CDOT STRUCTURE F-20-L REPLACEMENT

Preliminary Hydraulic Design Report

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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-20-L. Structure F-20-L is a 39-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-20-L bridge is located just east of the town of Byers, Colorado in Arapahoe County on US 40 approximately a quarter mile east of mile marker 321 and at Latitude 39.687752 North and Longitude 104.128781 West. The legal location of the structure is the North East 1/4 of the South West 1/4 of Section 20, Township 4 South, Range 60 West of the Sixth Meridian. **Figure 1** presents the location of Bridge F-20-L.

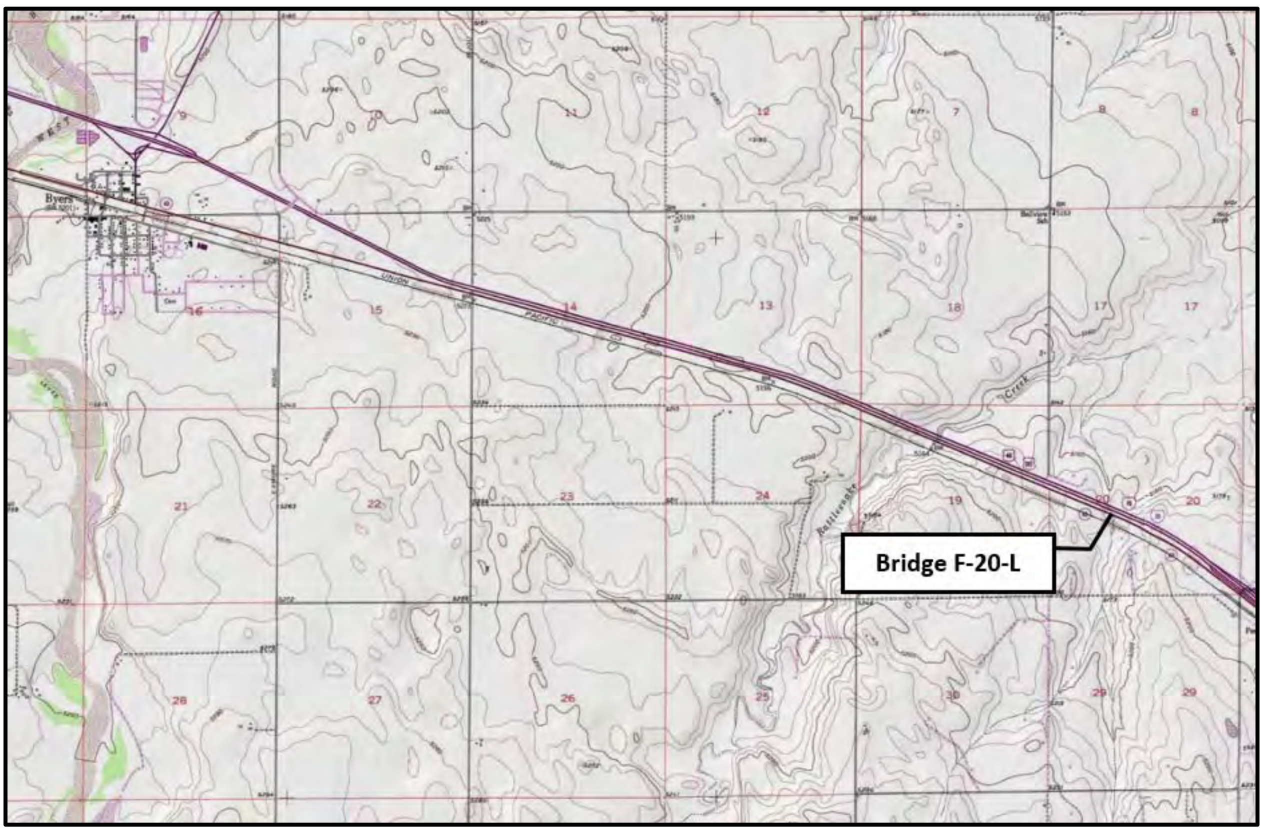


Figure 1. Vicinity Map

Structure F-20-L lies in a rural, primarily agricultural area with the surrounding topography generally consisting of hilly to gently sloping terrain. Bridge F-20-L spans an unnamed ephemeral tributary to Rattlesnake Creek with stormwater runoff passing from south to north through the bridge. A Union Pacific Railroad (UPRR) track is located approximately 250 feet upstream of Bridge F-20-L and generally parallels US 40. Interstate 70 is located downstream of Bridge F-20-L with the eastbound and westbound lanes being approximately 100 feet and 200 feet downstream of Bridge F-20-L, respectively. 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) B with some HSG Type C 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 08005C0325K effective December 17, 2010. Bridge F-20-L 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-20-L spans has no designated floodway. A FEMA Firmette is included in **Appendix B** that presents FEMA floodplain mapping near Bridge F-20-L.

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-20-L is approximately 3,240 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-20-L 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-20-L 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-20-L are summarized in **Table 1.**

Table 1. Summary of Peak Discharges at Bridge F-20-L

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)
885	1,935	2,485	4,010

No stream gauge data was available near Bridge F-20-L to confirm flows through a stream gauge analysis. 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-20-L

The existing F-20-L structure is an 89-year-old bridge constructed in 1931. The bridge has two equal 19-foot spans supported by 1-foot diameter cross-braced timber, six and seven pile, bents and abutments. The bridge superstructure is comprised of timber stringers with a concrete deck and asphalt overlay. Currently the bridge carries approximately 220 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-20-L'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. Standing water was present in a small scour hole near the west bridge abutment (left abutment) during the visit. 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 that "scour conditions are minimal although the alignment of the upstream railroad bridge and the skew of flow to the bridge results in flow and scour being directed primarily at the left bridge abutment". 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-20-L 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-20-L 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 2,200 feet upstream of the bridge past the UPRR embankment to a private gravel driveway. The model's downstream limit is approximately 2,100 feet downstream of the bridge and includes both the eastbound and westbound I-70 bridges. The mesh consists of approximately 90,900 predominantly triangular elements, with quadrilateral elements representing Bridge F-20-L, the US 40 embankment, the I-70 embankment, and UPRR embankment. Element sizes range from 3-foot elements at the bridge to 50-foot elements along mesh boundaries. The existing trestle bent is likely to catch debris and was modeled as a continuous obstruction within the model. The I-70 bridge piers were also modeled as obstructions in 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
Railroad Embankment/Ballast	0.033
Natural Grass	0.040
Row Crops	0.035
Structures	0.250
Asphalt/Concrete	0.013
Homestead (Light Debris)	0.030
Gravel Drive	0.030
Riprap	0.040

Boundary conditions used within the model include inflow boundaries for the unnamed ephemeral channel spanned by Bridge F-20-L as well as an outflow water surface elevation at the downstream end of the model. The 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-20-L to the downstream model limit, and the two I-70 bridge locations, 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-20-L. 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	5142.01
50-Year	5142.46
100-Year	5142.65
500-Year	5143.10

3.2.3 F-20-L Existing Conditions Hydraulic Modeling Results

A summary of the hydraulic properties immediately upstream and downstream of Structure F-20-L 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 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	5160.06	5159.54	3.64	0.76	4.52	1.18
50-Year	5163.08	5161.66	5.23	1.21	7.04	0.99
100-Year	5164.36	5162.85	5.27	1.48	7.55	1.24
500-Year	5166.07	5165.98	6.38	2.06	5.50	1.42

The existing structure experiences pressure flow during the 50-year event, and the US 40 roadway embankment is overtopped during the 100- and 500-year events. The downstream 80-foot span skewed I-70 bridges induce a tailwater on Bridge F-20-L, and this tailwater controls the hydraulic capacity of Bridge F-20-L. In comparison to the I-70 bridges, Bridge F-20-L has a reduced bridge open area and a lower embankment and roadway crest elevation. **Figure 2** presents the 100-year water depth plot at the existing structure.

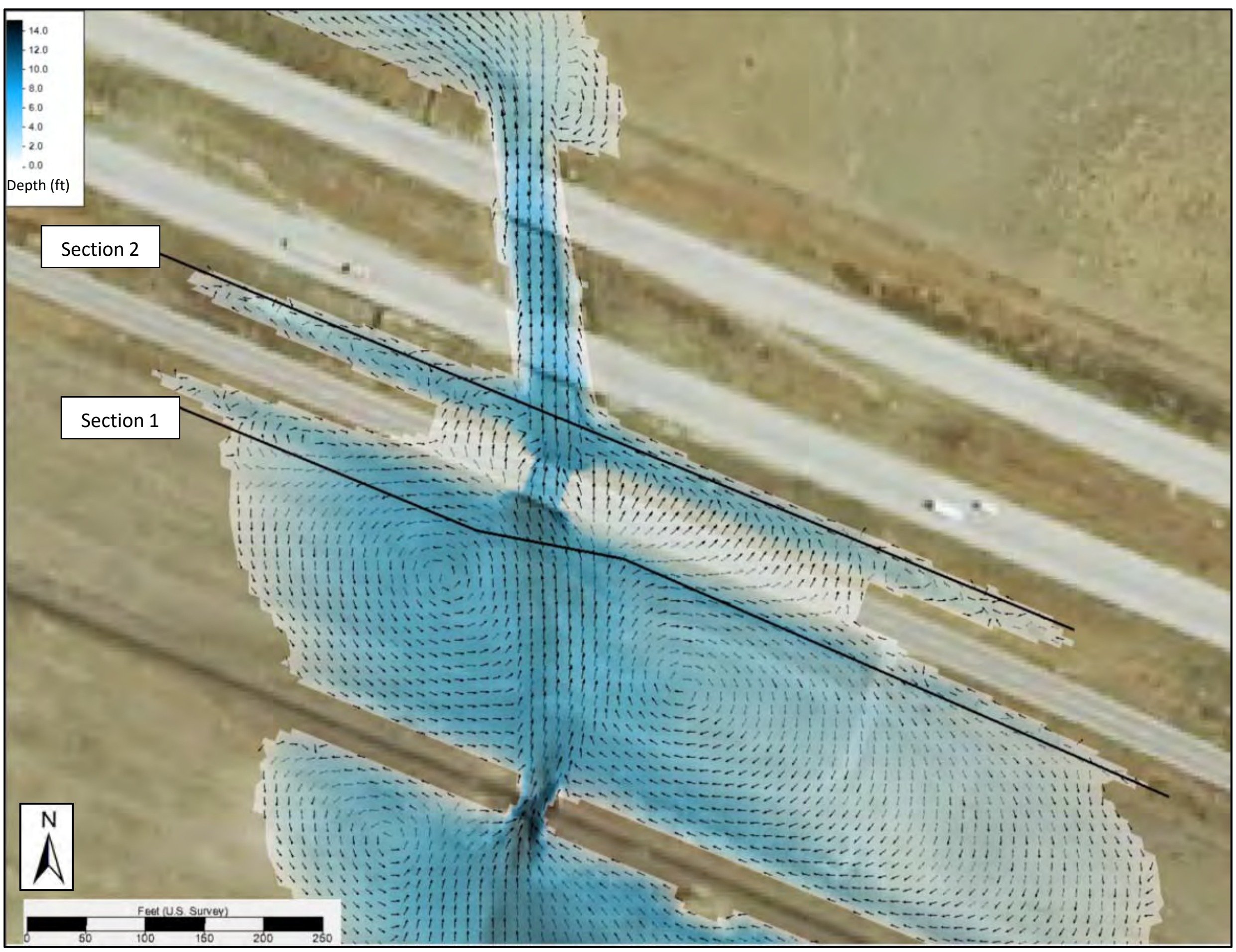


Figure 2. Existing Conditions 100-Year Depth

The ephemeral tributary spanned by Bridge F-20-L is controlled upstream by the UPRR bridge, and downstream by the I-70 bridges. Eddying is observed within the model between Bridge F-20-L and the upstream UPRR Bridge, as well as upstream of the UPRR bridge. Modeling of the existing conditions confirmed that no insurable structures are located within the floodplain for Bridge F-20-L and indicated that velocities through Bridge F-20-L would reach approximately 10.8 ft/s and 9.1 ft/s for the 100- and 500-year events, respectively. This decrease in velocity through Structure F-20-L between the 100- and 500-year events is due to the greater overtopping depth for the US 40 embankment and the induced tailwater from the I-70 bridges.

The downstream I-70 bridges are not overtopped, and do not experience pressure flow for events up to the 500-year event. The upstream UPRR embankment is not overtopped during the 100-year event but is overtopped during the 500-year event, just to the west of the UPRR bridge at a low point in the embankment.

4 DESIGN DISCUSSION

4.1 Replacement Consideration

The existing F-20-L bridge is 89 years old and decades past its design life. Bridge F-20-L requires frequent repair and closure and CDOT has determined that the bridge will be replaced.

