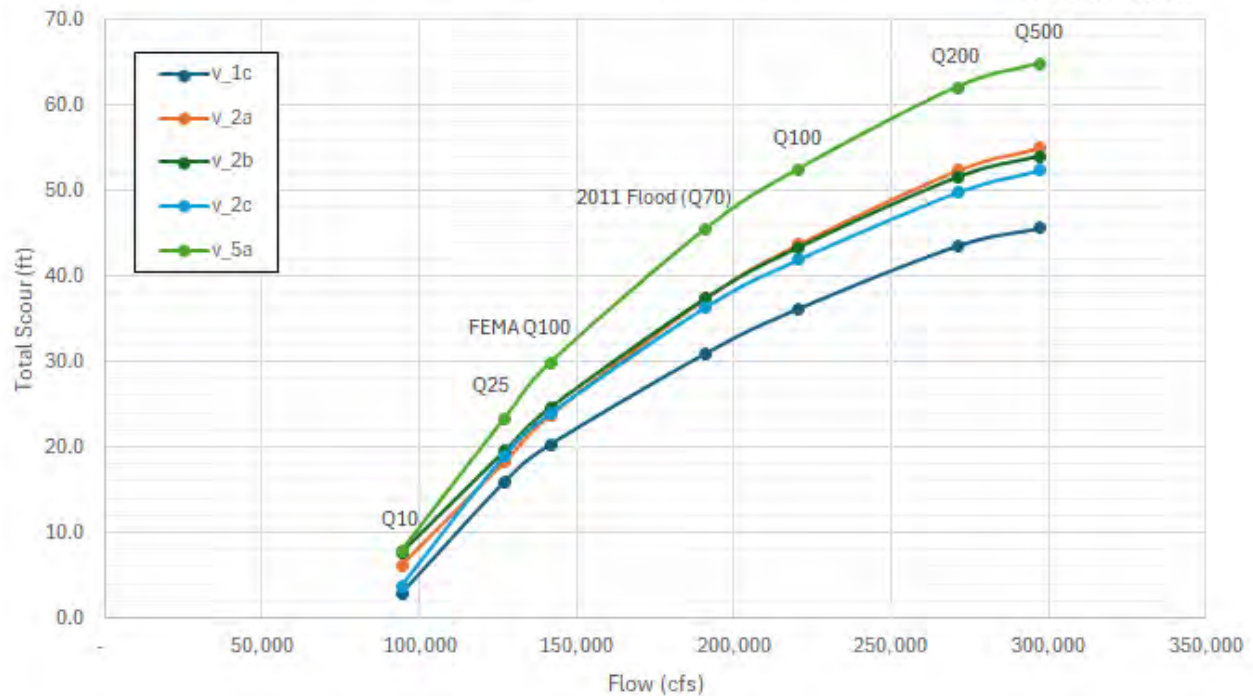
- contraction based on "clear water" conditions

Zone 1 - Total Scour at Piers

Total = contraction + local pier scour

Design Event: Q200

Check Event: Q500



Notes:

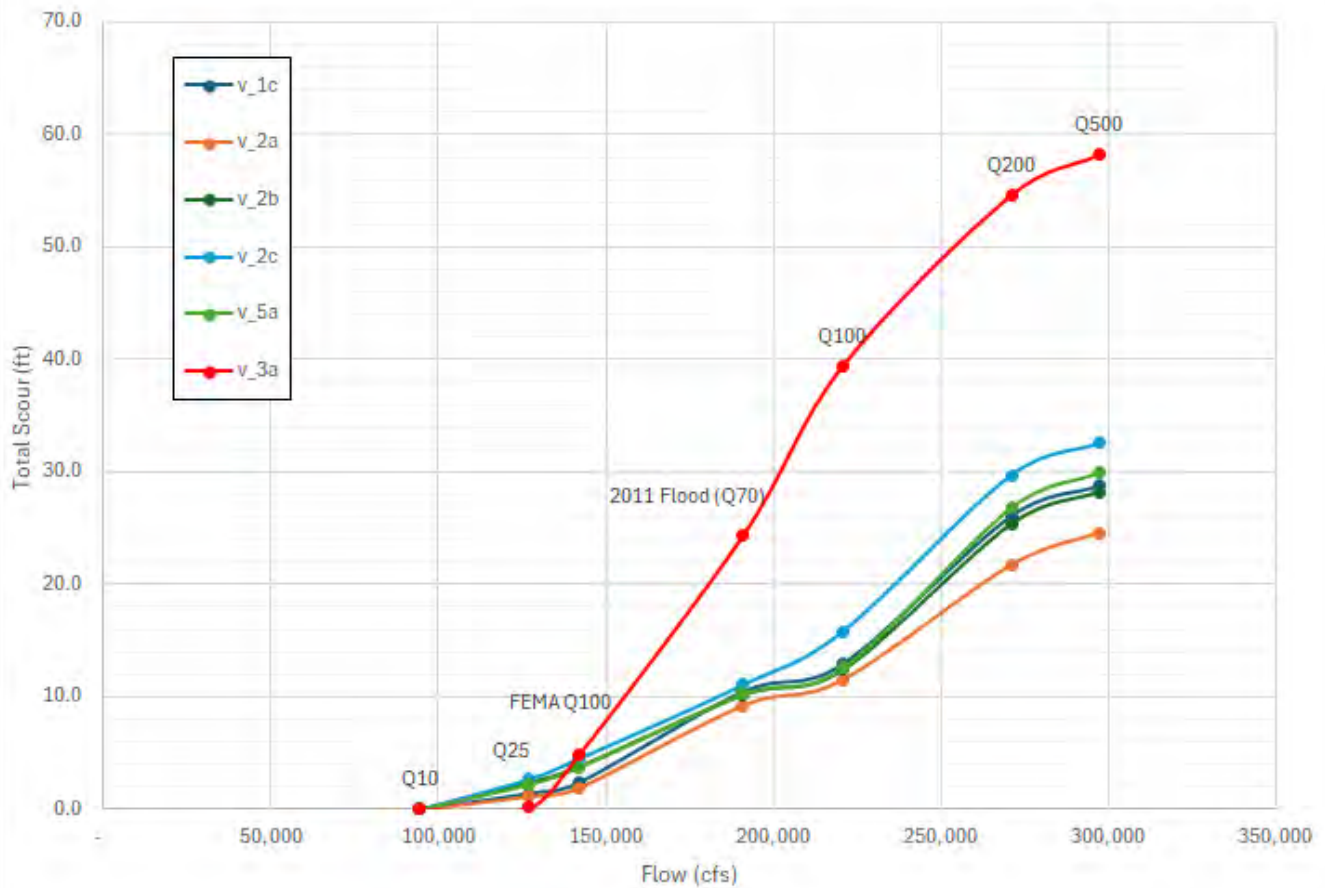
- contraction based on "clear water" conditions

Zone 2 - Total Scour at Piers

Total = contraction + local pier scour

Design Event: Q200

Check Event: Q500



Notes:

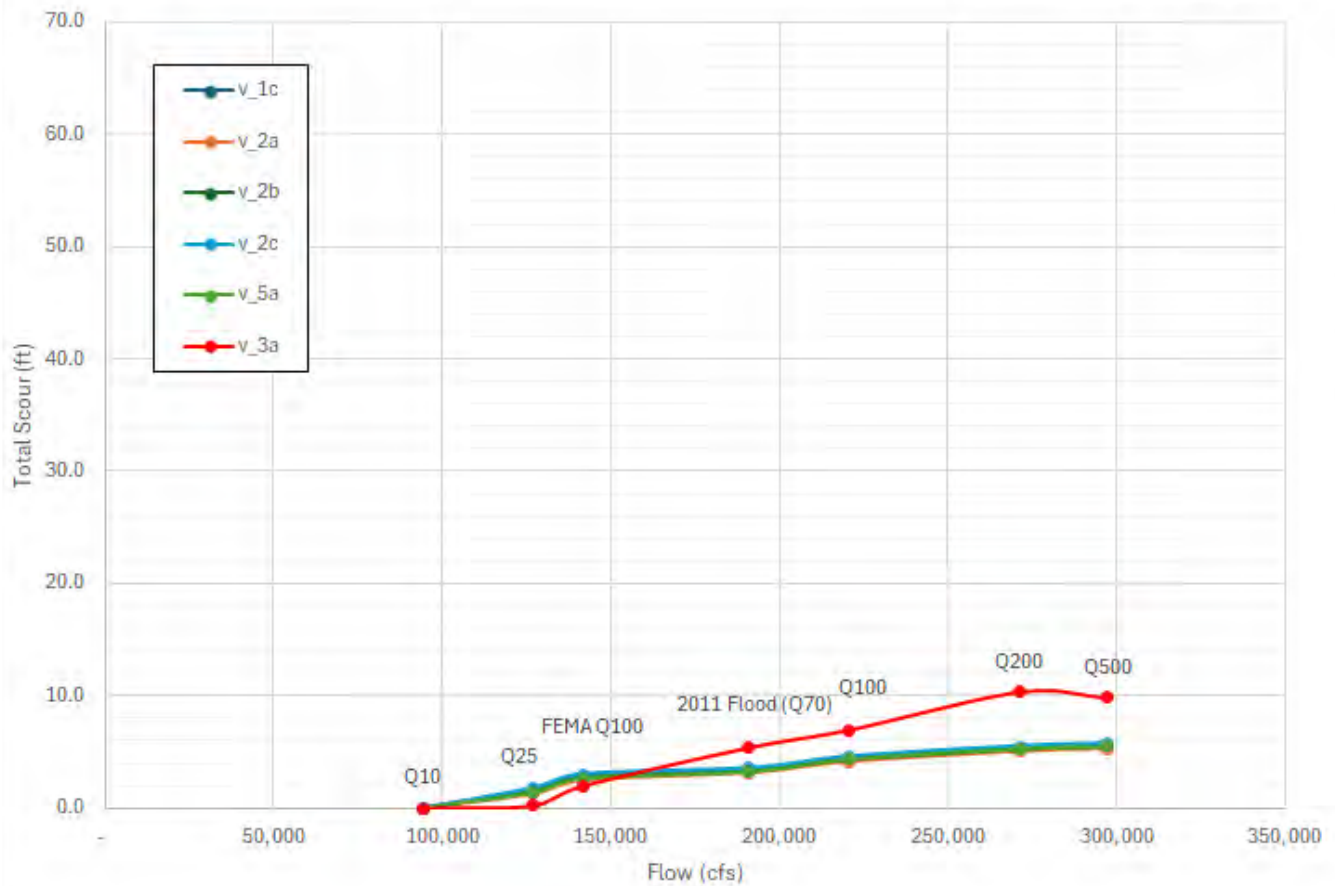
- contraction based on "clear water" conditions

