CDOT STRUCTURE F-19-E REPLACEMENT

Preliminary Hydraulic Design Report

March 2020

Prepared for:

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1 INTRODUCTION

1.1 Background and Project Description

Muller Engineering has prepared the following preliminary hydraulic design report for the replacement of CDOT Structure F-19-E. Structure F-19-E is an 82-foot long timber pile bridge and has been identified for replacement by the Colorado Department of Transportation (CDOT), Region 4. This report describes the existing site conditions, the proposed drainage design improvements recommended for the project at a Field Inspection Review (FIR) design level, and the hydrologic and hydraulic analyses related to both the existing and proposed site conditions.

1.2 Location and Project Area Description

The F-19-E bridge is located just east of the town of Strasburg, Colorado in Arapahoe County on US 36 near mile marker 98 and at Latitude 39.725452 North and Longitude 104.275026 West. The legal location of the structure is the South East 1/4 of the South West 1/4 of Section 1, Township 4 South, Range 62 West of the Sixth Meridian. **Figure 1** presents the location of Bridge F-19-E.

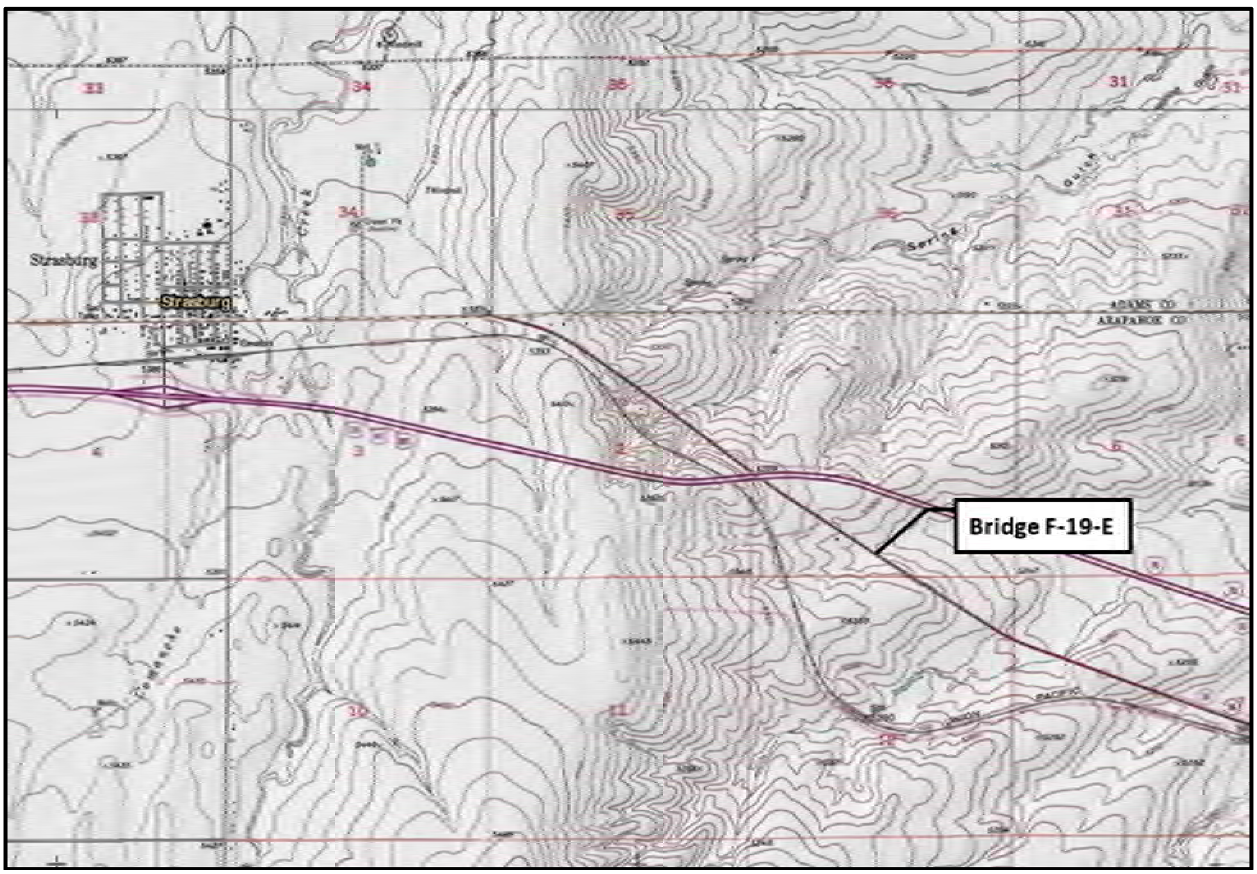

Figure 1. Vicinity Map

Structure F-19-E lies in a rural, primarily agricultural area with the surrounding topography generally consisting of hilly to gently sloping terrain. Bridge F-19-E spans an unnamed ephemeral tributary to West Bijou Creek with stormwater runoff passing from southwest to northeast through the bridge. A Union Pacific Railroad (UPRR) track is located approximately 0.3 miles upstream of Bridge F-19-E and generally parallels US 36. Based on information obtained from the Natural Resources Conservation

Services (NRCS) Web Soil Survey website, soils in the project vicinity are primarily classified as hydrologic soil group (HSG) C/D with some HSG Type B soils also being present.

2 HYDROLOGY

2.1 Flood History

No recorded floods have caused significant damage to this bridge, and no estimation of historic discharge events have been recorded at this location.

2.2 Regulatory Floodplain

The floodplain for the project site is mapped on FEMA community-panel number 08005C0300K effective December 17, 2010. Bridge F-19-E is located within a FEMA mapped Zone X, which is outside the 0.2% annual chance floodplain, in which base flood elevations (BFEs) and a floodplain have not been established. The unnamed tributary that Bridge F-19-E spans has no designated floodway. A FEMA Firmette is included in Appendix B that presents FEMA floodplain mapping near Bridge F-19-E.

According to Arapahoe County's *Stormwater Management Manual* a regulatory floodplain is defined as any drainageway with a drainage tributary area of 130 acres or more. The tributary area for Bridge F-19-E is approximately 320 acres and is therefore designated as a regulatory floodplain according to Arapahoe County criteria. Arapahoe County requires that development not increase the water surface elevation from the base flood elevation by more than one-half foot during the 100-year (1% annual risk) event, or by more than 0.00 feet at an insurable structure.

2.3 Design Flood Frequency

The CDOT Drainage Design Manual Table7.2 indicates that the scour design flood for the F-19-E structure should be the 100-year (1% annual risk) event and the scour check flood should be the 500-year (0.2% annual risk) event. A hydrologic analysis for Bridge F-19-E was performed by CDOT and technically reviewed by Muller Engineering. This analysis consisted of routing flows using the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center's, Hydrologic Modeling System (HEC-HMS) program. The hydrologic analysis used the Soil Conservation Service (SCS) method to assign sub-basin parameters, and a Type II, 24-hour storm distribution was assumed. The resulting design discharges at Bridge F-19-E are summarized in **Table 1.**

Table 1. Summary of Peak Discharges at Bridge F-19-E

10% Annual Risk 10-Year (cfs)	2% Annual Risk 50-Year (cfs)	1% Annual Risk 100-Year (cfs)	0.2% Annual Risk 500-Year (cfs)
85	185	240	385

A drainage basin map as well as a hydrologic calculation summary is presented in **Appendix B.**

3 EXISTING CONDITION HYDRAULICS

3.1 Existing Structure F-19-E

The existing F-19-E structure is an 89-year-old bridge constructed in 1931. The bridge has three equal spans supported by 1-foot diameter cross-braced timber, six pile, bents and abutments. Some of the bridge's stringers are supported near the west abutment with temporary wooden bents. The bridge superstructure is comprised of timber stringers with a concrete deck and asphalt overlay. Currently the bridge carries approximately 4,000 vehicles per day.

A field reconnaissance was performed by Muller Engineering on August 1, 2019 to evaluate the existing bridge, document the study site, identify hydraulically significant features, and estimate Manning roughness coefficients. Photographs of the site during this visit are presented in **Appendix A.** This field reconnaissance confirmed that Bridge F-19-E's drainage basin is primarily range and agricultural land with minimal potential for debris generation. Also, no evidence of debris accumulation was present at the bridge during the visit. The ephemeral tributary spanned by the bridge was noted as not being well defined during the visit. A berm and significant grade drop, approximately five feet, are present immediately downstream of Bridge F-19-E. This berm and grade drop are roughly at the CDOT right of way limit and could present a channel headcutting risk for Bridge F-19-E. Two excavations approximately one foot deep were noted near the helper bents at the west abutment. These excavations were assumed to be remnants of previous repair work and did not appear to be related to scour. No insurable structures appeared to be within the bridge's floodplain.

A field reconnaissance and desktop geomorphic assessment was performed by Olsson Associates in August and September of 2019. This evaluation indicates that "there is no readily apparent evidence of channel instability immediately upstream or downstream of the bridge site that would threaten the stability of a new replacement bridge." The geomorphic analysis also noted the headcutting potential at the bridge due to the downstream berm and grade drop as noted during the site visit. The geomorphic report for the site is presented in **Appendix E.**

3.2 Existing Conditions Hydraulic Modeling

3.2.1 Survey and Topography

Light Detection and Ranging (LiDAR) data was collected by the State of Colorado in May of 2018 and provided to Muller Engineering in June of 2019. Ground truthing and collection of photo identifiable points (PIDs) were used to calibrate the aerial data. The ephemeral channel spanned by Bridge F-19-E appeared to be correctly represented by the LiDAR data and a correction to model a low flow channel, sometimes obstructed during LiDAR data collection by water, was not required. Additional topographic ground survey information was collected by 105 West in September of 2019 at and around the F-19-E structure. This survey was merged into a combined terrain dataset by Ayres and Associates to create a continuous surface of best-available data throughout the model extents. All collected data have been transformed to Colorado State Plane Central (NAD-83) coordinates on the North American Vertical Datum of 1988 (NAVD-88) for the purposes of this analysis and design. This combined terrain dataset was provided to Muller Engineering by Ayres in October of 2019.

3.2.2 Existing Conditions Hydraulic Modeling Approach

Aquaveo's SMS version 13.0.8 was used to develop a 2-Dimensional (2D) hydraulic model for this effort.

SMS allows the user to develop a flexible computational mesh consisting of triangular or quadrilateral elements. SMS uses SRH-2D version 3.2, developed by the U.S. Bureau of Reclamation, to solve the 2D shallow water dynamic wave (depth-averaged St. Venant) equations.

The existing conditions model represents conditions prior to any changes as a result of the project. The model extends approximately 2,000 feet upstream of the bridge to the UPRR embankment, and approximately 2,000 feet downstream of the bridge to the embankment of I-70. The mesh consists of approximately 100,200 predominantly triangular elements, with quadrilateral elements representing the bridge and US 36 embankment. Element sizes range from 4-foot elements at the bridge to 40-foot elements along mesh boundaries. The existing trestle bents are likely to catch debris and were modeled as continuous obstructions within the model. A materials coverage was developed to represent ground cover within the model domain. Materials coverage and the associated Manning roughness values were developed using a combination of aerial imagery and site review. The Manning's n roughness values used for modeling efforts are presented in **Table 2.**

Table 2. Summary of Manning's n Roughness Values

Coverage Description	Manning's n Value
Mowed Grass	0.035
Natural Grass	0.040
Row Crops	0.035
Trees	0.070
Structures	0.250
Asphalt/Concrete	0.013
Homestead (Light Debris)	0.030
Homestead (Heavy Debris)	0.100
Earth (bare)	0.030

Boundary conditions used within the model include inflow boundaries for the unnamed ephemeral channel spanned by Bridge F-19-E, as well as an outflow water surface elevation at the downstream end of the model. The upstream boundary of the model coincides with the UPRR embankment which passes stormwater from upstream to downstream through the embankment via two culverts with similar size drainage basins. To stabilize flow further upstream of Bridge F-19-E and minimize negative depths within the model, it was assumed that inflows would be split equally between the discharge locations of the culverts passing through the UPRR embankment. The total upstream boundary condition inflows matched those developed by CDOT and presented previously in **Table 1** within Section 2.3.

The outflow boundary conditions were represented as normal depths, calculated based on the existing topography and material coverages for the site. No known drainage studies are available within the project limits to tie the downstream boundary conditions to a known water surface elevation. Given the distance from Bridge F-19-E to the downstream model limit it is not anticipated that altering the normal depth used as the downstream boundary condition would have an appreciable effect on the hydraulics for Bridge F-19-E. The downstream Boundary conditions for the 10-, 50-, 100-, and 500-year simulations are presented in **Table 3**.

Table 3. Downstream Boundary Conditions

Recurrence Interval	Downstream Water Surface Elevation (ft)
10-Year	5252.38
50-Year	5252.64
100-Year	5252.74
500-Year	5252.98

3.2.3 F-19-E Existing Conditions Hydraulic Modeling Results

A summary of the hydraulic properties immediately upstream and downstream of Structure F-19-E for the existing conditions is shown in **Table 4** with summary location cross sections illustrated in **Figure 2.**

Table 4. Existing Conditions Model Results

Recurrence	Section 1	Section 2	Section 1 - Velocity (ft/s)		Section 2 - Velocity	
Interval	WSEL (ft)	WSEL (ft)	Max (ft/s)	Average (ft/s)	Max (ft/s)	Average (ft/s)
10-Year	5283.73	5281.26	2.35	1.83	4.00	3.04
50-Year	5283.96	5281.42	2.99	2.21	5.27	4.42
100-Year	5284.08	5281.49	3.26	2.23	5.78	4.76
500-Year	5284.38	5281.65	3.60	2.40	6.91	5.34

The existing structure passes the 100-year and 500-year events without experiencing pressure flow through the bridge opening or overtopping of US 36. **Figure 2** presents the 100-year water depth plot at the existing structure.

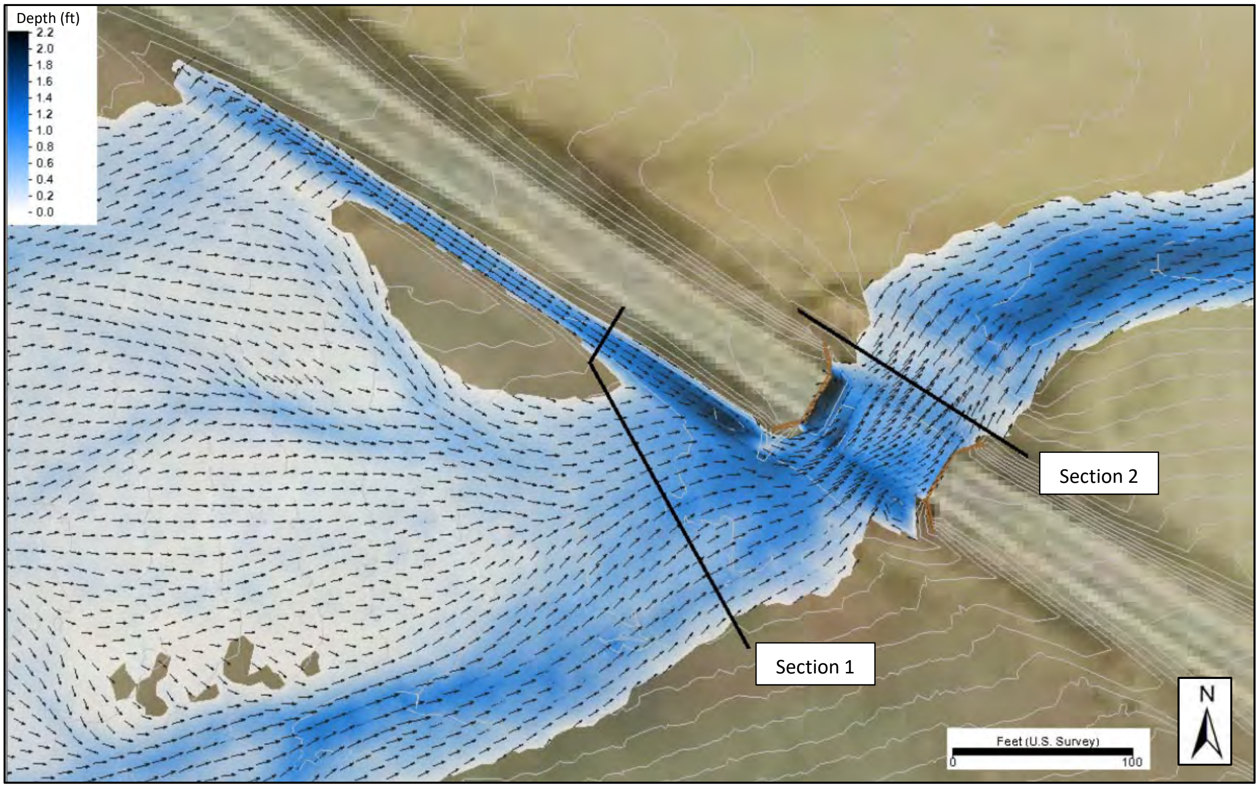

Figure 2. Existing Conditions 100-Year Depth

The ephemeral tributary spanned by Bridge F-19-E is a braided, not well defined, channel upstream of the bridge. The existing conditions modeling indicates that flow from some of the channel braids would be intercepted by the US 36 embankment and then directed to Bridge F-19-E via a roadside ditch. There is minimal eddying observed within the model near Bridge F-19-E due to the relatively large span of the bridge compared to the limited amount of runoff that is being conveyed, even during the 500-year event. Modeling of the existing conditions confirmed that no insurable structures are located within the floodplain for Bridge F-19-E. Modeling indicated that velocities would reach a maximum of approximately 9.7 ft/s and 11.1 ft/s for the 100- and 500-year events, respectively, near the downstream berm and grade drop.

4 DESIGN DISCUSSION

4.1 Replacement Consideration

The existing F-19-E bridge is 89 years old and decades past its design life. Bridge F-19-E requires frequent repair and closure and CDOT has determined that the bridge will be replaced.

4.2 Design Frequency and Floodplain Impacts

Bridge F-19-E is to be designed to meet the 100-year 24-hour storm event per Table 7.2 in the CDOT drainage criteria manual. The existing structure is not within a FEMA mapped floodplain but is within a floodplain as designated by Arapahoe County standards. Therefore, Arapahoe County requires that development not increase the water surface elevation from the base flood elevation by more than one-half foot during the 100-year (1% annual risk) event.

4.3 Design Alternatives and Proposed F-19-E Structure

Various bridge span lengths were reviewed to determine their feasibility as design solutions. These design alternatives included bridges with a centerline-of-bearing to centerline-of-bearing span of 60-feet, 50-feet, and 40-feet. All three configurations met the CDOT freeboard criteria. However, the 40-foot span produced a rise in the upstream floodplain greater than the allowable 0.5-foot rise. Therefore, the 50-foot span bridge was carried forward as the recommended design alternative.

The roadway over the proposed structure consists of two travel lanes with shoulders in both the eastbound and westbound directions. The proposed bridge opening will be centered on the existing abutments and contracted by approximately 15-feet from each of the existing abutment backwalls. The proposed bridge has one 50-foot span from centerline-of-bearing to centerline-of-bearing, with a clear span of 47.5-feet. It is anticipated that the bridge superstructure will have a thickness of 35.1 inches, or 2.92 feet. The low chord elevation will be 5285.23 feet (NAVD-88) for the west abutment and 5285.85 feet (NAVD-88) for the east abutment. The road grade of the proposed structure will be the same as the existing roadway, and the road width will be increased to 43 feet. The proposed bridge will have deep-foundation spill-through abutments with 2:1 (H:V) slopes. The terrain directly upstream of the bridge is expected to remain the same as in the existing conditions. The 50-foot span bridge meets both the CDOT freeboard requirements for the 100-year event as well as the floodplain rise limitations dictated by Arapahoe County.

5 PROPOSED CONDITION HYDRAULICS

5.1 Proposed Conditions Hydraulic Modeling Approach

The proposed conditions model was based off the existing conditions model and adjusted to represent any changes due to project conditions. The proposed bridge opening centerline was held at its existing location, and few modifications to the numerical mesh configuration near the structure were necessary. The obstructions representing the existing bridge bents were removed, and the materials coverage was adjusted to represent minor changes to ground cover at the bridge. Modifications to the terrain at and around the bridge were incorporated into the mesh to represent the bridge narrowing and grading within the channel.

5.2 Proposed Conditions Hydraulic Modeling Results

A summary of the hydraulic properties immediately upstream and downstream of Structure F-19-E for the proposed conditions are presented in **Table 5,** with summary cross section locations illustrated in **Figure 3.**

Table 5. Proposed Conditions Model Results

Recurrence Interval	Section 1 WSEL (ft)	Section 2 WSEL (ft)	Section 1 - Velocity (ft/s)		Section 2 - Velocity	
			Max (ft/s)	Average (ft/s)	Max (ft/s)	Average (ft/s)
10- Year	5283.73	5281.17	2.36	1.80	5.37	2.82
50-Year	5284.02	5281.34	2.79	2.03	7.08	4.02
100-Year	5284.19	5281.49	2.88	1.92	7.70	4.87
500-Year	5284.61	5281.64	3.07	1.77	8.92	5.66

The proposed structure passes the 100-year and 500-year events without experiencing pressure flow through the bridge opening or overtopping of US 36. **Figure 3** presents the 100-year water depth plot at the existing structure.

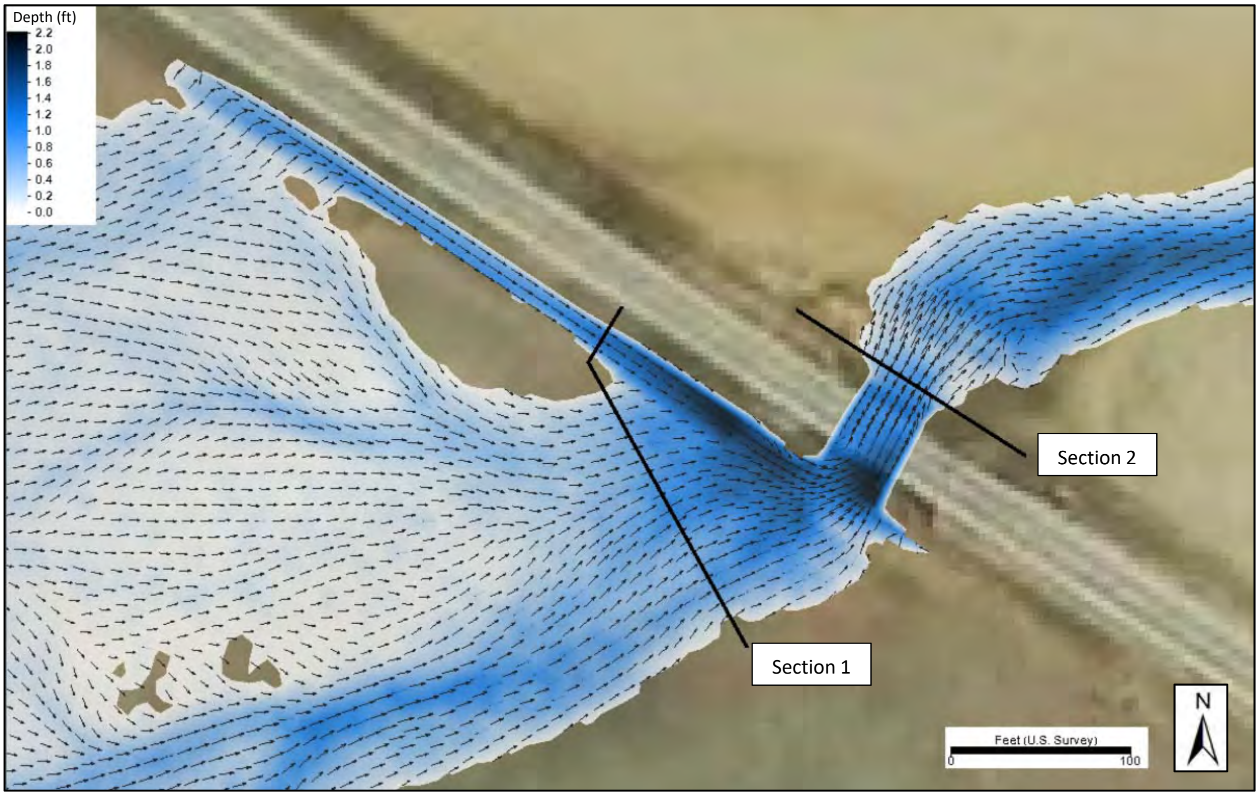

Figure 3. Proposed Conditions 100-Year Depth

In general, flow interacts with the proposed bridge in much the same way as the existing conditions bridge. Some minor eddying is noticed near the eastern abutment both upstream and downstream of the bridge due to the more constricted bridge opening. Velocities near the downstream berm and grade drop increase in magnitude slightly compared to the existing conditions model, with velocities reaching 11.2 and 13.0 ft/s for the 100- and 500-year events, respectively. This slight increase is due to the flow being concentrated near the grade drop in the proposed condition.