4.2 Design Frequency and Floodplain Impacts

Bridge F-20-L 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-20-L Structure

Various bridge span length and grade raise combinations 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 to 100-feet and grade raises up to 5.2-feet. Existing conditions hydraulic modeling indicated that Bridge F-20-L has a tailwater induced on it by the downstream I-70 bridges and embankment. Based on water surface elevations near the I-70 bridges, a grade adjustment for US 40 was required to accommodate the induced tailwater and meet CDOT freeboard requirements at Bridge F-20-L. Design guidance from CDOT required that the proposed Bridge F-20-L not be skewed to aid in construction techniques and to limit road closure time during construction. Shifting the proposed bridge slightly east to better align with the skew of the downstream I-70 bridges was considered. However, this concept was not carried forward as shifting the bridge east from its current location would result in flow impacting the west abutment more directly and potentially creating scour concerns at that abutment, similar to the existing condition.

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 expanded by approximately 31-feet from each of the existing abutment backwalls. The proposed bridge has one 100-foot span from centerline-of-bearing to centerline-of-bearing, with a clear span of 97.5-feet. It is anticipated that the bridge superstructure will have a thickness of 53.1 inches, or 4.42 feet. The low chord elevation will be 5164.48 feet (NAVD-88) for the west and east abutments. The road grade of the proposed structure will be raised by approximately 5.2-feet, 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 100-foot span bridge with the 5.2-foot grade raise 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 obstruction representing the existing F-20-L bridge bent was 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 widening, grading within the channel, and proposed grade raise.

5.2 Proposed Conditions Hydraulic Modeling Results

A summary of the hydraulic properties immediately upstream and downstream of Structure F-20-L 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	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	5159.77	5159.62	3.76	1.02	3.49	0.91
50-Year	5162.10	5161.95	5.65	1.12	4.88	0.64
100-Year	5163.13	5162.91	6.68	1.32	5.53	0.57
500-Year	5165.19	5164.89	8.97	2.11	6.86	0.67

The proposed structure passes the 100-year event without experiencing pressure flow through the bridge opening or overtopping of US 40. During the 500-year event the structure experiences pressure flow but is not overtopped. **Figure 3** presents the 100-year water depth plot at the proposed structure.

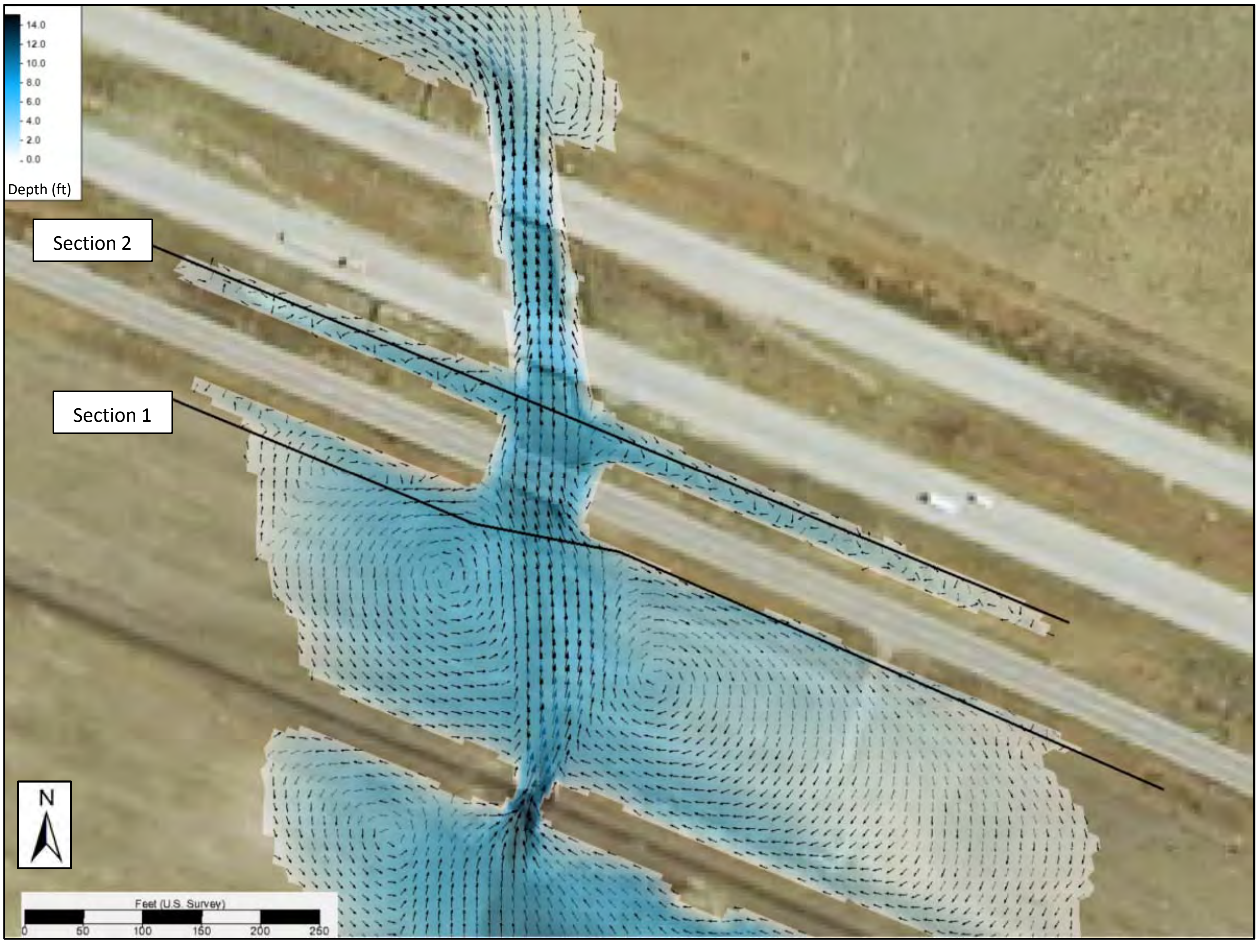


Figure 3. Proposed Conditions 100-Year Depth

In general, flow interacts with the proposed bridge in much the same way as the existing conditions bridge with the exception of the US 40 embankment not being overtopped. Eddying is noticed in the same locations as the existing conditions model. Velocities through the downstream I-70 bridges remain relatively the same as experienced in the existing conditions. The UPRR embankment is no longer overtopped during the 500-year event due to the US 40 embankment not being overtopped in the proposed condition and inducing as much of a tailwater on the upstream UPRR bridge.

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 for the 100-year event, and it was confirmed that no insurable structures were located within the proposed floodplain. In **Figure 4**, increases in WSEL for the 100-year event are shown as shades of red, while decreases in WSEL are shown as shades of blue. Green indicates WSEL changes less than ±0.01 foot. Between the UPRR embankment and Bridge F-20-L the floodplain is lowered by more than 1-foot during the 100-year event. This is due to the US-40 embankment not being overtopped and acting as a large weir controlling water surface elevations upstream. Upstream of the UPRR embankment water surface elevations also decrease compared to the existing conditions. Water surface elevations through the I-70 bridges are increased slightly but remain relatively the same with the hydraulic flow regimes remaining the same as in the existing conditions. Downstream of the I-70 embankment, water surface elevations remain relatively unchanged from the existing conditions.

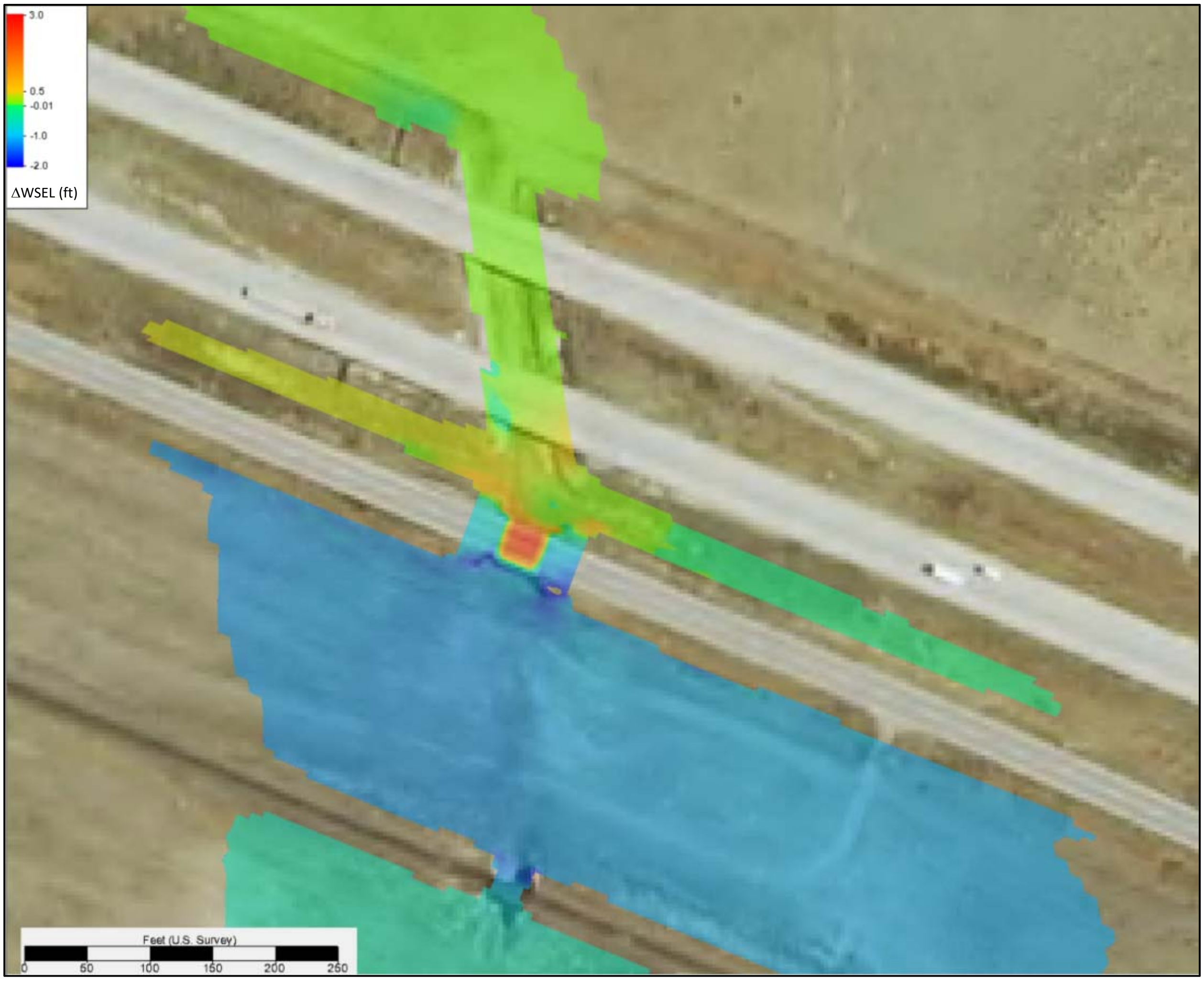


Figure 4. 100-Year Proposed vs Existing Water Surface Change

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 2,485 cfs with a mean velocity of 5.2 ft/s and a water surface elevation of 5163.13 (NAVD-88). The result of these calculations is a low chord elevation no lower than 5164.39 ft (NAVD-88). The Proposed design low chord is anticipated to be 5164.48 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-20-L flows through a low-plasticity clayey sand bed. Due to the alignment of the upstream UPPR railroad bridge, flows towards the US 40 bridge are skewed. The skewed flows are the probable cause of the 1 to 2 feet of scour under the exiting bridge, between the middle pile and the left abutment.

The stream is vegetated with dense grasses and weeds on the upstream and downstream ends of the bridge. Aerial imagery from 1953 was explored by Olsson and Associates to determine potential for stream migration throughout the years; however, because the channel is poorly defined and the floodplain rather wide, the only noticeable change to the stream was the construction of I-70. Olsson and Associates' geomorphic report identified the channel as having no readily apparent evidence of channel instability. Olsson did note that any scour experienced will be "directed primarily at the left bridge abutment."

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 only 21% of channel material passing the #200 sieve, indicates that the channel is composed of primarily sands. Due to the coarse nature of the channel sediment and higher flows during the 100-year and 500-year events, scour is expected to be live-bed.

5.3.3 Scour Variables

<u>Critical Velocity</u>

The velocity associated with the initiation of bed mobility, critical velocity, was determined based of the D_{50}, or reference particle size, of the channel bed. The critical velocity for the median diameter was determined to be 6.34 ft/s and 6.63 ft/s for the 100-year and 500-year storm events, respectively. Average velocities in the approach section upstream of the Bridge were taken from the SMS model to be 6.86 ft/s and 9.14 ft/s for the 100- and 500-yer events; because both velocities are larger than the critical velocity and the soils are non-cohesive in nature, live-bed scour dominates, and bed transport is anticipated.