Zone 3 - Total Scour at Piers

Total = contraction + local pier scour

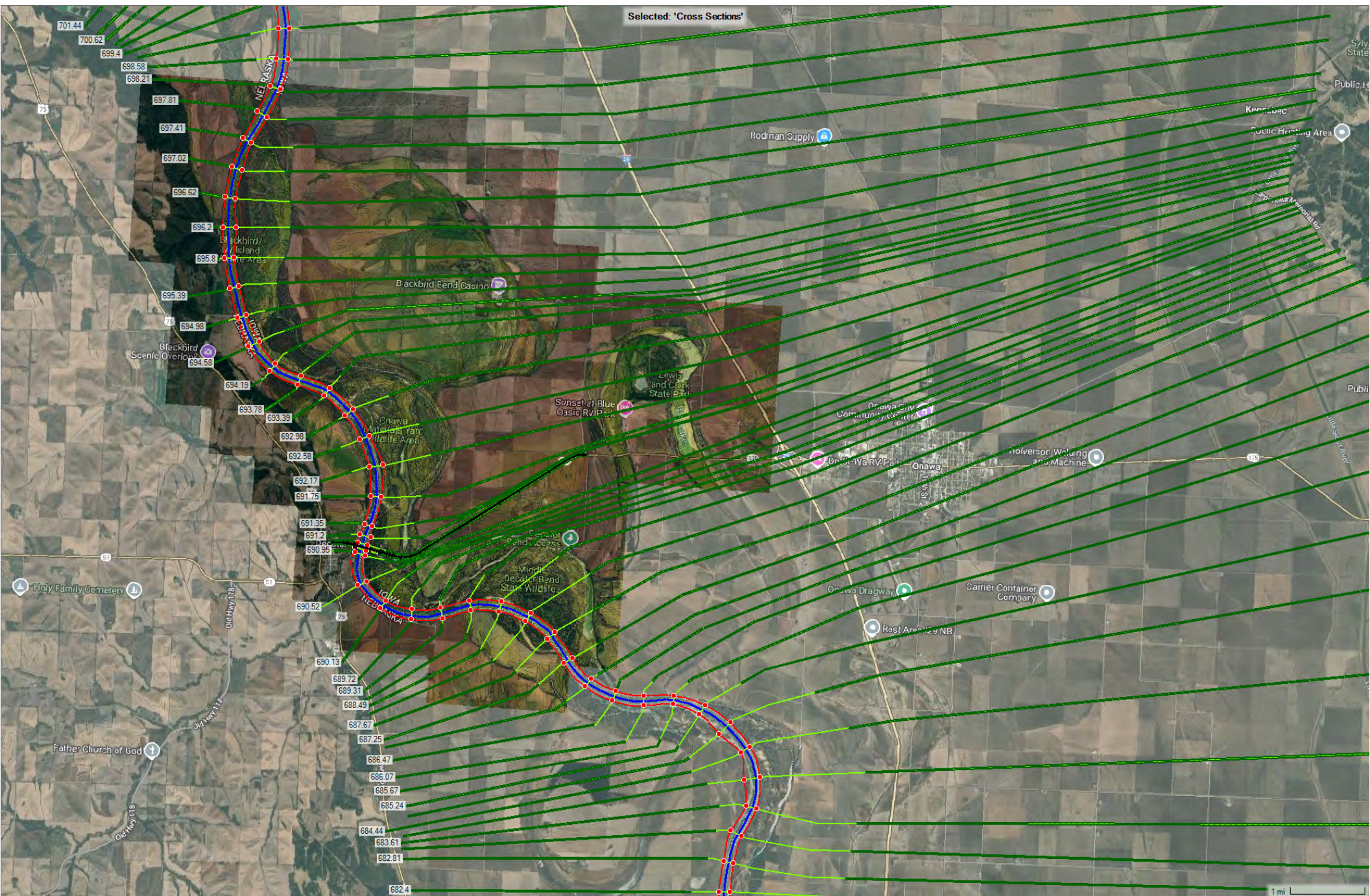
Design Event: Q200

Check Event: Q500



Appendix F – 1D Input & Results

F.1



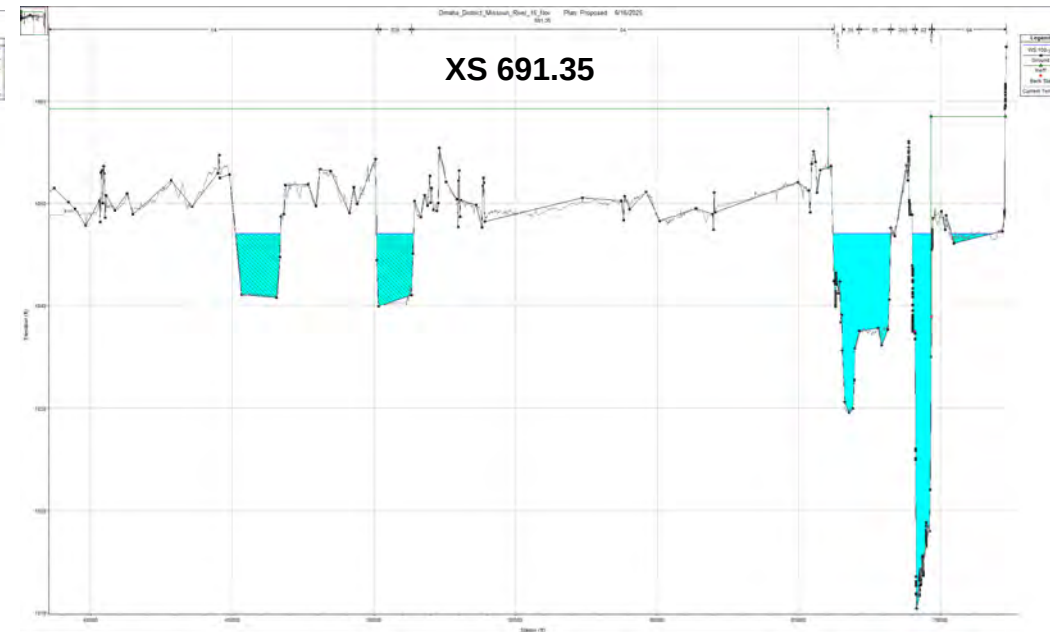
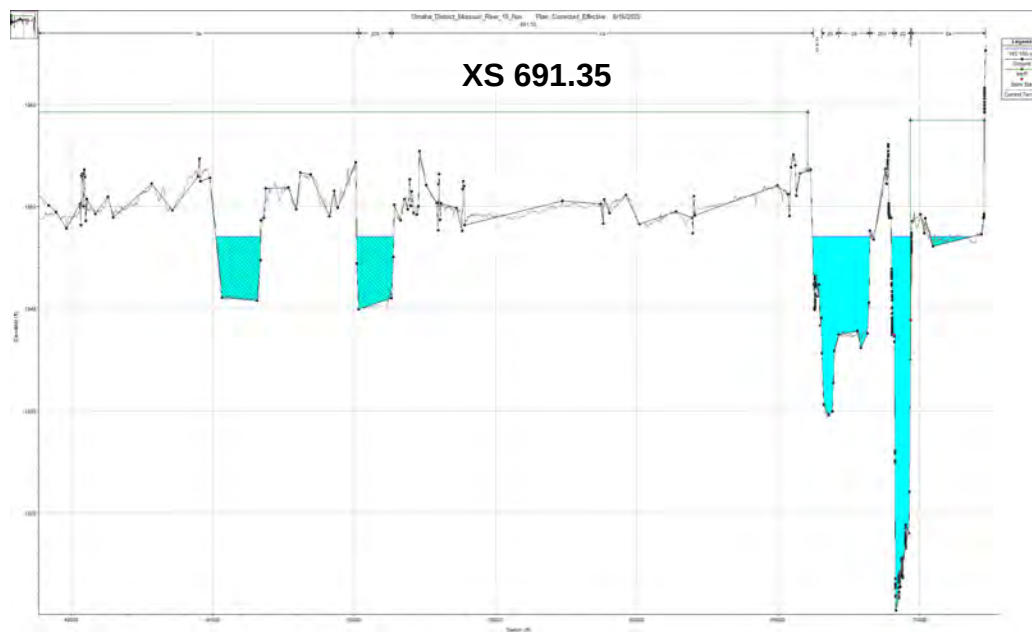
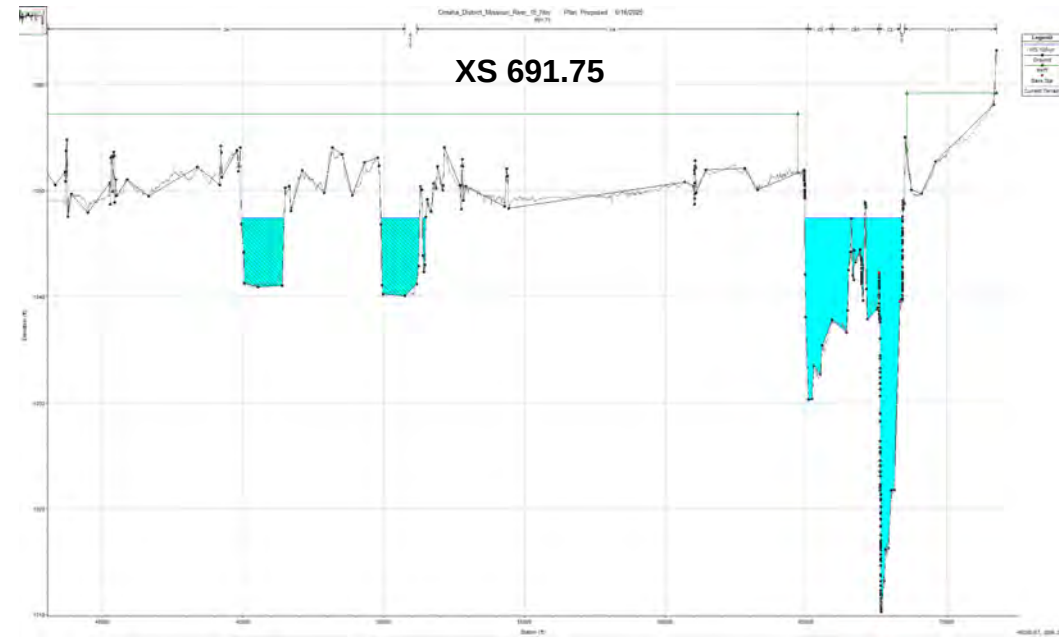
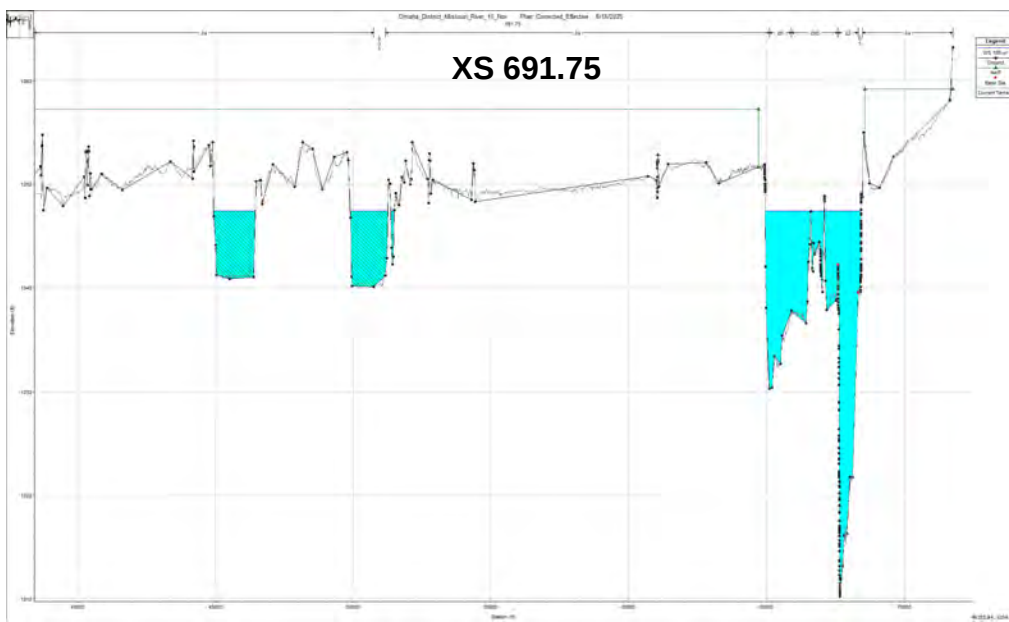
F.2



Corrected Effective Geometry

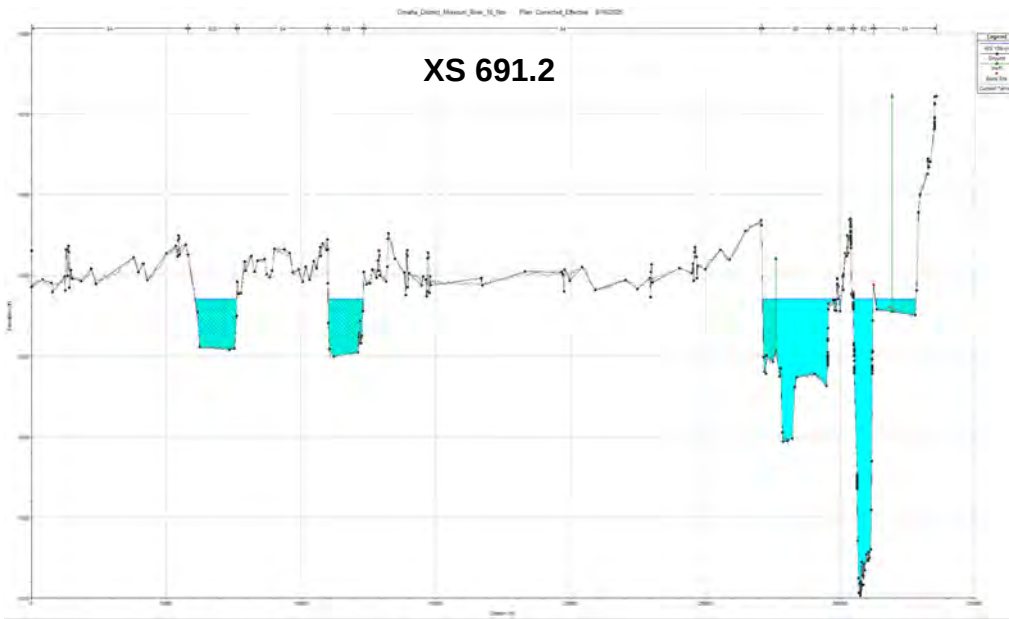
Proposed Geometry

F.3

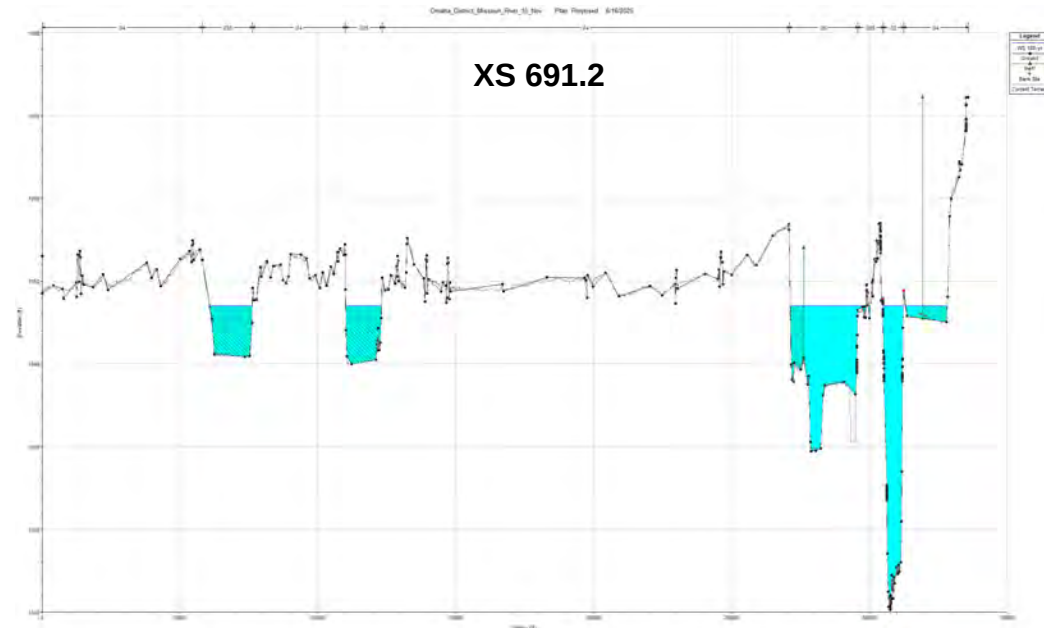


Note: Cross Sections only plotted in area with Lidar from the 2D model. Outside of these limits there are no changes from the Duplicate Effective Model. Only showing Cross-Sections that Terrain was edited from Duplicate Effective Model.