5.2.1 Existing Versus Proposed Water Surface Elevations

An analysis was performed using the Dataset Calculator within SMS to ensure no-adverse impacts and improved hydraulic conditions for the proposed condition versus the existing condition. **Figure 4** presents a comparison of the 100-year WSELs around the bridge. Outside of the immediate vicinity of the proposed structure there is no water surface elevation increase greater than 0.5-foot. It was confirmed that no insurable structures were located within the proposed floodplain. In **Figure 4**, increases in WSEL are shown as shades of red, while decreases in WSEL are shown as shades of blue. White indicates WSEL changes less than ±0.01 foot. Notice that the hydraulic impacts of the proposed bridge are limited to the areas immediately upstream and downstream of the bridge, with small areas of increase just upstream of the bridge and downstream of the downstream grade change.

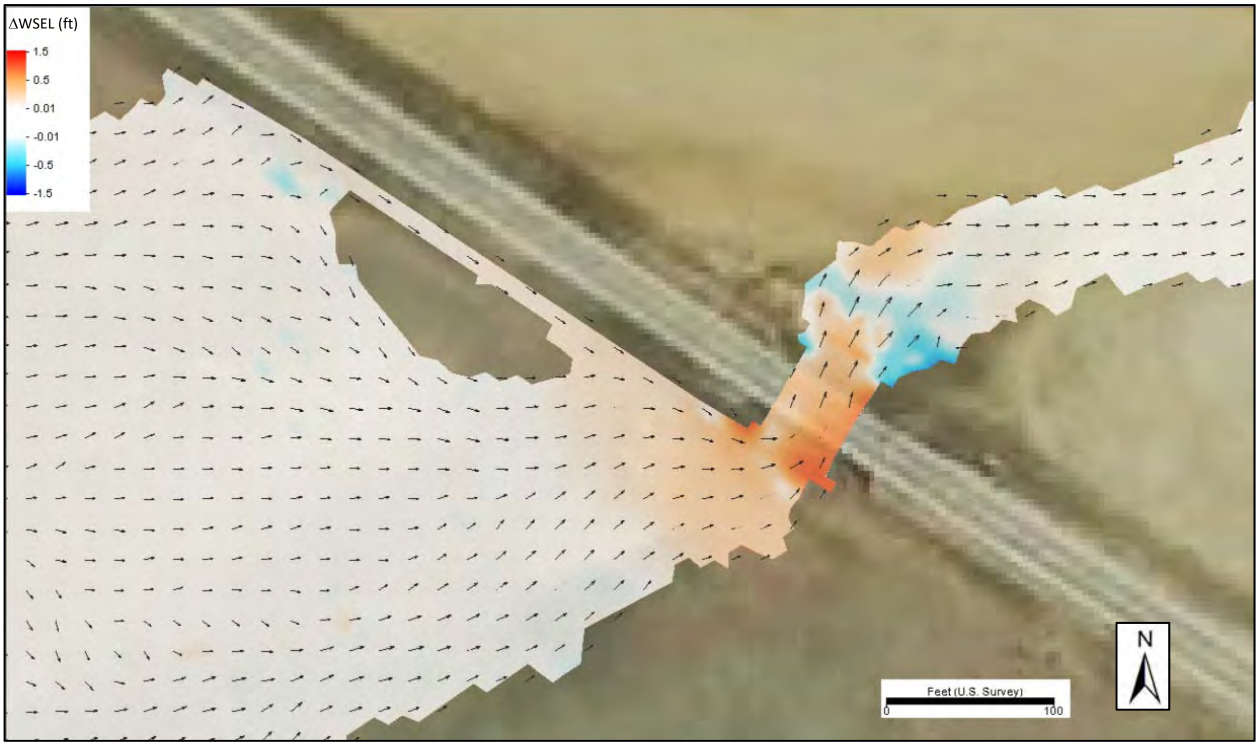

Figure 4. 100-Year Proposed vs Existing Water Surface Change

5.2.2 Proposed Conditions Freeboard Requirements

The minimum required freeboard for the proposed bridge was calculated using the CDOT freeboard equation for low to moderate debris streams,

$$Freeboard = 0.1Q_{design}^{0.3} + 0.008V_{design}^{2}$$

where Q_{design} is the design discharge (cfs) and V_{design} is the mean velocity of the design flow through the bridge (ft/s). In the proposed 100-year condition the flow rate is 240 cfs with a mean velocity of 6.0 ft/s through the bridge opening and a water surface elevation of 5284.19 (NAVD-88) upstream of the bridge. The result of these calculations is a low chord elevation no lower than 5285.00 ft (NAVD-88). The proposed design low chord is anticipated to be 5285.44 at the location of two-thirds along the bridge's length.

5.3 Scour and Countermeasures

5.3.1 Stream Stability

The unnamed stream at Bridge F-19-E flows through a low-plasticity, lean clay channel bed. There is some braiding in the floodplain due to a combination the upstream UPRR embankment culvert discharge locations, undulations of the existing ground, likely due to farming practices over the years, and the rapid constriction of the floodplain at the US 36 embankment near the Bridge.

The stream is vegetated with grasses and weeds on the upstream and downstream ends of the bridge. Aerial imagery from 1953 was explored to determine potential for stream migration throughout the years, but the channel experiences minor flows from grazed prairie lands and wasn't easily defined in the aerial. Olsson and Associates' geomorphic report identified the channel as having no readily apparent evidence of channel instability. There is, however, an approximate 13.4% drop at the downstream right of way limit, approximately 35-feet downstream of Bridge F-19-E. This drop presents the potential to turn into an active headcut if high flows are experienced by the bridge, and some form of grade control is not installed at or near the bridge.

5.3.2 Scour Potential

Scour potential at the proposed structure was analyzed for the 100-year scour design flood and the 500-year scour check flood using the methods described in Hydraulic Engineering Circular Number 18 (HEC-18), *Evaluating Scour at Bridges* fifth edition engineering manual, published by the Federal Highway Administration. The bulk bed sample taken during Olsson and Associates' field visit, with 55% of channel material passing the #200 sieve, indicates that the channel is composed of primarily silt and clay. Due to the small nature of the channel sediment, and the vegetation upstream and downstream of the bridge, scour is expected to be clear-water contraction scour.

5.3.3 Scour Variables

<u>Critical Velocity</u>

Clear-water scour assumes sand-bed channels with no cohesive properties. The bulk sediment sample from the channel; however, shows that the cohesive properties are prevalent and as such the channel will have a higher critical velocity than would otherwise be determined using traditional critical velocity calculations. To account for this, the critical velocity was estimated based on properties of soil using Mirtskhoulava's simplified expression for cohesive sediments (Hoffmans and Verheij, 1997):

$$U_c = \log\left(\frac{8.8h}{d_a}\right)\sqrt{\frac{0.4}{\rho}\left[(\rho_s - \rho)gd_a + 0.6C_f\right]}$$

$$C_f = 0.035C_o$$

Where U_c is the critical depth-averaged velocity, h is equilibrium depth of flow (post-scour), d_a is the detaching aggregate size, C_f is the rupture strength of clay, and C_o is the cohesion of clay. C_o is estimated based on the liquidity index and voids ratio of a soil using Table 2.5 reported in Hoffmans and Verheij's Scour Manual. Because these two parameters are relative unknowns for this soil, conservative estimates were made to estimate C_o. The liquidity index is expected to be well below 1 (between 0-0.25) and a voids ratio of 0.95 was assumed to give slightly more conservative results. Post-scour equilibrium depth of flow was determined iteratively to determine a final estimation of depth-averaged critical velocity. Parameters and results of this analysis are given in **Table 6**.

Table 6. Depth-Averaged Critical Velocity for Cohesive Soils (Mirtskhoulava 1988)

Description	Variable	100-Year	500-Year
Equilibrium depth of flow (m) POST-SCOUR	h	0.3	0.4
Detaching aggregate size (m)	d_a	0.004	0.004
Density of water (kg/m3)	ρ	1,000	1,000
Density of particle solid (kg/m3)	ρ_s	2,650	2,650
Acceleration due to gravity (m/sec2)	g	9.81	9.81
Fatigue rupture strength of clay (N/m2). $C_f = 0.035C_o$	C_f	1.4	1.4
Depth averaged critical velocity (m/sec)	U_c	0.47	0.48
Depth averaged critical velocity (ft/sec)	U_c	1.53	1.56

Time-Rate Contraction Scour

Because the channel has cohesive properties, scour is expected to occur at a much slower rate than it would in a sand-channel bed. To take this into account clear-water contraction scour calculations have been adjusted using Yang's total load sediment transport equation for sand with the limiting value of erosion being the clear-water scour. Hydrographs for both the 100-year and 500-year storm events were used to determine storm duration. Since the 10-year storm flows were sufficient enough to result in velocities that transport bed material this flow-rate is the ideal value at which to take the storm duration measurement; however, to be conservative a slightly larger duration was taken at 50 cfs, see **Figure 5** and **Figure 6**. Time dependent scour for the 100-year event is presented in **Figure 7**.

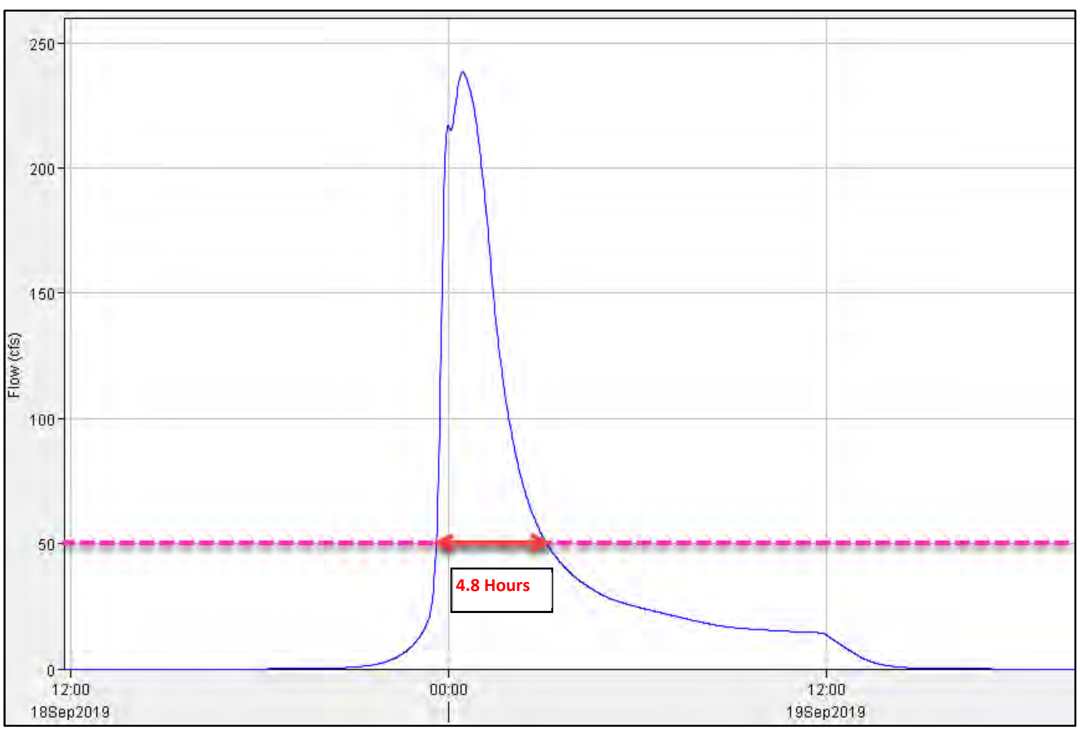

Figure 5. Flood Hydrograph: 100-Year Event

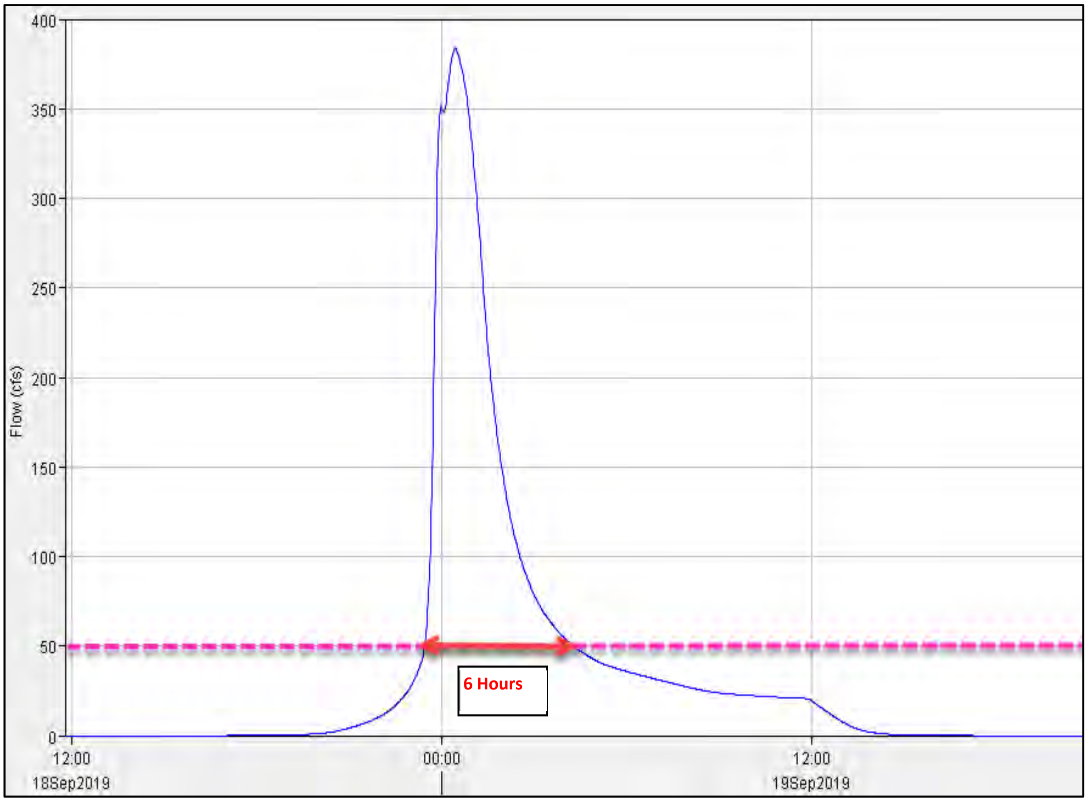

Figure 6. Flood Hydrograph: 500-Year Event

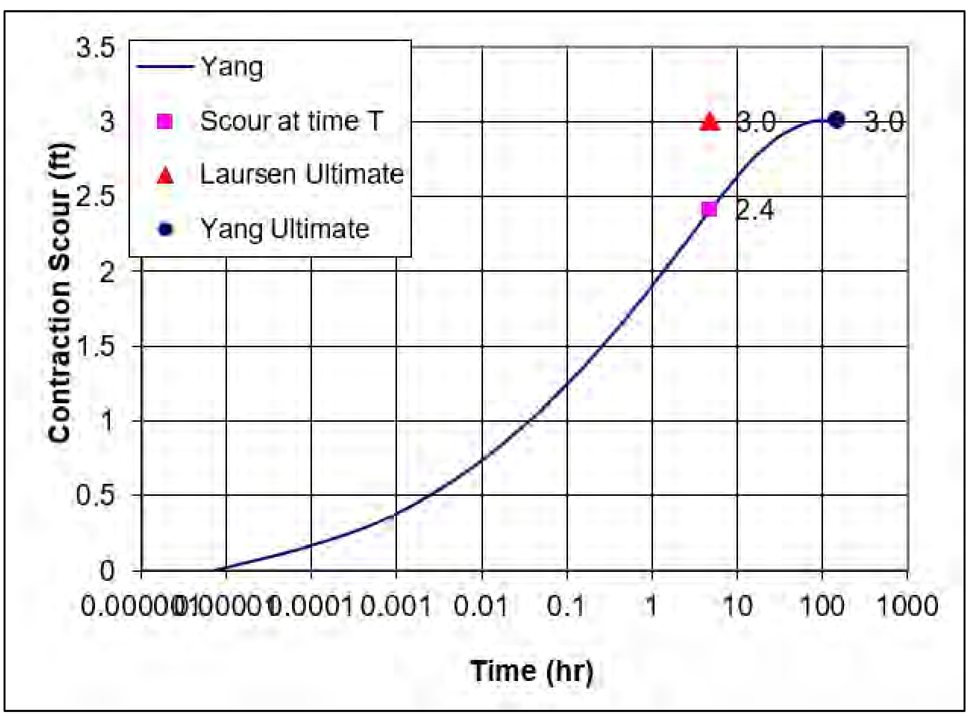

Figure 7. 100-Year Time Dependent Contraction Scour

Clear-Water Abutment Scour

HEC-18, Figure 8.11 (spill-through abutments) was used in conjunction with the unit discharge ratio of the approach and contraction sections to determine the amplification factor to be used in calculating abutment scour for the 100-year and 500-year events. This factor was applied to the predicted average flow depth, post-scour.

Long-Term Degradation

The terrain data captures a significant drop in the channel located approximately 35 feet downstream of the proposed bridge, as can be seen in **Figure 8**. This drop has an approximate 13.4% grade and is likely caused by agricultural practices. While there is not presently an active headcut at this location, there is potential for one to initiate during high flows. The channel slope downstream of the drop is approximately 1.11% and the channel slope upstream of the drop is 1.60%.

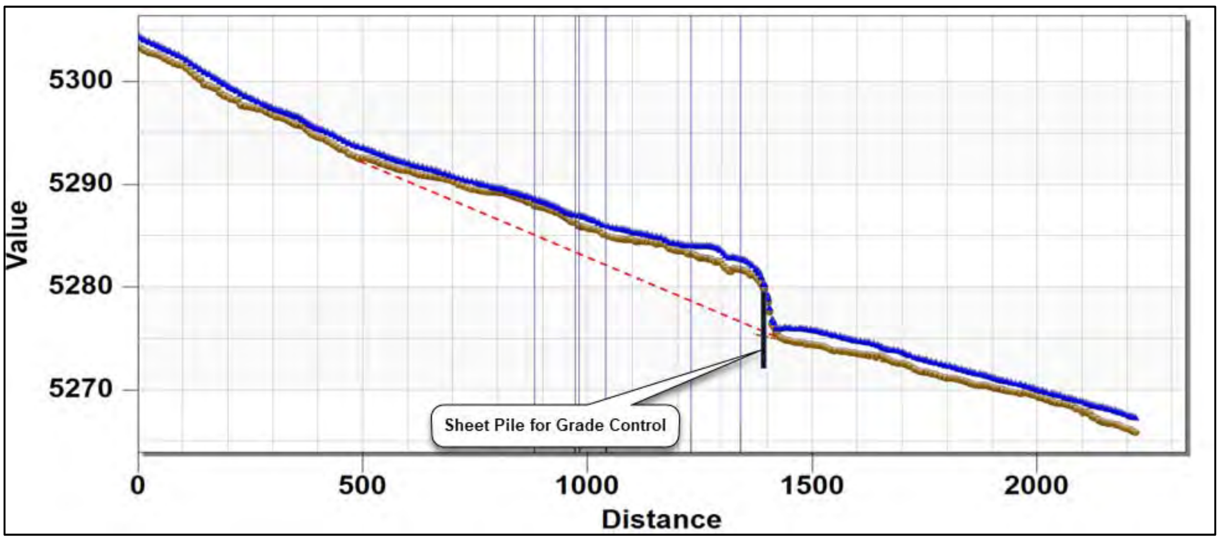

Figure 8. Channel Bed Profile

To determine the long-term degradation that threatens the stream, the slope downstream of the potential headcut would, ideally, be projected upstream until it intersects the thalweg upstream of the potential headcut. However, this situation does not present itself based on ground data upstream of the proposed bridge. **Figure 8** presents a potential scenario for propagation of the potential headcut upstream, which would result in a 1.9% channel slope. The vertical channel degradation that could potentially be experienced at the proposed bridge location is shown in **Table 7**. A potential location for a grade control structure is shown in **Figure 8.**

Table 7. Long Term Degradation

Parameter	Value	Units	Notes
Long-Term Degradation	4.96	ft	
Channel Invert Elevation at Bridge	5281.46	ft	
Long-Term Degradation Channel Invert Elev. at Bridge	5276.50	ft	Assumes 1.9% channel slope from potential headcut location

Scour Summary

A summary of scour depths is provided in **Table 8** and a calculation packet, which includes scour calculations and additional clarifications of variables, has been included in **Appendix F**. Unit discharges for the 100-year event, as well as the location under the bridge where scour calculations were applied, are shown in **Figure 9**.