<u>Contraction Scour</u>

The modified version of Laursen's 1960 equation for live-bed scour was utilized to determine contraction scour at the Bridge. The mode of transport material for the 100-year and 500-year scour events was determined to be mostly suspended bed material. The results of both storm events revealed a negative scour depth result, indicating that no scour hole is expected to form in the main channel at the bridge. Since the restriction of the upstream railroad bridge is approximately 70 feet smaller than the proposed constriction at the US 40 Bridge, water expands rapidly on the downstream end of the railroad bridge and begins to form shallow eddies along the left and right overbanks of the main channel. Due to the recirculation of these flows, contraction at the bridge is not as severe as would be expected if the channel were more incised and conveyed all flows uniformly and perpendicularly towards the Bridge. See **Appendix F** for additional contraction scour details and calculations.

Pressure Scour

The bridge is under pressure flow, but not overtopping, during the 500-year event and as such pressure scour calculations were carried out. Pressure scour is equal to the sum of the separation zone thickness (t) and average depth in the contracted section (y_2) less the vertical size of the bridge opening prior to scour (h_b). The bridge opening was determined by cutting a cross-section at the upstream face of the bridge and subtracting the invert elevation from the low-chord elevation. **Figure 5**, taken from HEC- 18, illustrates where each of the hydraulic variables mentioned above are located in relation to the bridge. It should be noted that $h_{b,}$ in a non-overtopping condition, is equal to the effective upstream channel flow depth for live-bed (h_{ue}), or as Equation 6.2 in HEC-18 refers to it—y_1. The proposed scour depth calculated at Bridge F-20-L utilizing the ratio of the upstream main channel bottom width to the contracted section's channel bottom width (W_1/W_2) resulted in a negative scour depth (- 0.05-ft); to test sensitivity and produce a more conservative pressure scour depth, the calculation was carried out assuming a ratio of 1 for W_1/W_2, which resulted in a scour depth of 0.85-ft. Additional details and the equation utilized to determine zone thickness can be found in the **Appendix F**.

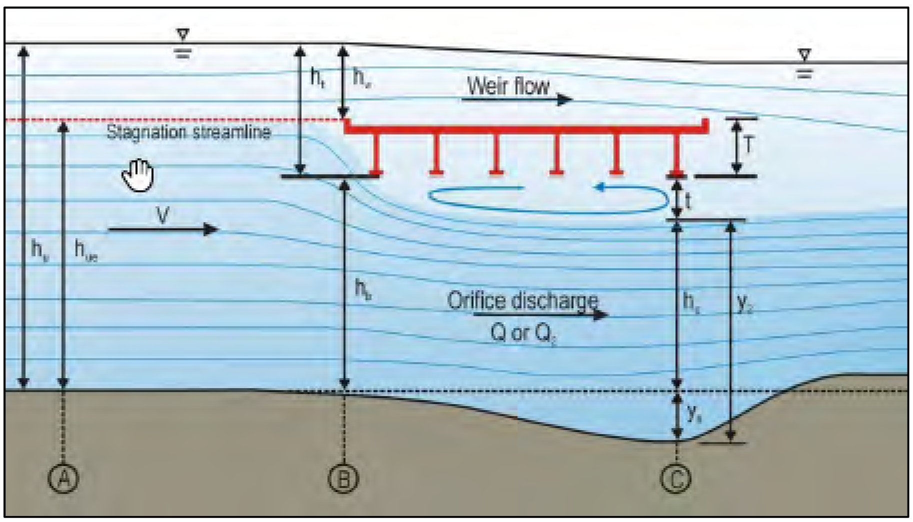


Figure 5. Pressure Scour: Hydraulic Variables

Live-Bed Abutment Scour

Initially, the NCHRP abutment scour approach for live-bed scour was used to determine abutment scour; the results of the NCHRP abutment calculations were negative, indicating that no abutment scour would be experienced. Due to the high flows during the 100- and 500-year events, skewed flow vectors approaching the bridge, and the 1-2-ft scour depth observed during site reconnaissance, these results did not seem realistic.

Froehlich's abutment scour equation was used to calculate an alternate abutment scour depth that is more realistic. These calculations resulted in 11.5-ft and 11.1-ft of scour for both the left and right abutments during the 100-year storm event. The 500-year event abutment scour depths are 14.8-ft and 14.3-ft for left and right abutments, respectively. See **Appendix F** for calculation details and **Table 6** for detailed scour results.

Scour Summary

The maximum scour depth elevation realized at the left abutment for the 100-year and 500-year events is 5142.50 and 5139.18. A summary of scour depths is provided in **Table 6** 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 6**.

Table 6. Proposed Condition Scour Results

Recurrence Interval	Flow Rate (cfs)	Critical Velocity (ft/s)	Contraction Scour	Left Abutment Scour	Right Abutment Scour	Pressure Scour
100-Year	2,485	6.34	0	11.45	11.10	———
500-Year	4,010	6.63	0	14.77	14.33	0.85*

* Pressure scour assumed $W_1/W_2 = 1$

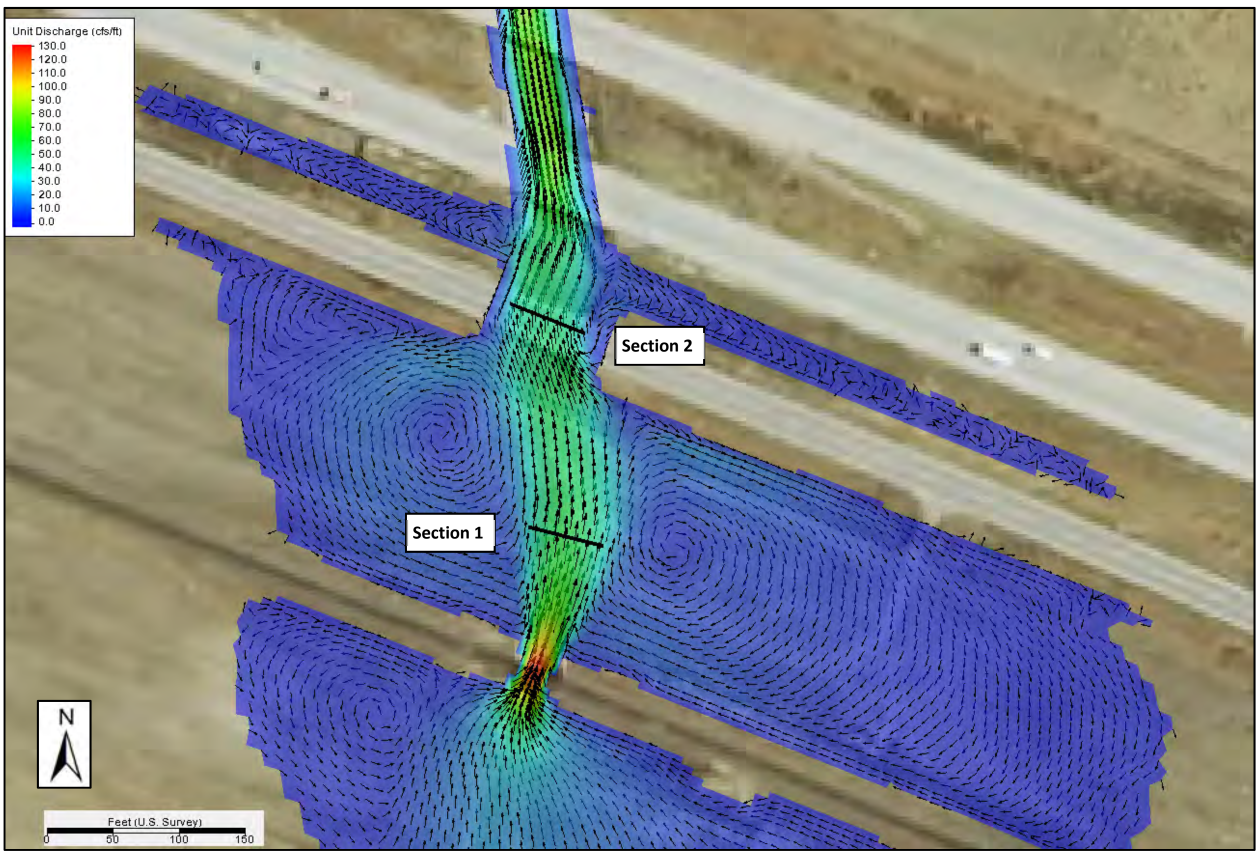


Figure 6. Approach and Contracted Sections with Unit Discharge

5.3.4 Proposed Scour Countermeasures

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.

While the 100-year riprap sizes for protection against scour has a D_{50} of 5.07 inches, the 500-year size for protection against scour was calculated as a D_{50} of 10-½ inches. However, to conform with standard riprap sizing it is recommended that 12-inch riprap be installed at a depth of 2-feet. Two different options are available for the protection. One option is riprap installed 2-ft below the channel invert, which will allow for the granular channel bed to naturally scour during the rising stage of a runoff event and fill during the falling stage, without restricting flow through the bridge during the falling stages. A second option of 12-inch riprap extended down the abutments at 2:1 to a depth of 11.5-ft was also considered. HEC-23 suggests that the riprap extend parallel to the channel bed to a width of $2Y_0$ or 16-ft, but this extension from left and right abutment would intersect at a distance less than 16-ft (approximately 13-ft). Because abutment scour is so deep, it may be more cost effective and easier to install riprap all the way across the channel then for the full 11.5-ft depth. However, since both options appear to be feasible, they are provided in **Figure 7** and **Figure 8**.

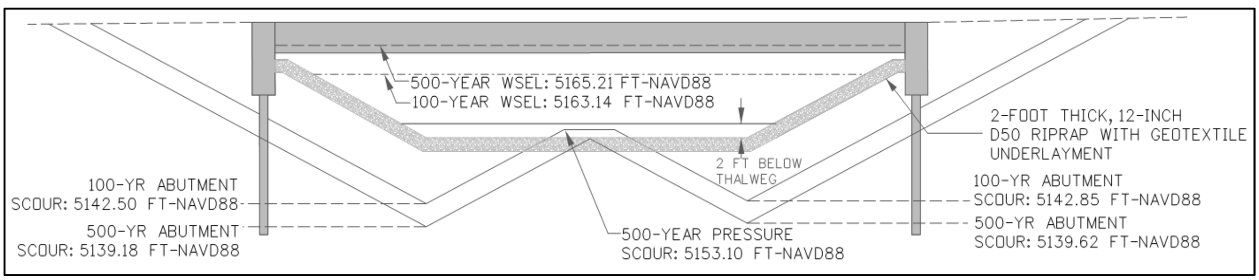


Figure 7. Riprap Armoring for Scour: Option 1

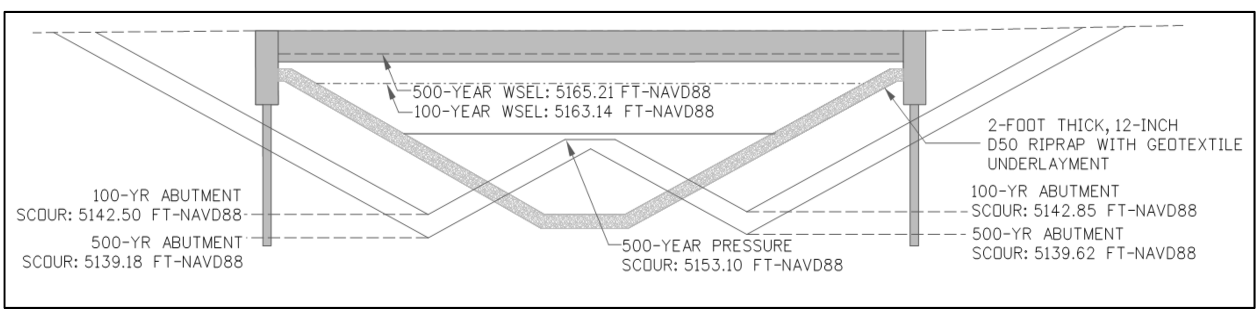


Figure 8. Riprap Armoring for Scour: Option 2

6 CONCLUSION

The existing F-20-L structure along US 40 spanning an unnamed ephemeral channel will be replaced by CDOT. This hydraulic analysis has concluded that a 100-foot, centerline-of-bearing to centerline-of-bearing, single span bridge, with a grade raise of approximately 5.2-feet at the 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 5142.50 ft and 5139.18 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 either a 2-foot thick section of 12-inch, riprap extending across the full channel bottom-width or a 2H:1V slope down to a depth of 11.5 feet.