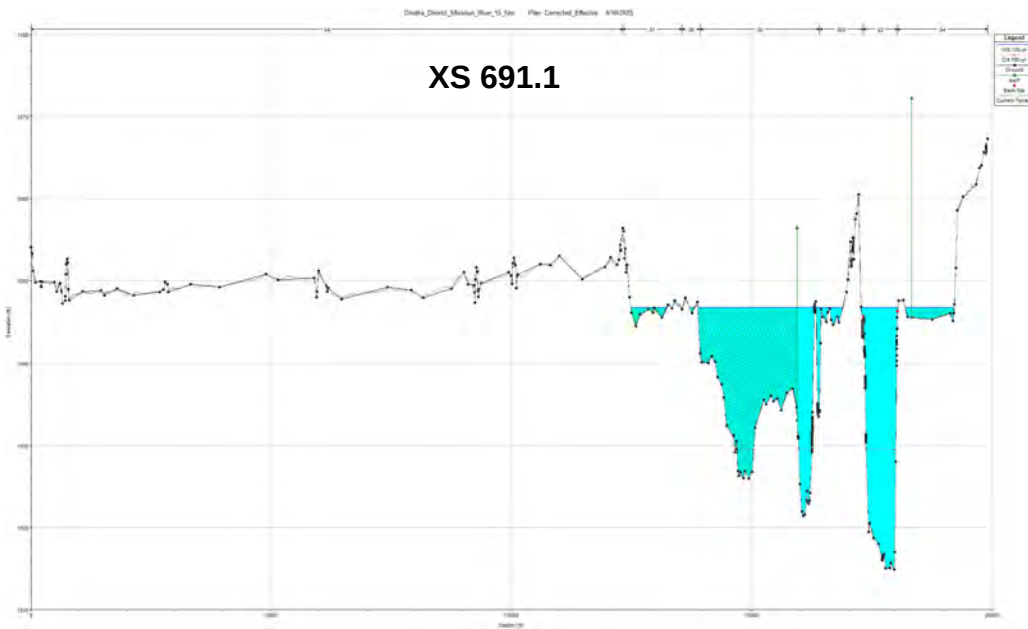
XS 691.2



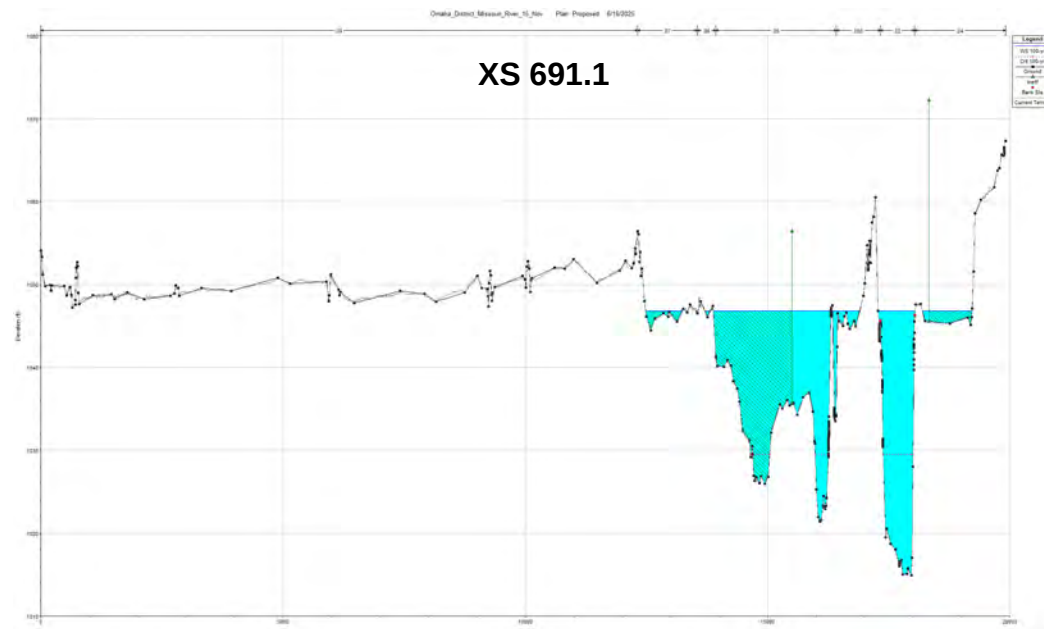
XS 691.2



XS 691.1

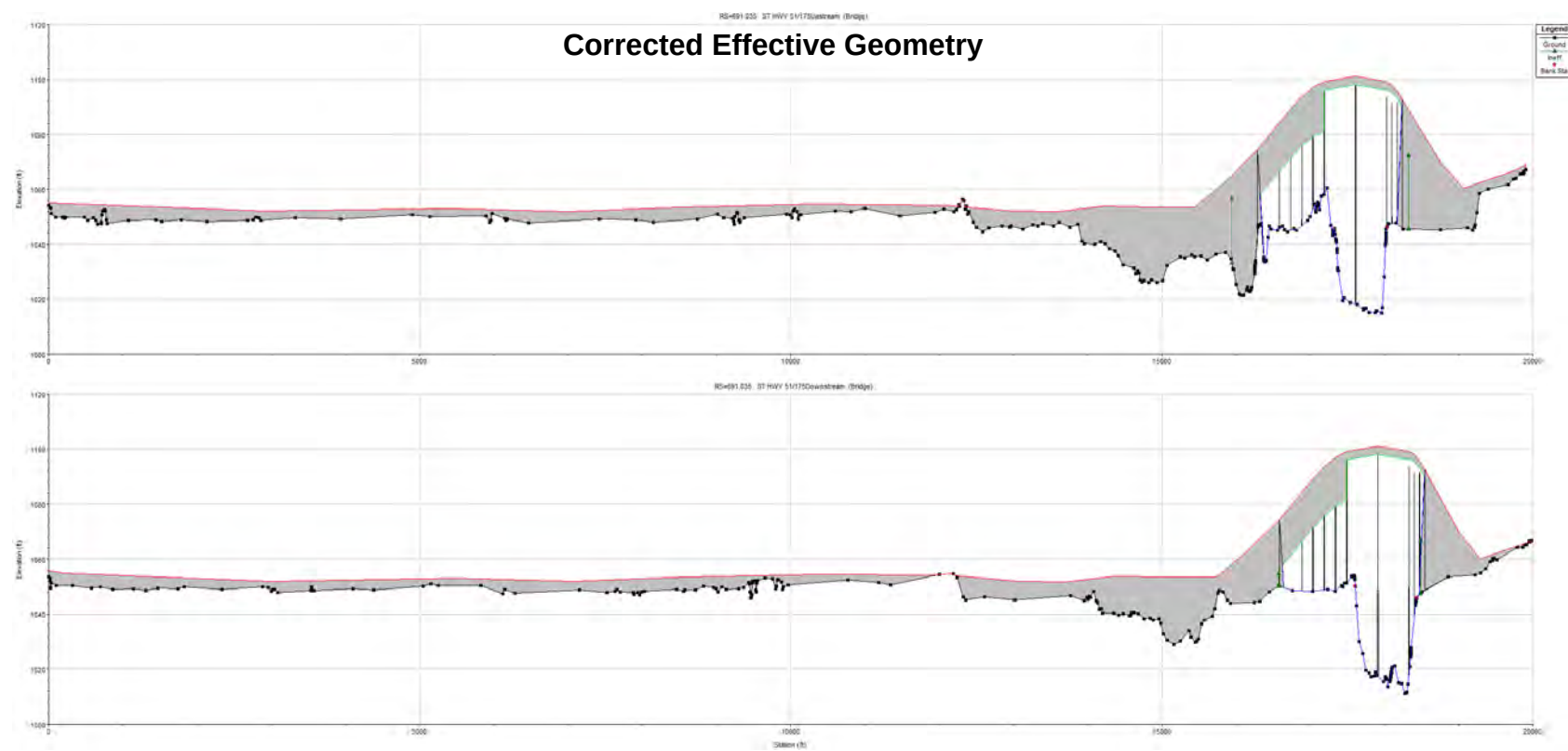


XS 691.1

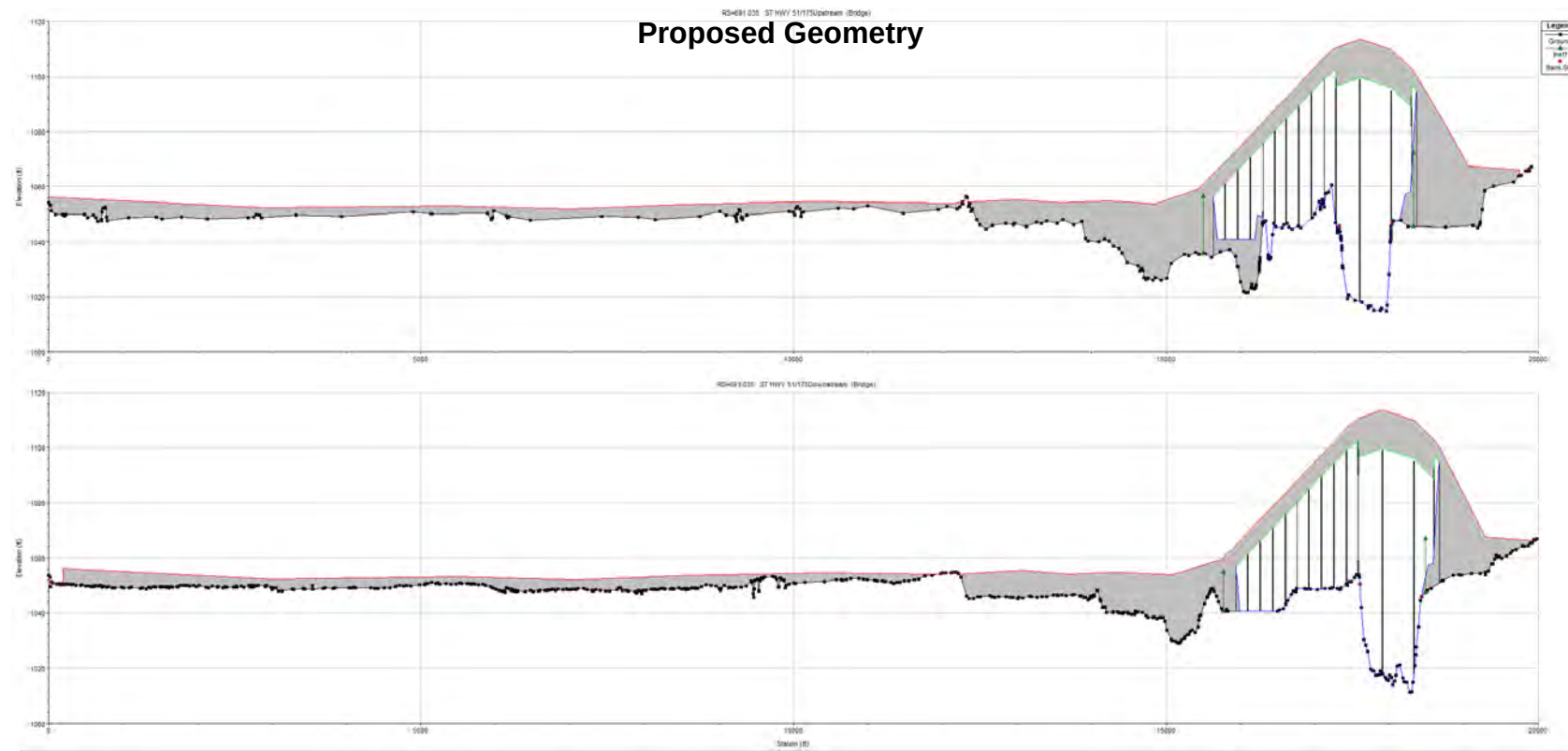


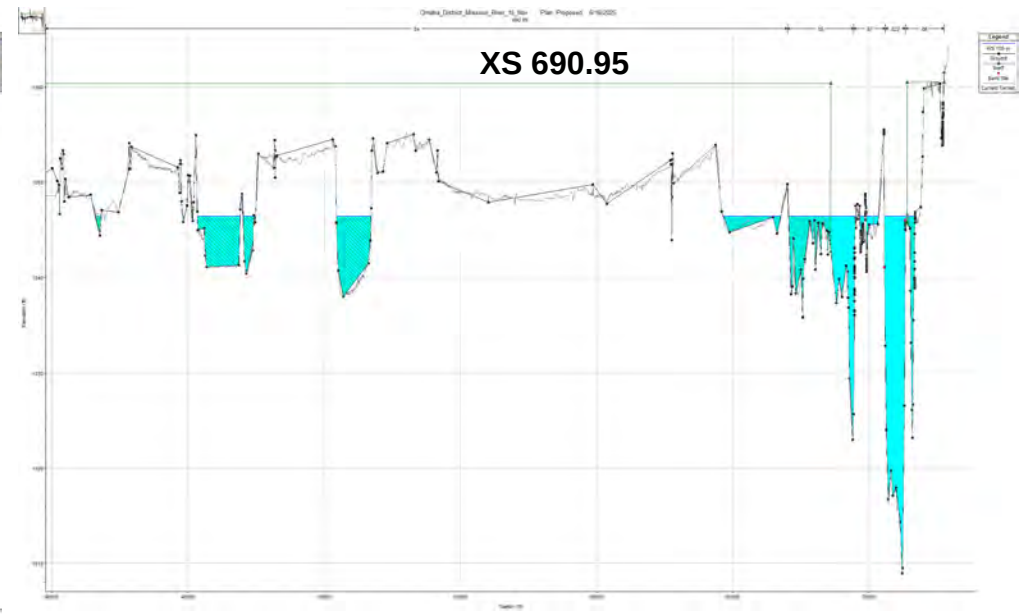
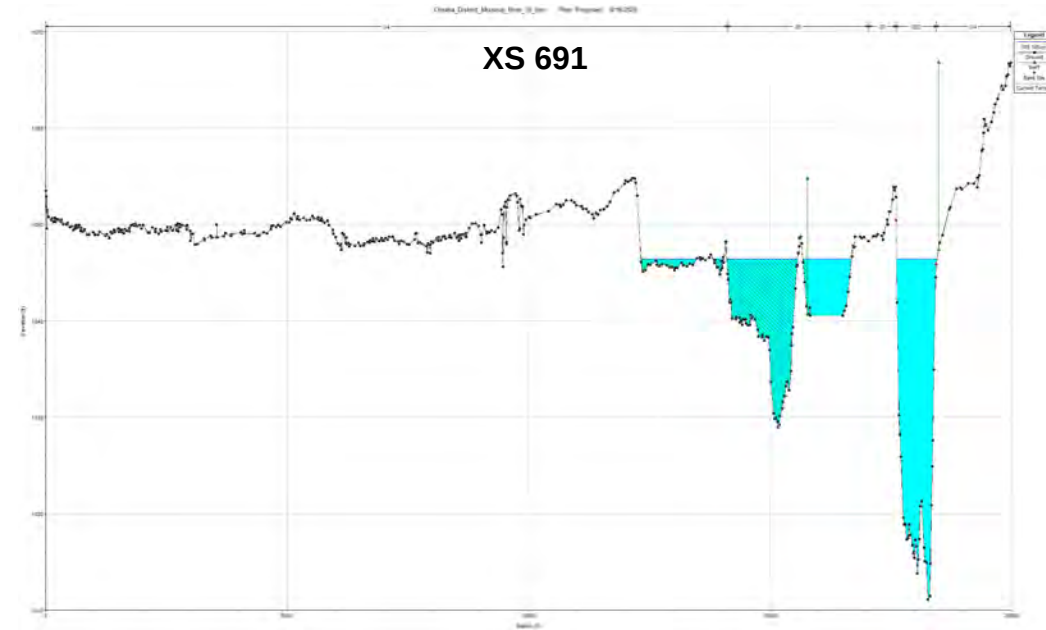
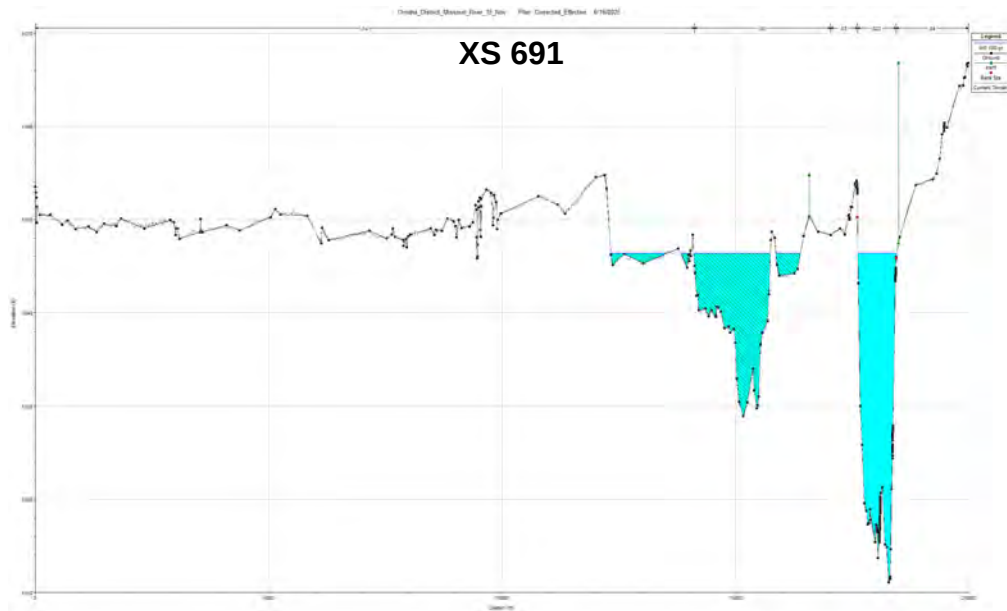
Note: Cross Sections only plotted in area with Lidar from the 2D model. Outside of these limits there are no changes from the Duplicate Effective Model. Only showing Cross-Sections that Terrain was edited from Duplicate Effective Model.