Table 8. Proposed Condition Scour Results

Recurrence Interval	Flow Rate (cfs)	Critical Velocity (ft/s)	Storm Duration (hrs)	Long-Term Scour	Contraction Scour	Abutment Scour	Long-Term Degradation+ Contraction Scour
100-Year	240	1.53	4.8	4.96	2.40	3.00	7.36
500-Year	385	1.56	6.0	4.96	3.40	4.30	8.36

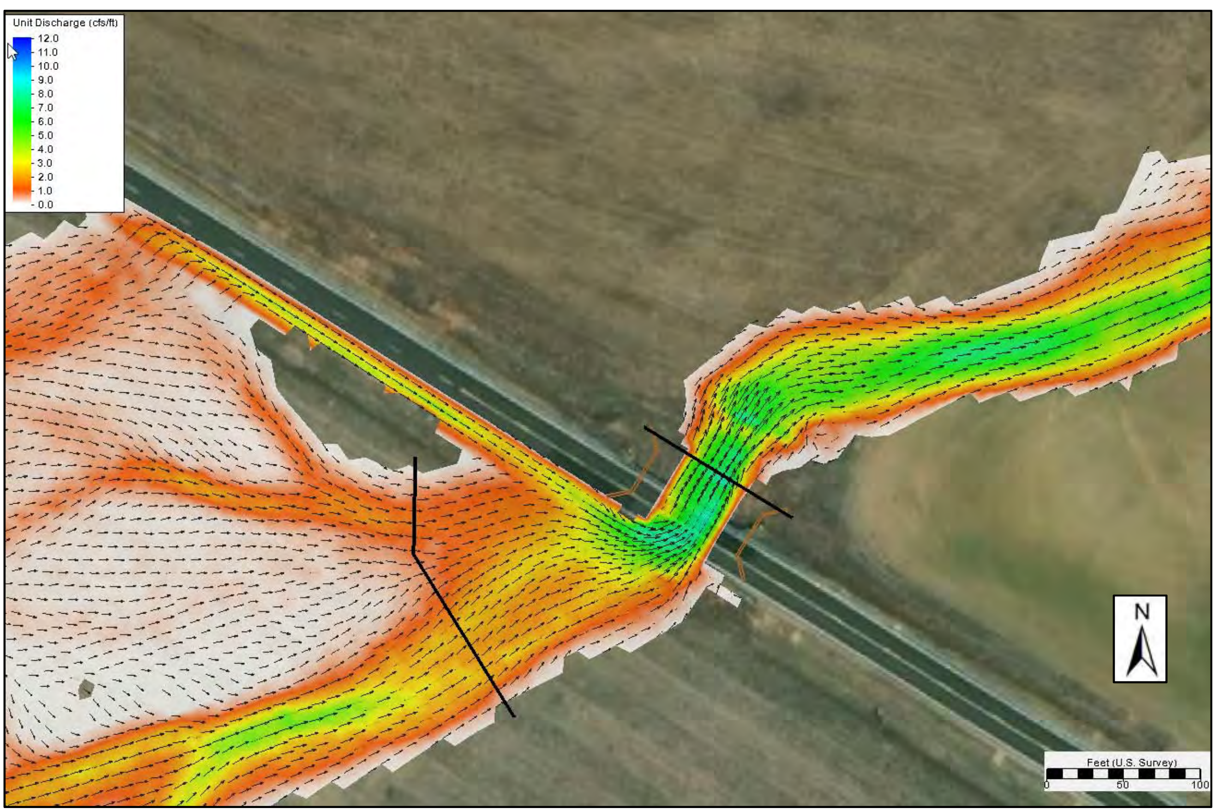

Figure 9. Approach Arc with Unit Discharge

5.3.4 Bridge Scour Countermeasure Design

Abutment and channel armoring countermeasures were designed using the guidance set forth in FHWA Hydraulic Engineering Circular Number 23 (HEC-23), Volume 2, *Bridge Scour and Stream Instability Countermeasures.* The proposed abutments of the structure will be designed to remain stable during the conditions of the scour design (100-year) and scour check (500-year) floods, and the proposed bridge will be protected by abutment rock riprap countermeasures.

The required riprap size for protection against scour was calculated as a D_{50} of 10-inches. However, to conform with standard riprap sizing it is recommended that 12-inch riprap be installed at a minimum. If protection is only anticipated for abutment and contraction scour, and ignoring long term degradation, the riprap section should be installed at a 2-foot depth and extend into the channel for a width of $2Y_0$ or 2-foot along the 100-year contraction scour elevation as presented in **Figure 10**. If no grade control is to be installed downstream of the bridge to control long-term degradation, the 2-foot deep, 12-inch riprap should be extended to the full depth of long-term degradation plus the contraction scour. This protection, to elevation 5274.10, is presented in **Figure 11**.

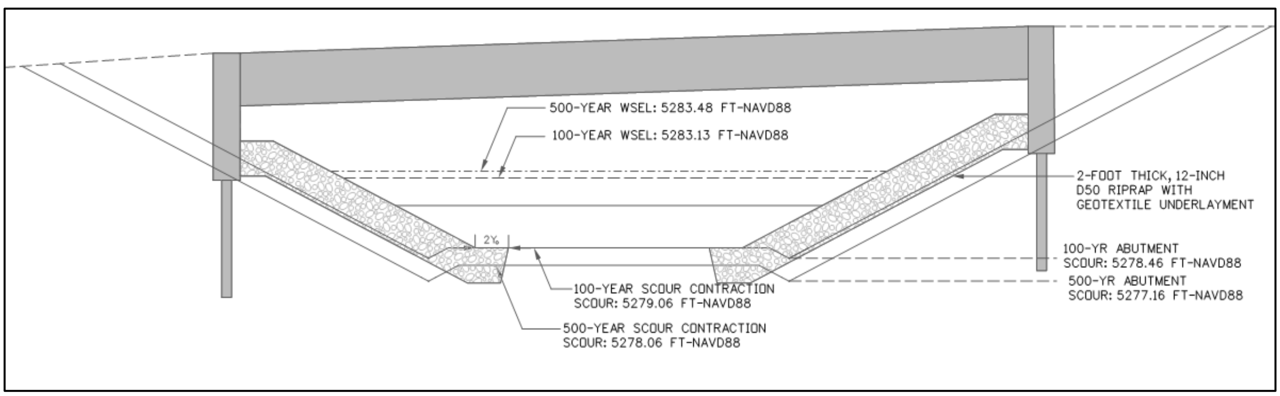

Figure 10. Riprap Armoring for Contraction Scour

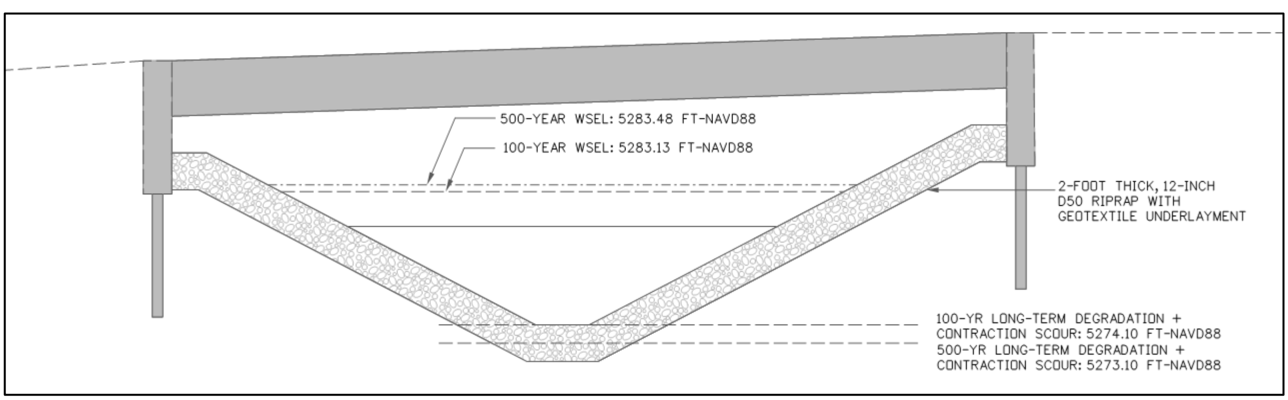

Figure 11. Riprap Armoring for Long-Term Degradation and Contraction Scour

6 CONCLUSION

The existing F-19-E structure along US 36 spanning an unnamed ephemeral channel will be replaced by CDOT. This hydraulic analysis has concluded that a 50-foot, centerline-of-bearing to centerline-of-bearing, single span bridge will be a suitable replacement. This replacement will cause no WSEL increase in excess of the maximum 0.5-foot allowed for unmapped floodplains within Arapahoe County, nor any adverse impact during the 100-year flood event to insurable structures. A scour analysis has been performed for the 100- and 500-year floods producing scour elevations of 5274.1 ft and 5273.1 ft (NAVD-88), respectively. It is understood that the proposed abutments of the structure will be designed to remain stable during the conditions of the scour design (100-year) and scour check (500-year) floods. A countermeasure design has been performed to protect embankment fill during the 100-year flood event and recommends a 2-foot thick section of 12-inch rock riprap, with the riprap extents determined based on long-term channel degradation mitigation.

7 REFERENCES

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Appendix A

Site Photographs

Photograph 1. Upstream Embankment Looking Northwest

Photograph 2. Downstream Embankment Looking Northwest

Photograph 3. Upstream Embankment Looking Southeast

Photograph 4. Downstream Embankment Looking Southeast

Photograph 5. Bridge Deck Looking Northwest

Photograph 6. Bridge Deck Looking Southeast

Photograph 7. Upstream Bridge Face Looking Northeast

Photograph 8. Downstream Bridge Face Looking Southwest

Photograph 9. Upstream Bridge Face Looking East

Photograph 10. Upstream Bridge Face Looking North

Photograph 11. Downstream Bridge Face Looking South

Photograph 12. Downstream Bridge Face Looking West

Photograph 13. East Abutment Backwall (Downstream) Looking South

Photograph 14. East Abutment Backwall (Upstream) Looking East

Photograph 15. West Abutment Backwall (Downstream) Looking West

Photograph 16. West Abutment Backwall (Upstream) and Excavation Looking North

Photograph 17. Bridge Stringers and Bents Looking Northwest

Photograph 18. Helper Bents Looking Northwest

Photograph 19. Typical Pier Cap and Pier

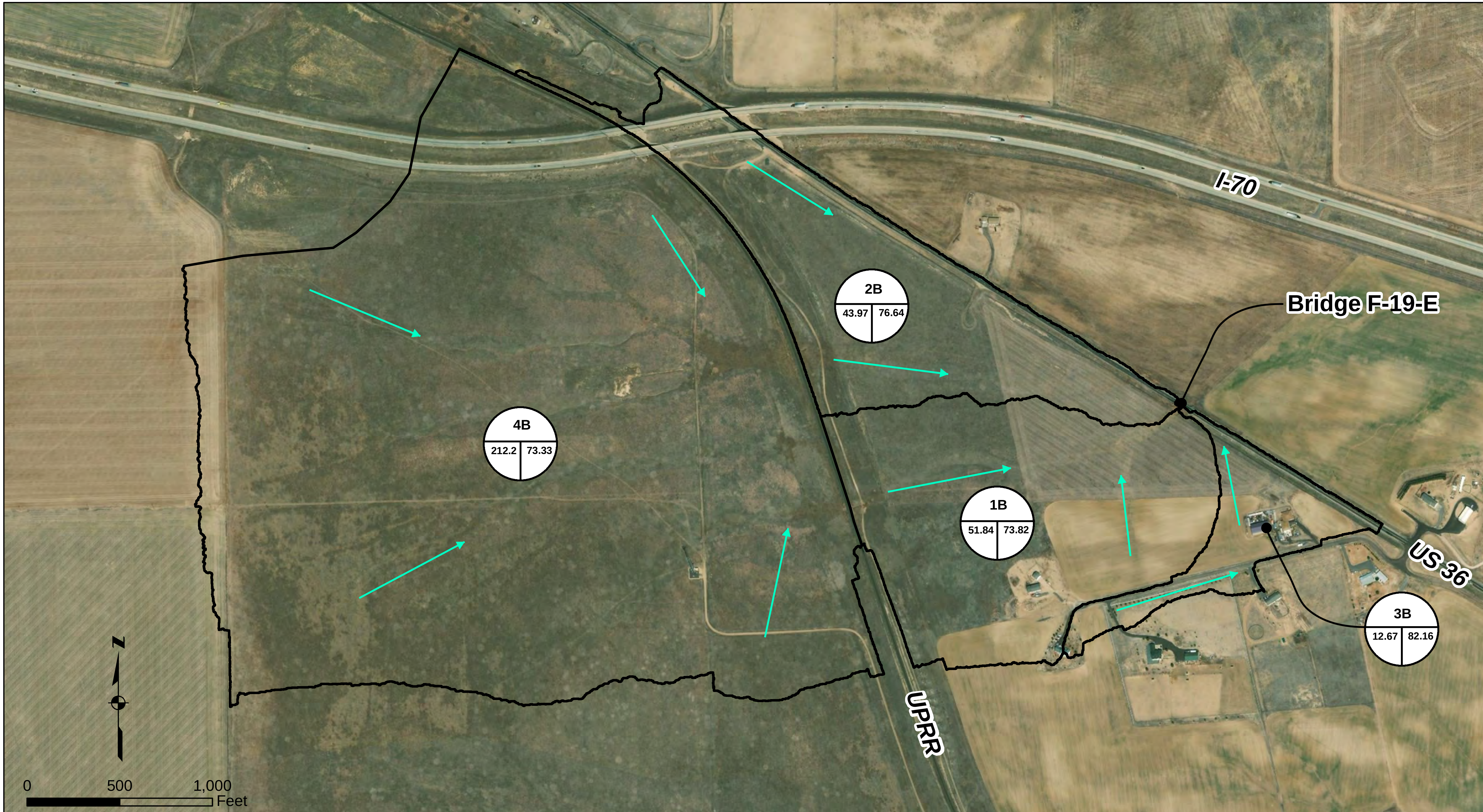
Photograph 20. Downstream Berm and Grade Drop Looking Northeast

Photograph 21. Upstream UPRR Embankment and Floodplain Looking West

Photograph 22. Downstream Floodplain Looking Northeast

Appendix B

Basin Maps and Hydrology Calculation Summary



LEGEND

Drainage Basin

Flow Direction

BASIN ID
AREA (AC)
CN

Drainage Basin Label

As Constructed	DRAINAGE BASIN MAP BRIDGE F-19-E EASTERN PLAINS I-70 CORRIDOR BRIDGES			Project No. / Code
No Revisions:				
Revised:	Designer: MAG	Structure Numbers		23010
	Detailer: MAG			
Void:	Sheet Subset:	Subset Sheets: 1 of 1		Sheet Number

4 BASINS

Sheet Flow

$$T_T = \frac{0.007 * (nL)^{0.8}}{P_2^{0.5} * S^{0.4}}$$

Where:
T_T = Travel time (hr)
n = Mannings roughness coefficient (Table 3-1, TR-55 manual)
L = Flow length
P_2 = 2 yr, 24-hr rainfall (in)
s = Slope of hydraulic grade line (est. as land slope, ft/ft)

Sub-Basin ID	Flowpath Length (ft)	Manning n	Slope (ft/ft)	P_2 (in)	T_T (hr)	
1B	300	0.24	0.019	1.93	0.7528075	
2B					0	***No sheet flow present***
3B	300	0.24	0.025	1.93	0.6745426	
4B	300	0.24	0.009	1.93	1.0150522	

Shallow Concentrated Flow

$$V = 16.1345\sqrt{S} \qquad \text{(Unpaved)}$$

$$V = 20.3282\sqrt{S} \qquad \text{(Paved)}$$

where:
V = average velocity (ft/s)
S = slope of hydraulic grade line (watercourse slope, ft/ft)

$$T_T = \frac{L}{3600 * V}$$

Sub-Basin ID	Flowpath Length (ft)	Slope (ft/ft)	Velocity (ft/s)	T_T (hr)	
1B	600	0.019	2.223987	0.07494	
2B	1400	0.02	2.281763	0.170434	
3B	0	0	0	0	***No shallow concentrated flow***
4B	3050	0.02	2.281763	0.371302	

Channel Flow

$$V = \frac{1.49r^{\frac{2}{3}}\sqrt{S}}{n}$$

Sub-Basin ID	Typical Reach Bottom Width (ft)	Typical Reach Side Slope z	Typical Reach Depth, ft	Cross-Sectional Flow Area (ft^2)	Wetted Perimeter (ft)	Hydraulic Radius (ft)	Manning n	Slope (ft/ft)	Velocity (ft/s)	Open Channel Length (ft)	T_T (hr)
1B	80	45	1	125	170.02222	0.735198	0.048	0.019	3.485444	985	0.078501
2B	85	40	0.75	86.25	145.01875	0.594751	0.048	0.02	3.104679	2000	0.178941
3B	24	2	2	56	32.944272	1.69984	0.048	0.011	4.637103	2680	0.160541
4B	40	30	0.5	27.5	70.016662	0.392764	0.048	0.001	0.526461	790	0.41683

Lag Time

$$T_L(hr) = 0.6 * T_C$$

Sub-Basin ID	T_C (hr)	T_L (hr)
1B	0.906249	0.5437494
2B	0.349375	0.209625
3B	0.835083	0.5010501
4B	1.803184	1.0819101

Muskingum-Cunge Flow Routing

No routing performed for this watershed

Reach	Reach Length (ft)	Energy Slope (ft/ft)	Bottom Width (ft)	Side Slope z	Manning n

RESULTS

Recurrence Flood	Qpeak (cfs)
10 yr	85
50 yr	185
100 yr	240
500 yr	385

AREA

0.5 mi^2

National Flood Hazard Layer FIRMette

FEMA

39°43'45.50"N

104°16'48.86"W

104°16'11.41"W

39°43'17.83"N

Feet 1:6,000

0 250 500 1,000 1,500 2,000

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- B 20.2 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- 8 Coastal Transect
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 1/15/2020 at 4:26:22 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