7 REFERENCES

Arneson, L.A, Zevenbergen, L.W., Lagasse, P.F., and Clopper, P.E., 2013. "Evaluating Scour at Bridges, "Federal Highways Administration Hydraulic Engineering Circular 18, Fifth Edition, Washington, D.C.

Arneson, L.A, Zevenbergen, L.W., Lagasse, P.F., and Clopper, P.E., 2009. "Bridge Scour and Stream Instability Countermeasures, "Federal Highways Administration Hydraulic Engineering Circular 23, Volume 2, Washington, D.C.

Aquaveo LLC. (2020). Surface-Water Modeling System [Software], Version 13.0, Provo, Utah, 2011.

Chow, V.T., 1959. “Open Channel Hydraulics.” New York, McGraw-Hill Book Company.

Colorado Department of Transportation, “Drainage Design Manual”. 2004.

Federal Emergency Management Agency (FEMA), Flood Insurance Study of Arapahoe County, Colorado and Incorporated Areas. Flood Insurance Map Panel Number 08005C0325K, December 17, 2010.

“Geomorphic Assessment of Stream Stability, BE Bridge F-20-L (US36 MP321.288) over Unnamed Creek near Peoria, CO”, Project No. 017-1690. Olsson. September 23, 2019.

Soil Survey Staff, Natural Resources Conservation Service, United States Department of Agriculture. Web Soil Survey. Available online at the following link: https://websoilsurvey.sc.egov.usda.gov/. Accessed August, 2019.

U.S. Army Corps of Engineers (USACE). (2017). Hydrologic Modeling System HEC-HMS (Version 4.2.1) [Software]. Available from http://www.hec.usace.army.mil/software/hec-hms/downloads.aspx.

U.S. Bureau of Reclamation (USBR). (2019). Sedimentation and River Hydraulics – Two-Dimension (Version 3.2.4) [Software]. Available from http://www.usbr.gov/techreferences/computer/models/srh2d/index.html.

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. West Abutment Backwall (Upstream) Looking North

Photograph 10. East Abutment Backwall (Upstream) Looking East

Photograph 11. Bridge Bent Looking North

Photograph 12. Upstream UPRR Bridge Looking Southwest

Photograph 13. Upstream Face of Downstream West Bound I-70 Bridge Looking North

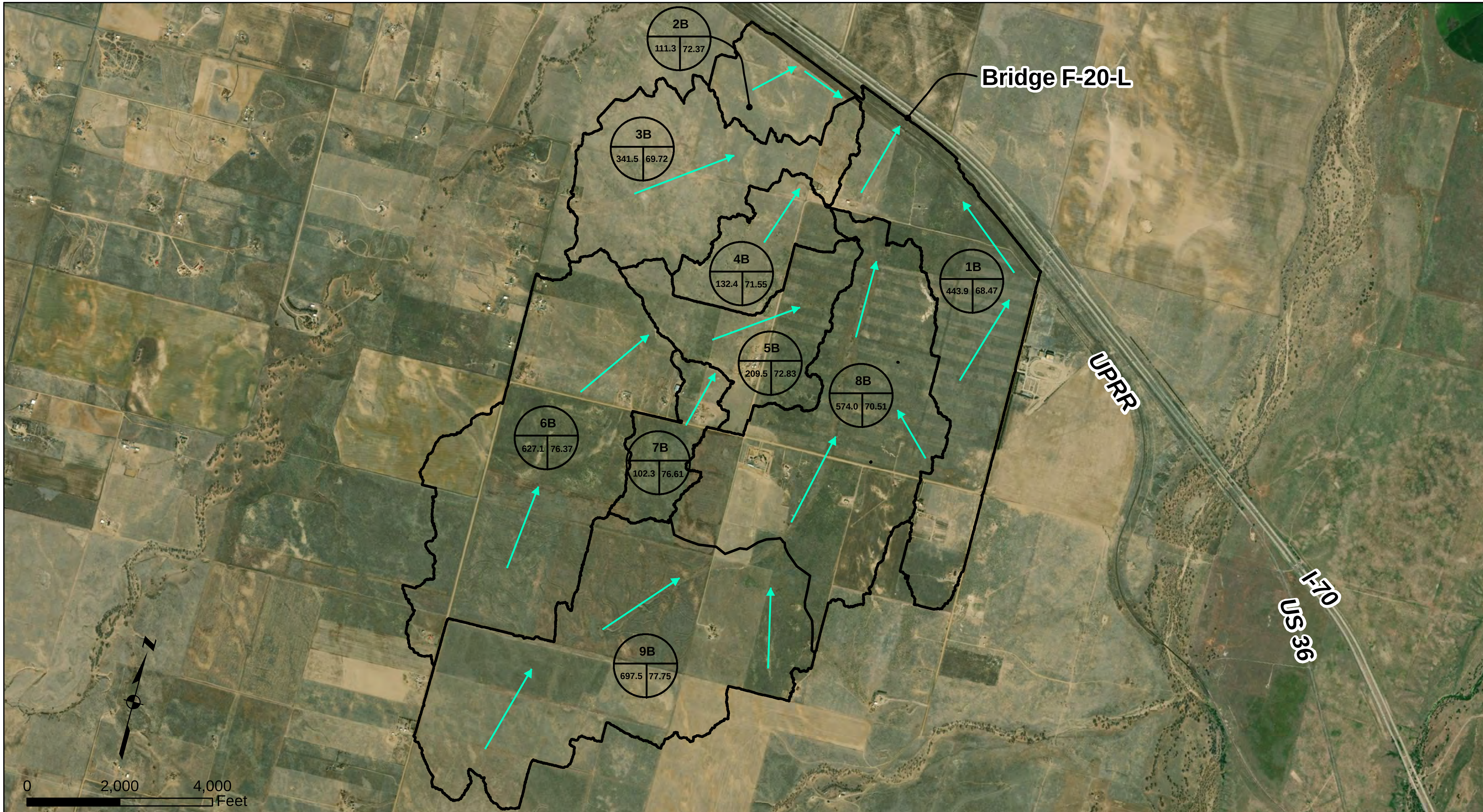
Photograph 14. Upstream Face of Downstream East Bound I-70 Bridge Looking Northeast

Photograph 15. Upstream UPRR Embankment and Floodplain Looking Southeast

Photograph 16. Downstream Floodplain Looking Northwest

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-20-L 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

9 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	0.0318	1.91	0.615852
2B	300	0.24	0.0303	1.91	0.627871
3B	300	0.24	0.0255	1.91	0.672715
4B	300	0.24	0.0416	1.91	0.553108
5B	300	0.24	0.0383	1.91	0.5717
6B	300	0.24	0.0258	1.91	0.669575
7B	300	0.24	0.0429	1.91	0.546342
8B	300	0.24	0.0451	1.91	0.535521
10B	300	0.24	0.0416	1.91	0.553108

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	1640	0.0138	1.8953746	0.240351
2B	2325	0.018	2.1646703	0.298352
3B	2367	0.018	2.1646703	0.303741
4B	1595	0.039	3.1863084	0.13905
5B	1717	0.032	2.8862271	0.165248
6B	4486	0.019	2.2239873	0.560305
7B	1398	0.0109	1.6844913	0.230534
8B	2262	0.0114	1.7226932	0.364739
10B	818	0.0146	1.9495391	0.116552

Channel Flow

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

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	10	1.18	1.5	17.655	14.64022	1.205925	0.048	0.0026	1.793268	8665	1.342211
2B	3	4.43	1	7.43	12.08293	0.614917	0.048	0.0032	1.269788	11625	2.543076
3B	10	9.25	1	19.25	28.60779	0.672894	0.048	0.0036	1.43019	10121	1.965744
4B	6.5	6.75	0.75	8.671875	16.73551	0.518172	0.048	0.0027	1.040582	8418	2.24714
5B	3	15.5	1	18.5	34.06445	0.543088	0.048	0.0018	0.876654	9127	2.891994

6B	2.5	5	1	7.5	12.69804	0.590642	0.048	0.0047	1.49811	6670	1.236743
7B	1.2	5.9	1	7.1	13.16829	0.539174	0.048	0.0043	1.348441	12940	2.66563
8B	3.2	5	0.5	2.85	8.29902	0.343414	0.048	0.0051	1.087115	18360	4.691316
10B	15	5	0.75	14.0625	22.64853	0.620901	0.048	0.0211	3.281718	6709	0.567877

Lag Time

$T_L(hr) = 0.6 * T_C$

Sub-Basin ID	T_C (hr)	T_L (hr)
1B	2.1984146	1.31904877
2B	3.4692989	2.08157933
3B	2.9422012	1.76532069
4B	2.9392978	1.76357865
5B	3.6289418	2.17736506
6B	2.4666238	1.47997429
7B	3.4425065	2.06550392
8B	5.5915761	3.35494567
10B	1.2375365	0.74252191

Muskingum-Cunge Flow Routing

Reach	Reach Length (ft)	Energy Slope (ft/ft)	Bottom Width (ft)	Side Slope z	Manning n
6R	10449	0.00409	10	9.25	0.048
7R	7643	0.00514	2.5	5	0.048
2R	3409	0.00391	10	1.18	0.048
4R	7237	0.00572	3	4.43	0.048
10R	9171	0.00414	3	4.33	0.048

RESULTS

Recurrence Flood	Qpeak (cfs)
10 yr	515
50 yr	1570
100 yr	2255
500 yr	4385

AREA

16.4 mi^2

National Flood Hazard Layer FIRMette

FEMA

39°41'29.54"N

104°8'2.37"W

104°7'24.91"W

39°41'1.85"N

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

1:6,000

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 1/21/2020 at 11:04:42 AM 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