Corrected Effective Geometry



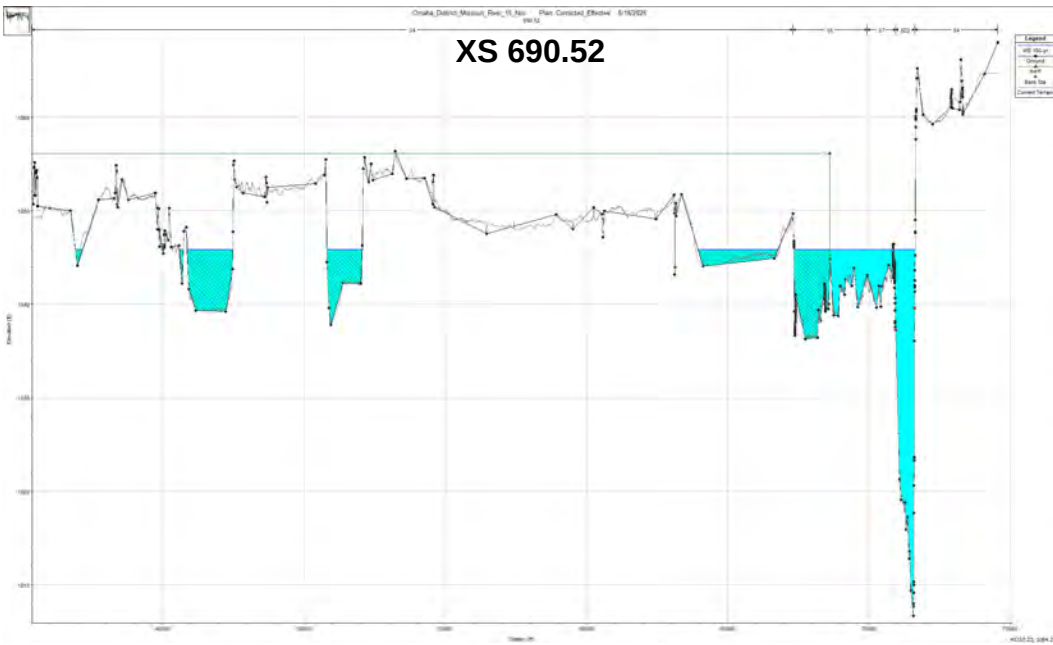
Proposed Geometry





Note: Cross Sections only plotted in area with Lidar from the 2D model. Outside of these limits there are no changes from the Duplicate Effective Model. Only showing Cross-Sections that Terrain was edited from Duplicate Effective Model.

Corrected Effective Geometry



Proposed Geometry

F.7



Note: Cross Sections only plotted in area with Lidar from the 2D model. Outside of these limits there are no changes from the Duplicate Effective Model. Only showing Cross-Sections that Terrain was edited from Duplicate Effective Model.

HEC-RAS 1D Input Data Requested from NDOT Hydraulic Analysis Guideline:

Corrected Effective Geometry

Proposed Geometry

Edit Downstream Reach Lengths

River: ☒ Edit Interpolated XS's

Reach:

Selected Area Edit Options

	River Station	LOB	Channel	ROB
306	695.39 BBU	2022.8	2120	2168.9
307	694.98 BBT	1947.6	2114	2290.3
308	694.58 BBS	1634.2	2116	2560.9
309	694.19 BBR	2093	2107	2234.9
310	693.78 BBQ	2156.2	2124	1877.7
311	693.39 BBP	2673.8	2139	1926.8
312	692.98 BBO	2337.1	2122	1845.9
313	692.58 BBN	2322.9	2130	1878.4
314	692.17 BBM	2404.1	2125	1760.2
315	691.75 BBL	2882.5	2882.5	2882.5
316	691.35 BBK	985	782	805
317	691.2	1150	598.8	500
318	691.1	487.8	487.8	487.8
319	691.035 ST HWY 51/17	Bridge		
320	691	332	338.1	308
321	690.95 BBH	1808	2203.02	2313.9
322	690.52 BBG	1849	2141.5	2275.31
323	690.13 BBF	1484.9	2128	2337.5
324	689.72 BBE	2104	2140	2531.3
325	689.31 BBD	2474.4	2091	2186.4
326	688.91 BBC	2491.9	2136	1785
327	688.49 BBB	2221.1	2120	1891.2
328	688.08 BBA	2269.2	2126	1793.5
329	687.67 BAZ	2251.8	2118	1987.3

Edit Downstream Reach Lengths

River: ☒ Edit Interpolated XS's

Reach:

Selected Area Edit Options

	River Station	LOB	Channel	ROB
306	695.39 BBU	2022.8	2120	2168.9
307	694.98 BBT	1947.6	2114	2290.3
308	694.58 BBS	1634.2	2116	2560.9
309	694.19 BBR	2093	2107	2234.9
310	693.78 BBQ	2156.2	2124	1877.7
311	693.39 BBP	2673.8	2139	1926.8
312	692.98 BBO	2337.1	2122	1845.9
313	692.58 BBN	2322.9	2130	1878.4
314	692.17 BBM	2404.1	2125	1760.2
315	691.75 BBL	2882.5	2882.5	2882.5
316	691.35 BBK	985	782	805
317	691.2	1150	598.8	500
318	691.1	487.8	487.8	487.8
319	691.035 ST HWY 51/175	Bridge		
320	691	332	338.1	308
321	690.95 BBH	1808	2203.02	2313.9
322	690.52 BBG	1849	2141.5	2275.31
323	690.13 BBF	1484.9	2128	2337.5
324	689.72 BBE	2104	2140	2531.3
325	689.31 BBD	2474.4	2091	2186.4
326	688.91 BBC	2491.9	2136	1785
327	688.49 BBB	2221.1	2120	1891.2
328	688.08 BBA	2269.2	2126	1793.5
329	687.67 BAZ	2251.8	2118	1987.3

Edit Manning's n or k Values

River: ☒ Edit Interpolated XS's

Reach:

Selected Area Edit Options

Channel n Values have a light green background

ver	Statio	rcn (n/k)	n #1	n #2	n #3	n #4	n #5	n #6	n #7	n #8	n #9	n #10	n #11	n #12	n
306	695.39 E	n	0.04	0.035	0.05	0.022	0.04								
307	694.98 E	n	0.04	0.048	0.022	0.05									
308	694.58 E	n	0.04	0.048	0.022	0.05									
309	694.19 E	n	0.04	0.05	0.022	0.05	0.04								
310	693.78 E	n	0.04	0.05	0.022	0.05									
311	693.39 E	n	0.04	0.05	0.022	0.05	0.04	0.05							
312	692.98 E	n	0.04	0.05	0.04	0.05	0.035	0.04	0.035	0.04	0.05	0.022	0.05	0.04	
313	692.58 E	n	0.04	0.035	0.055	0.06	0.065	0.02	0.06	0.05					
314	692.17 E	n	0.04	0.035	0.055	0.06	0.065	0.02	0.06	0.05					
315	691.75 E	n	0.04	0.035	0.04	0.05	0.055	0.02	0.05	0.04					
316	691.35 E	n	0.04	0.035	0.04	0.045	0.05	0.055	0.02	0.05	0.04				
317	691.2	n	0.04	0.035	0.04	0.035	0.04	0.05	0.055	0.02	0.04				
318	691.1	n	0.04	0.07	0.06	0.05	0.055	0.02	0.04						
319	691.035	Bridge													
320	691	n	0.04	0.05	0.07	0.022	0.04								
321	690.95 E	n	0.04	0.04	0.05	0.07	0.022	0.06							
322	690.52 E	n	0.04	0.04	0.05	0.07	0.022	0.04							
323	690.13 E	n	0.05	0.07	0.022	0.07	0.05								
324	689.72 E	n	0.05	0.04	0.06	0.07	0.024	0.07	0.05						
325	689.31 E	n	0.05	0.06	0.07	0.024	0.07	0.06							
326	688.91 E	n	0.05	0.06	0.2	0.06	0.07	0.024	0.09	0.2	0.07				
327	688.49 E	n	0.04	0.04	0.07	0.2	0.07	0.024	0.07	0.2	0.06				
328	688.08 E	n	0.05	0.07	0.2	0.09	0.07	0.024	0.09	0.2	0.06				
329	687.67 E	n	0.05	0.06	0.2	0.07	0.024	0.09	0.06	0.2	0.06				

Edit Manning's n or k Values

River: ☒ Edit Interpolated XS's

Reach:

Selected Area Edit Options

Channel n Values have a light green background

ver	Statio	rcn (n/k)	n #1	n #2	n #3	n #4	n #5	n #6	n #7	n #8	n #9	n #10	n #11	n #12	n
306	695.39 E	n	0.04	0.035	0.05	0.022	0.04								
307	694.98 E	n	0.04	0.048	0.022	0.05									
308	694.58 E	n	0.04	0.048	0.022	0.05									
309	694.19 E	n	0.04	0.05	0.022	0.05	0.04								
310	693.78 E	n	0.04	0.05	0.022	0.05									
311	693.39 E	n	0.04	0.05	0.022	0.05	0.04	0.05							
312	692.98 E	n	0.04	0.05	0.04	0.05	0.035	0.04	0.035	0.04	0.05	0.022	0.05	0.04	
313	692.58 E	n	0.04	0.035	0.04	0.035	0.04	0.035	0.04	0.05	0.022	0.05	0.04		
314	692.17 E	n	0.04	0.035	0.055	0.06	0.065	0.02	0.06	0.05					
315	691.75 E	n	0.04	0.035	0.04	0.05	0.055	0.02	0.05	0.04					
316	691.35 E	n	0.04	0.035	0.04	0.045	0.05	0.055	0.02	0.05	0.04				
317	691.2	n	0.04	0.035	0.04	0.035	0.04	0.05	0.055	0.02	0.04				
318	691.1	n	0.04	0.07	0.06	0.05	0.055	0.02	0.04						
319	691.035	Bridge													
320	691	n	0.04	0.05	0.07	0.022	0.04								
321	690.95 E	n	0.04	0.04	0.05	0.07	0.022	0.06							
322	690.52 E	n	0.04	0.04	0.05	0.07	0.022	0.04							
323	690.13 E	n	0.05	0.07	0.022	0.07	0.05								
324	689.72 E	n	0.05	0.04	0.06	0.07	0.024	0.07	0.05						
325	689.31 E	n	0.05	0.06	0.07	0.024	0.07	0.06							
326	688.91 E	n	0.05	0.06	0.2	0.06	0.07	0.024	0.09	0.2	0.07				
327	688.49 E	n	0.04	0.04	0.07	0.2	0.07	0.024	0.07	0.2	0.06				
328	688.08 E	n	0.05	0.07	0.2	0.09	0.07	0.024	0.09	0.2	0.06				

No channel slope input because model has Known WS boundary condition

No channel slope input because model has Known WS boundary condition

Note: Only showing input and results for cross-sections around the bridge. The model runs from Gavin's Point Dam in Yankton, SD to Rulo, NE. There is too much input/output to show in the report. Only cross-sections in the vicinity of the bridge are effected with this project.

Corrected Effective

Bridge Output

FileTypeOptionsHelp

River:MissouriProfile:100-yrReach1RS:691.035Plan:CorrEff

Plan: CorrEff Missouri 1 RS: 691.035 Profile: 100-yr				
E.G. US. (ft)	1047.36	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	1046.74	E.G. Elev (ft)	1047.22	1047.12
Q Total (cfs)	141500.00	W.S. Elev (ft)	1046.27	1046.37
Q Bridge (cfs)	141500.00	Crit W.S. (ft)	1029.86	1028.99
Q Weir (cfs)		Max Chl Dpth (ft)	31.33	35.25
Weir Sta Lft (ft)		Vel Total (ft/s)	7.46	6.92
Weir Sta Rgt (ft)		Flow Area (sq ft)	18962.64	20447.37
Weir Submerg		Froude # Chl	0.27	0.24
Weir Max Depth (ft)		Specif Force (cu ft)	288641.80	317445.10
Min El Weir Flow (ft)	1056.45	Hydr Depth (ft)	15.78	25.11
Min El Prs (ft)	1098.00	W.P. Total (ft)	1278.49	952.29
Delta EG (ft)	0.29	Conv. Total (cfs)	10938100.0	10776900.0
Delta WS (ft)	0.38	Top Width (ft)	1201.35	814.40
BR Open Area (sq ft)	87314.74	Frctn Loss (ft)	0.00	0.03
BR Open Vel (ft/s)	7.46	C & E Loss (ft)	0.10	0.01
BR Sluice Coef		Shear Total (lb/sq ft)	0.15	0.23
BR Sel Method	Energy only	Power Total (lb/ft s)	1.16	1.60

Errors, Warnings and Notes

Profile Output Table - Standard Table 1

FileOptionsStd.TablesLocationsHelp

HEC-RAS Plan: CorrEff River: Missouri Reach: 1

Reach	River Sta	Profile	Q Total (cfs)	Min Chl (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
1	693.78 BBQ	100-yr	41500.00	1014.97	1048.50		1049.62	0.000239	8.66	19510.94	17314.20	0.31
1	693.78 BBQ	Floodway	41500.00	1014.97	1049.60		1050.63	0.000211	8.30	19736.95	1151.17	0.29
1	693.39 BBP	100-yr	41500.00	1019.05	1048.17		1049.11	0.000197	8.01	22626.72	21260.66	0.28
1	693.39 BBP	Floodway	41500.00	1019.05	1049.35		1050.18	0.000165	7.57	23983.09	1467.71	0.26
1	692.98 BBO	100-yr	41500.00	1015.30	1047.98		1048.61	0.000171	7.11	35189.79	28644.61	0.26
1	692.98 BBO	Floodway	41500.00	1015.30	1049.21		1049.76	0.000139	6.63	36071.52	3401.60	0.24
1	692.58 BBN	100-yr	41500.00	1020.40	1047.91		1048.24	0.000099	5.54	49164.07	22631.59	0.20
1	692.58 BBN	Floodway	41500.00	1020.40	1049.13		1049.45	0.000089	5.42	46903.30	4238.27	0.19
1	692.17 BBM	100-yr	41500.00	1016.10	1047.65		1048.02	0.000104	5.54	51547.27	18736.91	0.22
1	692.17 BBM	Floodway	41500.00	1016.10	1048.91		1049.26	0.000088	5.31	49681.22	4047.59	0.20
1	691.75 BBL	100-yr	41500.00	1010.25	1047.40		1047.81	0.000082	5.79	44571.57	14044.55	0.20
1	691.75 BBL	Floodway	41500.00	1010.25	1048.56		1049.05	0.000086	6.10	38230.40	2788.52	0.21
1	691.35 BBK	100-yr	41500.00	1010.41	1047.04		1047.56	0.000082	6.49	39175.63	15429.97	0.21
1	691.35 BBK	Floodway	41500.00	1010.41	1047.83		1048.73	0.000112	7.73	21053.10	1151.81	0.24
1	691.2	100-yr	41500.00	1010.29	1047.03		1047.44	0.000075	5.69	42839.19	7908.33	0.19
1	691.2	Floodway	41500.00	1010.29	1048.04		1048.39	0.000064	5.37	46597.55	8413.11	0.18
1	691.1	100-yr	41500.00	1014.94	1046.74	1029.48	1047.36	0.000108	6.71	27295.61	5684.76	0.23
1	691.1	Floodway	41500.00	1014.94	1047.73	1029.48	1048.31	0.000096	6.48	29172.30	6496.67	0.22
1	691.035 ST HWY 51/175	Bridge										
1	691	100-yr	41800.00	1011.12	1046.36		1047.07	0.000137	6.80	20864.30	4428.60	0.24
1	691	Floodway	41800.00	1011.12	1047.40		1048.06	0.000120	6.53	21750.94	4921.12	0.22
1	690.95 BBH	100-yr	41800.00	1008.94	1046.38		1046.95	0.000136	6.28	26719.69	15231.26	0.21
1	690.95 BBH	Floodway	41800.00	1008.94	1047.36		1048.01	0.000140	6.51	22137.71	1428.19	0.22
1	690.52 BBG	100-yr	41800.00	1006.69	1045.84		1046.62	0.000148	7.31	27919.81	13513.53	0.25
1	690.52 BBG	Floodway	41800.00	1006.69	1046.82		1047.68	0.000147	7.45	19989.93	976.21	0.25
1	690.13 BBF	100-yr	41800.00	1016.20	1045.47		1046.27	0.000190	7.72	30794.87	13754.26	0.28
1	690.13 BBF	Floodway	41800.00	1016.20	1046.25		1047.28	0.000217	8.33	21410.77	1377.16	0.30
1	689.72 BBE	100-yr	41800.00	1009.40	1045.15		1045.86	0.000186	7.11	30662.60	13066.84	0.25
1	689.72 BBE	Floodway	41800.00	1009.40	1045.95		1046.79	0.000198	7.43	22206.95	1318.79	0.26
1	689.31 BBD	100-yr	41800.00	1015.15	1044.54		1045.40	0.000232	7.62	22280.42	13454.24	0.28
1	689.31 BBD	Floodway	41800.00	1015.15	1045.53		1046.35	0.000208	7.41	22142.24	1200.00	0.27
1	688.91 BBC	100-yr	41800.00	1012.70	1044.18		1044.90	0.000196	7.26	35959.84	12457.20	0.26
1	688.91 BBC	Floodway	41800.00	1012.70	1045.18		1045.91	0.000183	7.21	34199.60	2902.75	0.25
1	688.49 BBB	100-yr	41800.00	1010.75	1043.91		1044.47	0.000159	6.59	52020.47	11900.73	0.23
1	688.49 BBB	Floodway	41800.00	1010.75	1044.89		1045.51	0.000158	6.73	47828.29	5590.61	0.23