Appendix C

Existing Conditions Hydraulic Modeling Results

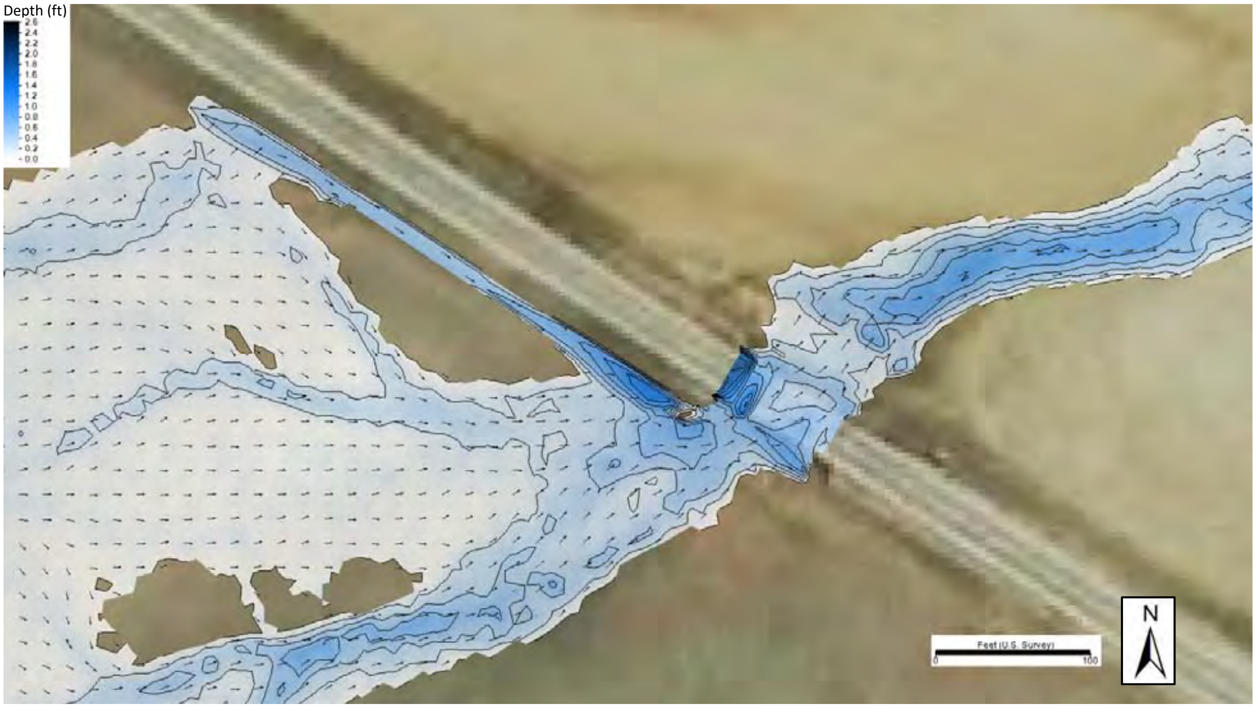


Figure 1. Existing Conditions Depth, 10-Year

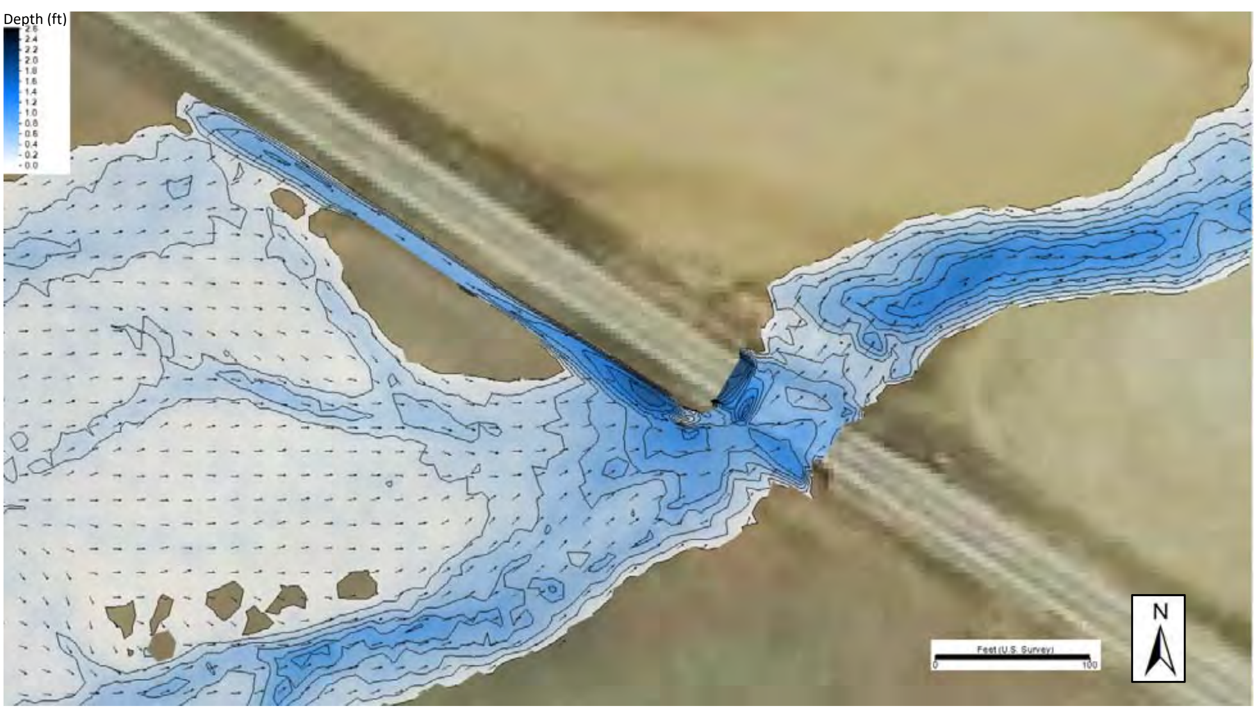


Figure 2. Existing Conditions Depth, 50-Year

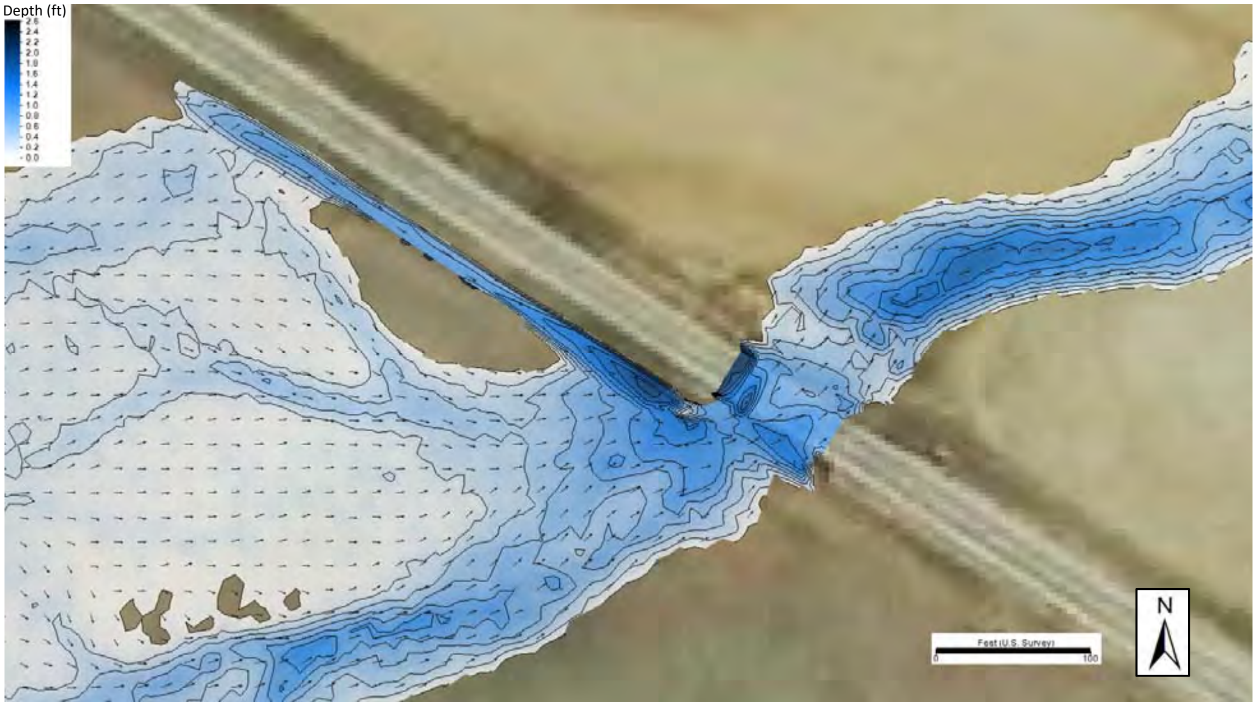


Figure 3. Existing Conditions Depth, 100-Year

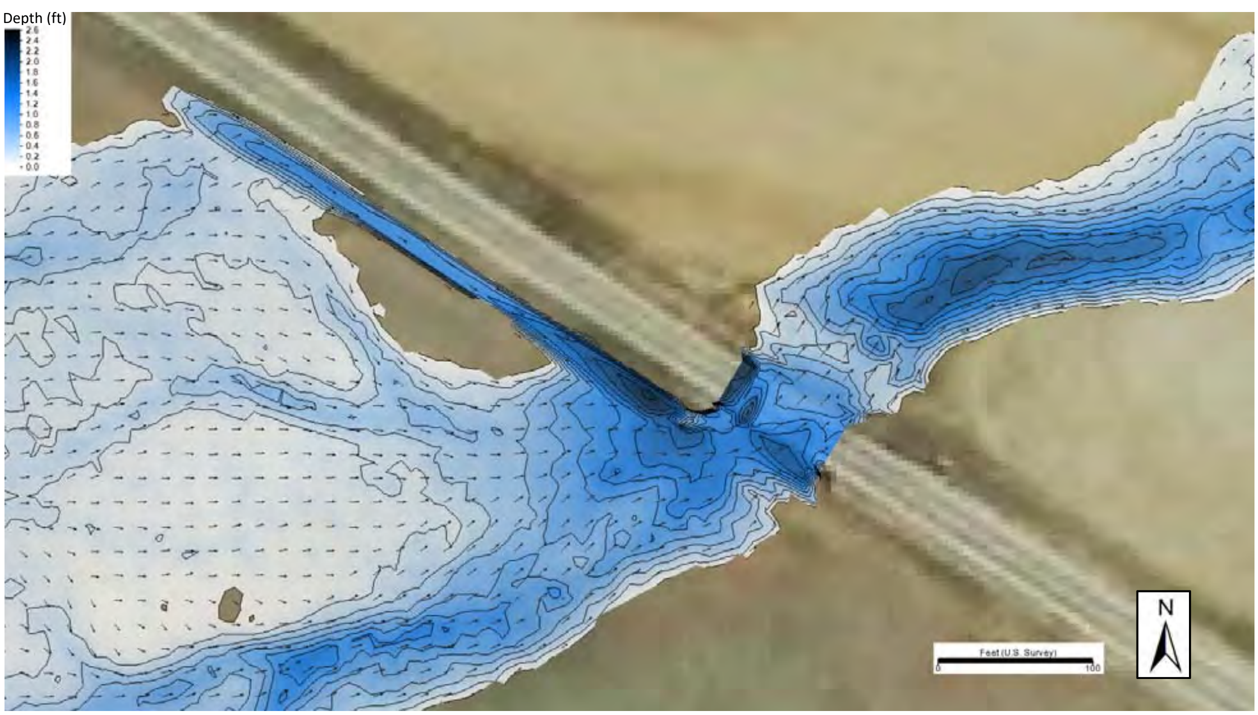


Figure 4. Existing Conditions Depth, 500-Year

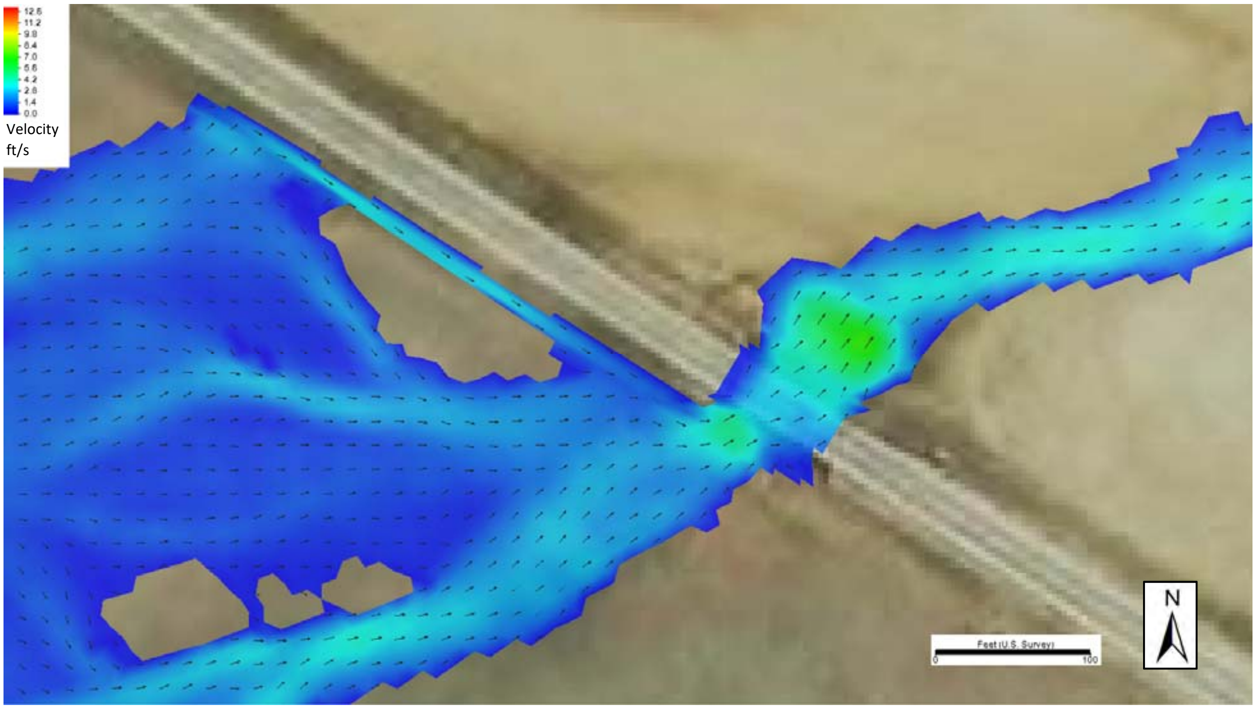


Figure 5. Existing Conditions Velocity, 10-Year

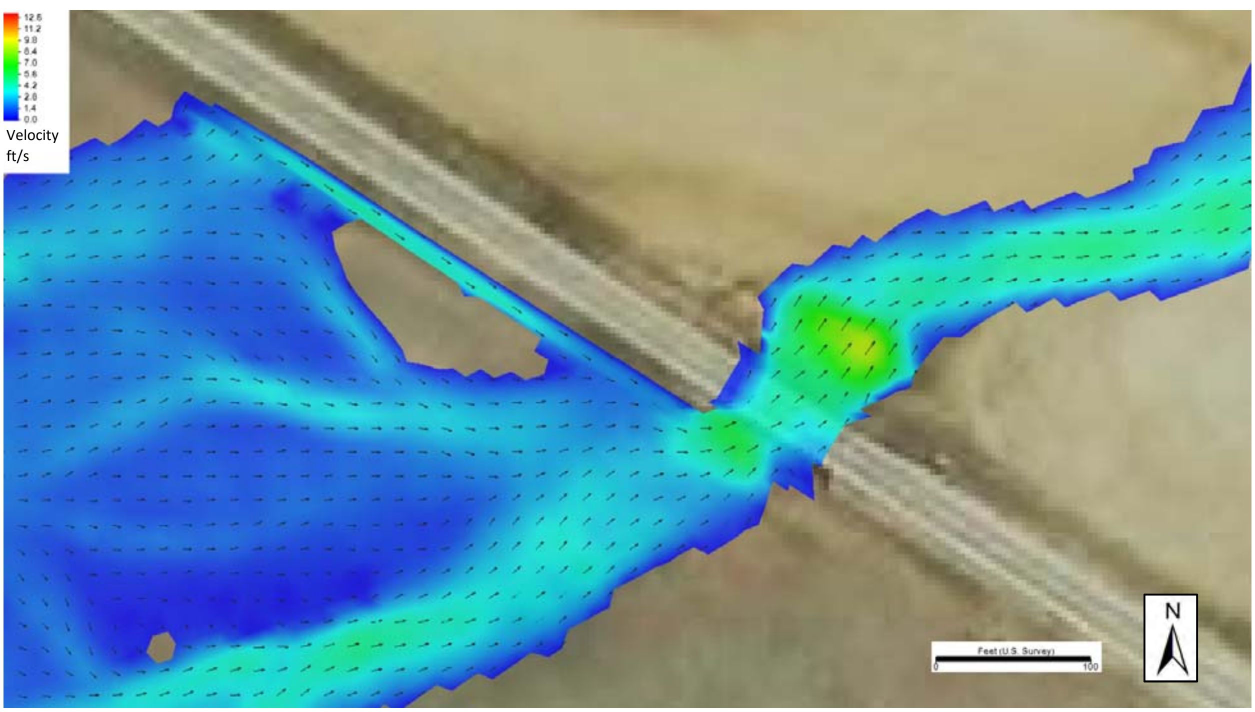


Figure 6. Existing Conditions Velocity, 50-Year

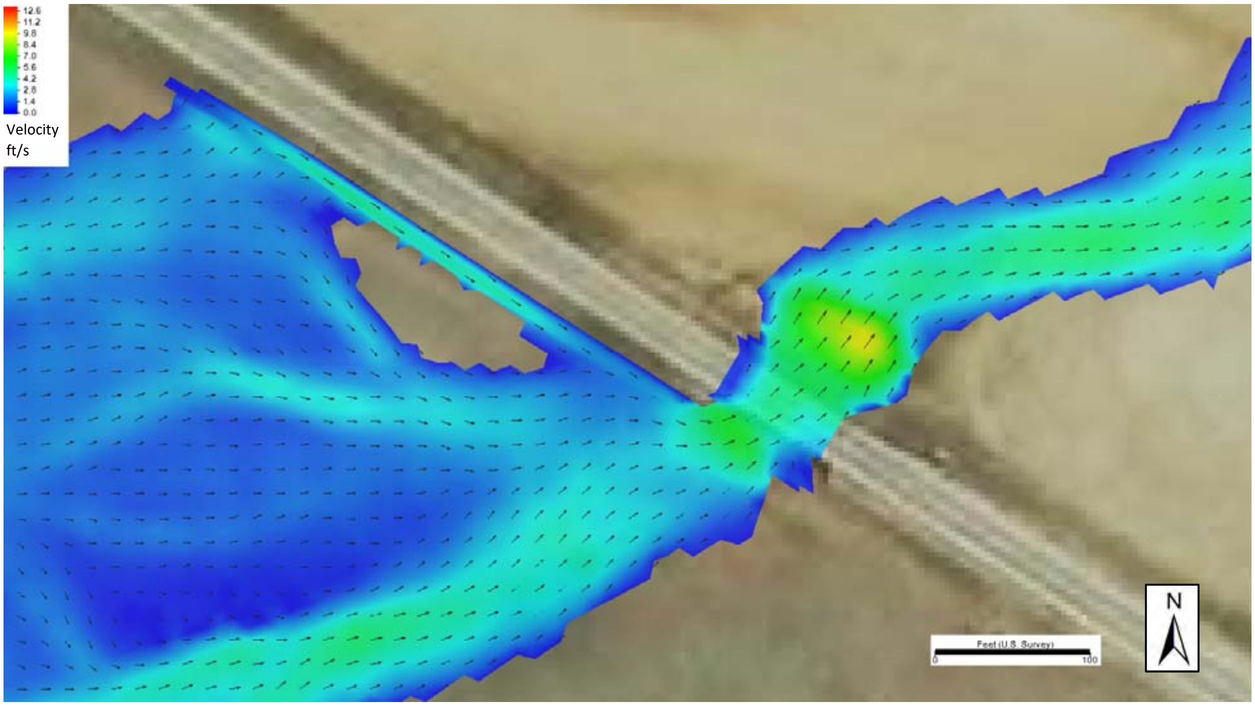


Figure 7. Existing Conditions Velocity, 100-Year

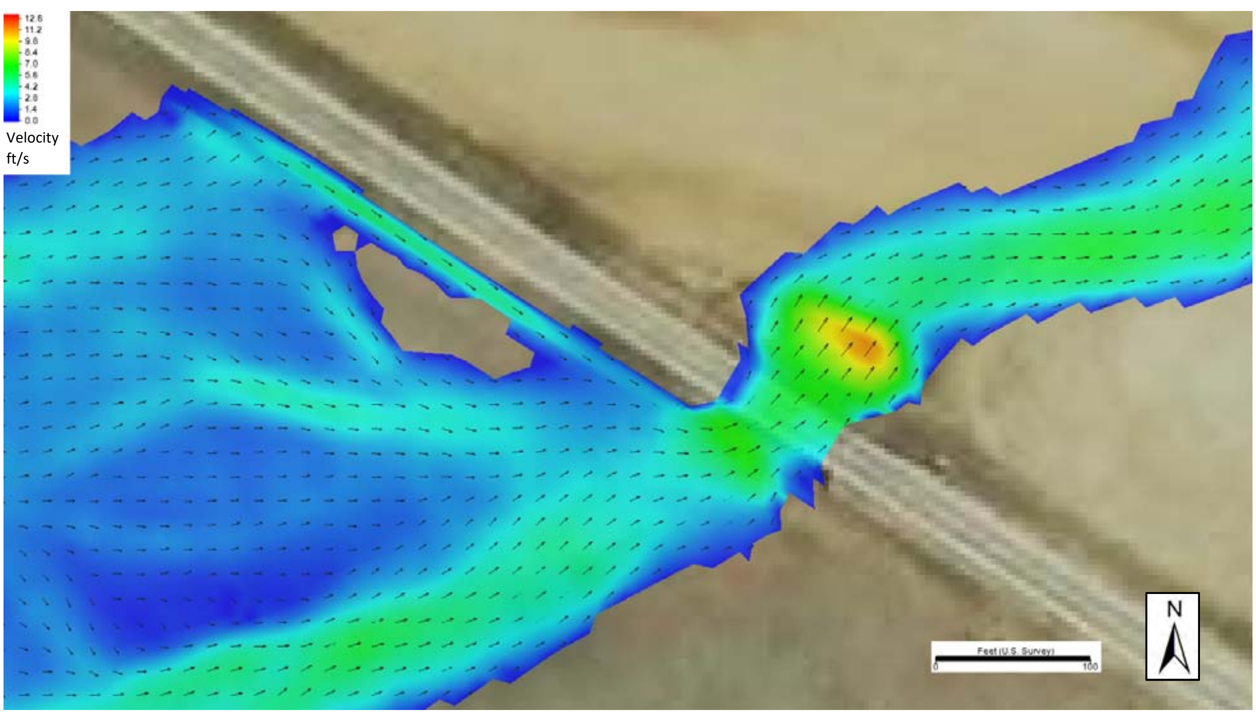


Figure 8. Existing Conditions Velocity, 500-Year

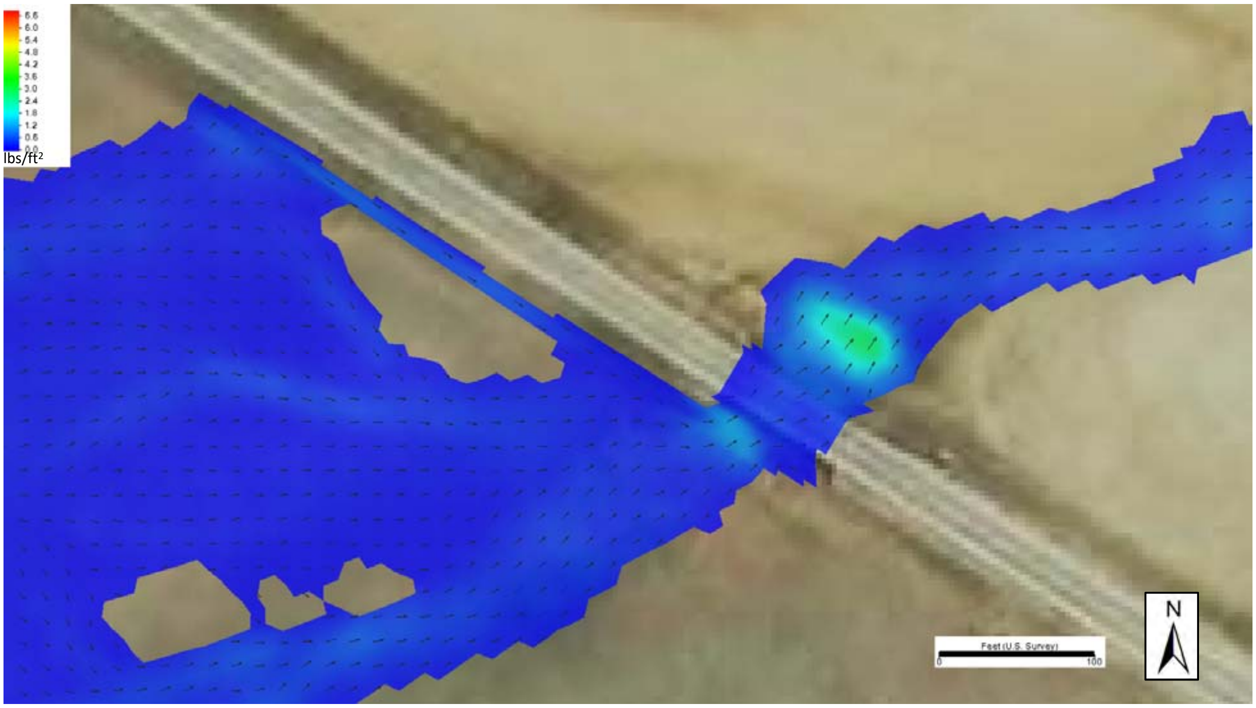


Figure 9. Existing Conditions Bed Shear Stress, 10-Year

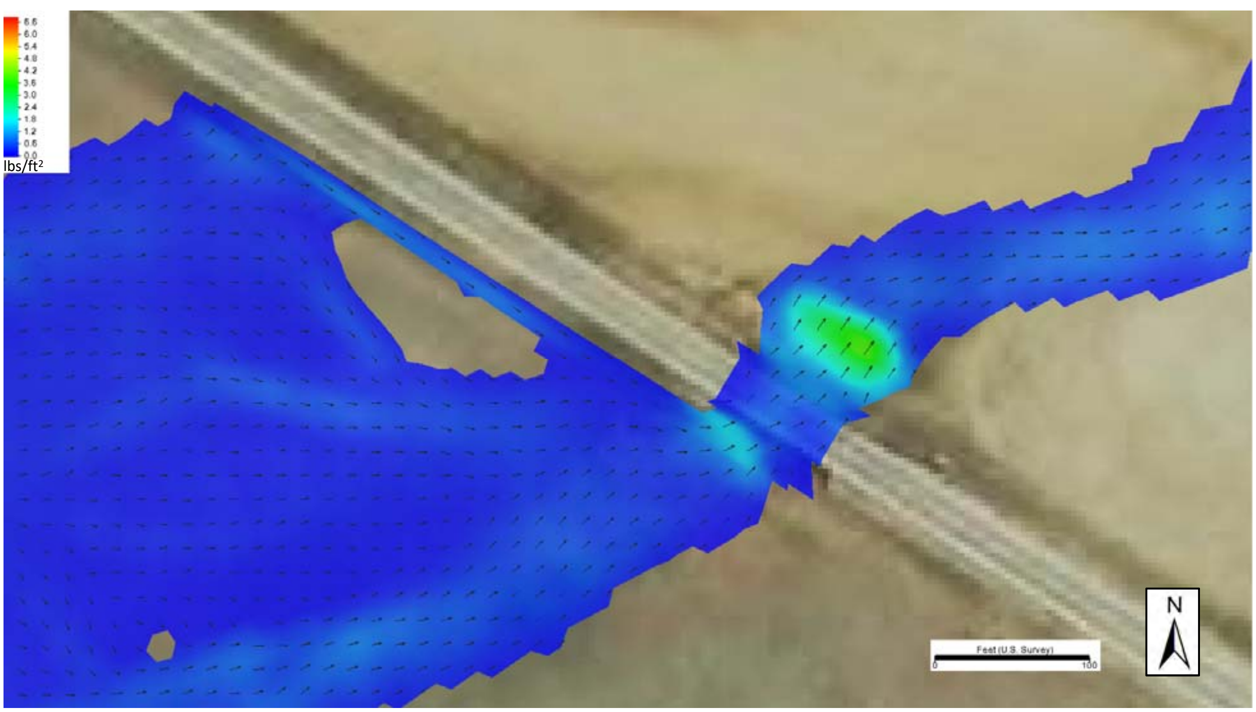


Figure 10. Existing Conditions Bed Shear Stress, 50-Year

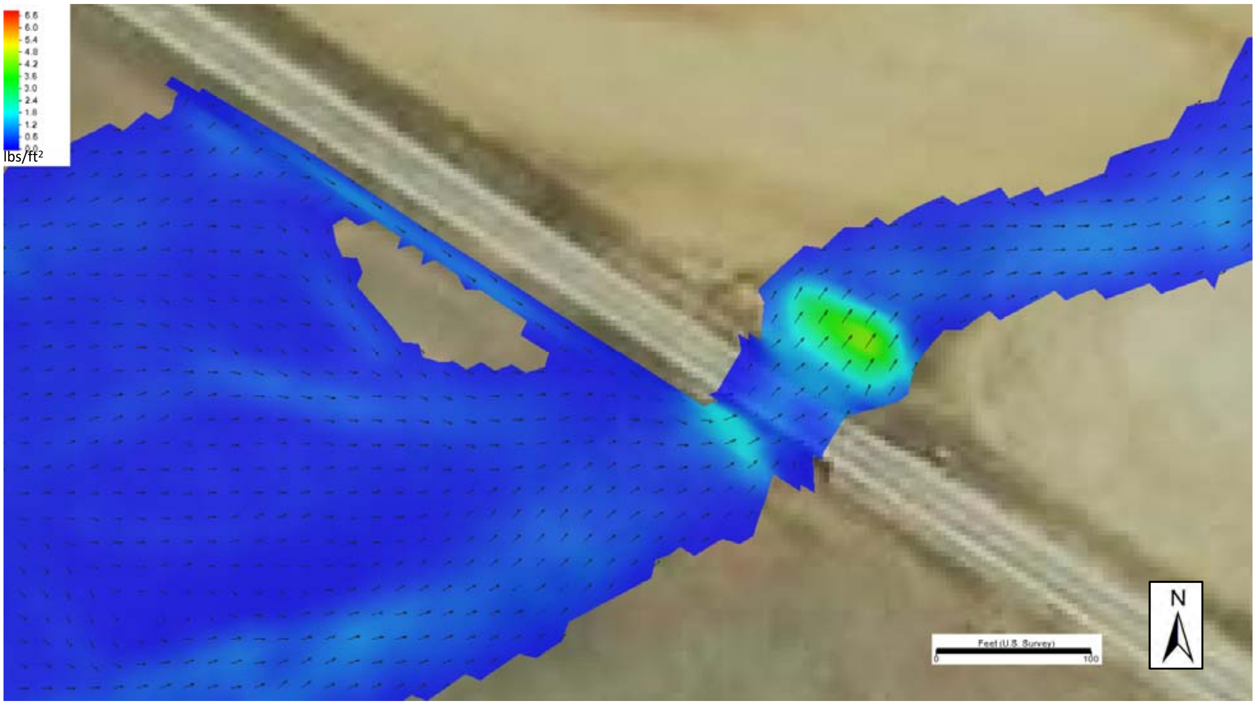


Figure 11. Existing Conditions Bed Shear Stress, 100-Year

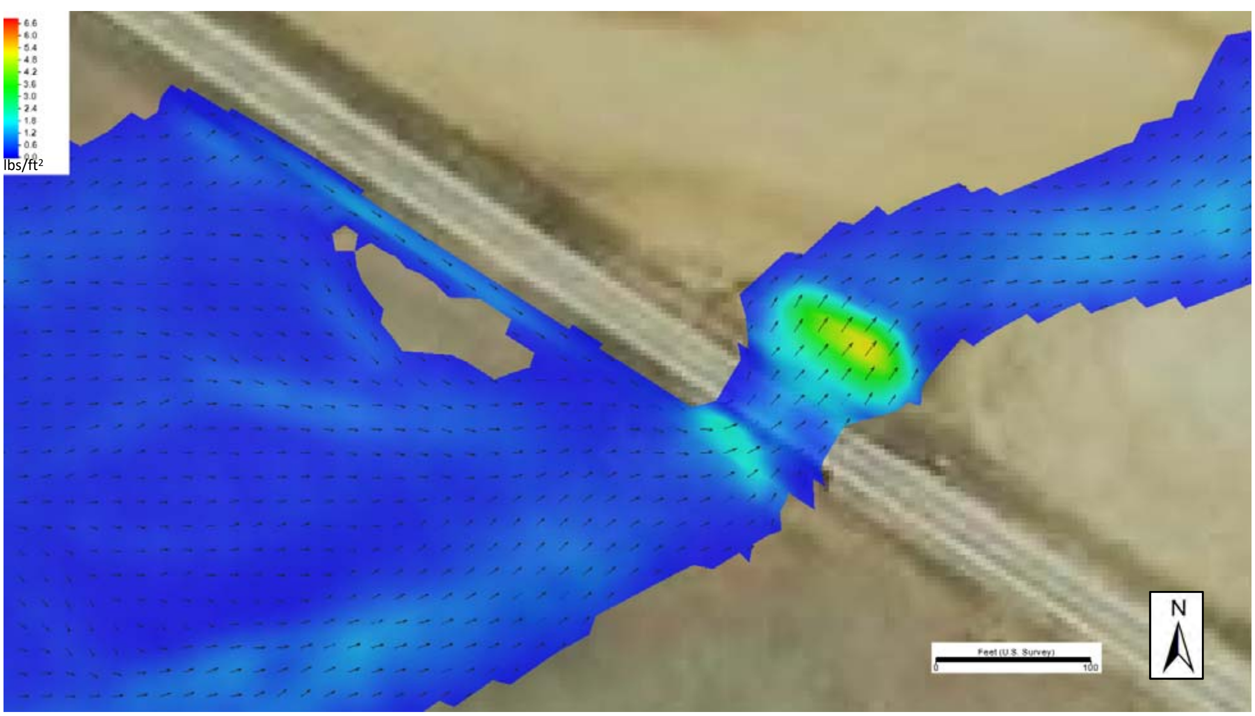


Figure 12. Existing Conditions Bed Shear Stress, 500-Year

Appendix D

Proposed Conditions Hydraulic Modeling Results

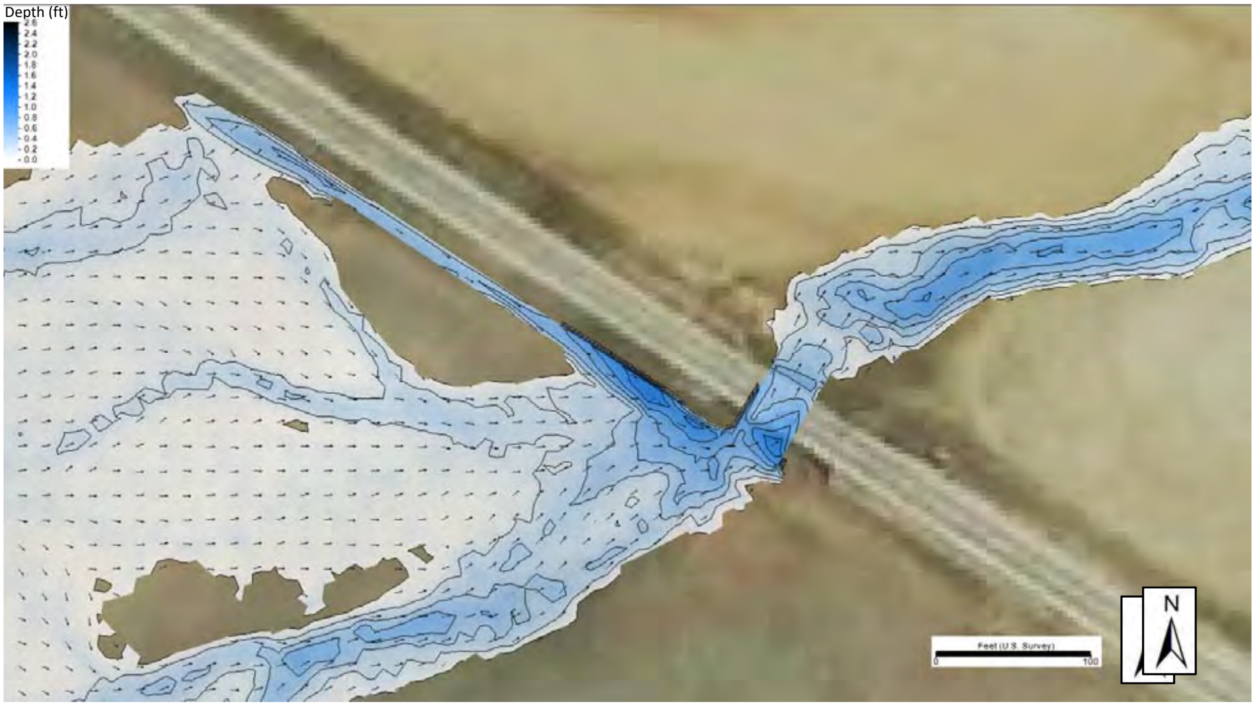


Figure 1. Proposed Conditions Depth, 10-Year

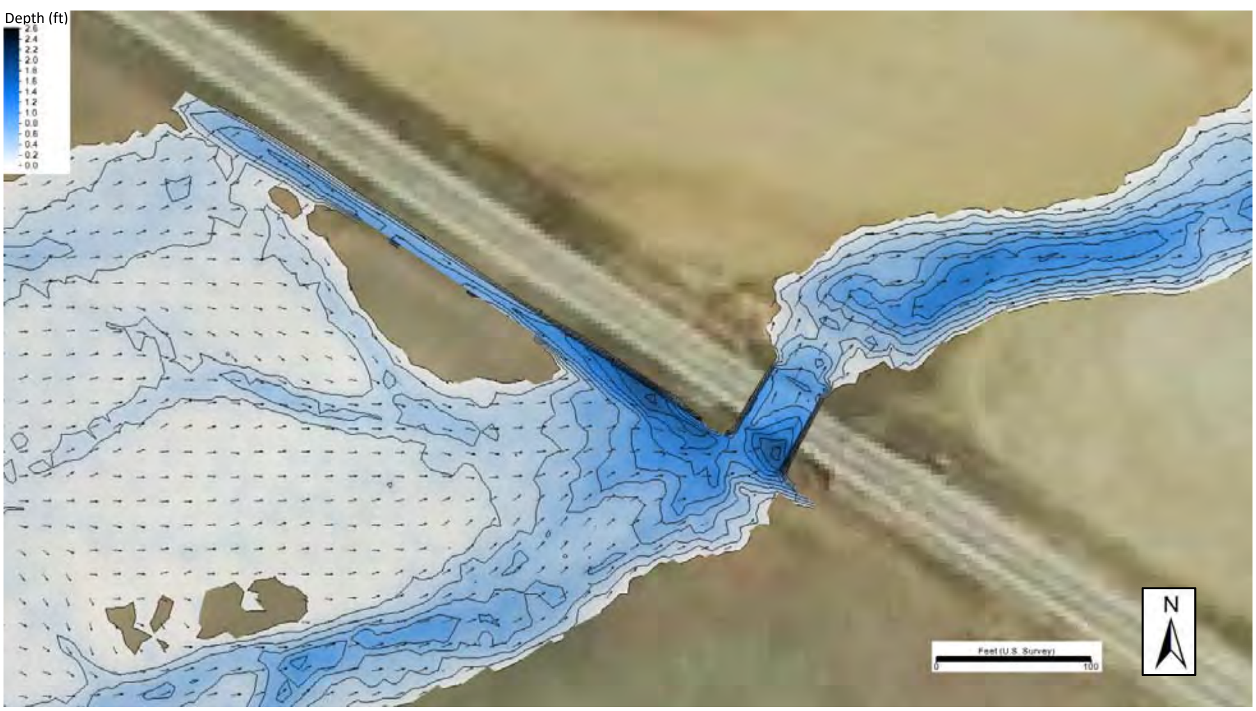


Figure 2. Proposed Conditions Depth, 50-Year

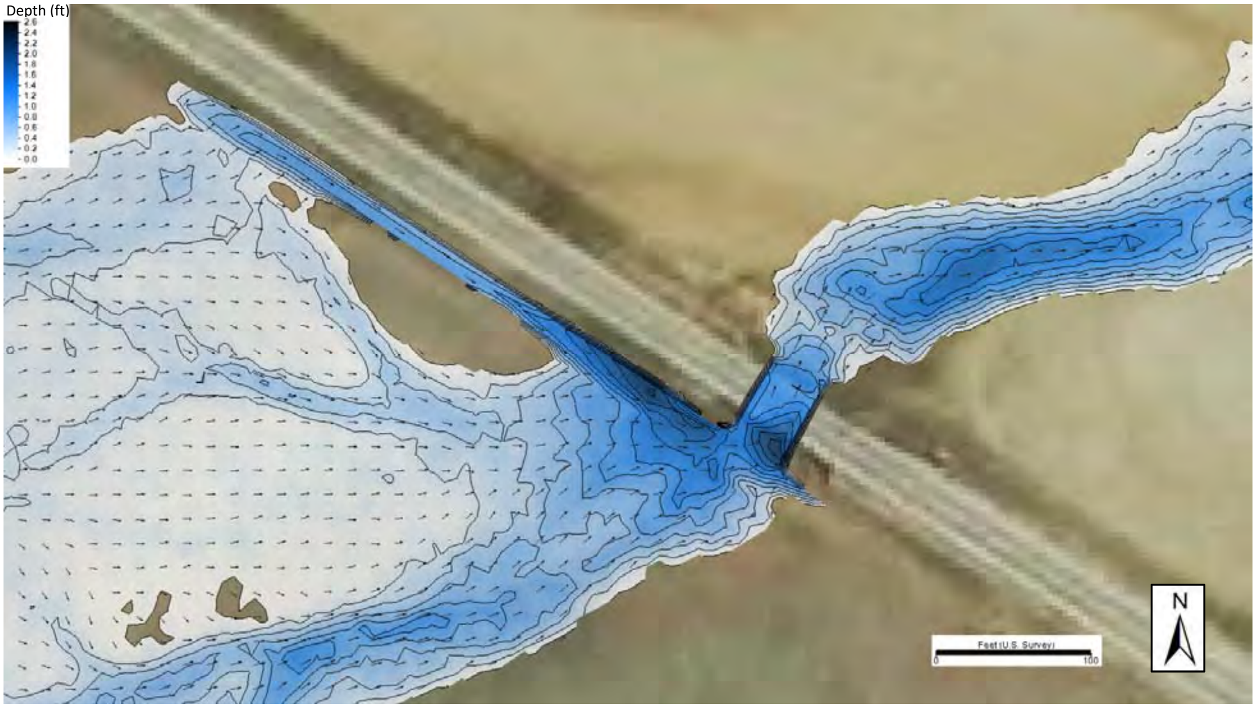


Figure 3. Proposed Conditions Depth, 100-Year

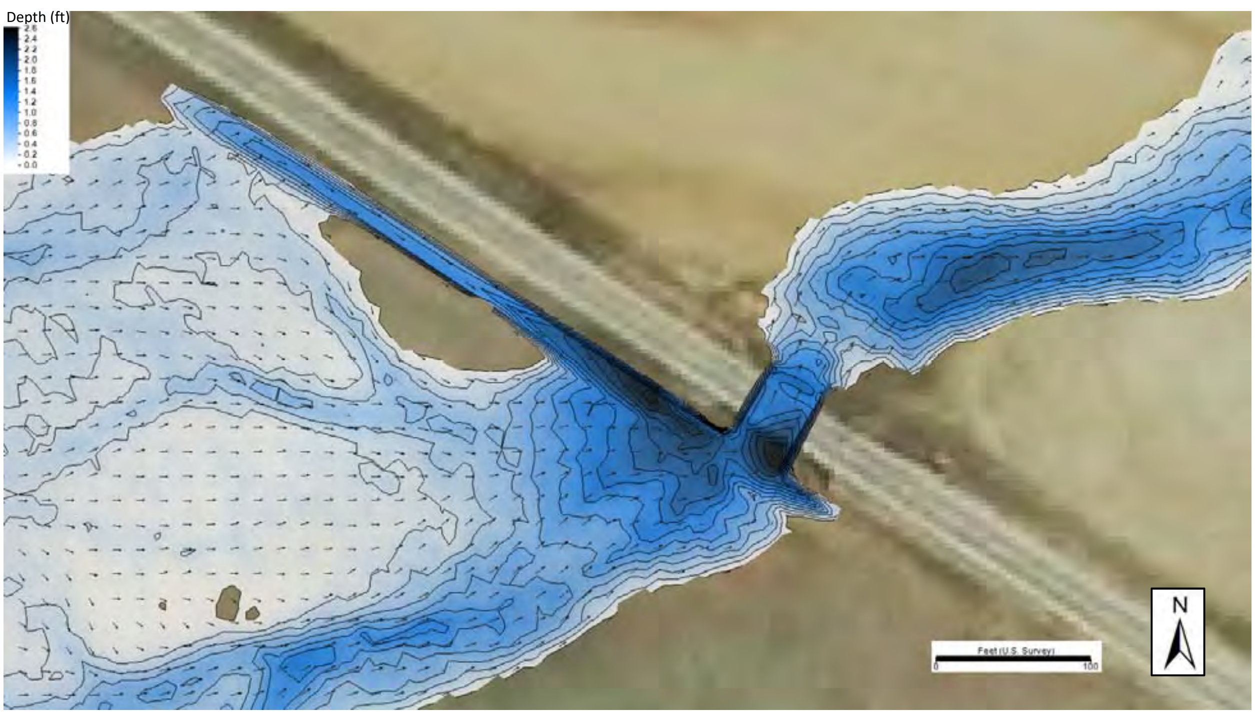


Figure 4. Proposed Conditions Depth, 500-Year

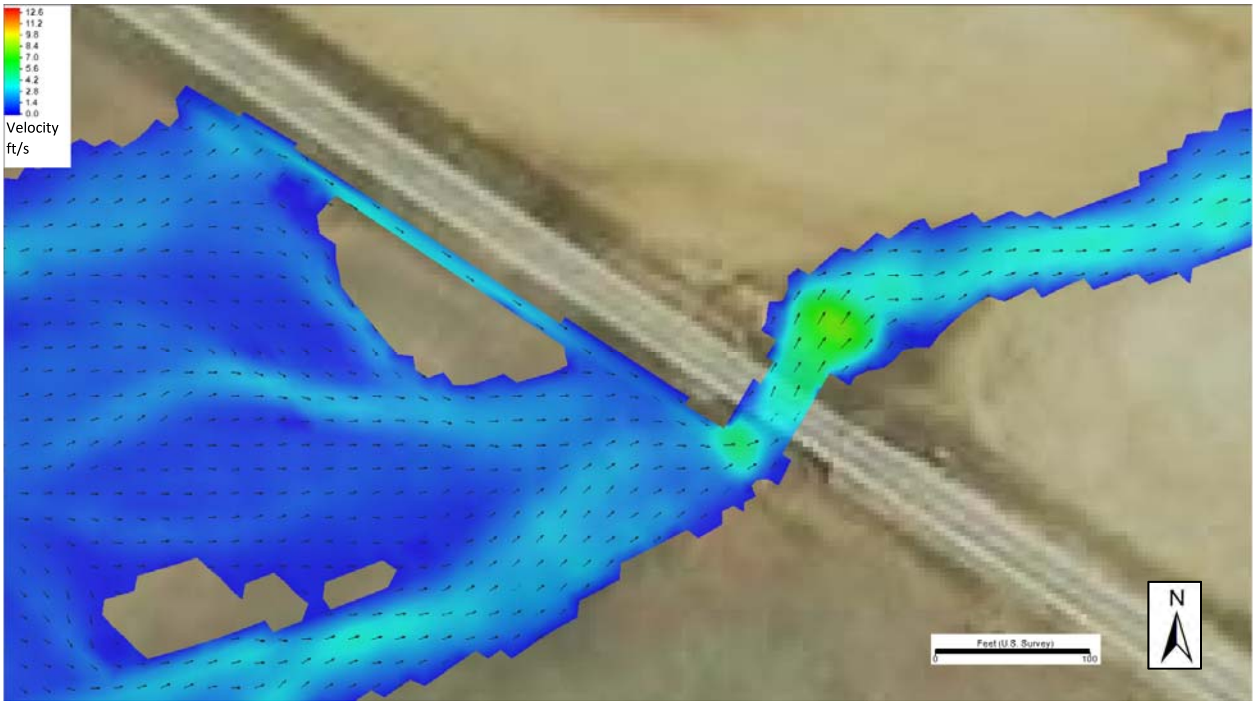


Figure 5. Proposed Conditions Velocity, 10-Year

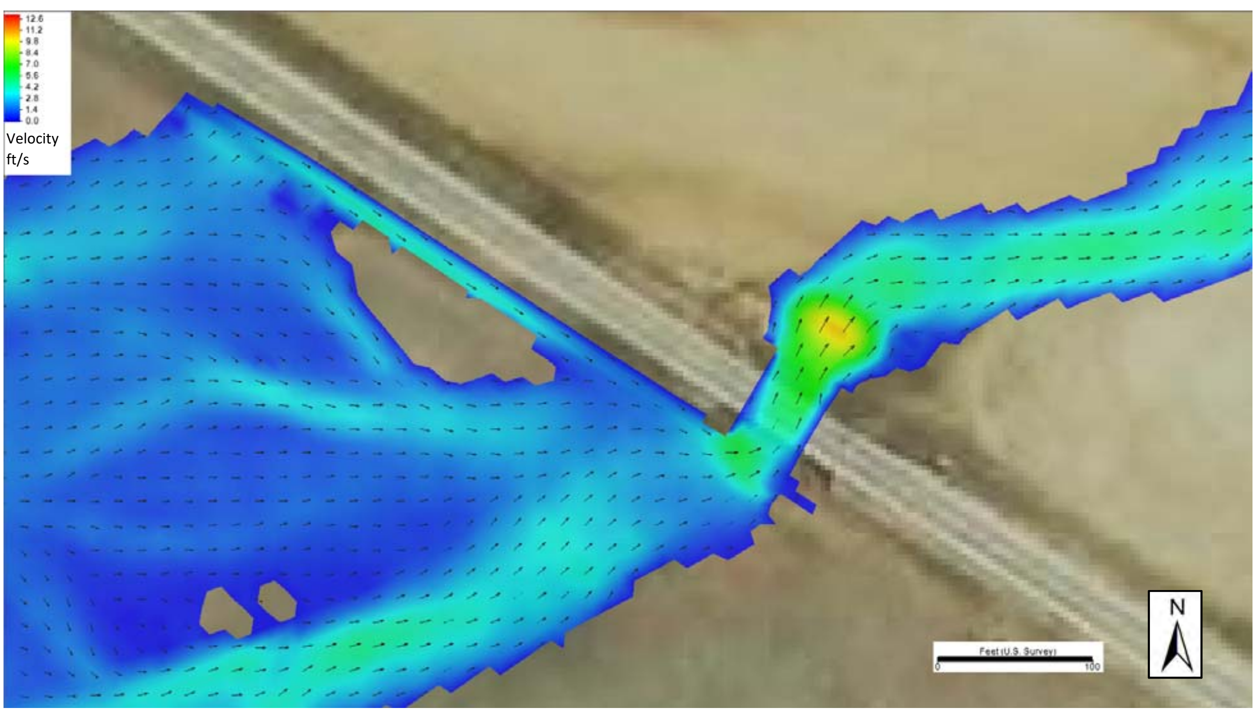


Figure 6. Proposed Conditions Velocity, 50-Year

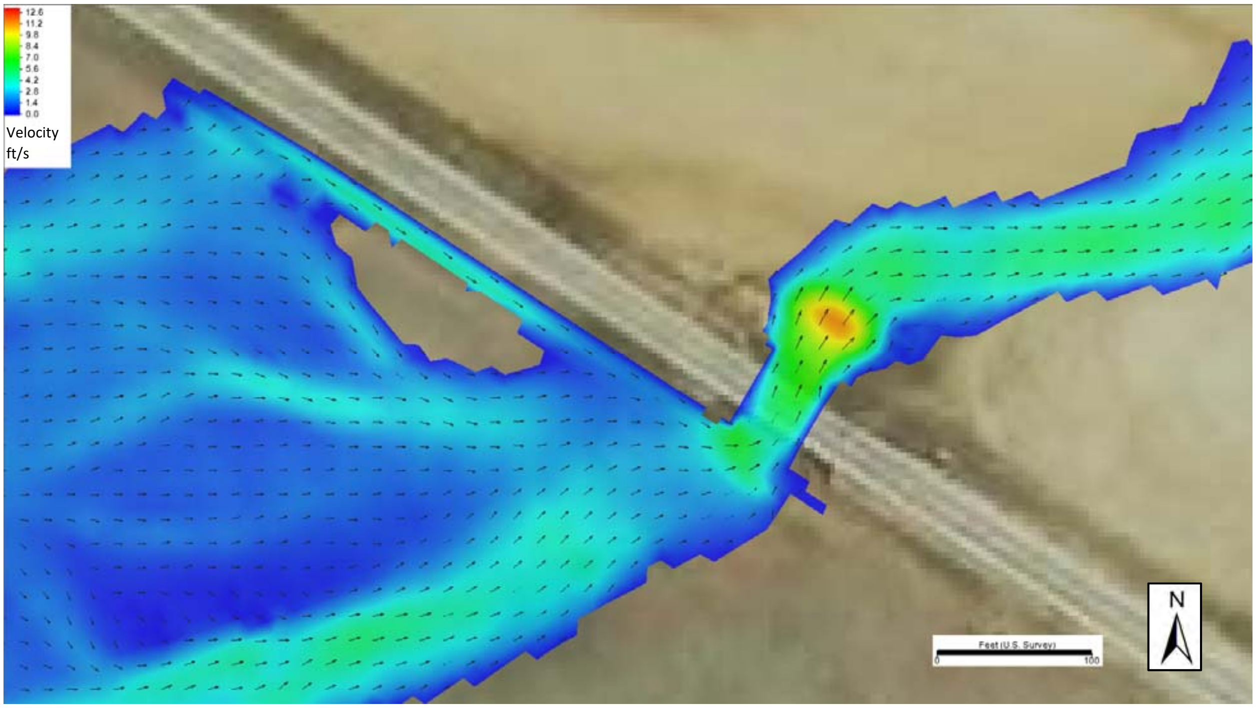


Figure 7. Proposed Conditions Velocity, 100-Year

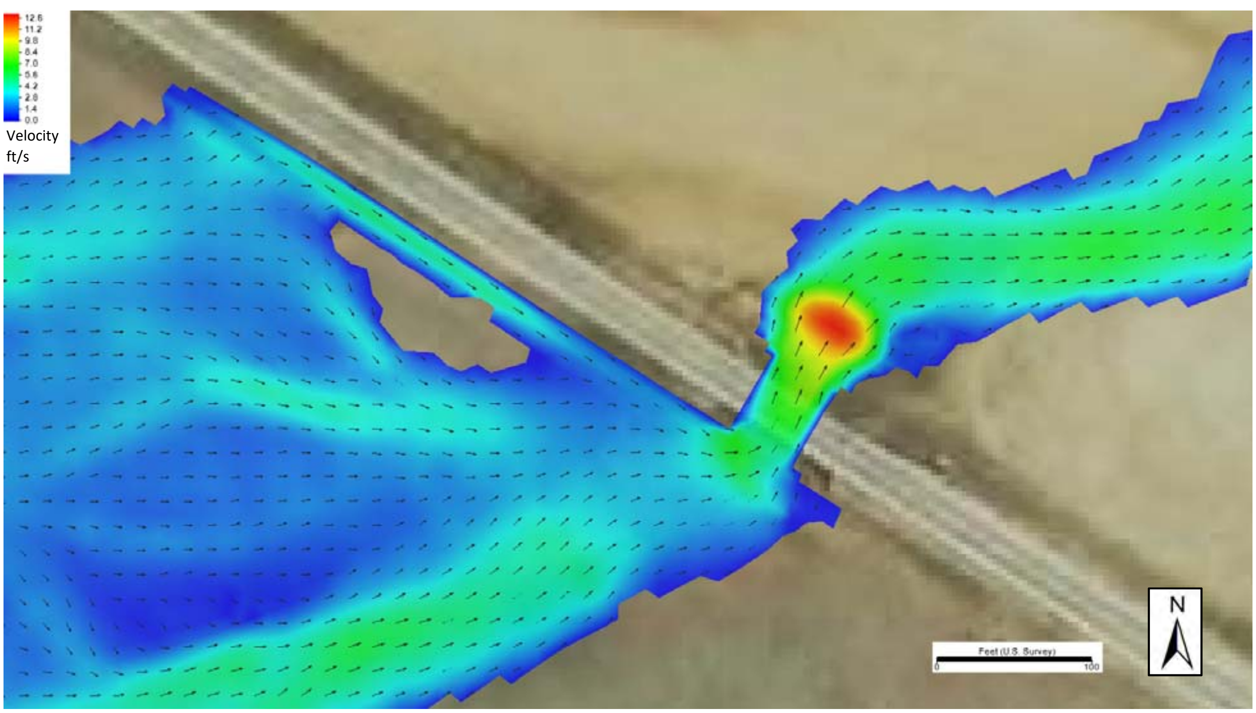


Figure 8. Proposed Conditions Velocity, 500-Year

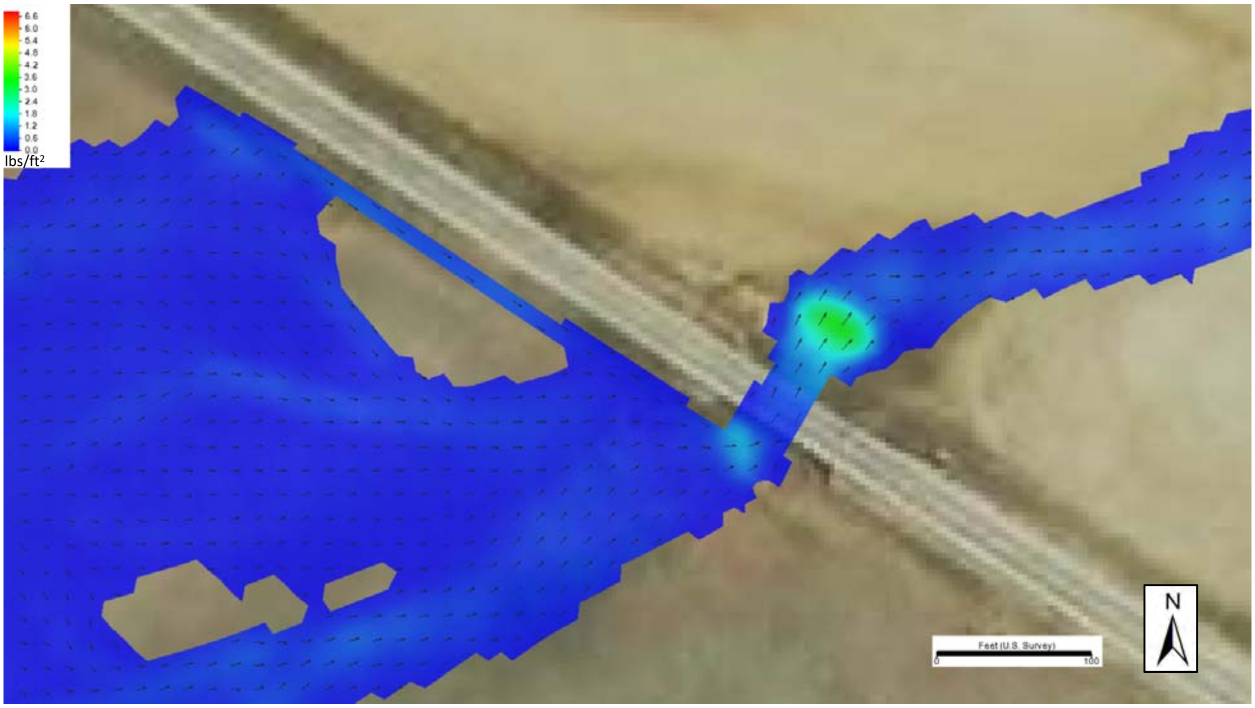


Figure 9. Proposed Conditions Bed Shear Stress, 10-Year

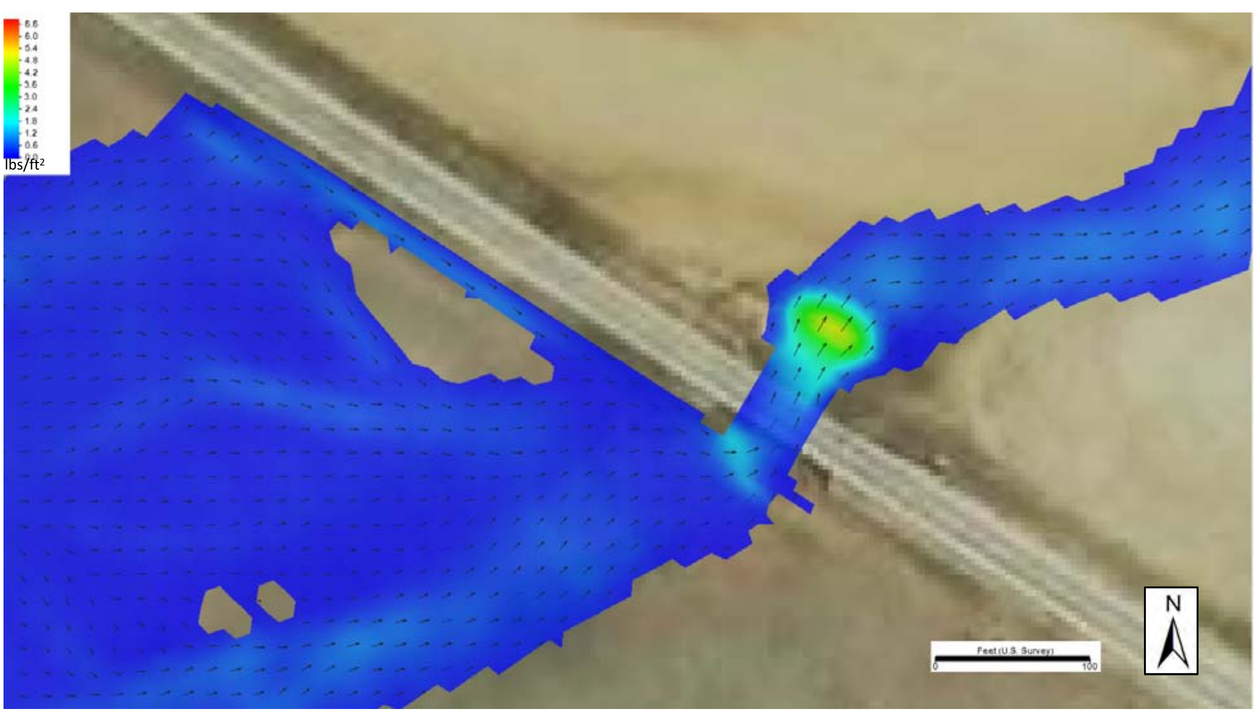


Figure 10. Proposed Conditions Bed Shear Stress, 50-Year

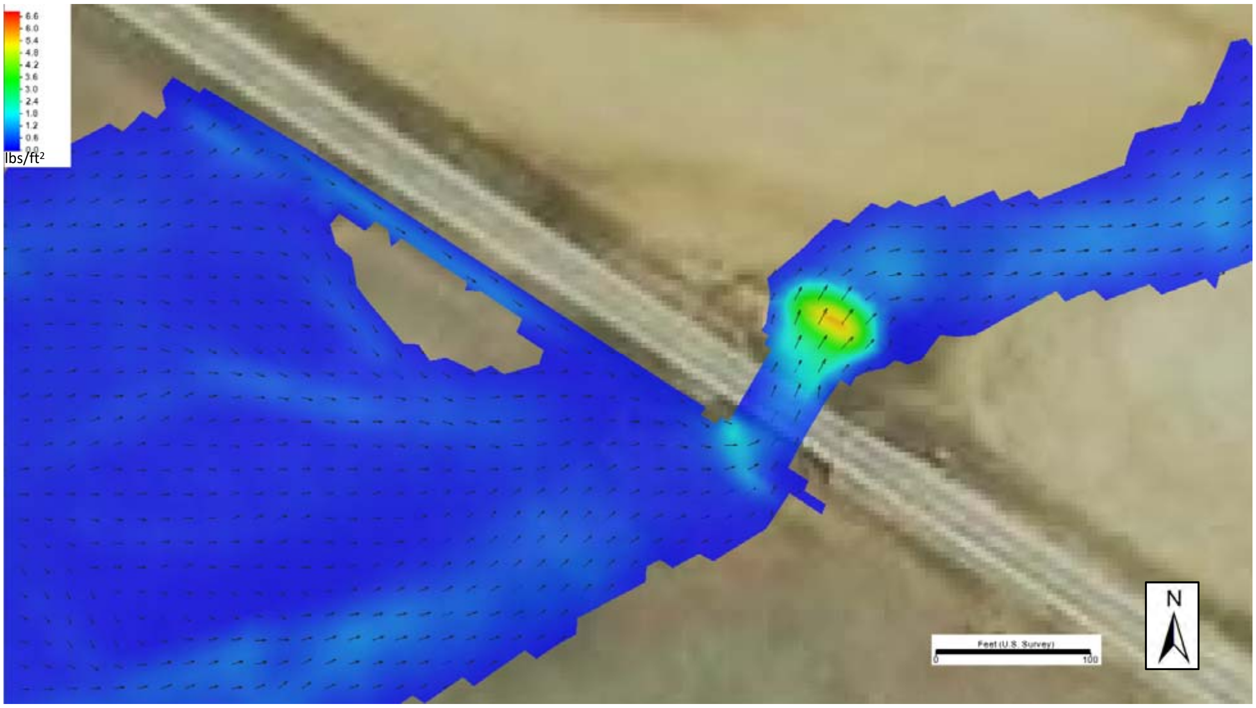


Figure 11. Proposed Conditions Bed Shear Stress, 100-Year

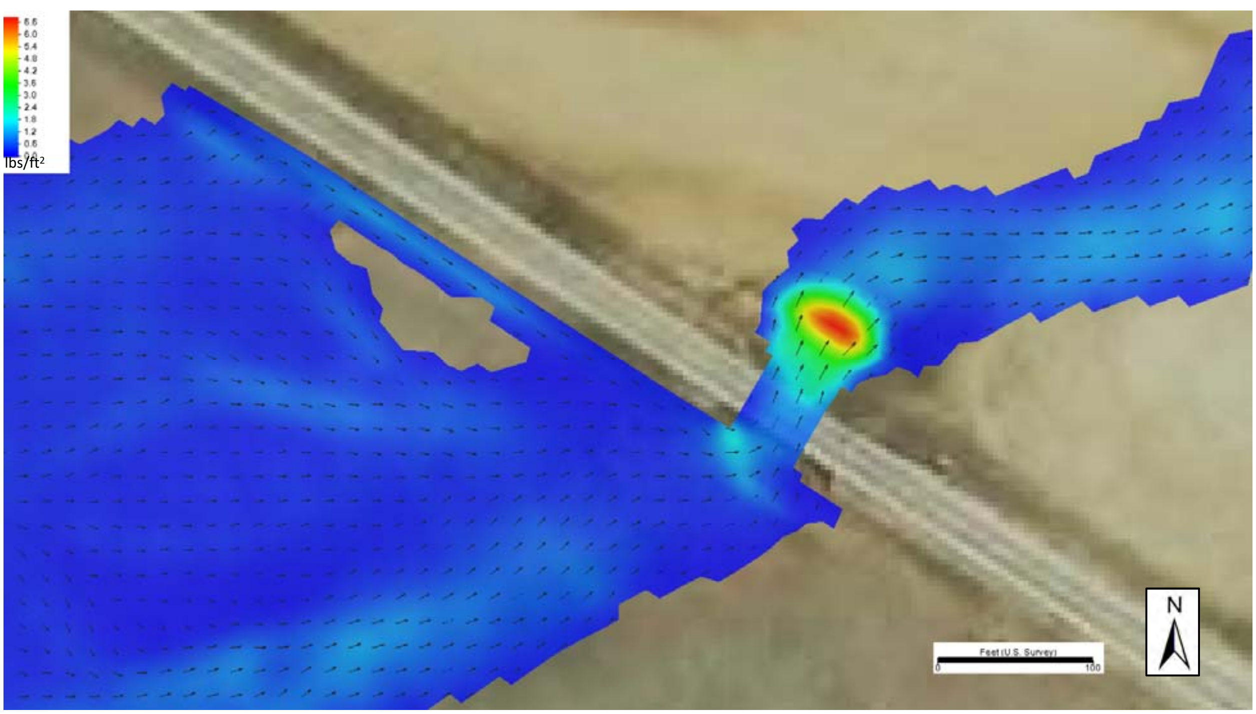


Figure 12. Proposed Conditions Bed Shear Stress, 500-Year

Appendix E

Geomorphic Assessment Memorandum

MEMO

TO:	Anthony Alvarado, PE (Ayres Associates)
FROM:	William Spitz, PG
RE:	BE Bridge F-19-E (MP 98.029) Geomorphic Assessment
DATE:	September 23, 2019
PROJECT #:	017-1690
CDOT PROJECT #:	20252
CDOT TO #:	22

Geomorphic Assessment of Stream Stability
BE Bridge F-19-E (US36 MP98.029) over Unnamed Creek near Strasburg, CO