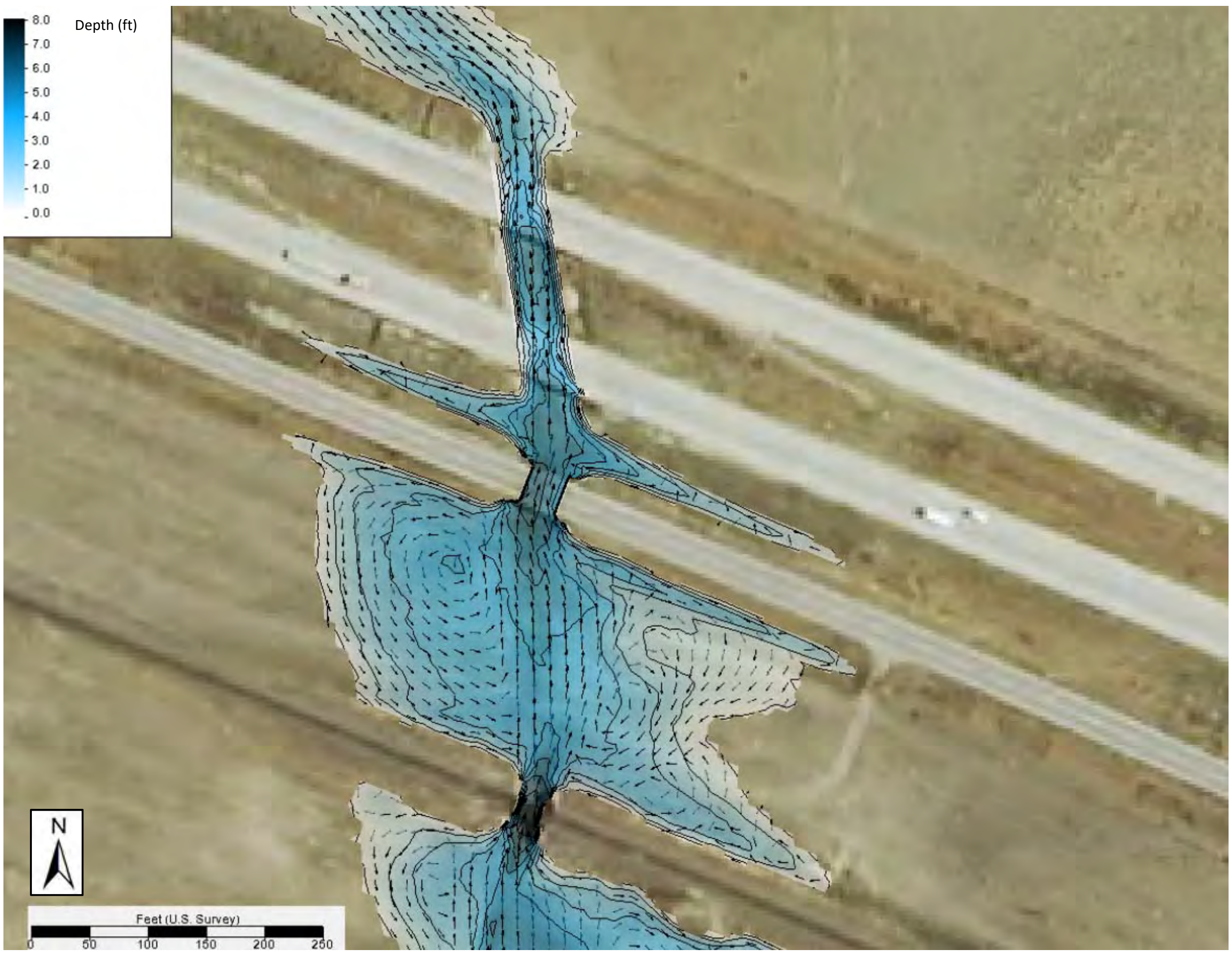


Figure 1. Existing Conditions Depth, 10-Year

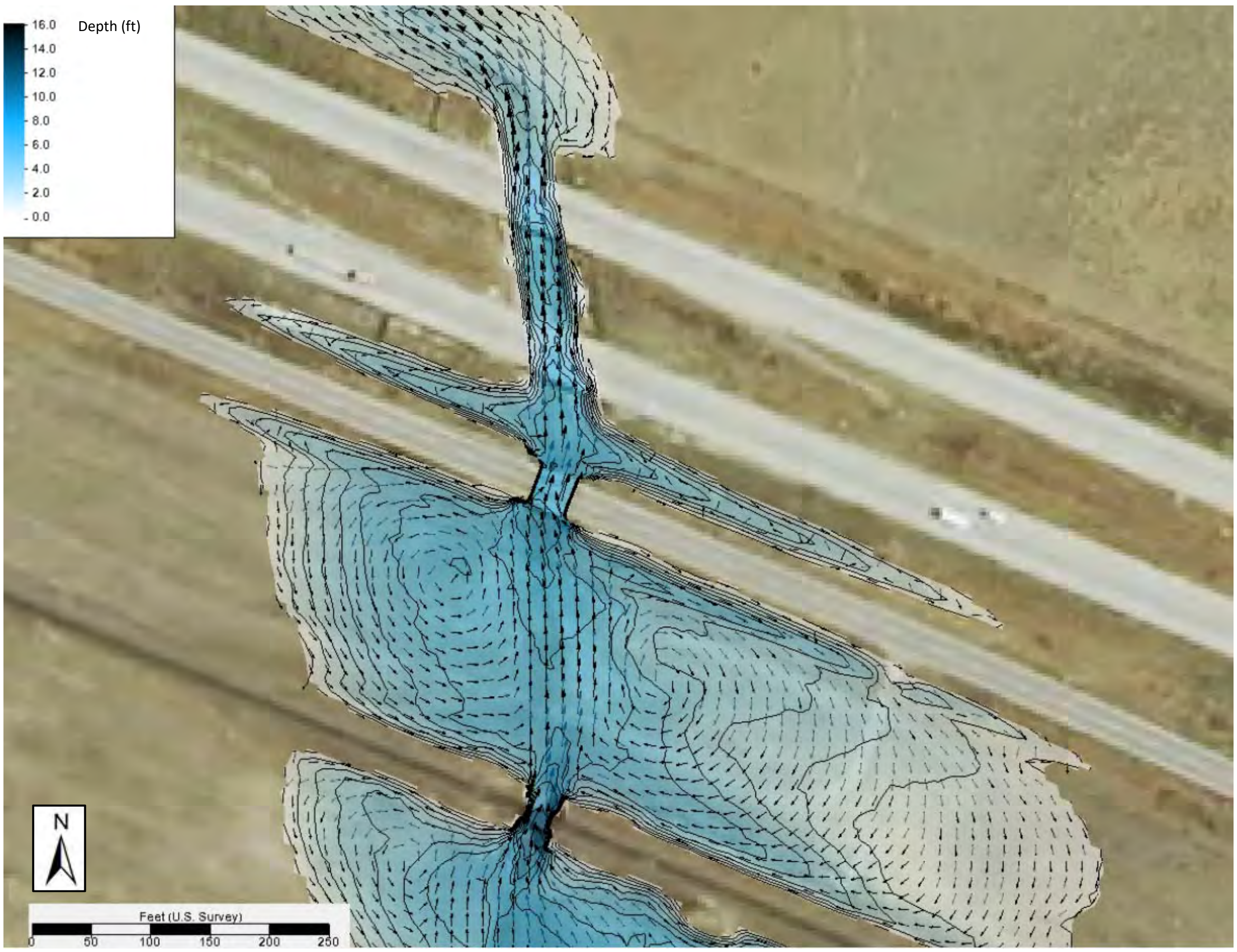


Figure 2. Existing Conditions Depth, 50-Year

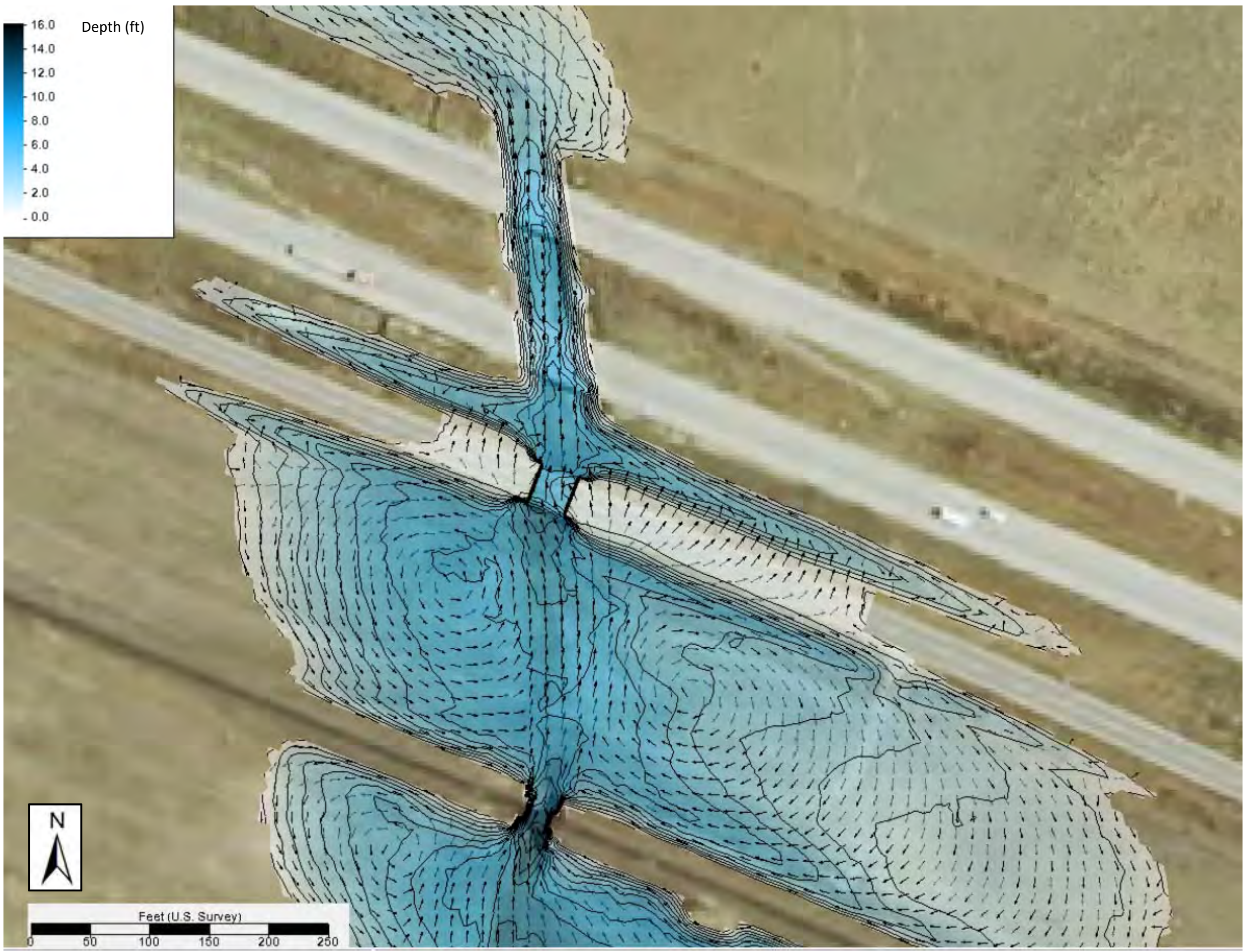


Figure 3. Existing Conditions Depth, 100-Year

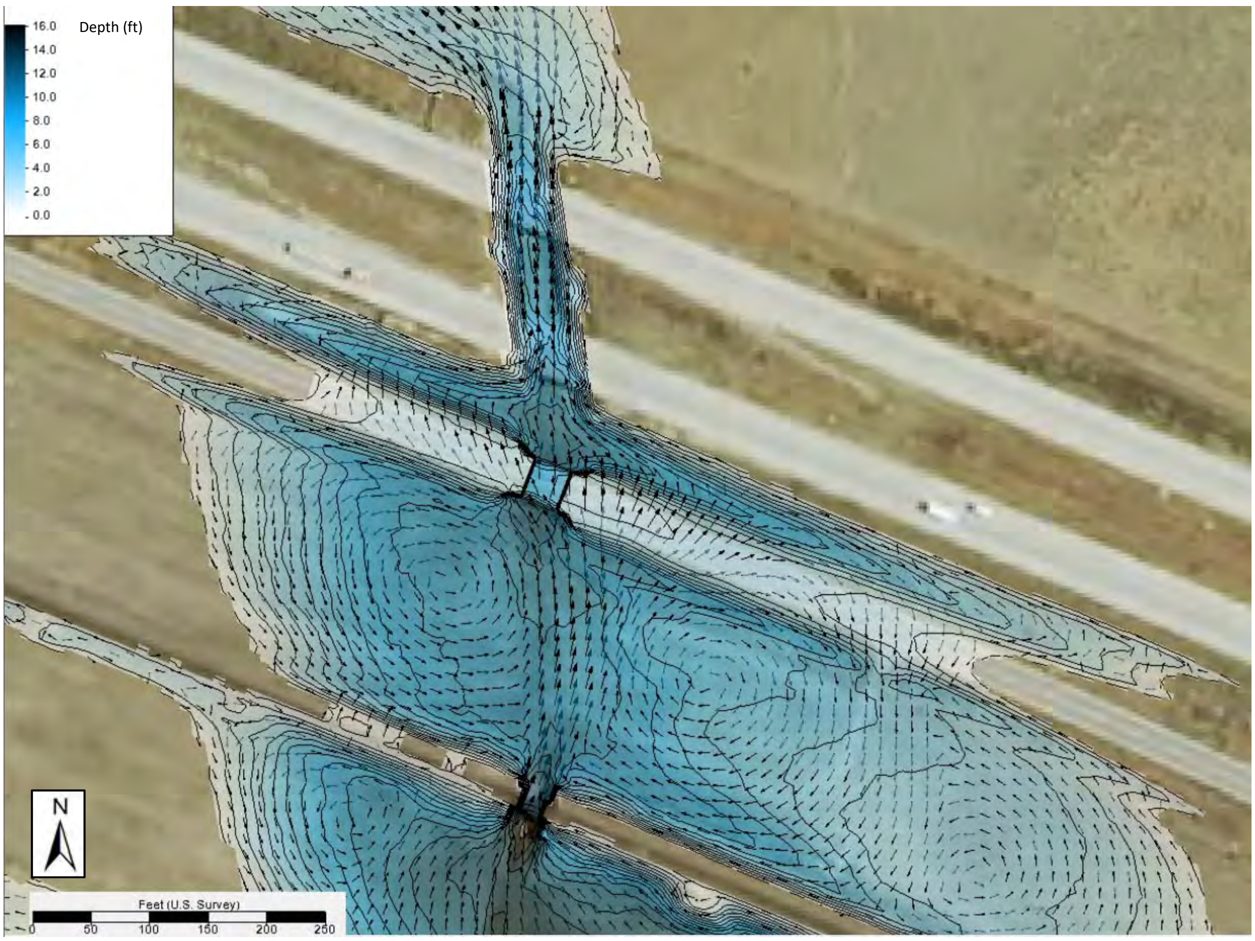


Figure 4. Existing Conditions Depth, 500-Year