Proposed Conditions

Bridge Output

FileTypeOptionsHelp

River:MissouriProfile:100-yrReach1RS:691.035Plan:Prop

Plan: Prop Missouri 1 RS: 691.035 Profile: 100-yr				
E.G. US. (ft)	1047.35	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	1046.81	E.G. Elev (ft)	1047.21	1047.10
Q Total (cfs)	141500.00	W.S. Elev (ft)	1046.31	1046.41
Q Bridge (cfs)	141500.00	Crit W.S. (ft)	1029.89	1029.09
Q Weir (cfs)		Max Chl Dpth (ft)	31.37	35.29
Weir Sta Lft (ft)		Vel Total (ft/s)	6.50	5.94
Weir Sta Rgt (ft)		Flow Area (sq ft)	21765.10	23812.69
Weir Submerg		Froude # Chl	0.27	0.24
Weir Max Depth (ft)		Specif Force (cu ft)	294979.50	323185.90
Min El Weir Flow (ft)	1056.45	Hydr Depth (ft)	12.51	16.04
Min El Prs (ft)	1102.49	W.P. Total (ft)	1849.64	1650.83
Delta EG (ft)	0.30	Conv. Total (cfs)	11240500.0	11075560.0
Delta WS (ft)	0.40	Top Width (ft)	1739.98	1484.76
BR Open Area (sq ft)	121573.90	Frctn Loss (ft)	0.01	0.03
BR Open Vel (ft/s)	6.50	C & E Loss (ft)	0.10	0.03
BR Sluice Coef		Shear Total (lb/sq ft)	0.12	0.15
BR Sel Method	Energy only	Power Total (lb/ft s)	0.76	0.87

Errors, Warnings and Notes

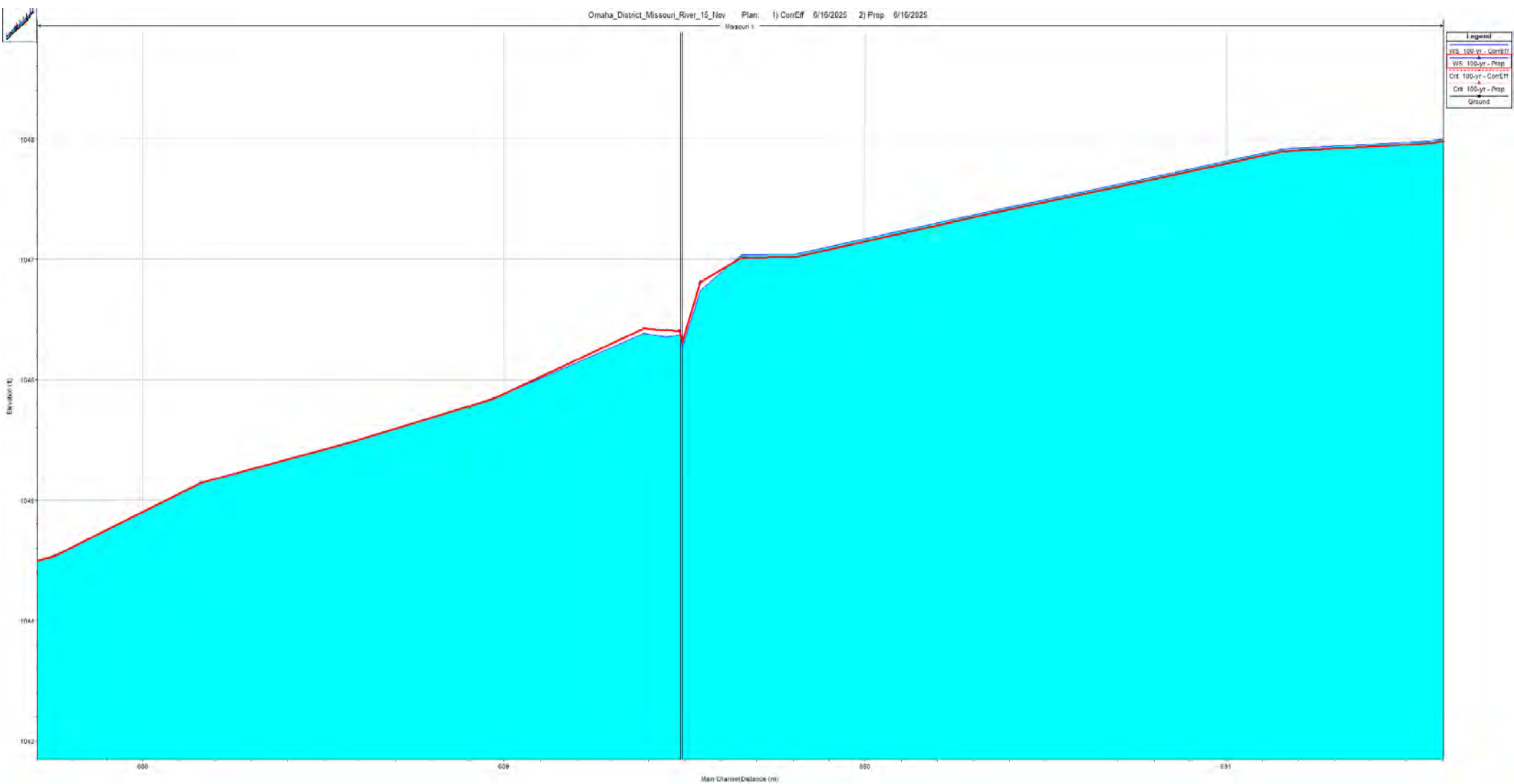
Profile Output Table - Standard Table 1

FileOptionsStd.TablesLocationsHelp

HEC-RAS Plan: Prop River: Missouri Reach: 1

Reach	River Sta	Profile	Q Total (cfs)	Min Chl (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
1	693.78 BBQ	100-yr	41500.00	1014.97	1048.48		1049.61	0.000239	8.67	19489.24	17214.51	0.31
1	693.78 BBQ	Floodway	41500.00	1014.97	1049.62		1050.65	0.000210	8.30	19756.76	1151.17	0.29
1	693.39 BBP	100-yr	41500.00	1019.05	1048.15		1049.09	0.000197	8.01	22599.62	21038.38	0.28
1	693.39 BBP	Floodway	41500.00	1019.05	1049.37		1050.19	0.000165	7.57	24009.61	1467.76	0.26
1	692.98 BBO	100-yr	41500.00	1015.30	1047.96		1048.60	0.000172	7.12	35113.90	28550.38	0.26
1	692.98 BBO	Floodway	41500.00	1015.30	1049.23		1049.77	0.000138	6.62	36137.54	3401.78	0.24
1	692.58 BBN	100-yr	41500.00	1020.40	1047.90		1048.23	0.000100	5.55	49050.20	22524.80	0.20
1	692.58 BBN	Floodway	41500.00	1020.40	1049.15		1049.47	0.000089	5.41	46987.63	4238.43	0.19
1	692.17 BBM	100-yr	41500.00	1016.10	1047.63		1048.00	0.000105	5.55	51430.34	18648.13	0.22
1	692.17 BBM	Floodway	41500.00	1016.10	1048.93		1049.28	0.000088	5.30	49764.23	4047.73	0.20
1	691.75 BBL	100-yr	41500.00	1010.25	1047.38		1047.79	0.000082	5.80	44499.42	13976.92	0.20
1	691.75 BBL	Floodway	41500.00	1010.25	1048.58		1049.07	0.000086	6.09	38289.98	2788.98	0.21
1	691.35 BBK	100-yr	41500.00	1010.41	1047.02		1047.54	0.000082	6.49	39114.16	15197.45	0.21
1	691.35 BBK	Floodway	41500.00	1010.41	1047.85		1048.75	0.000112	7.72	21077.93	1153.77	0.24
1	691.2	100-yr	41500.00	1010.29	1047.01		1047.42	0.000075	5.70	42771.66	7903.76	0.19
1	691.2	Floodway	41500.00	1010.29	1048.06		1048.41	0.000064	5.36	46701.24	8427.60	0.18
1	691.1	100-yr	41500.00	1014.94	1046.81	1029.48	1047.35	0.000098	6.40	32368.92	5748.89	0.22
1	691.1	Floodway	41500.00	1014.94	1047.86	1029.48	1048.35	0.000085	6.11	34839.63	6643.35	0.21
1	691.035 ST HWY 51/175	Bridge										
1	691	100-yr	41800.00	1011.12	1046.41		1047.05	0.000126	6.54	25825.41	4771.42	0.23
1	691	Floodway	41800.00	1011.12	1047.51		1048.08	0.000108	6.21	27777.61	5235.68	0.21
1	690.95 BBH	100-yr	41800.00	1008.94	1046.42		1046.95	0.000128	6.11	30749.13	15491.20	0.21
1	690.95 BBH	Floodway	41800.00	1008.94	1047.36		1048.01	0.000140	6.51	22137.71	1428.19	0.22
1	690.52 BBG	100-yr	41800.00	1006.69	1045.84		1046.62	0.000148	7.31	27919.81	13513.53	0.25
1	690.52 BBG	Floodway	41800.00	1006.69	1046.82		1047.68	0.000147	7.45	19989.93	976.21	0.25
1	690.13 BBF	100-yr	41800.00	1016.20	1045.47		1046.27	0.000190	7.72	30794.87	13754.26	0.28
1	690.13 BBF	Floodway	41800.00	1016.20	1046.25		1047.28	0.000217	8.33	21410.77	1377.16	0.30
1	689.72 BBE	100-yr	41800.00	1009.40	1045.15		1045.86	0.000186	7.11	30662.60	13066.84	0.25
1	689.72 BBE	Floodway	41800.00	1009.40	1045.95		1046.79	0.000198	7.43	22206.95	1318.79	0.26
1	689.31 BBD	100-yr	41800.00	1015.15	1044.54		1045.40	0.000232	7.62	22280.42	13454.24	0.28
1	689.31 BBD	Floodway	41800.00	1015.15	1045.53		1046.35	0.000208	7.41	22142.24	1200.00	0.27
1	688.91 BBC	100-yr	41800.00	1012.70	1044.18		1044.90	0.000196	7.26	35959.84	12457.20	0.26
1	688.91 BBC	Floodway	41800.00	1012.70	1045.18		1045.91	0.000183	7.21	34199.60	2902.75	0.25
1	688.49 BBB	100-yr	41800.00	1010.75	1043.91		1044.47	0.000159	6.59	52020.47	11900.73	0.23
1	688.49 BBB	Floodway	41800.00	1010.75	1044.89		1045.51	0.000158	6.73	47828.29	5590.61	0.24

Water Surface Profile Plot: Corrected Effective vs Proposed Conditions



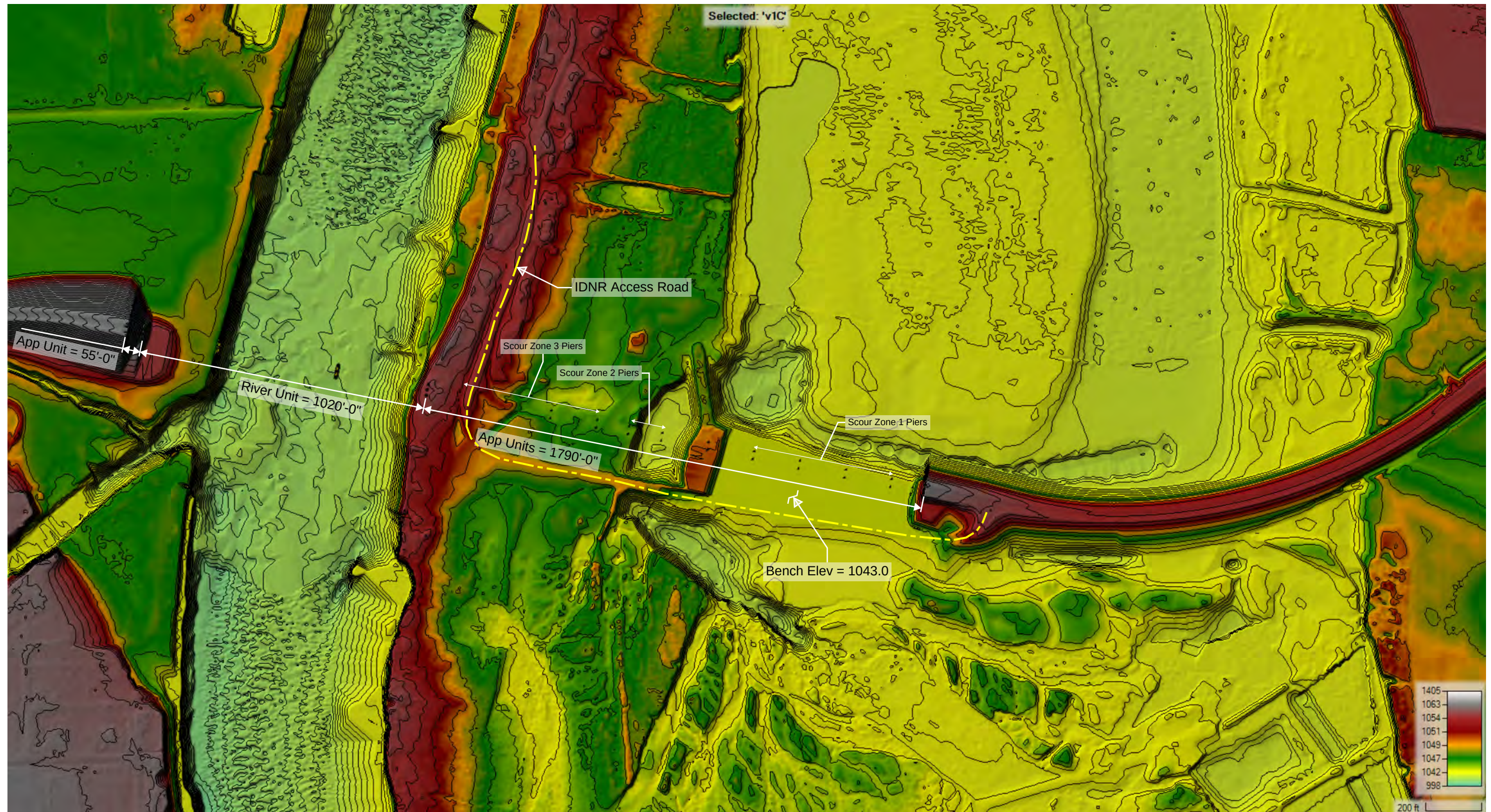
Appendix G – Other Strategy Terrain Plots