The following memo describes the geomorphic assessment of the stability of the unnamed drainage channel and the US36 (SH40) crossing of the channel by the Bridge Enterprise (BE) Bridge F-19-E, which is located in Arapahoe County about 2 ¾ miles east of Strasburg, Colorado at Mile Post 321.288 (**Figure 1**). The unnamed drainage channel flows from the southwest to the northeast and appears to be tributary to a drainage channel along the south side of I-70 which eventually passes under the interstate via a small box culvert that eventually drains to West Bijou Creek. The bridge site and US36 are surrounded by low rolling hills and with the creek passing under the railroad tracks/embankment about 1,800 feet to the southwest and intersecting the eastbound I-70 roadside drainage about ½ mile to the northeast.

The following assessment includes the findings and conclusions from a desktop analysis and field reconnaissance of the site.

DESKTOP ANALYSIS AND GEOMORPHIC ASSESSMENT

The desktop analysis includes a review and analysis of historic aerial photography and maps, geology, soils, and general hydrology of the area. A sediment sample was also collected at the bridge site. A comparison of historic aerial photographs provides information on the long term lateral stability of the channel and can help identify potential geomorphic or man-made features that have had an impact in the past or can have an impact in the future on the vertical stability of the channel. Changes in vegetation and land use can also have an impact in the stability of the channel. The local geology and soils provide insight into local topographic controls and the characteristics and caliber of sediment delivered to and transported by the channel.

Geology

Since there is no geologic quadrangle map for the area, the general geology of the area was obtained from the Geologic Map of Colorado (Tweto 1979). The bedrock underlying most of the watershed is the early Tertiary/late Cretaceous age Denver Formation (or lower part of Dawson

Arkose) which consists of arkosic sandstone, shale, mudstone, conglomerate, and local coal beds. Overlying the Denver Formation in the area are eolian (windblown) deposits as noted in the NRCS Soils Report for Arapahoe County.

Figure 1. Location of US36 Bridge F-19-E over an unnamed drainage channel near Strasburg, CO.

Soils

Descriptions of the soils of the area were obtained from the NRCS's Web Soil Survey website. The principal soils of the watershed above the bridge consist of the Adena-Colby silt loams, 1 to 5 percent slopes (approx. 49% of the area) and Renohill-Buick loams, 3 to 9 percent slopes (approx. 41% of the area).

The Adena-Colby silt loams, 1 to 5 percent slopes, are farmland soils of statewide importance. The Adena and Colby components make up 65% and 25% of the complex, respectively, and are located on drainageways and hills. The parent material of the Adena component is eolian deposits. The parent material of the Colby component is fine-loamy eolian deposits and/or fine-silty eolian deposits. The unit is well drained, is in the low runoff class, with the Adena component belonging to the Hydrologic Soil Group C and the Colby component belonging to the Hydrologic Soil Group B.