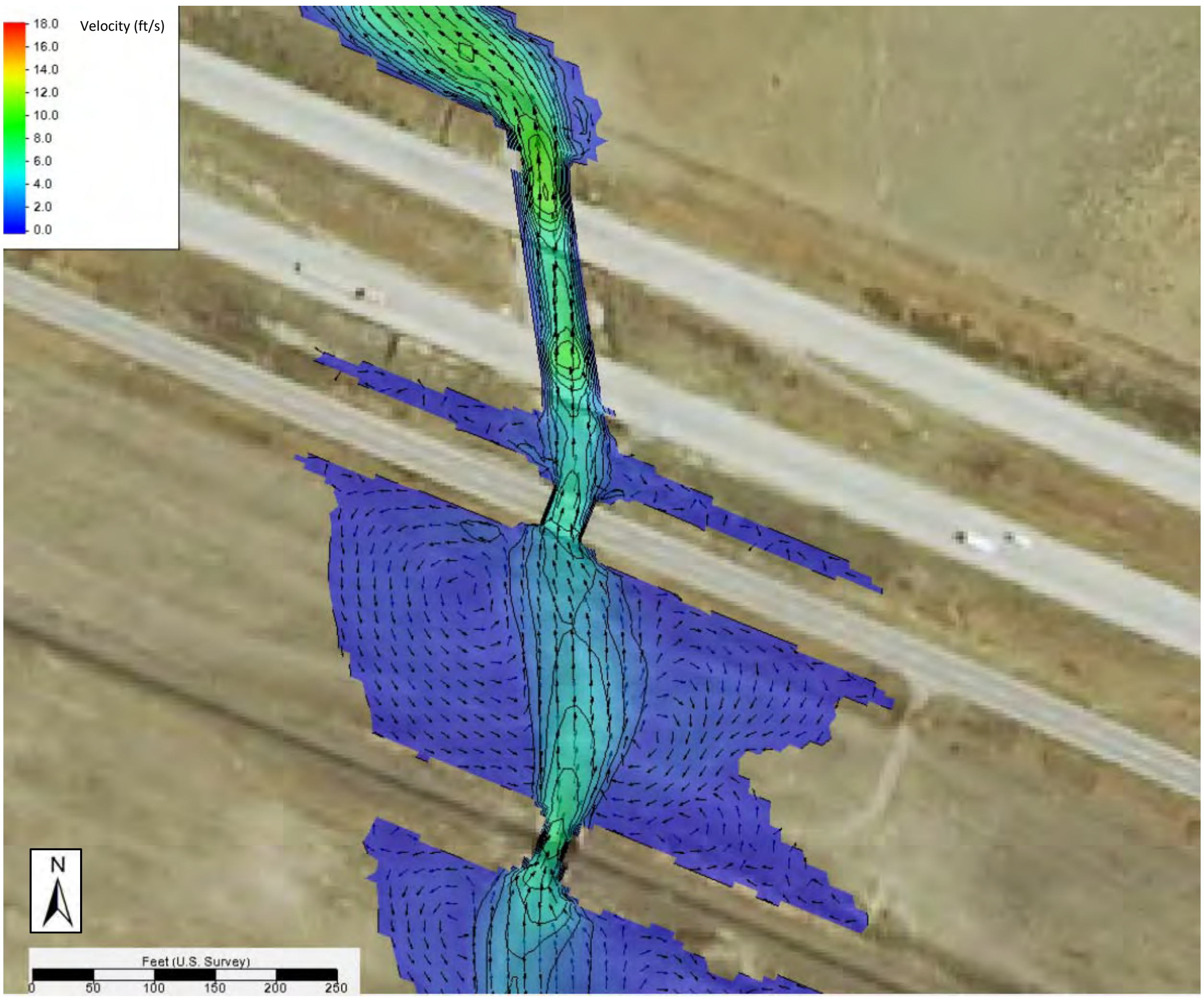


Figure 5. Existing Conditions Velocity, 10-Year

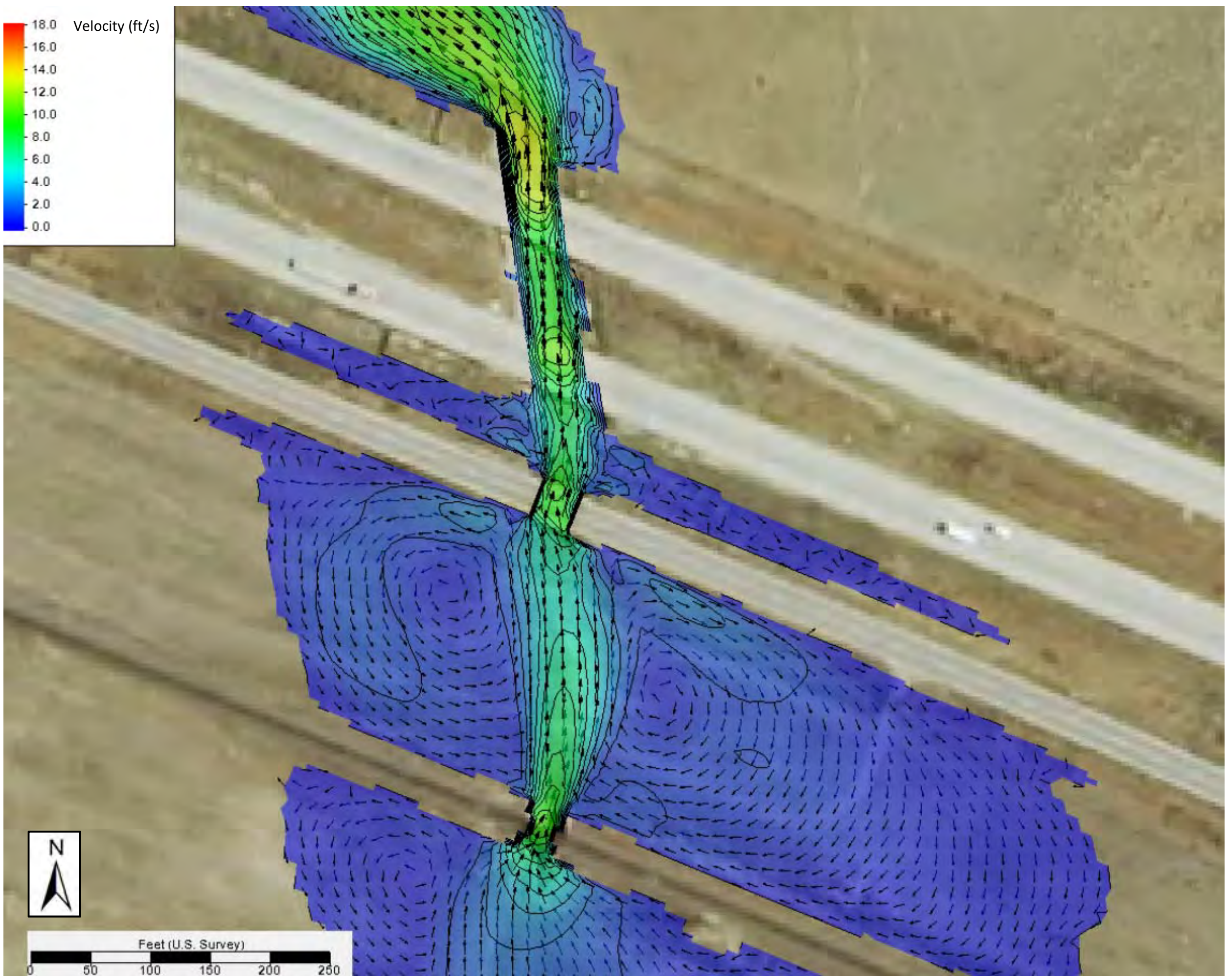


Figure 6. Existing Conditions Velocity, 50-Year

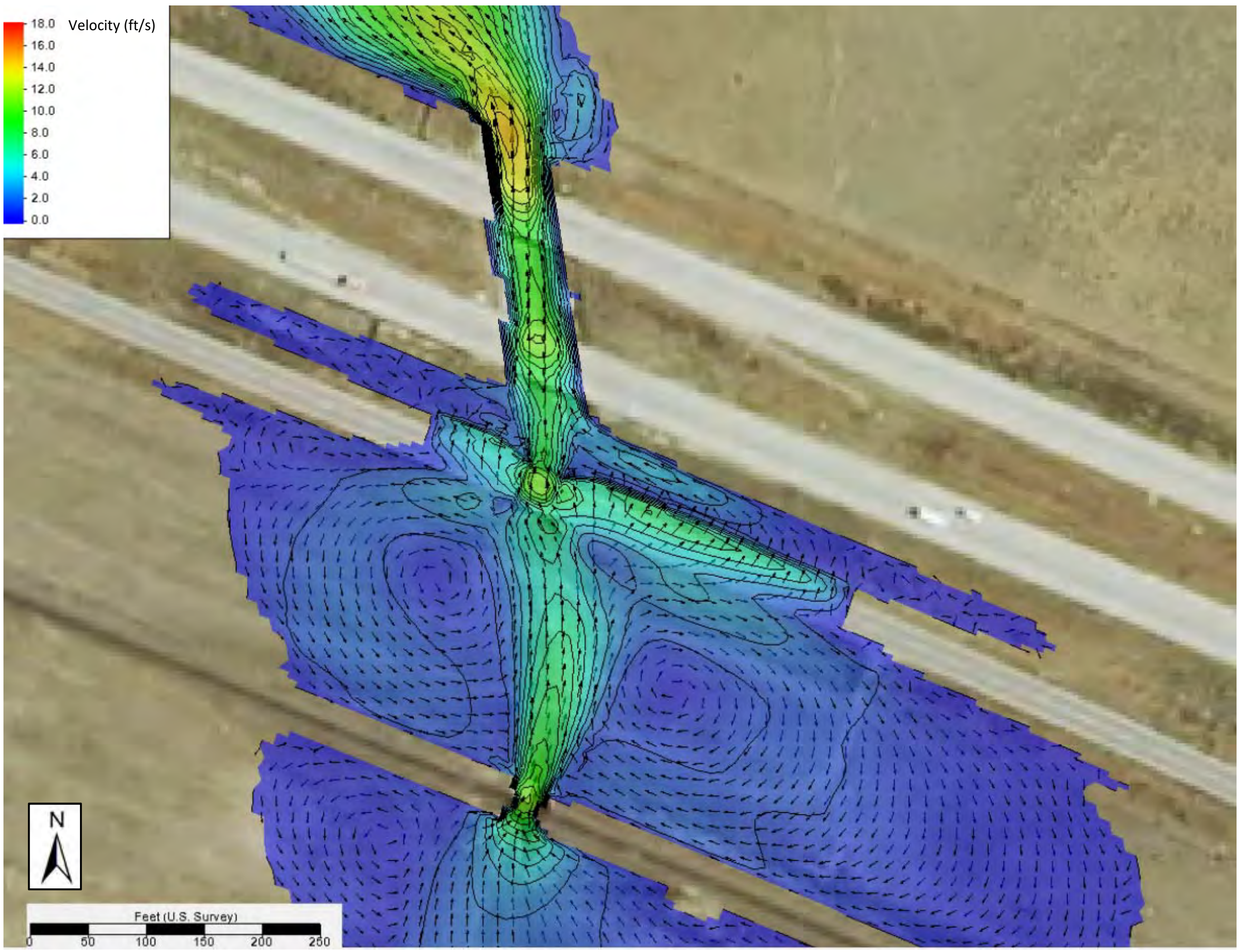


Figure 7. Existing Conditions Velocity, 100-Year

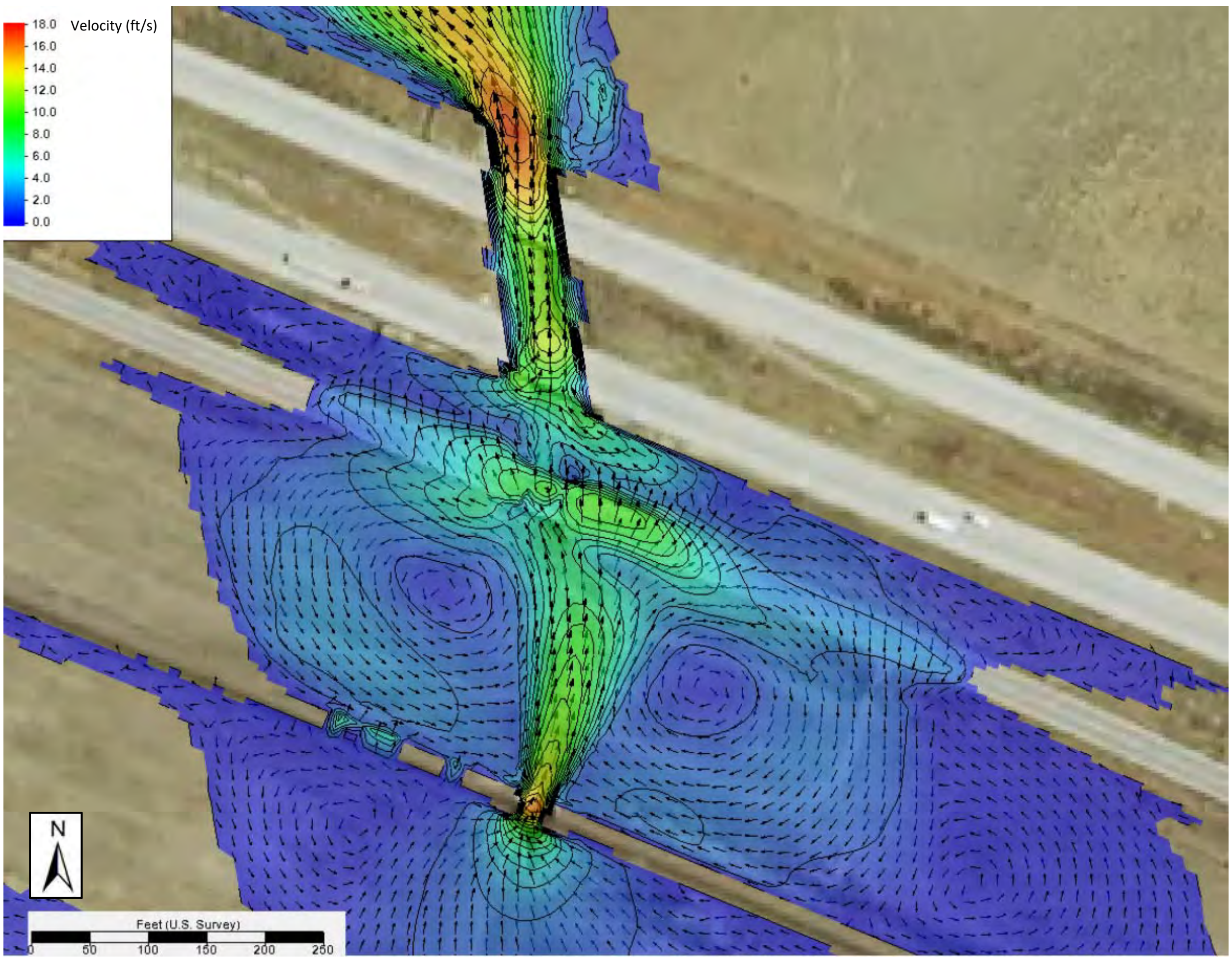