Strategy: v_1c

Total length of Iowa Approach Units = 1,790'-0"

New abutment re-location east of existing abutment = 800'-0"

Bench Elevation = 1043.0

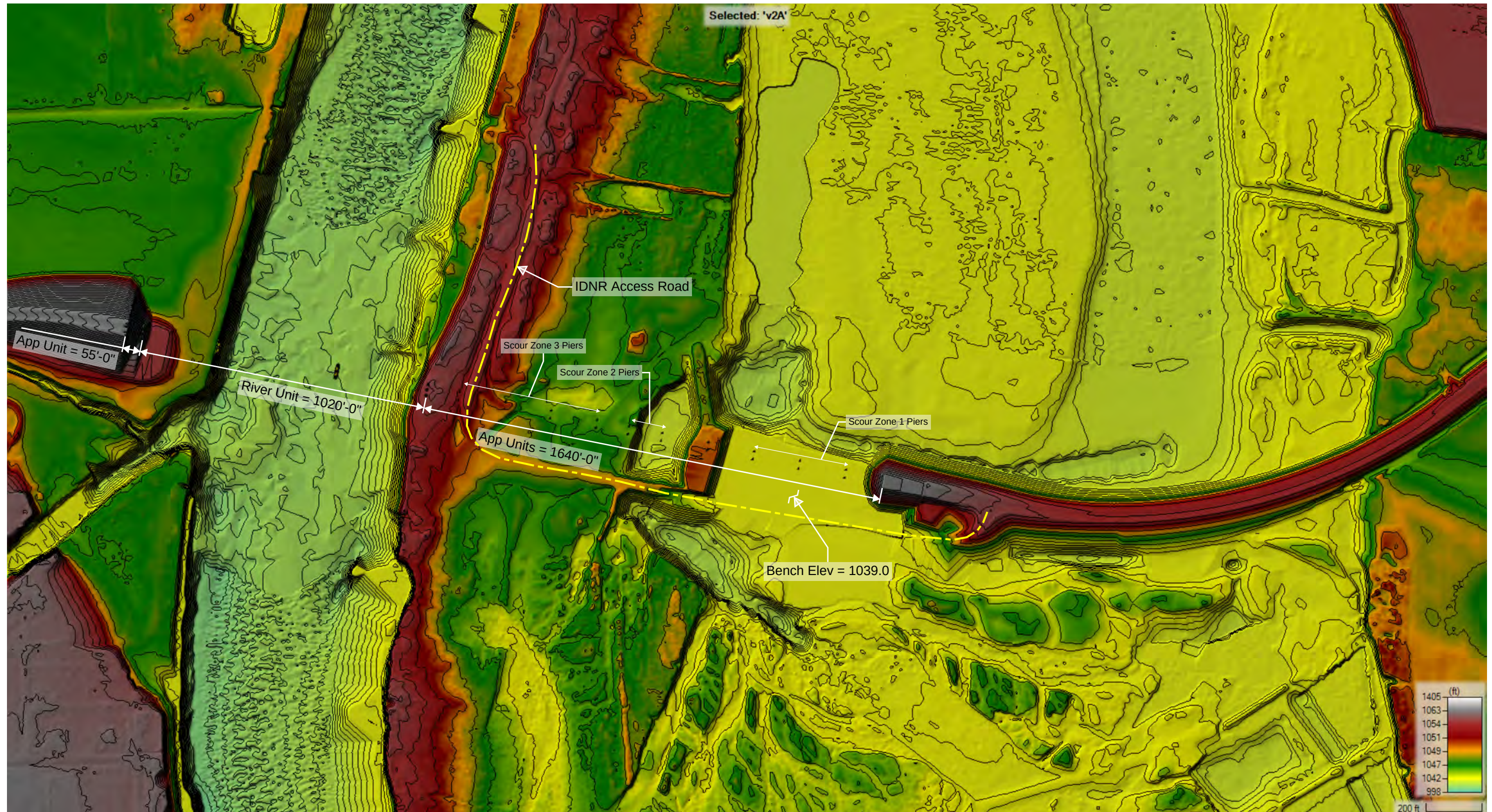


Strategy: v_2a

Total length of Iowa Approach Units = 1,640'-0"

New abutment re-location east of existing abutment = 650'-0"

Bench Elevation = 1039.0

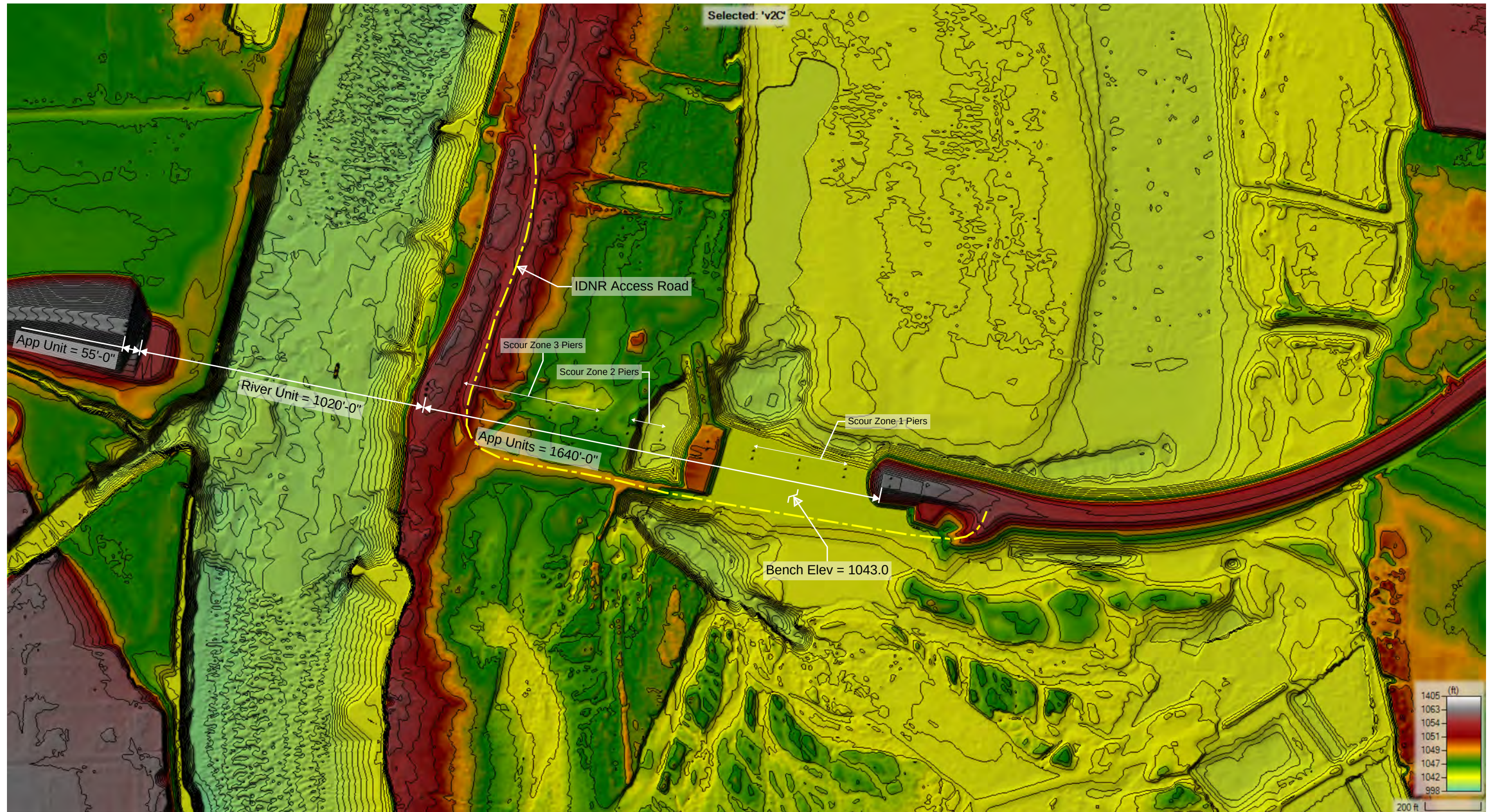


Strategy: v_2c

Total length of Iowa Approach Units = 1,640'-0"

New abutment re-location east of existing abutment = 650'-0"

Bench Elevation = 1043.0

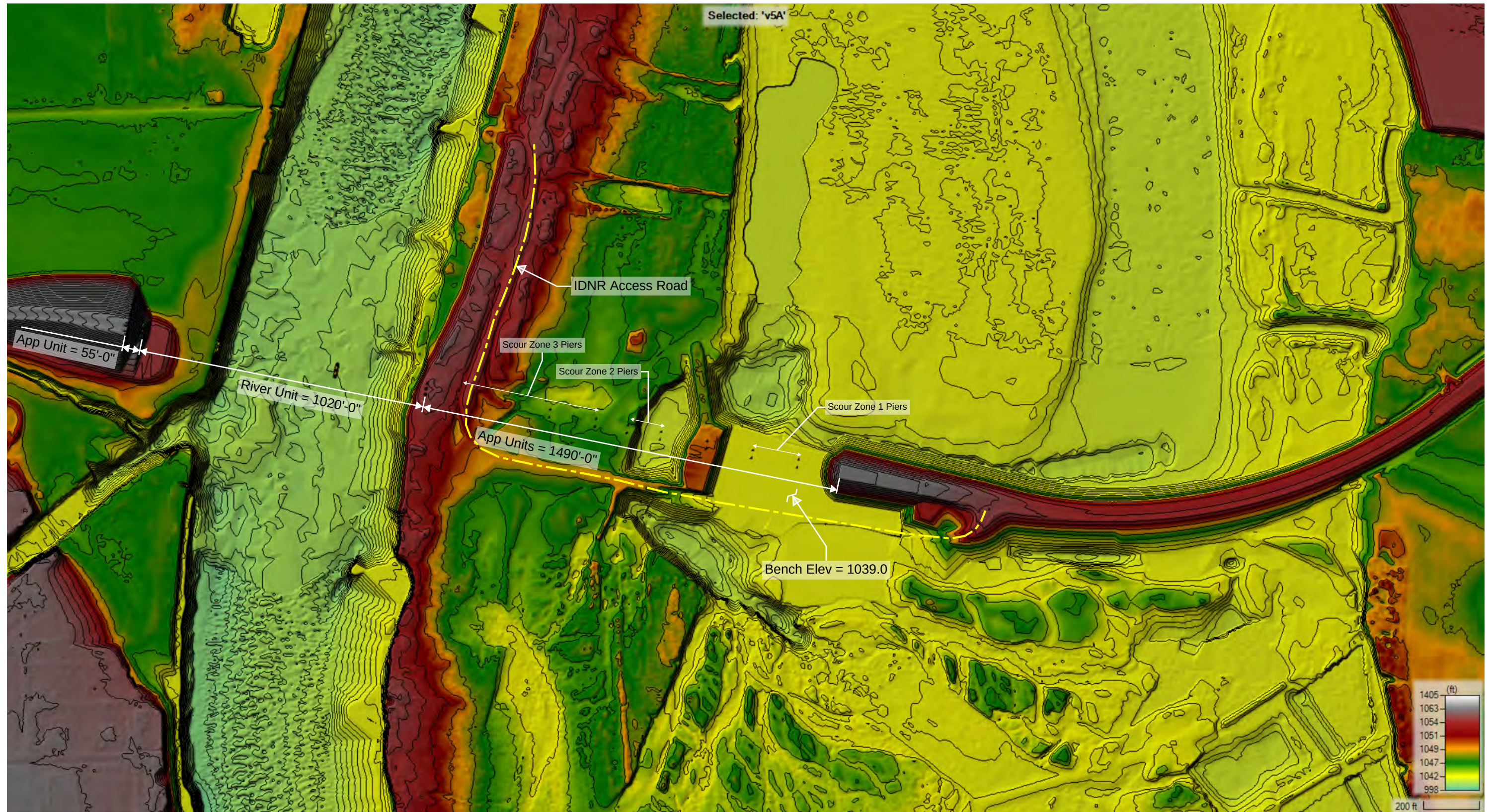


Strategy: v_5a

Total length of Iowa Approach Units = 1,490'-0"

New abutment re-location east of existing abutment = 500'-0"

Bench Elevation = 1039.0



Appendix H – Other Strategy Results

Hydraulic Strategy Evaluation

Notes

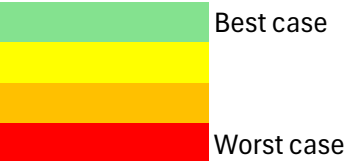
- All Strategies below use River Unit Option "Blue 3A"

- the "Effective FEMA Q100" is from the 2003 USACE UMFRRS

- the "New Q100" is from the 2023 USACE MRFFS

- the "Effective FEMA Q100" corresponds to ~ Q35 per 2023 USACE MRFFS

Legend



				Recommended				
Strategy	Units	v_1C	v_2A	v_2B	v_2C	v_5A	v_3A	
Extension Length (Approx.)	LF	800	650	650	650	500	155	
Iowa Approach Unit Length	LF	1,790	1,640	1,640	1,640	1,490	1,145	
Bench Elevation		1043	1039	1041	1043	1039	1047	
Resiliency Metrics								
Highway Overtopping								
Highway overtopped by Q100?	ft	no	no	no	no	no	no	
Velocity								
2011 Flood (~ Q70)	ft			See Velocity Heat Maps				
Q200	ft							
Zone 1 Scour - Piers in Newly Opened Channel								
Q10	ft	3	6	8	4	8	n/a	
Q25	ft	16	18	19	19	23		
Effective FEMA Q100	ft	20	24	25	24	30		
2011 Flood (~ Q70)	ft	31	37	37	36	46		
Q100	ft	36	44	43	42	53		
Q200	ft	43	52	51	50	62		
Q500	ft	46	55	54	52	65		
Zone 2 Scour - Piers in Existing Scour Hole								
Q10	ft	0	0	0	0	0	0	
Q25	ft	1	1	2	3	2	0	
Effective FEMA Q100	ft	2	2	4	4	4	5	
2011 Flood (~ Q70)	ft	10	9	10	11	10	24	
Q100	ft	13	12	12	16	13	39	
Q200	ft	26	22	25	30	27	55	
Q500	ft	29	25	28	33	30	58	
Zone 3 Scour - Piers West of Existing Scour Hole								
Q10	ft	0	0	0	0	0	0	
Q25	ft	2	1	2	2	1	0	
Effective FEMA Q100	ft	3	2	3	3	3	2	
2011 Flood (~ Q70)	ft	3	3	3	4	3	5	
Q100	ft	4	4	4	5	4	7	
Q200	ft	5	5	5	5	5	10	
Q500	ft	6	5	6	6	6	10	
Shear Stress								
2011 Flood (~ Q70)	ft			See Shear Stress Heat Maps				
Q200	ft							
Other Impacts								
Rise at Effective FEMA Q100		Yes, affects ~ 300 acres downstream						
Rise at new Q100		Yes. Moderate rise (over 0.1 ft) for ~300 acres downstream. Minimal rise (less than 0.1 ft) extending to south limits of model						Yes. Moderate rise for ~300 ac DS. Minimal rise extending to north & south limits of model
IDNR Access Road Flooded at:		Q10	Q5	Q5 (est.)	Q10	Q5	Q40 (est.)	
Cost Estimate								
Total Iowa Approach Unit Cost	\$	\$26.2 million	\$ 24.3 million	\$ 24.2 million	\$ 24.2 million	\$ 22.3 million	\$ 18.6 million	