Group B soils have a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C soils have a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. Group C soils have a slow rate of water transmission.

Hydrology

The general hydrology for the bridge site was obtained from the USGS's StreamStats website. StreamStats is a Web-based Geographic Information Systems (GIS) application that provides users with access to an assortment of analytical tools that are useful for a variety of water-resources planning and management purposes, and for engineering and design purposes. StreamStats users can select United States Geological Survey (USGS) data-collection station locations shown on a map and obtain previously published information for the stations. Users also can select any location along a stream and obtain the drainage-basin boundary, basin characteristics, and estimates of streamflow statistics for the location. Since there are no gages on Agate Creek, the creek at the bridge site was selected as the downstream end for the basin delineation which is used in obtaining the basin's hydrologic data.

The drainage area above the bridge site delineated by StreamStats is approximately 0.43 mi^2 (**Figure 2**). However, CDOT Mean annual precipitation is about 15.96 inches. The maximum 6-hour, 2-year recurrence precipitation is estimated to be 1.37 inches and the maximum 24-hr, 100-year recurrence precipitation is estimated to be 4.88 inches. The estimated peak flow statistics for the unnamed channel at Bridge F-19-E are provided in **Table 1**.

Table 1. Estimated Peak Flows for unnamed channel at Bridge F-19-E.

Recurrence	StreamStats Peak Flow (cfs)	CDOT Peak Flows (cfs)
2-yr	25.3	---
5-yr	91.7	---
10-yr	172	340
25-yr	324	---
50-yr	481	615
100-yr	692	755
500-yr	1,390	1,115

CDOT also developed hydrology for the bridge site using HEC-HMS. The drainage area above the bridge site as delineated by CDOT is approximately 0.7 mi^2, or about 40% larger than that delineated by StreamStats. The peak discharge estimated by CDOT, which are also shown in Table 1, reflect the larger drainage area delineated by CDOT.

Bed Sediment

A bulk sediment sample of the bed material was collected at the bridge site during the site visit. The grain size distribution from the dry sieve analysis indicates that the bed material is primarily 55% silt and clay (< 0.074 mm) and 36% sand (≤ 2.0 mm). The Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI) of 24, 13, and 11, respectively, indicating that the sample is a lean clay (CL).

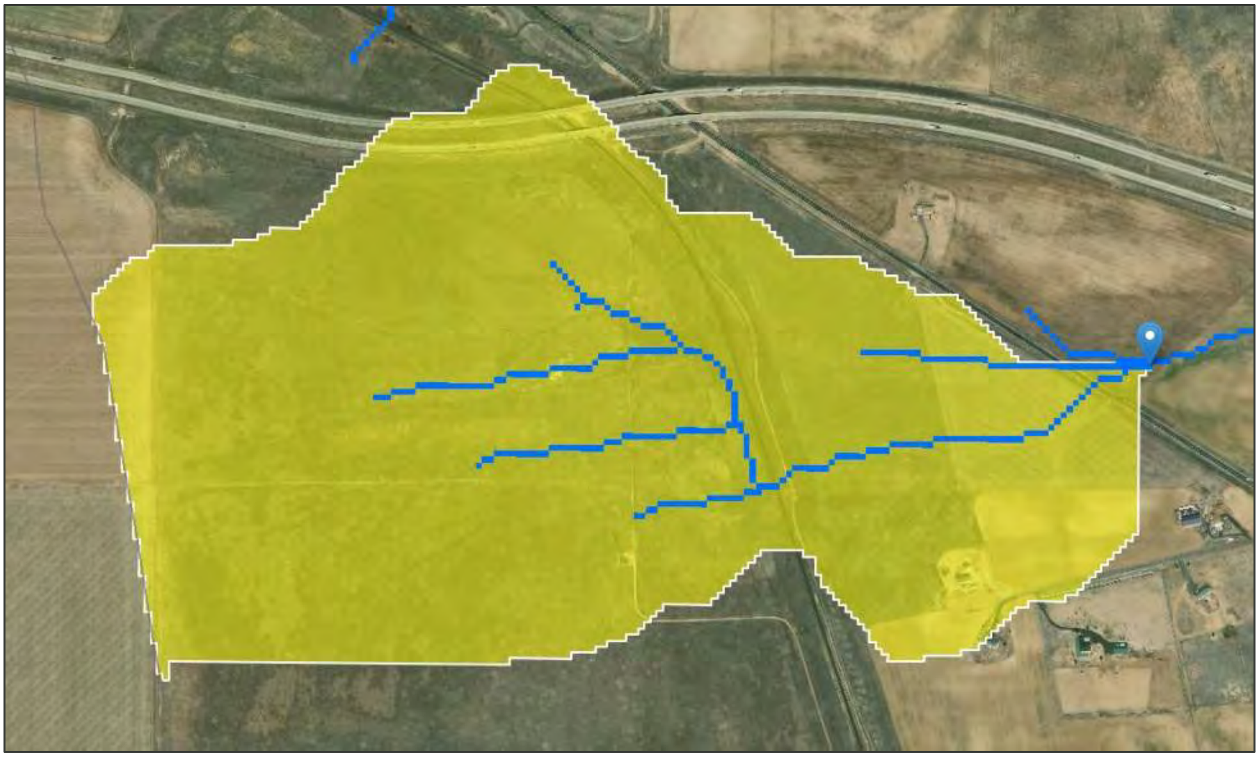


Figure 2. Drainage area for unnamed channel at US36 Bridge F-19-E near Strasburg, CO.