Figure 8. Existing Conditions Velocity, 500-Year

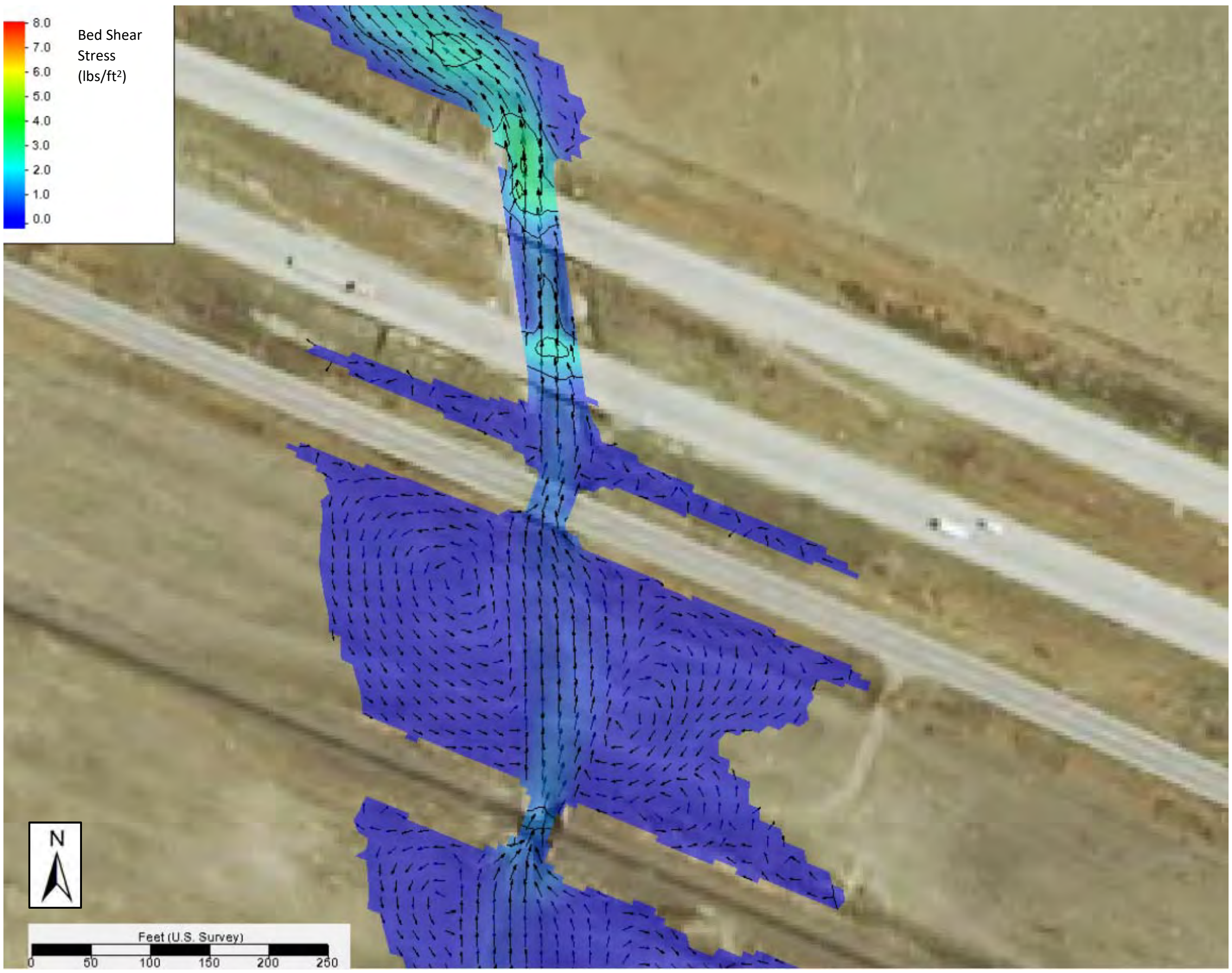


Figure 9. Existing Conditions Bed Shear Stress, 10-Year

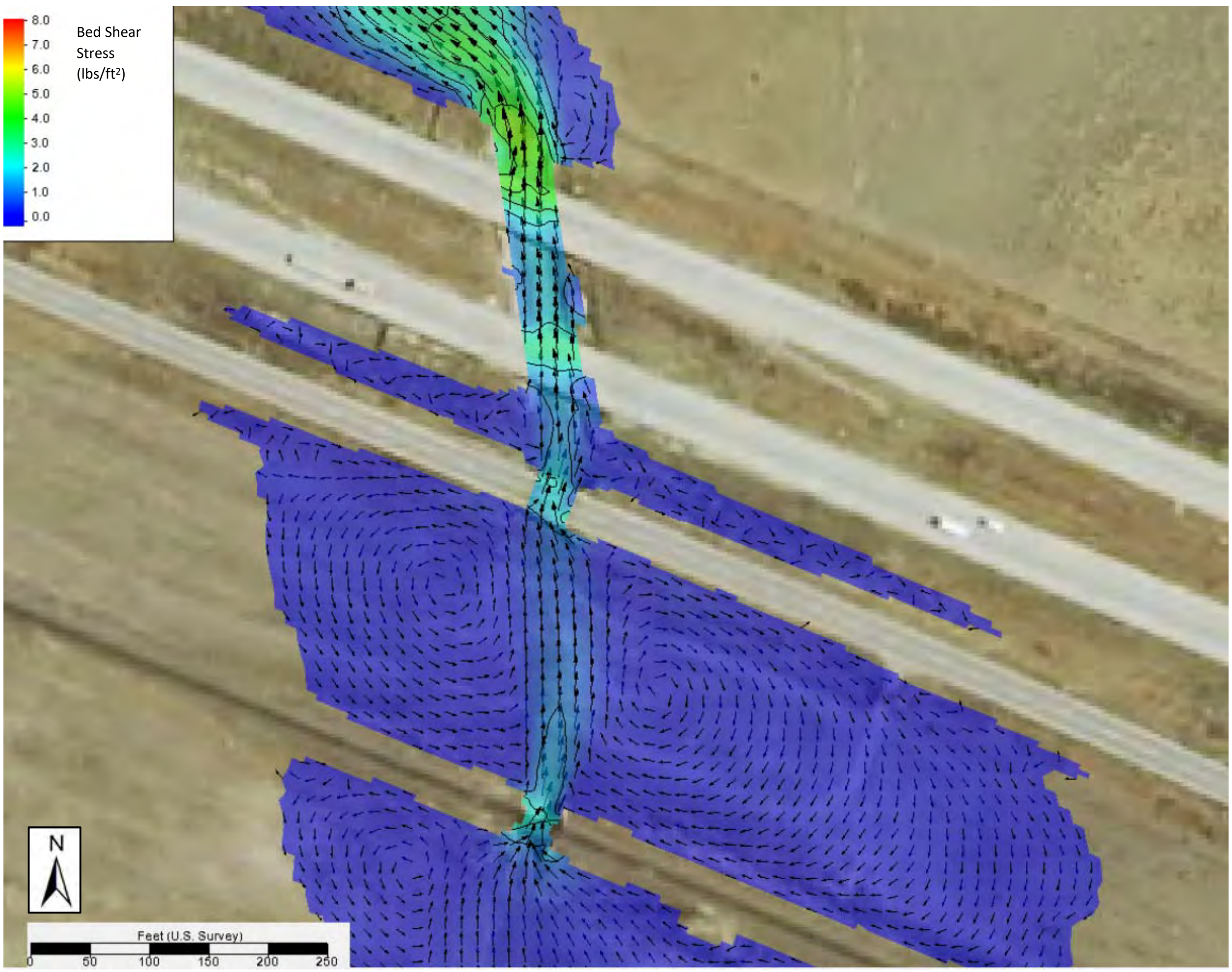


Figure 10. Existing Conditions Bed Shear Stress, 50-Year

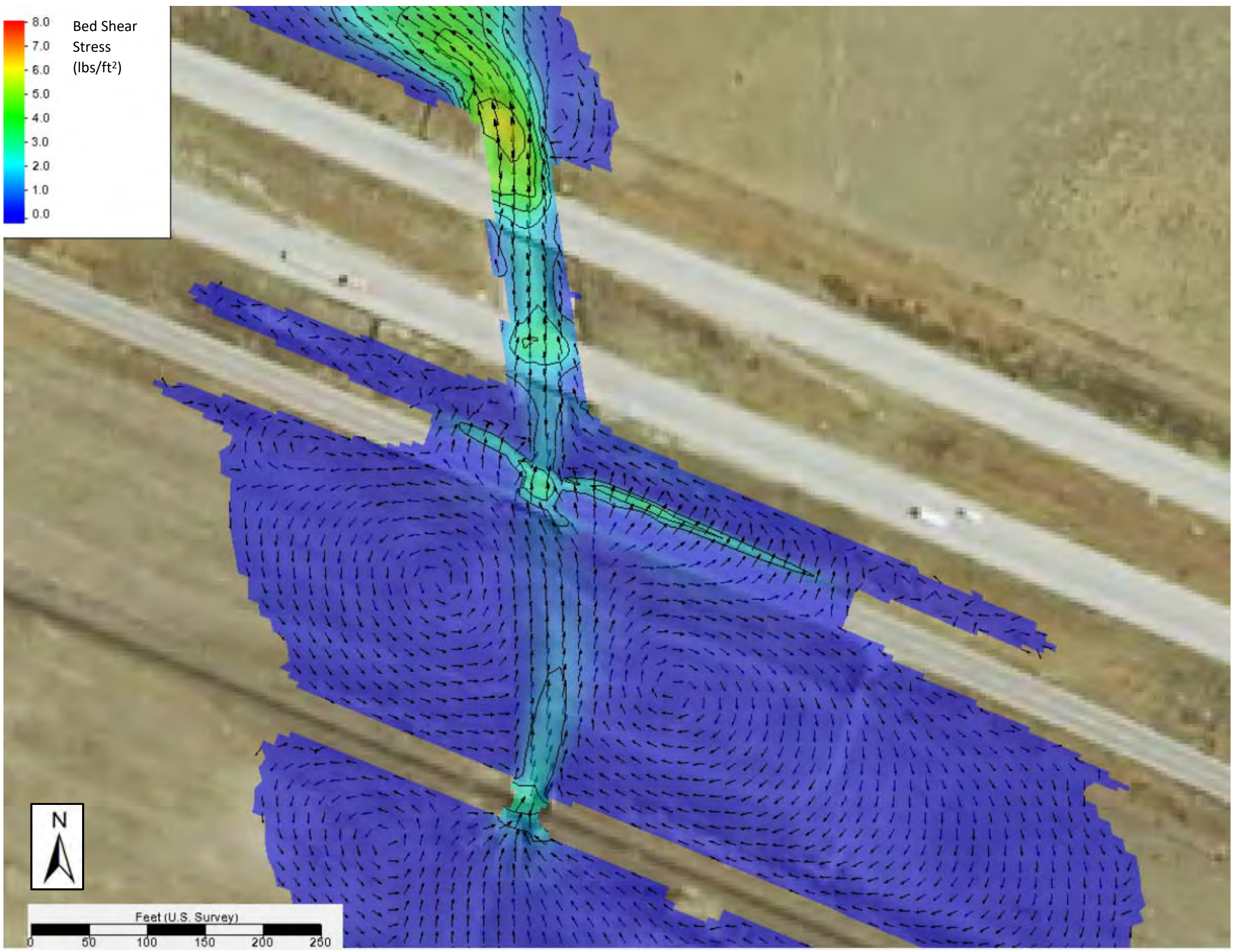


Figure 11. Existing Conditions Bed Shear Stress, 100-Year