Cost Estimate Notes

- Unit prices from Decatur Preliminary Cost Estimate (updated as of **date in header**)

- Iowa Approach Unit Cost includes: embankment removal and additional length of embankment for the shorter bridge options

- Cost includes mobilization (10%)

- Cost does not reflect potential decrease in pier shaft size at piers with less scour (i.e. same shafts assumed for all approach piers)

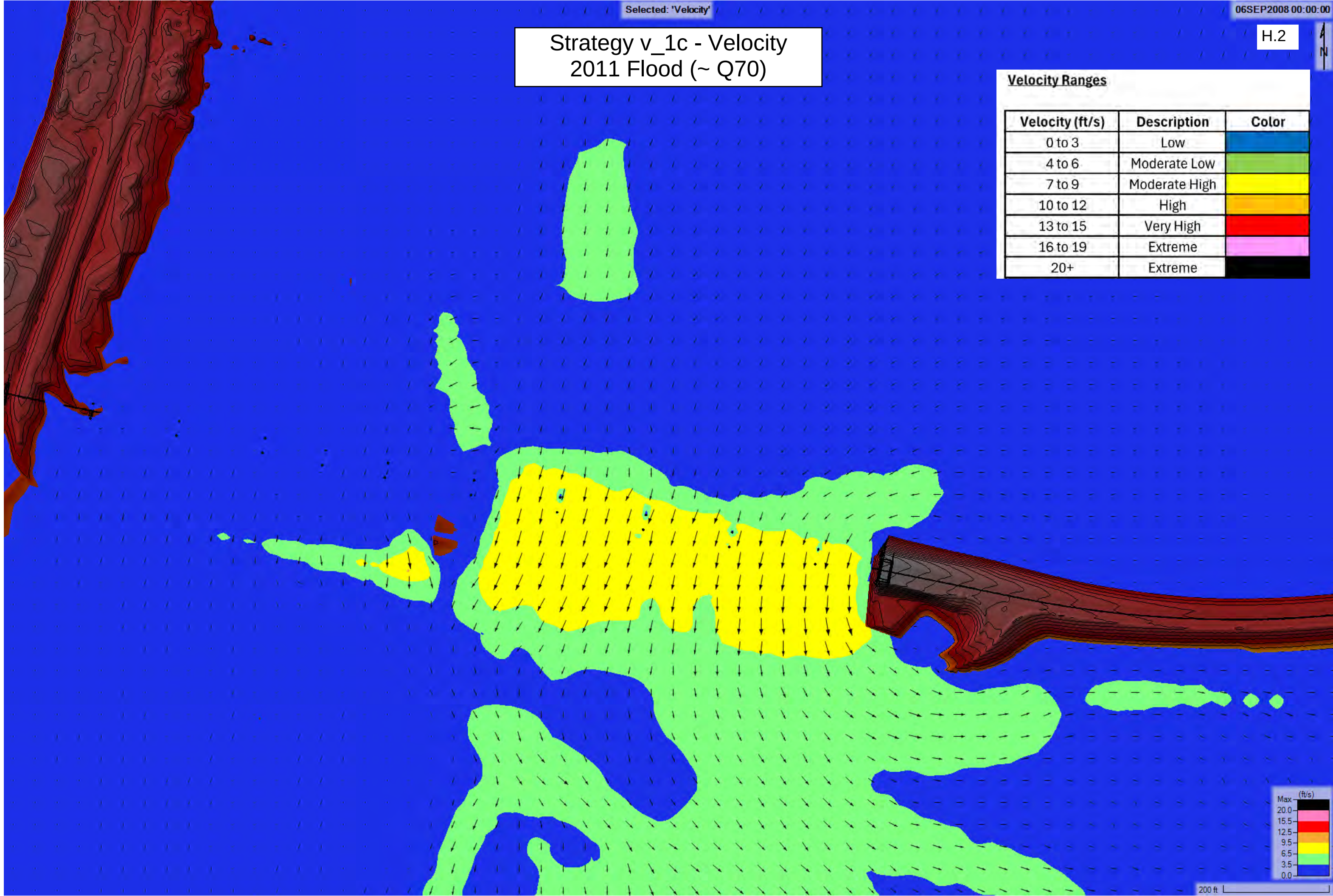
- Cost does not include scour countermeasure riprap



Strategy v_1c - Velocity
2011 Flood (~ Q70)

Velocity Ranges

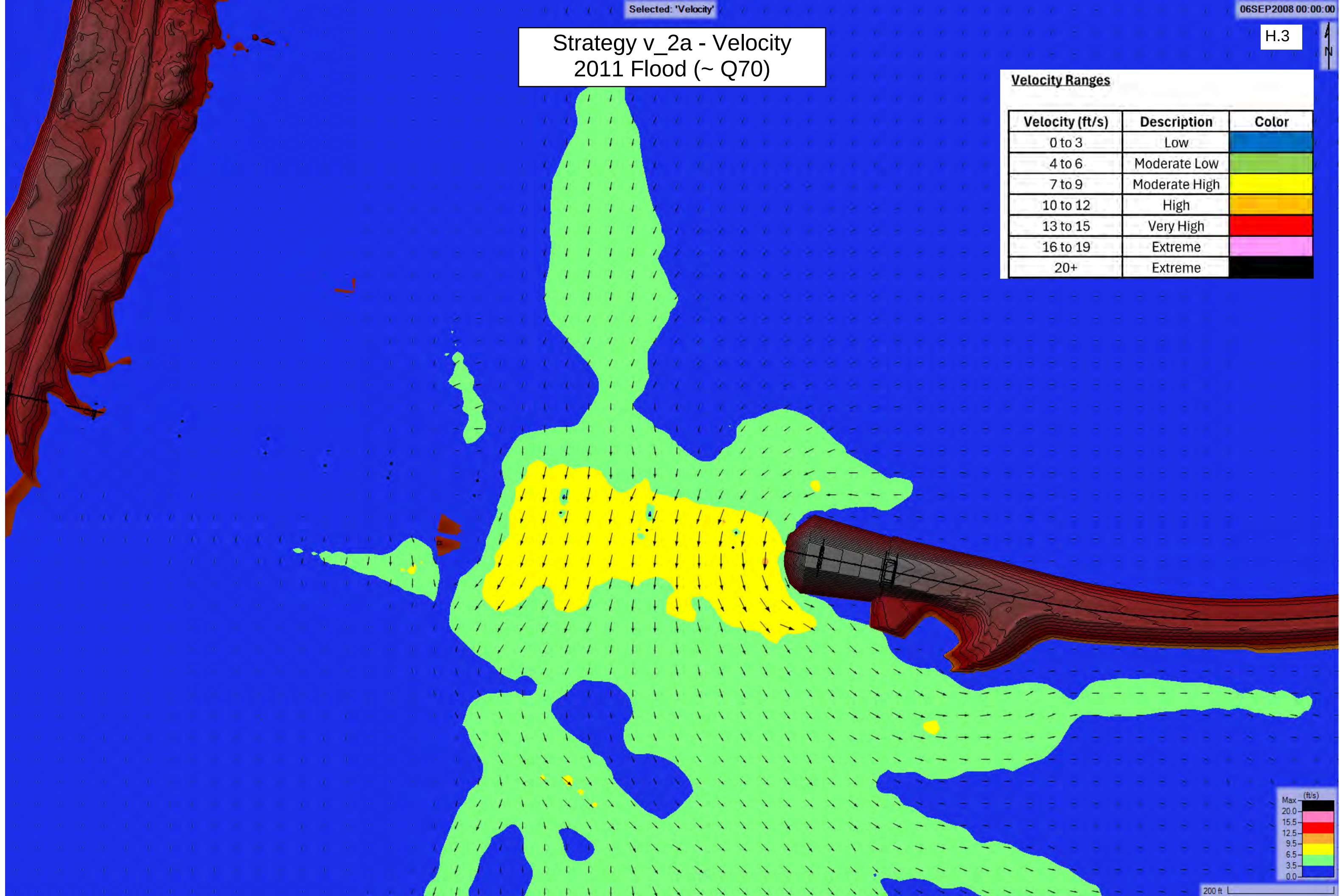
Velocity (ft/s)	Description	Color
0 to 3	Low	Blue
4 to 6	Moderate Low	Green
7 to 9	Moderate High	Yellow
10 to 12	High	Orange
13 to 15	Very High	Red
16 to 19	Extreme	Pink
20+	Extreme	Black



Strategy v_2a - Velocity 2011 Flood (~ Q70)

Velocity Ranges

Velocity (ft/s)	Description	Color
0 to 3	Low	Blue
4 to 6	Moderate Low	Light Green
7 to 9	Moderate High	Yellow
10 to 12	High	Orange
13 to 15	Very High	Red
16 to 19	Extreme	Pink
20+	Extreme	Black

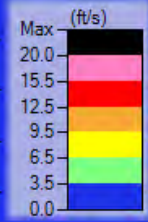
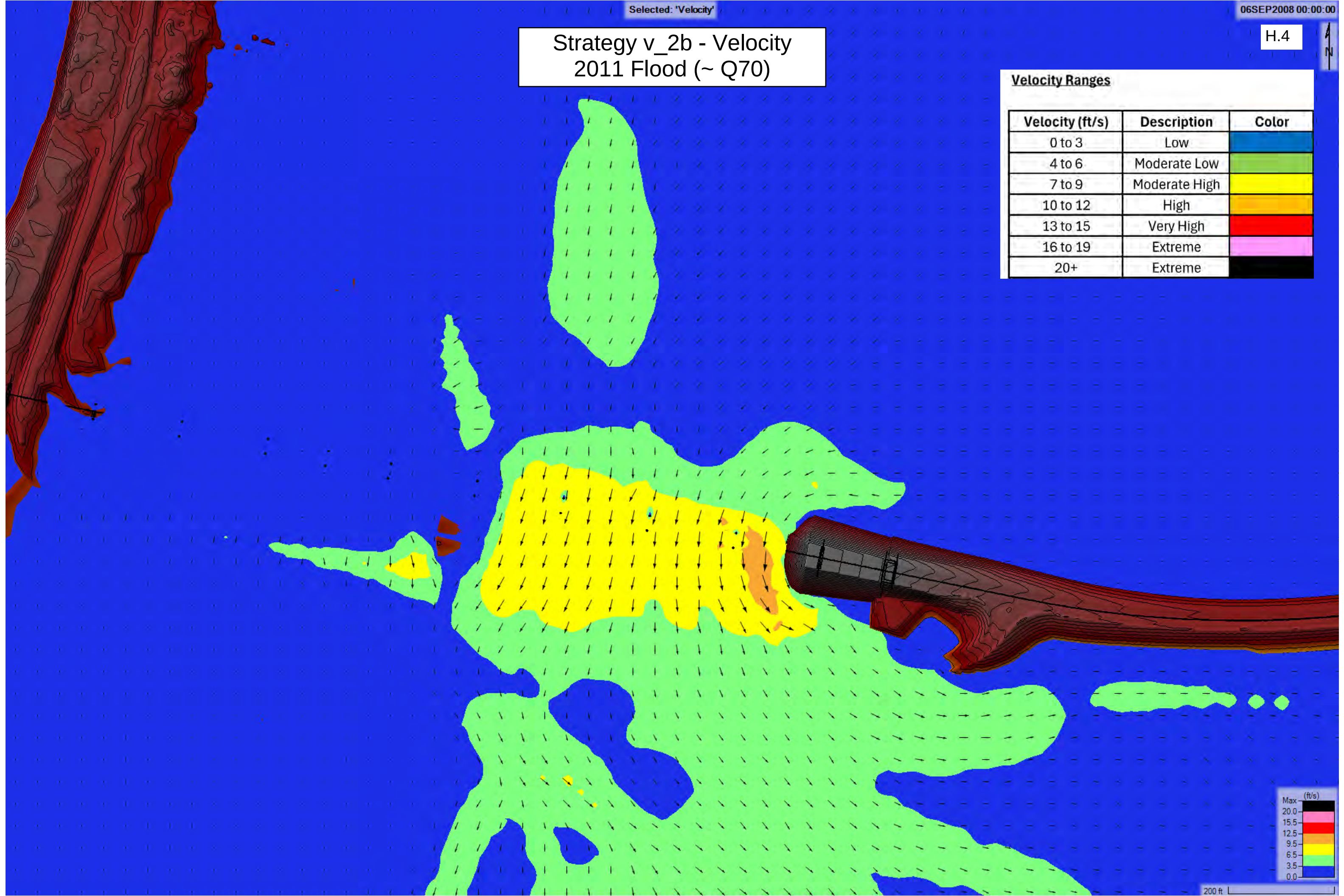


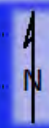


Strategy v_2b - Velocity
2011 Flood (~ Q70)

Velocity Ranges

Velocity (ft/s)	Description	Color
0 to 3	Low	Blue
4 to 6	Moderate Low	Light Green
7 to 9	Moderate High	Yellow
10 to 12	High	Orange
13 to 15	Very High	Red
16 to 19	Extreme	Pink
20+	Extreme	Black