Aerial Photo Analysis
A poor resolution 1953 aerial photo of the area was obtained from the USGS's Earth Explorer website and compared to aerial photos from Google Earth that span the period from 1993 to 2017. The aerial photo comparison can assist in identifying any planform changes and man-made and geomorphic features within the bridge reach that may affect the stability of the stream and, consequently, the bridge.

The aerial photo comparison reveals that this unnamed channel is poorly defined both upstream (west) and downstream (east) and drains predominately grazed prairie lands. Changes noted since 1953 was the construction of I-70 and changes in land use, with the area upstream between the highway bridge and the railroad tracks being cultivated in 1953. Extensive vehicle tracking on the upstream and downstream side of the bridge was noted in the 2008 aerial photo and appears to correspond with repairs at the west end of the bridge as noted below in the discussion of the site visit.

Site Visit and Assessment
Bridge F-19-E, which is about 80 feet wide, consists of vertical timber retaining wall abutments with timber wingwalls. The bridge sits on 3 pile bents which contains 7 wood piles that are 12 inches in diameter. Wood cross braces are present on the pile bent. There are 2 short crutch bents with two 12-inch square wood piles with a 12-inch square wood header each at the southwest corner of the bridge and appear to be associated with repairs conducted in 2008. Currently, both crutch bents do not contact the low chord of the bridge and appear to have only provided support to that corner of the bridge during repairs. In addition, 2 trenches appear to have been excavated under the west span and may have been used to allow movement of equipment under the bridge during the repairs. The pavement above the crutch bents appears

to have been repaired as well, although the reason for the repair is unknown. The pile bents are skewed about 10° relative to the upstream and downstream channel. **Figure 3** shows the configuration of the bridge.

Figure 3. View looking downstream at US36 Bridge F-19-E near Strasburg, CO.

The channel upstream and downstream of the bridge contains a dense growth of grasses and weeds. **Figure 4** shows the upstream swale channel. Just upstream of the bridge is a mound of dirt and ground up asphalt in the swale channel that may have been left behind during the repairs. There is also a long berm along the downstream side of the bridge with about a 6-7 foot drop from the berm to the downstream swale channel (**Figure 5**), which could be problematic for the bridge if the berm were to breached during high flows.

Figure 4. View looking at the drainage upstream from Bridge F-19-E.

Figure 5. View looking at Bridge F-19-E and the high berm and drop to the swale channel downstream of the bridge.

CONCLUSIONS

Based on the desktop analysis and geomorphic assessment, there is no readily apparent evidence of channel instability immediately upstream or downstream of the bridge site that would threaten the stability of a new replacement bridge. However, given the berm and significant drop below the existing bridge, there may be need for some form of grade control to be constructed on the downstream side of the replacement bridge to control the grade change and prevent potential headcutting of the channel at and upstream of the bridge if the downstream berm is breached during a high flow event.

Appendix F

Scour and Countermeasure Calculations

Project Name	East Timber Bridges – Bridge F-19-E	Project No.	17-030.22
Design Calculation	Scour Analysis	Version	2
Originator	NLN	Date:	February 27, 2020
Checker	ALR	Date:	February 28, 2020

PURPOSE:

The Colorado Department of Transportation has identified the need to replace the existing US 36 Bridge (structure No. F-19-E) over a draw at mile point 98.03 and west of the town of Byers, Colorado. This calculation memorandum presents the scour analysis performed to inform the preliminary design of the replacement structure. The analysis utilizes data taken from the proposed SRH-2D hydraulic model developed by Muller Engineering.

REFERENCES:

Criteria Manual(s):

Colorado Department of Transportation, *Drainage Design Manual*. 2004.

Federal Highway Administration, H*ydraulic Engineering Circular No. 18, Evaluating Scour at Bridges.* Fifth ed., L.A. Arneson et al., 2012.

Federal Highway Administration, H*ydraulic Engineering Circular No. 23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance.* Third ed., vol 2. P.F. Lagasse et al., 2009.

Federal Highway Administration, *Hydraulic Considerations for Shallow Abutment Foundations.* Office of Bridge and Structures, FHWA-HIF-19-007. 2018.

Software:

Aquaveo SMS Version 13.0.8

Survey:

Topographic ground survey at and adjacent to the existing F-19-E structure (CDOT, 2019)

LiDAR (Ayres,2019)

Reports:

Yeh and Associates, Inc., *Draft Geotechnical Engineering Report: Eastern Plains Timber Bridge Replacement Project, US 36 Bridge West of the Town of Byers.* January 17, 2019.

Olsson Associates, Inc., *Geomorphic Assessment of Stream Stability BE Bridge F-19-E (US36 MP98.029) Over Unnamed Creek near Strasburg, CO.* September 23, 2019.

BRIDGE SCOUR ANALYSIS:

The proposed bridge design at structure No. F-19-E will be designed by CDOT Staff Bridge and adhere to both CDOT and FHWA criteria. The structure will convey the 100-year event with adequate freeboard. Riprap is required to protect the bridge foundation from failure by channel and embankment erosion and scour. Local scour at the proposed, 50-foot-span replacement bridge was computed for the 100- and 500-year events. FHWA's Hydraulic Engineering Circular No. 18 (HEC-18), SMS 13.0.8, and FHWA's Hydraulic Toolbox 4.4 were utilized for the scour analysis. Clips containing the approach and contracted sections are provided below.

Site Geology

Structure F-19-F is situated along a channel with a low-plasticity lean clay bed. A bulk sediment sample of the bed material was taken by Ayres and analyzed by CDOT during a site visit. The grain size distribution from the dry sieve analysis indicates that the bed material is primarily 55% silt and clay (< 0.074 mm) and 36% sand (≤ 2.0 mm). The Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI) of 24, 13, and 11, respectively. The D55 particle size was utilized in place of the D_{50} for purposes of critical velocity and scour calculations, see ***Figure 1*** for a summary of the bulk sediment sample. Based off the data taken from Section 1 shown below in ***Figure 2*** and the best available sediment data (D55), the channel's critical velocity was calculated using HEC-18 equations and results qualified the stream as a live-bed. The gradation of the bed sample; however, indicates that the channel is composed of cohesive soils and as such the design team decided to analyze the channel assuming clear-water scour equations. In an effort to capture a more realistic critical velocity, which would be higher in cohesive soils, Mirtskhoulava's simplified expression for cohesive sediments was used (Hoffmans and Verheij, 1997):

$$U_c = \log\left(\frac{8.8h}{d_a}\right)\sqrt{\frac{0.4}{\rho}\left[(\rho_s - \rho)gd_a + 0.6C_f\right]}$$

$$C_f = 0.035C_o$$

Table 1. *Depth-Averaged Critical Velocity for Cohesive Soils (Mirtskhoulava 1988)*

Description	Variable	100-Year	500-Year
Equilibrium depth of flow (m) POST-SCOUR	h	0.3	0.4
Detaching aggregate size (m)	d_a	0.004	0.004
Density of water (kg/m3)	ρ	1,000	1,000
Density of particle solid (kg/m3)	ρ_s	2,650	2,650
Acceleration due to gravity (m/sec2)	g	9.81	9.81
Fatigue rupture strength of clay (N/m2). $C_f = 0.035C_o$	C_f	1.4	1.4
Depth averaged critical velocity (m/sec)	U_c	0.47	0.48
Depth averaged critical velocity (ft/sec)	$\mathbf{U_c}$	**1.53**	**1.56**

Olsson Associate’s geomorphic assessment determined that there is no evidence of channel instability immediately upstream and downstream of the bridge site that will threaten the stability of the proposed bridge. Furthermore, the dense growth of grasses and weeds—in combination with the cohesive soils—both upstream and downstream of the existing bridge better justifies the clear-water scour assessment of the channel in the replacement bridge location.

Project ID	23010	**Location**	253379		
Project	FBR R400-371	**Source**	ROADWAY	**Report Date**	9/30/2019
F.S. #	253379	**Region**	04	**Construction**	3200
Engineer	Gary L. DeWitt - Region 4 Materials Engineer			**Working Days**	0
Comments					

Test #	Lab #	SP?	Station	Depth	LL	PL	PI	%Moist	R-Val	Group Class(GI)	mr
F-19-E	2019-4148	None	MP 98.029		24	13	11	2.0		A-6(3)	

Gradations:

mm	75	25	19	9.5	#4	#10	#40	#200
in	3	1	3/4	3/8				
%Pass						100	91	55
As Run						100	91	55

Proctor:

MDD :
OMC :
SpG :
Abs :

Lab Performing Work:

Atterberg	: CDOT	T180	:
Direct Shear	:	Mechanical Analysis	: CDOT
R-Value	:	Other	:
T99	:		

Figure 1. *Channel Bed Gradation*

Observation Cross-Sections

An approach section (Section 1), was taken approximately 145-ft upstream of the bridge location, see ***Figure 2*** below. This section was chosen because it captures velocities that will carry sediment towards the bridge (at or above critical velocity) and it is taken far enough downstream to avoid the high, sinusoidal ground elevations that result is shallow flow depths and braiding of the stream. The contraction section was cut closer to the downstream section of the bridge, but still captures the bridge geometry (Section 2). Initially this section was observed at the middle of the bridge; however, in an effort to capture some of the higher velocities that are experienced just downstream of the bridge it was moved to the location seen below. Average velocities, flow depths, and flow rates were taken at these sections for scour calculations.

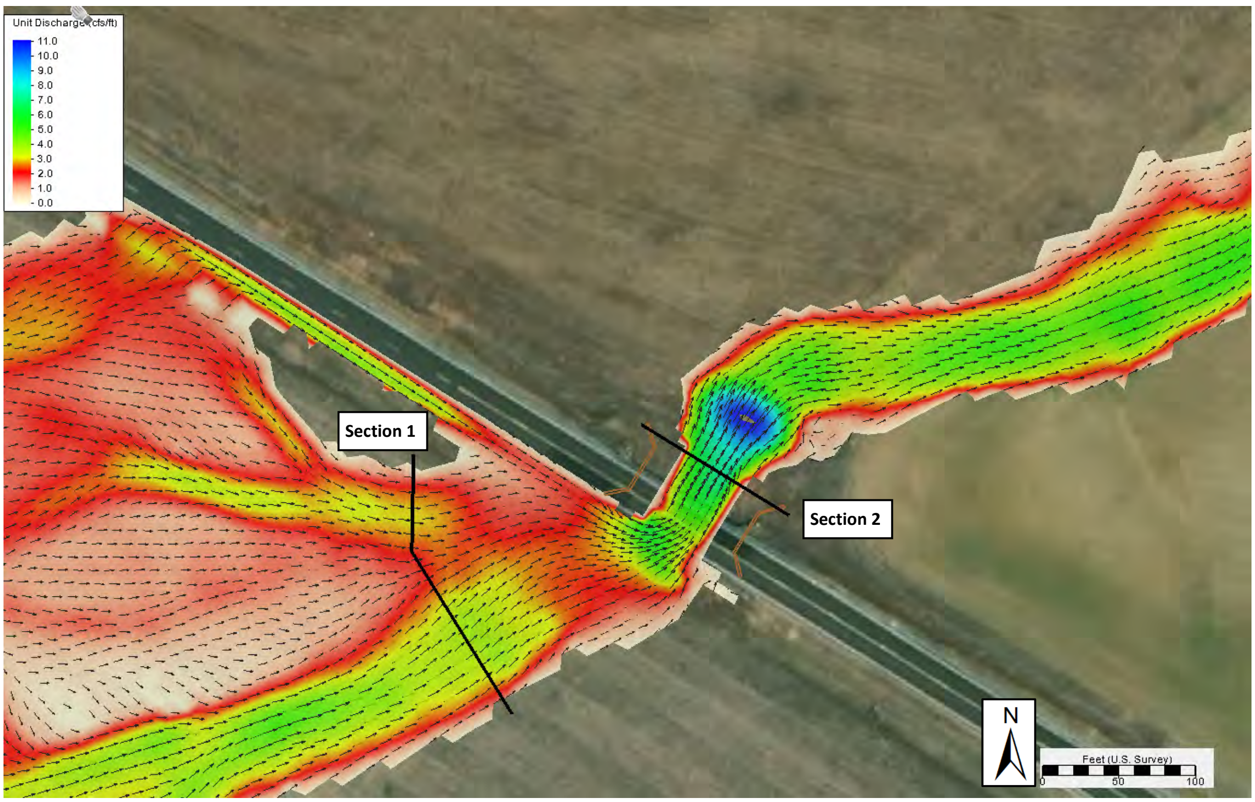


Figure 2. *Approach Sections Considered (Plan View)*

Contraction Scour

A time dependent contraction scour method was used to determine contraction scour at the replacement bridge. The D_{50} used in the time dependent calculation was modified until the critical depth (Y_2) matched the computed depth of the contracted section found by plugging Mirskoulava's critical velocity results into the contraction scour equation for cohesive soils. The rate of scour was estimated using Yang's total load sediment transport equation for sand with the limiting value of erosion being the clear-water scour. Hydrographs for both the 100-year and 500-year storm events were used to determine storm duration—since the 10-year storm flows were sufficient enough to result in velocities that transport bed material this flow-rate would be the ideal value at which to take the storm duration measurement; however, to be conservative a slightly larger duration was taken at 50 cfs, see ***Figure 3*** and ***Figure 4*** below. Time dependent scour for the 100-year and 500-year event can be seen in ***Figure 5*** and ***Figure 6***. Laursen's datapoints should be ignored, as these are live-bed results.

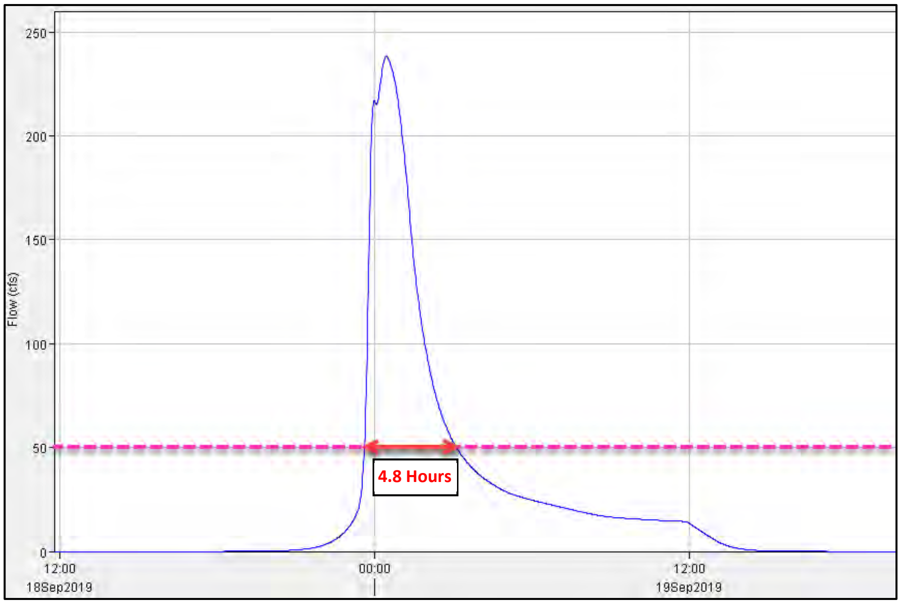


Figure 3. *Flood Hydrograph: 100-Year Event*

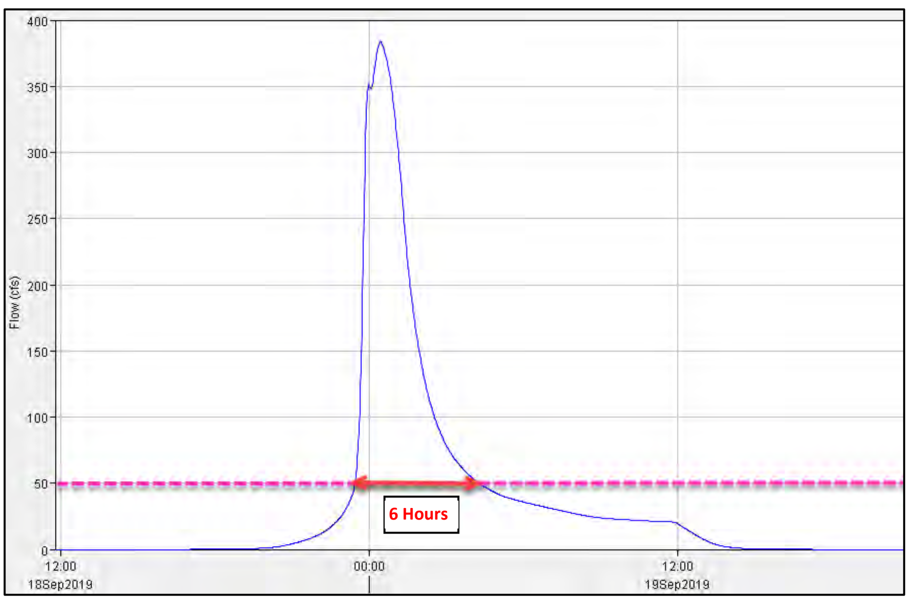


Figure 4. *Flood Hydrograph: 500-Year Event*

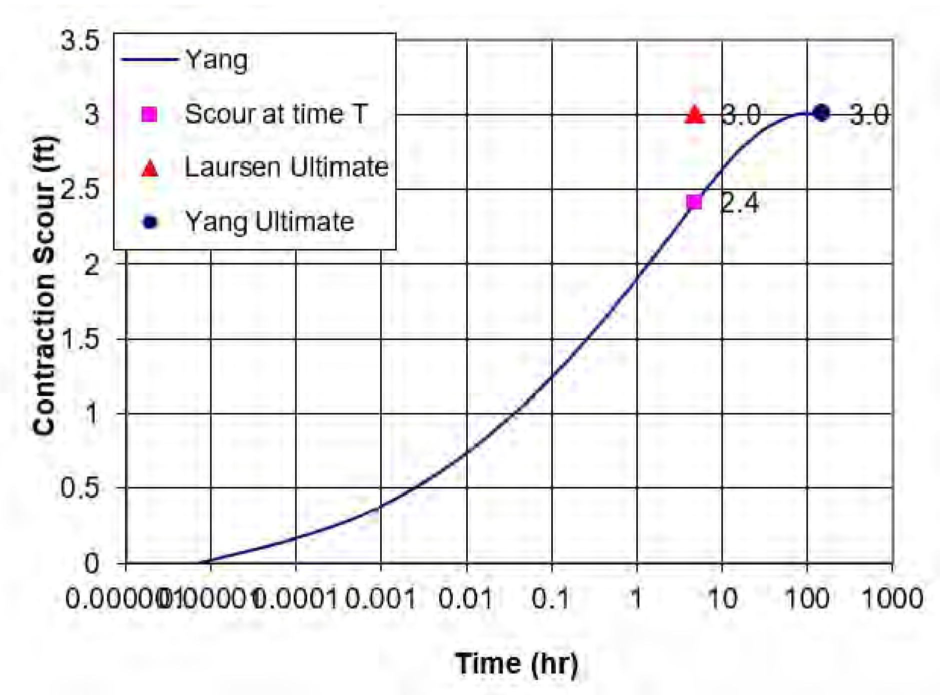


Figure 5. *100-Year Time Dependent Contraction Scour*

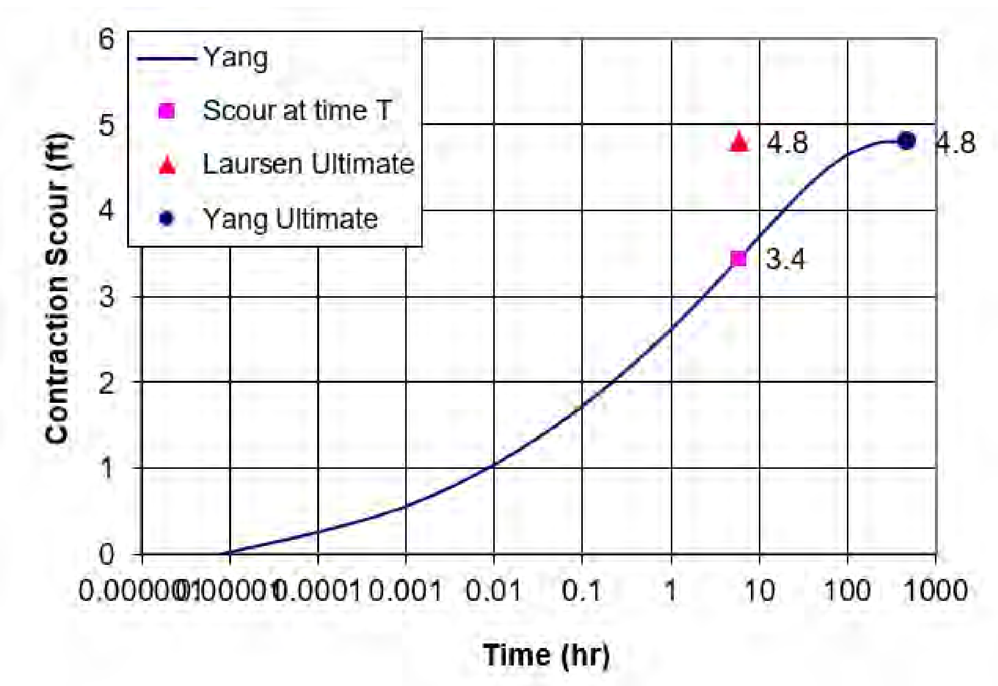


Figure 6. *500-Year Time Dependent Contraction Scour*

Clear-Water Abutment Scour

HEC-18, Figure 8.11 (spill-through abutments) was used in conjunction with the unit discharge ratio of the approach and contraction sections to determine the amplification factor to be used in calculating abutment scour for the 100-year and 500-year events. This factor was applied to the predicted average flow depth, post-scour.

Long Term Degradation

The survey provided by CDOT captures a significant drop in the channel located approximately 35 feet downstream of the proposed bridge, see ***Figure 7*** below. This drop has an approximate 13.4% grade and is caused by agricultural practices; while there is not presently an active headcut at this location, there is potential for one to start during high flows. The channel slope downstream of the drop is approximately 1.11% and the channel slope upstream of the cut is 1.60%.

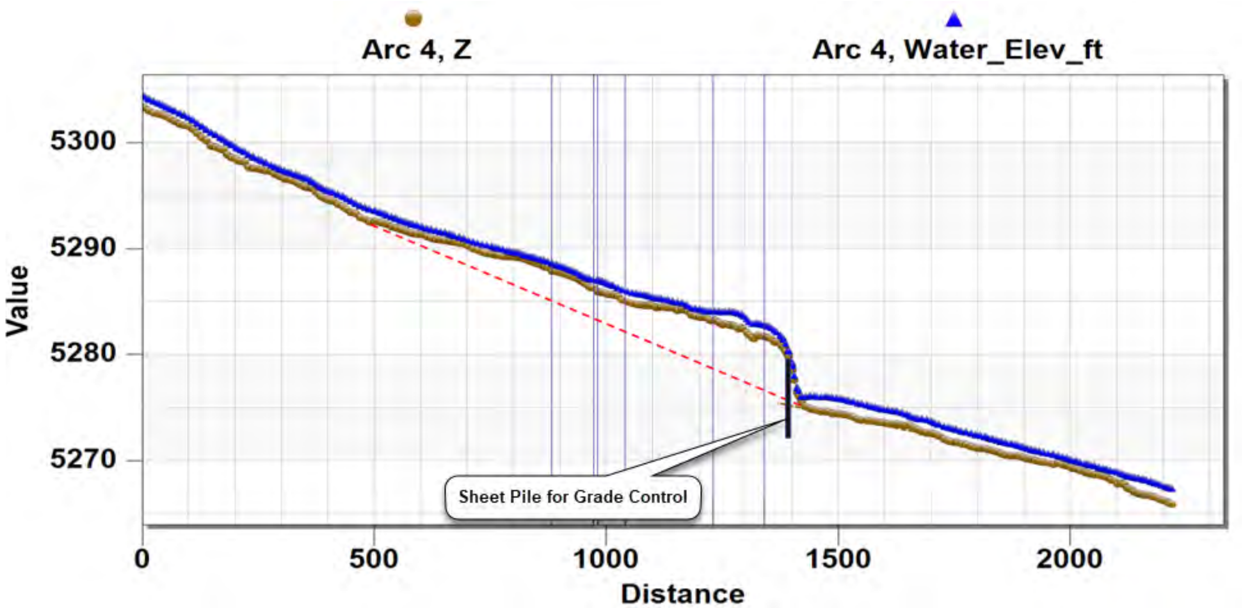


Figure 7. *Channel Bed Profile*

To determine the long-term degradation that threatens the stream the slope downstream of the headcut would, ideally, be projected upstream until it intersects the thalweg upstream of the headcut; however, this scenario does not present itself due to the limit of ground data upstream of the proposed bridge. ***Figure 7*** above shows a potential scenario for propagation of the headcut upstream, which would result in a 1.9% slope. The vertical channel degradation that could potentially be experienced at the proposed bridge location is shown below in ***Table 2***. A potential location for a grade control structure is shown above in ***Figure 7***.

Table 2. *Long Term Degradation Table*

Parameter	Value	Units	Notes
Input Parameters			
User-Specified Long-Term Degradation	4.96	ft	
Channel Inv. El. at Bridge	5281.46	ft	
Long-Term Degradation Channel Inv. El. at Bridge	5276.50	ft	Assumes 1.9% channel slope from existing headcut location

Scour Results

Estimated scour results for Structure F-19-E are provided in the table below.

Table 3. *Proposed Condition Scour Results*

Recurrence Interval	Long-Term Scour	Contraction Scour	Abutment Scour	Long-Term Degradation+ Contraction Scour
100-Year	4.96	2.40	3.00	7.36
500-Year	4.96	3.40	4.30	8.36

Using guidance recently published in FHWA's Hydraulic Considerations for Shallow Abutments Tech Brief, the maximum vertical contraction scour should be approximated by either long term degradation plus horizontal contraction scour or vertical contraction (pressure flow) scour, whichever is greater (see figure below). Since F-19-E is not under pressure flow conditions for either the 100-year or the 500-year events, long-term degradation plus horizontal contraction scour was used to approximate maximum potential scour depth.

Bridge Scour Countermeasure Design

Abutment and channel armoring countermeasures were designed using the guidance set forth in FHWA HEC-23. Table 2.1 of HEC-18 outlines that the 200-year return frequency event should be used to design scour countermeasures; however, 200-year flows were not provided in the hydrologic analysis. As a result, the 100-year and 500-year events were analyzed for scour countermeasures in order to approximate the magnitude of protection the 200-year event might require.

Abutment Riprap

The abutments of the structure will be designed to withstand scour from the 500-year scour event. The proposed bridge will be protected by abutment rock riprap countermeasures that have been outlined in in the Hydraulic Engineering Circular Number 23 (HEC-23), Volume 2, *Bridge Scour and Stream Instability Countermeasures.*

The required riprap size for protection against scour was calculated as a D_{50} of 10-inches; however, it is being suggested that 12-inch riprap be installed at a minimum. If protection will only be provided for abutment and contraction scour, the riprap section shall be installed at 2-ft depth and extend into the channel a width of $2Y_0$ or 2-ft along the 100-year contraction scour elevation. See ***Figure 8*** below. Should no grade control be installed downstream of the bridge to control long-term degradation, the 2-ft deep, 12-inch riprap should be extended to the full depth of long-term degradation plus the contraction scour—to elevation 5274.10. See ***Figure 9*** below.

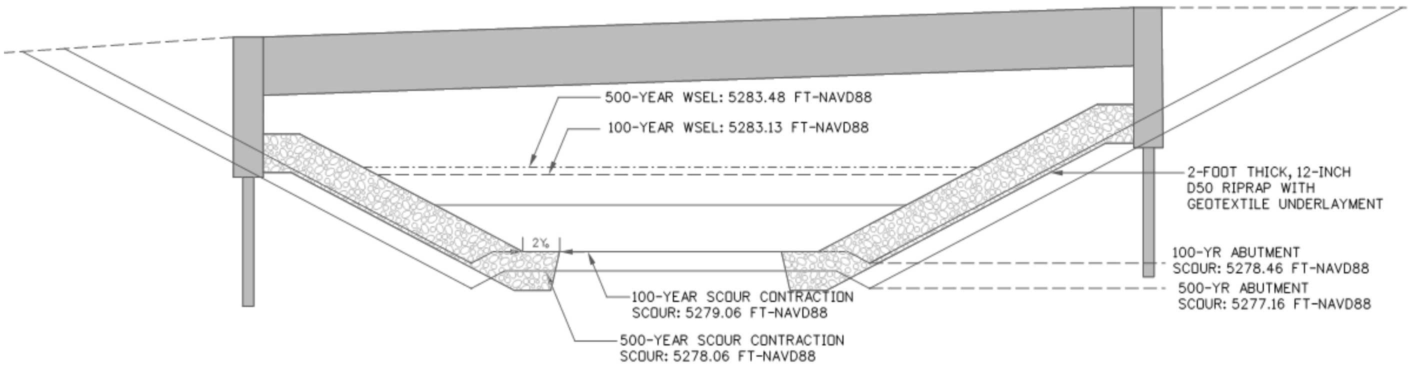


Figure 8. Riprap Armoring for Contraction Scour

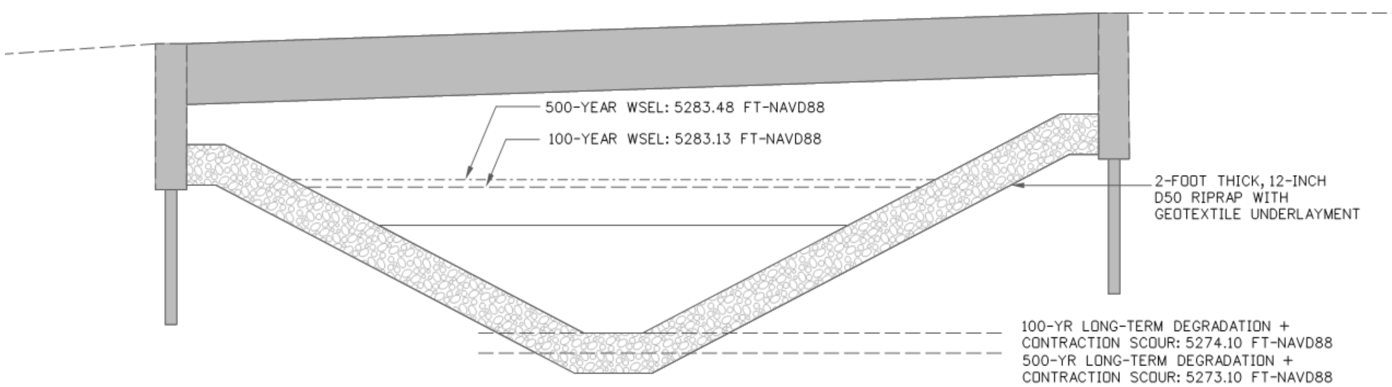


Figure 9. Riprap Armoring for Long-Term Degradation and Contraction Scour

100-yr. Flood Data					500-yr. Flood Data				
Hydraulic:	Y_o	=	0.98	ft. adjacent	Y_o	=	1.28	ft. adjacent
	Y_1	=	0.43	ft. adjacent	Y_1	=	0.60	ft. adjacent
	Left Y_0	=		ft. adjacent	Left Y_0	=		ft. adjacent
	Right Y_0	=		ft. adjacent	Right Y_0	=		ft. adjacent
	Q_1	=	198	cfs	Q_1	=	325	cfs
	Channel Q_2	=	247	cfs	Channel Q_2	=	388	cfs
	Left Q_2	=		cfs	Left Q_2	=		cfs
	Right Q_2	=		cfs	Right Q_2	=		cfs
	A_1	=	71.50	ft.^{2}	A_1	=	106.30	ft.^{2}
	A_2	=	40.45	ft.^{2}	A_2	=	52.42	ft.^{2}
	W_1	=	167.50	ft.	W_1	=	177.30	ft.
	Channel W_2	=	42.00	ft.	Channel W_2	=	44.0	ft.
	Left W_2	=		ft.	Left W_2	=		ft.
	Right W_2	=		ft.	Right W_2	=		ft.
	D_{50}		0.074	mm	D_{50}		0.074	mm
	Energy Slope	=	1.600E-02	ft/ft	Energy Slope	=	1.600E-02	ft/ft
	Gravity Acceleration	=	32.2	ft/sec^2	Gravity Acceleration	=	32.2	ft/sec2
	Fall Vel., ω	=	0.0184779	fps	Fall Vel., ω	=	0.0184779	fps
	V_m	=	2.77	fps	V_m	=	3.06	fps
	D_{50}	=	0.000243	ft.	D_{50}	=	0.000243	ft.

*D_{55} = 0.074 mm. D_{50} not reported by sieve analysis.

BY: **NLN**

DATE: **20-Feb-20**

CRITICAL AVERAGE VELOCITY CALCULATION
FOR STATE HWY 36 OVER UNNAMED CREEK
FOR BRIDGE F-19-E

FEBRUARY 2020

This analysis determines critical channel velocity based on Mirtskhoulava's simplified critical velocity relation (Hoffman, 1997).

$$U_c = \log\left(\frac{8.8h}{d_a}\right)\sqrt{\frac{0.4}{\rho}\left[(\rho_s - \rho)g d_a + 0.6C_f\right]} \quad \textit{Eqn. 2.9}$$

where

U_c	=	Critical Depth Averaged Velocity (m/sec).
h	=	Equilibrium depth of flow (m).
d_a	=	Detaching aggregate size (m).
		da = 0.004m for this relationship.
ρ	=	Density of water (kg/m^3).
ρ_s	=	Density of particle solid (kg/m^3).
g	=	Acceleration due to gravity (m/sec^2).
C_f	=	Fatigue rupture strength of clay (N/m^2). $C_f = 0.035C_o$.
C_o	=	Cohesion of clay. C_o is a function of the type of soil, its water content, and its geotechnical properties.

This analysis applies post-scour depths to the equilibrium depth of flow through iteration.
Liquidity index and void ratio is not known for this soil. Soil is assumed to be Lean Clay (Low Plasticity) with a liquidity index between 0 and 0.25 based on the geomorphic report. A void ratio of 0.95 was assumed for a slightly more conservative result [Table 2.5, Hoffman, 1997]

SOIL PROPERTIES

Porosity	n	=	
voids ratio	e	=	
Specific Gravity	G	=	2.65
Saturated Water Content	w_s	=	0.000
Liquid Limit	ll	=	0.24
Plastic Limit	pl	=	0.13
Plasticity Index	pi	=	0.11
Liquidity Index	li	=	
Soil Type (USCS)		=	C and B
Cohesion of Clay (Pa) [From Table 2.5, Hoffman, 1997].	C_o	=	40.2

COHESIVE SOIL CRITICAL VELOCITY

EVENT			**100-year**	**500-year**
Computed depth of contracted section (ft), Y_2	y_2	=	1.11	1.28
Equilibrium depth of flow (m) POST-SCOUR.	h	=	0.3	0.4
Detaching aggregate size (m).	d_a	=	0.004	0.004
Density of water (kg/m^3).	ρ		1000.0	1000.0
Density of particle solid (kg/m^3).	ρ_s	=	2,650.0	2,650.0
Acceleration due to gravity (m/sec^2).	g	=	9.81	9.81
Fatigue rupture strength of clay (N/m^2). $C_f = 0.035C_o$.	C_f	=	1.4	1.4
Depth averaged critical velocity (m/sec).	U_c	=	0.47	0.48
Depth averaged critical velocity (ft/sec)	U_c	=	1.53	1.56
Applied Depth Averaged Critical Velocity (ft/sec)	$\mathbf{U_c}$	**=**	**1.53**	**1.56**

Calc. By:	NLN	Date:	2/20/2020
Check By:		Date:	

Abutment Riprap Sizing Calculation
HEC-23, Volume 2 Edition 3, Equation 14.2

100-yr Riprap Sizing			
	Froude	**Max Velocity (ft/s)**	**Max Water Depth (ft)**
Max	**1.20**	**7.15**	**1.11**

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)}\left[\frac{V^2}{gy}\right]^{0.14}$$

K	0.89	
Ss	2.65	
V	7.15	ft/s
g	32.20	ft^3/s
y	1.11	
D50	7.55	in

500-yr Riprap Sizing			
	Froude	**Max Velocity (ft/s)**	**Max Water Depth (ft)**
Max	**1.24**	8.32	1.40

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)}\left[\frac{V^2}{gy}\right]^{0.14}$$

K	0.89	
Ss	2.65	
V	8.32	ft/s
g	32.20	ft^3/s
y	1.40	
D50	9.62	in

CONTRACTION SCOUR COMPUTATIONS
FOR STATE HWY 36 OVER UNNAMED CREEK
FOR BRIDGE F-19-E

COHESIVE SOIL ANALYSIS
FEBRUARY 2020

The following computations are made using the following relation for
Clear Water Contraction Scour:

Scour Depth, $Y_s=Y_2-Y_0$

where:

$Y_2= V_0Y_0/V_c$

and :

Vc is determined using Mirtskhoulava's simplified relation
for depth-averaged critical velocity in cohesive sediments.

100-YEAR DISCHARGE
MAIN CHANNEL
CLEAR-WATER CONTRACTION SCOUR CALCULATIONS

AVERAGE VELOCITY IN CONTRACTED SECTION (fps), V_0	=	6.10
AVERAGE WATER DEPTH (ft), Y_0	=	0.98
CRITICAL VELOCITY (fps), V_c	=	1.50
COMPUTED DEPTH OF CONTRACTED SECTION (ft), Y_2	=	3.99
DEPTH OF CONTRACTION SCOUR (ft), Y_s	**=**	**3.01**

Calc. By:	NLN	Date:	2/20/2020
Check By:		Date:	

100-YEAR TIME SCOUR SUMMARY
FOR STATE HWY 36 OVER UNNAMED CREEK
FOR BRIDGE F-19-E

FEBRUARY 2020

		Approach	Bridge (t=0)	Live-Bed Bridge (ultimate)	Clear-Water Bridge (ultimate)
Manning n	=	0.035			
Q (cfs)	=	198	247		
W (ft)	=	167.50	42.00		
Y (ft)	=	0.43	0.98	5.28	3.99
A (ft^2)	=	72	41.2	222	167
V (ft/s)	=	2.75	5.99	1.11	1.47
Sf	=	0.012924	0.020481	0.000075	0.000191

D50mm	=	0.264	*Iterate D_{50} until Y_2 (clear water) below matches cohesive clear-water Y_2
D50ft	=	0.000866	
Vcrit (ft/s)	=	7.50	HARD_WIRED, assumed Clear-Water
Sediment Sg	=	2.65	
Fall vel. (ft/s)	=	0.129	
Sediment Porosity	=	0.58	
Scour hole side slope	=	2.5	

		Approach	Bridge (t=0)	Live-Bed Bridge (ultimate)	Clear-Water Bridge (ultimate)
nu (ft^2/s)	=	1.08E-05			
g (ft/s^2)	=	32.2			
water wt (lb/ft^3)	=	62.4			
U* (ft/s)	=	0.423	0.804	0.113	0.156
Re*	=	33.96	64.53	9.04	12.56
Vcr/omega	=	2.36	2.09	3.45	3.07
log(C)	=	4.164	4.736	0.388	1.311
C ppm-wt	=	14594.0	54442.8	2.4	20.5
Qs(cfs)	=	1.091	5.069	0.000	0.002

Clear Water Controls

Y2 (clear water)	3.99
Y2 (ultimate live-bed)	5.28
Qs Ratio	0.000
	Run Goal Seek
Laursen Ys(ultimate)	3.0
Yang Ys(ultimate)	3.0

Scour Time T (hr)	4.8
Scour at Time T (ft)	**2.4**

Calc. By:	NLN	Date:	2/21/2020
Check By:		Date:	

CONTRACTION SCOUR COMPUTATIONS
FOR STATE HWY 36 OVER UNNAMED CREEK
FOR BRIDGE F-19-E

COHESIVE SOIL ANALYSIS
FEBRUARY 2020

The following computations are made using the following relation for
Clear Water Contraction Scour:

Scour Depth, $Y_s=Y_2-Y_0$

where:

$Y_2= V_0Y_0/V_c$

and Vc is determined using Mirtskhoulava's simplified relation for depth-averaged critical velocity in cohesive sediments.

500-YEAR DISCHARGE
MAIN CHANNEL
CLEAR-WATER CONTRACTION SCOUR CALCULATIONS

AVERAGE VELOCITY IN CONTRACTED SECTION (fps), V_0	=	7.41
AVERAGE WATER DEPTH (ft), Y_0	=	1.28
CRITICAL VELOCITY (fps), V_c	=	1.56
COMPUTED DEPTH OF CONTRACTED SECTION (ft), Y_2	=	6.08
DEPTH OF CONTRACTION SCOUR (ft), Y_s	**=**	**4.80**

Calc. By:	NLN	Date:	2/20/2020
Check By:		Date:	

Y 40 OVER
IDGE G-21

500-YEAR TIME SCOUR SUMMARY
FOR STATE HWY 36 OVER UNNAMED CREEK
FOR BRIDGE F-19-E

FEBRUARY 2020

		Approach	Bridge (t=0)	Live-Bed Bridge (ultimate)	Clear-Water Bridge (ultimate)
Manning n		0.035			
Q (cfs)	=	325	388		
W (ft)	=	177.30	44.00		
Y (ft)	=	0.60	1.28	8.03	6.08
A (ft^2)	=	106	56.3	353	268
V (ft/s)	=	3.06	6.90	1.10	1.45
Sf	=	0.010249	0.018986	0.000042	0.000105

D50mm	=	0.204	*Iterate D_{50} until Y_2 (clear water) below matches cohesive clear-water Y_2		
D50ft	=	0.000669			
Vcrit (ft/s)	=	7.50	HARD_WIRED, assumed Clear-Water		
Sediment Sg	=	2.65			
Fall vel. (ft/s)	=	0.093			
Sediment Porosity	=	0.58			
Scour hole side slope	=	2.5			
	=				
nu (ft^2/s)	=	1.08E-05			
g (ft/s^2)	=	32.2			
water wt (lb/ft^3)	=	62.4			
U* (ft/s)	=	0.445	0.885	0.104	0.144
Re*	=	27.60	54.87	6.44	8.91
Vcr/omega	=	2.47	2.15	4.00	3.47
log(C)	=	4.261	4.930	0.059	0.997
C ppm-wt	=	18243.4	85173.3	1.1	9.9
Qs(cfs)	=	2.239	12.484	0.000	0.001

Clear Water Controls

Y2 (clear water)	6.08
Y2 (ultimate live-bed)	8.03
Qs Ratio	0.000
	Run Goal Seek
Laursen Ys(ultimate)	4.8
Yang Ys(ultimate)	4.8

Scour Time T (hr)	6
Scour at Time T (ft)	**3.4**

Calc. By:	NLN	Date:	2/20/2020
Check By:		Date:	

TIME-DEPENDENT ABUTMENT SCOUR COMPUTATIONS FOR COHESIVE SOILS
FOR STATE HWY 36 OVER UNNAMED CREEK
FOR BRIDGE F-19-E

February 2020

The following computations are made using the HEC-18 equation for NCHRP 24-20 Abutment Scour:

$Y_{max} = \alpha_A Y_2$ or $Y_{max} = \alpha_B Y_2$ (α_A for Live-Bed Conditions; α_B for Clearwater Conditions)

$Y_s = Y_{max} - Y_0$

100-YEAR & 500-YEAR DISCHARGE
Main Channel
CLEAR-WATER ABUTMENT SCOUR COMPUTATIONS

		100-yr CONDITION	500-yr CONDITION
UPSTREAM SECTION UNIT DISCHARGE (ft^2/s), q_f	=	1.18	1.83
CONTRACTED SECTION UNIT DISCHARGE (ft^2/s), q_{2f}	=	5.87	8.83
DEPTH-AVERAGED CRITICAL VELOCITY (ft/s), V_c	=	1.50	1.56
COMPUTED DEPTH OF CONTRACTED SECTION (ft), Y_2	=	3.99	6.08
STORM DURATION (hrs), t	=	4.8	6.0
TIME RATE FLOW DEPTH (ft), Y_{2t}	=	3.39	4.72
DISCHARGE RATIO, q_{2f}/q_f	=	4.97	4.81
AMPLIFICATION FACTOR CLEAR-WATER CONDITIONS, α_B	=	1.17	1.19
COMPUTED WATER DEPTH AT ABUTMENT (ft), Y_{max}	=	3.97	5.62
AVERAGE WATER DEPTH AT BRIDGE (ft), Y_0	=	0.98	1.28
AVERAGE ABUTMENT SCOUR DEPTH, Y_s	=	**3.0**	**4.3**

Calc. By:	NLN	Date:	2/20/2020
Check By:		Date:	

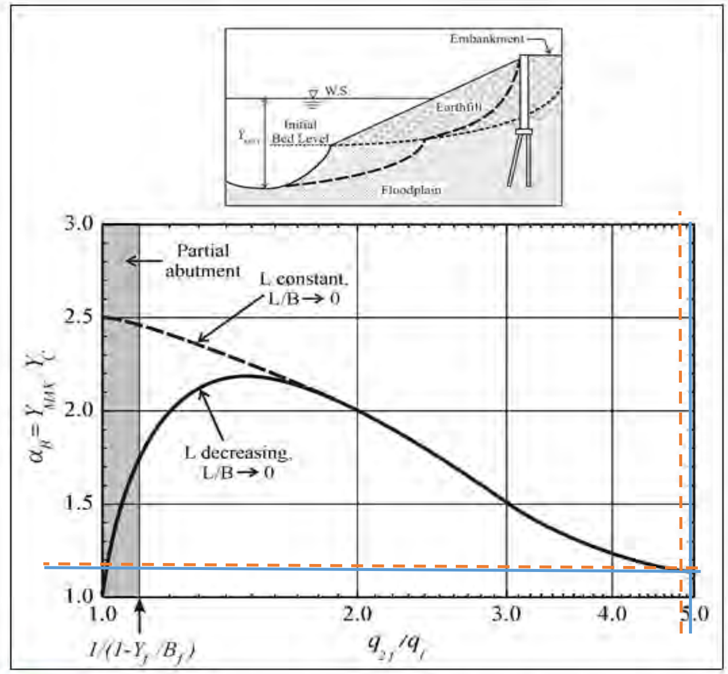


Figure 8.11. Scour amplification factor for spill-through abutments and clear-water conditions (NCHRP 2010b).