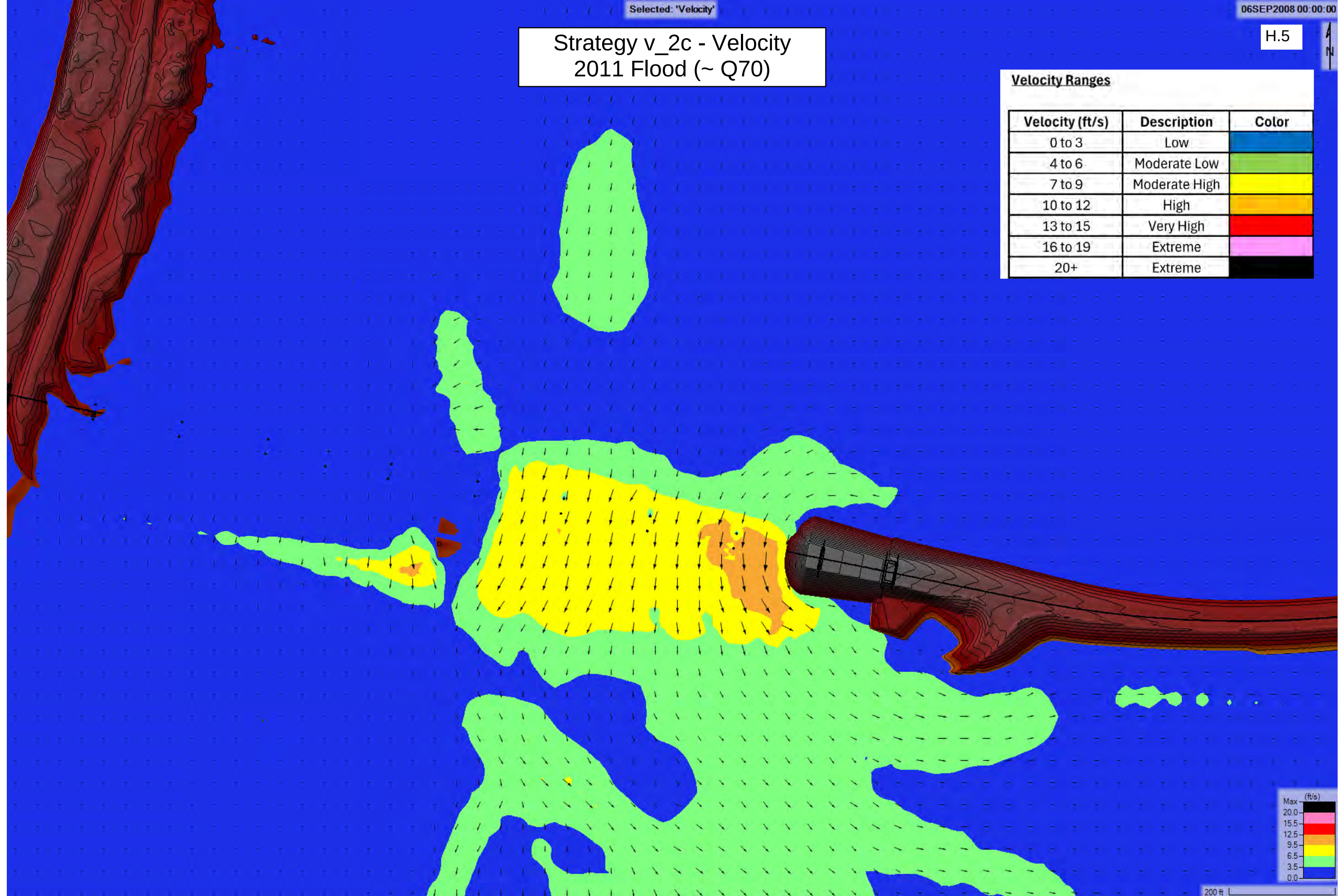




Strategy v_2c - Velocity
2011 Flood (~ Q70)

Velocity Ranges

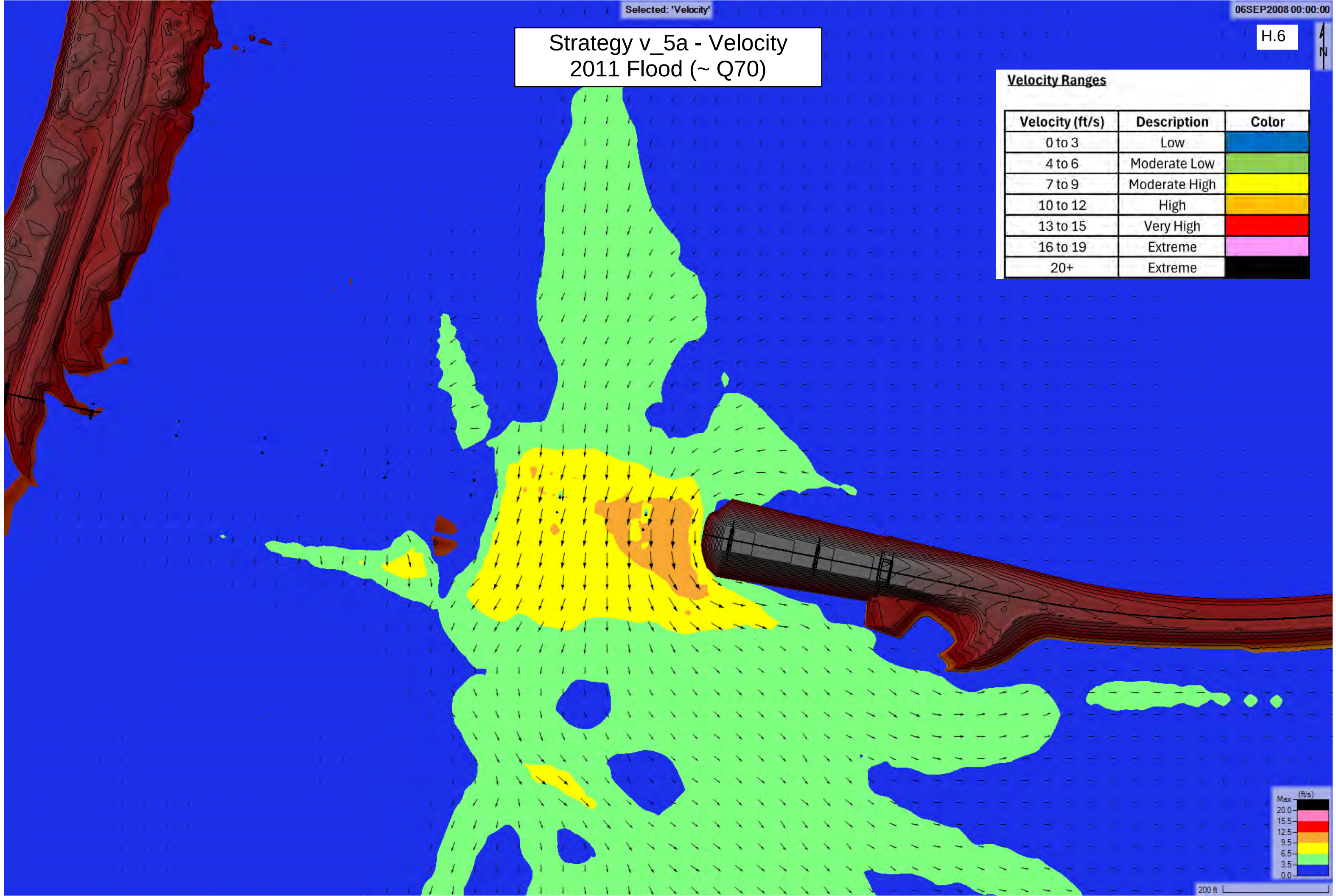
Velocity (ft/s)	Description	Color
0 to 3	Low	Blue
4 to 6	Moderate Low	Green
7 to 9	Moderate High	Yellow
10 to 12	High	Orange
13 to 15	Very High	Red
16 to 19	Extreme	Pink
20+	Extreme	Black

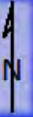


Strategy v_5a - Velocity
2011 Flood (~ Q70)

Velocity Ranges

Velocity (ft/s)	Description	Color
0 to 3	Low	Blue
4 to 6	Moderate Low	Green
7 to 9	Moderate High	Yellow
10 to 12	High	Orange
13 to 15	Very High	Red
16 to 19	Extreme	Pink
20+	Extreme	Black

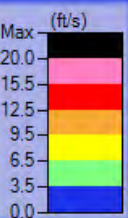
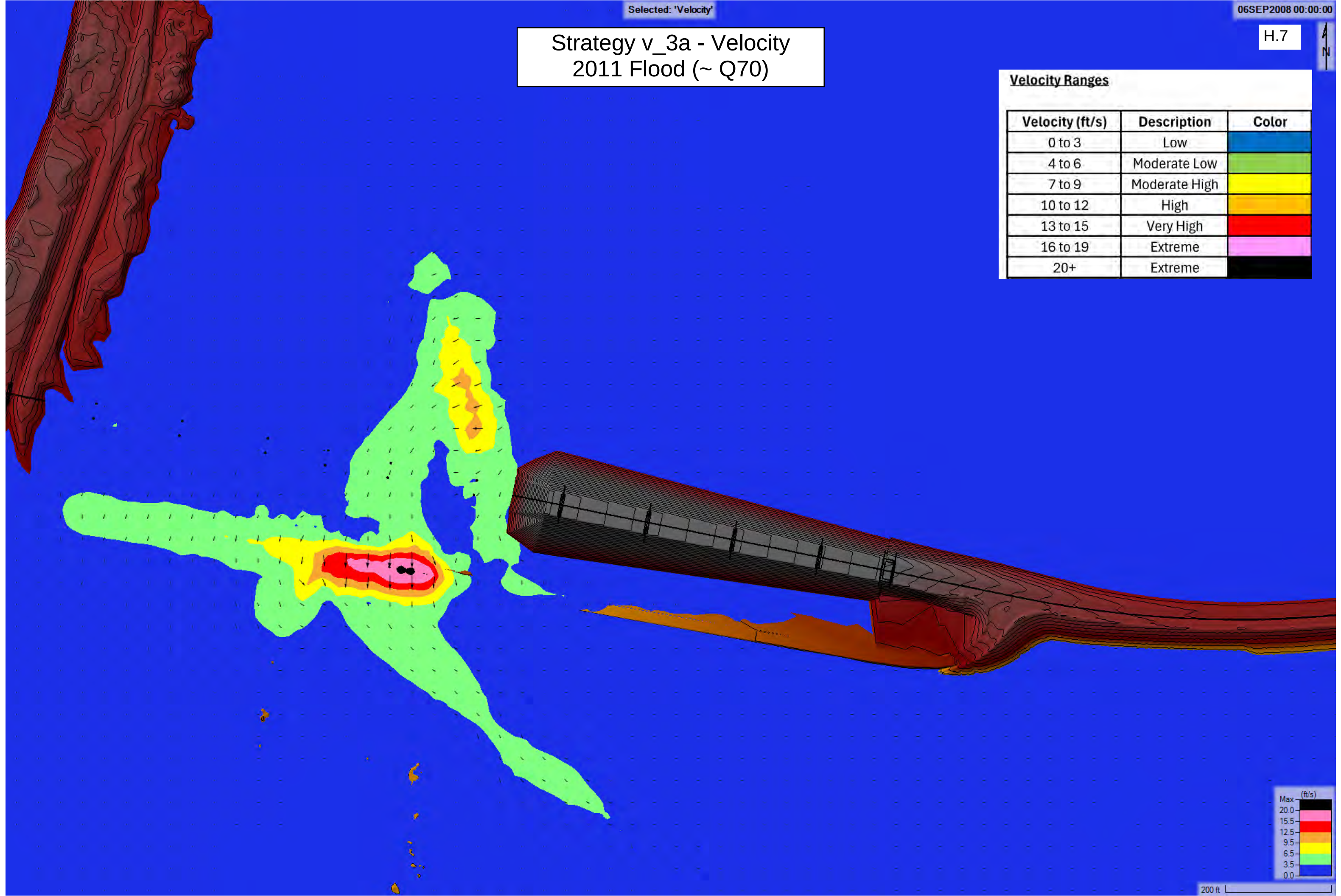




Strategy v_3a - Velocity 2011 Flood (~ Q70)

Velocity Ranges

Velocity (ft/s)	Description	Color
0 to 3	Low	Blue
4 to 6	Moderate Low	Light Green
7 to 9	Moderate High	Yellow
10 to 12	High	Orange
13 to 15	Very High	Red
16 to 19	Extreme	Pink
20+	Extreme	Black

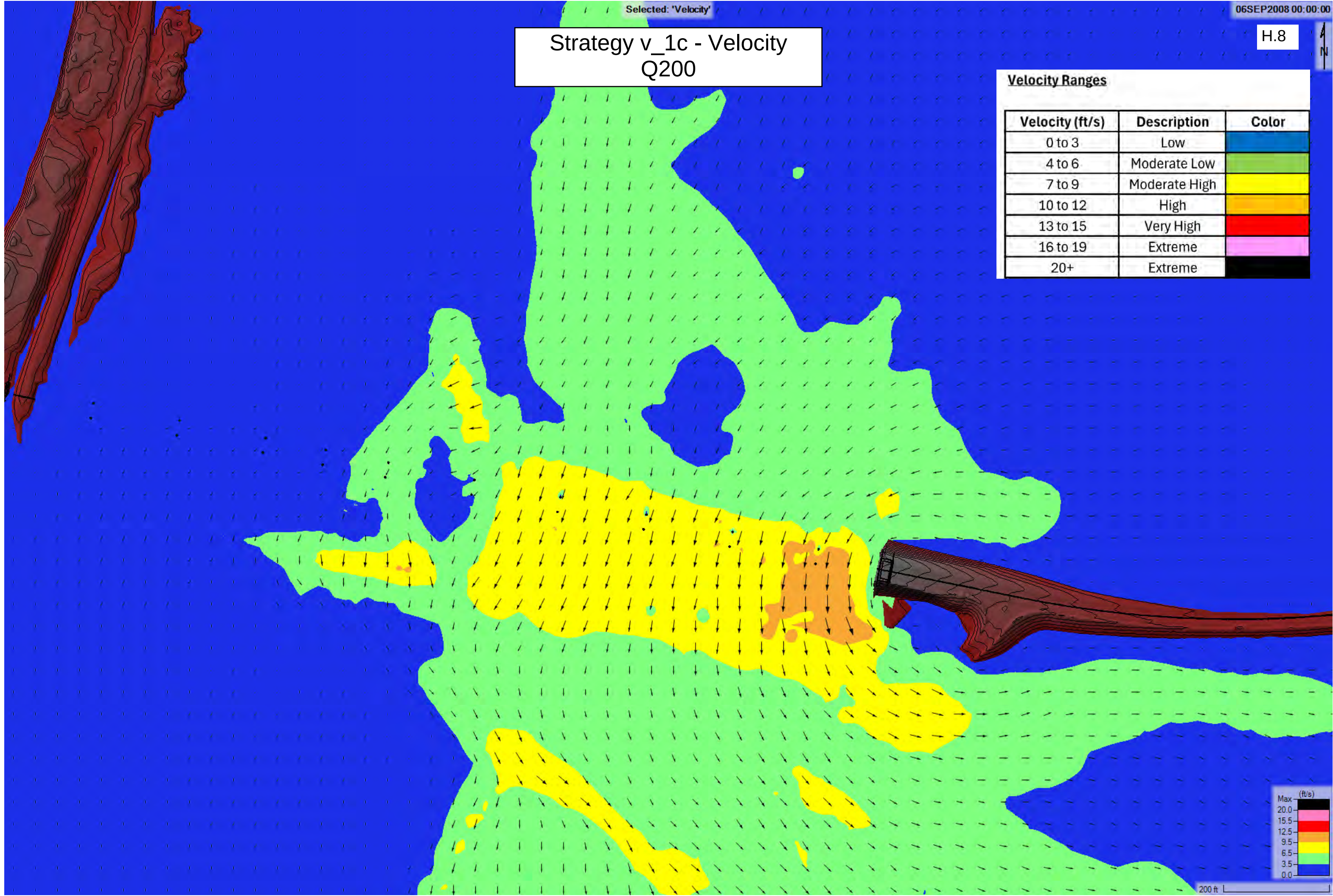




Strategy v_1c - Velocity
Q200

Velocity Ranges

Velocity (ft/s)	Description	Color
0 to 3	Low	Blue
4 to 6	Moderate Low	Light Green
7 to 9	Moderate High	Yellow
10 to 12	High	Orange
13 to 15	Very High	Red
16 to 19	Extreme	Pink
20+	Extreme	Black



200 ft