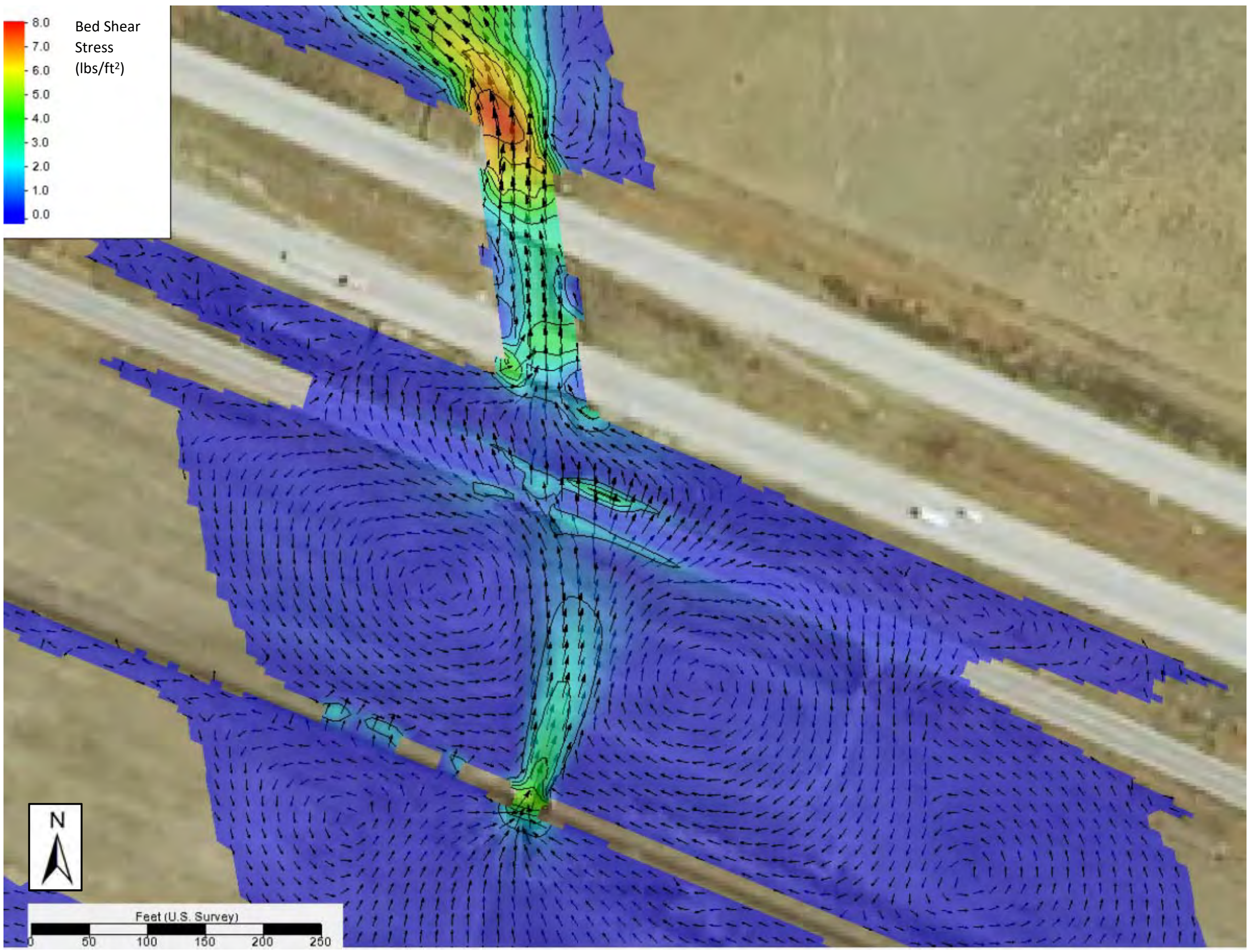


Figure 12. Existing Conditions Bed Shear Stress, 500-Year

Appendix D

Proposed Conditions Hydraulic Modeling Results

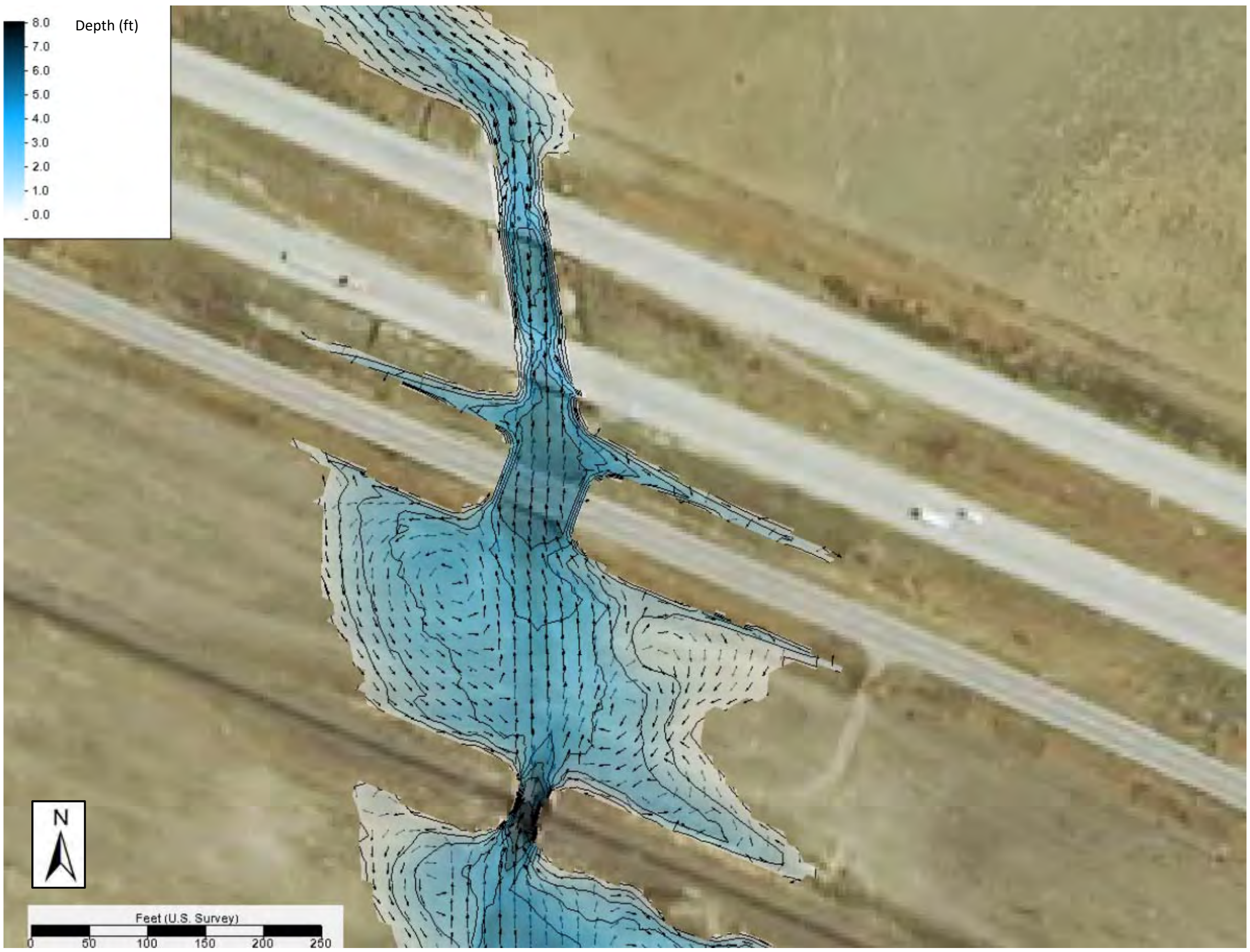


Figure 1. Proposed Conditions Depth, 10-Year

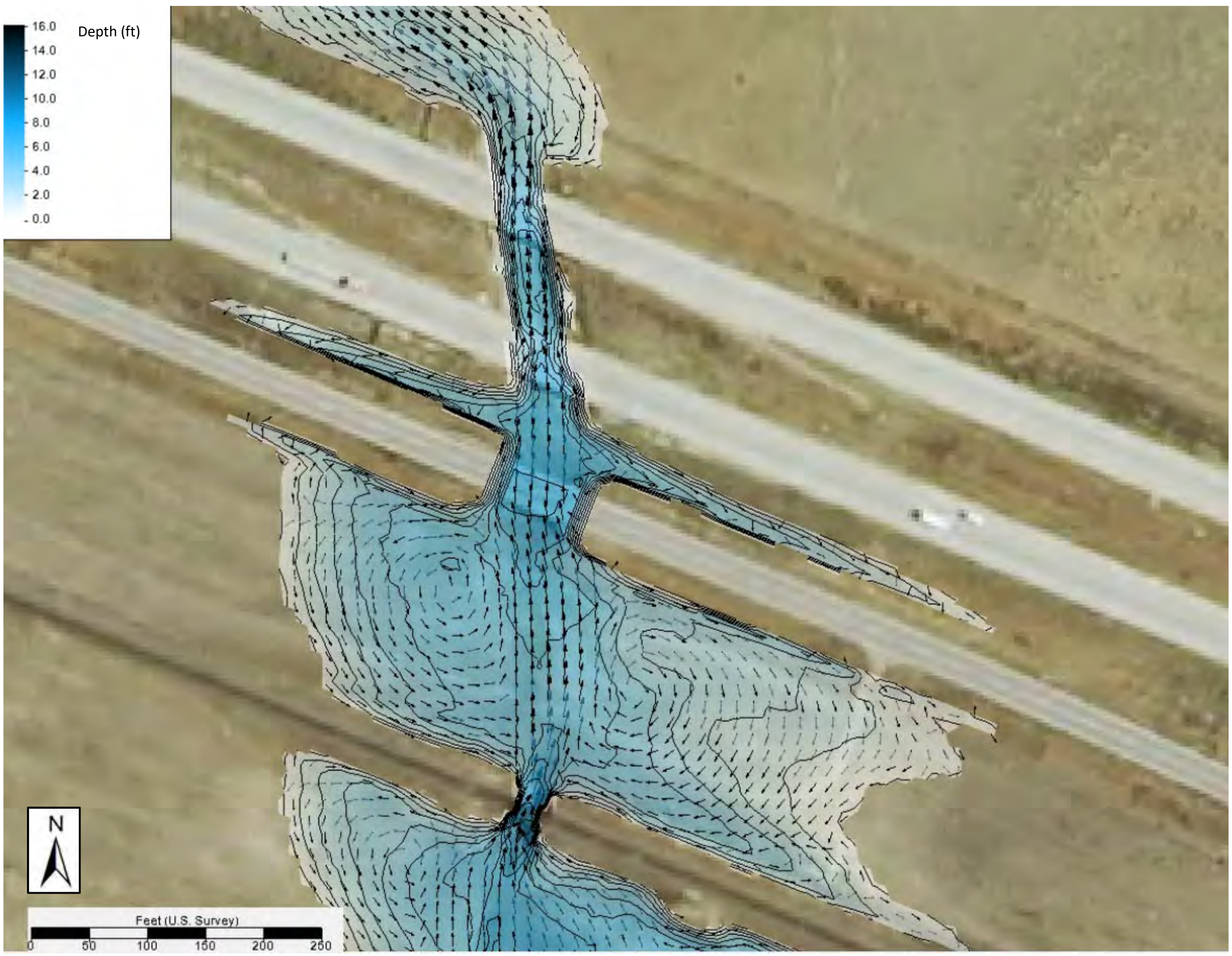


Figure 2. Proposed Conditions Depth, 50-Year

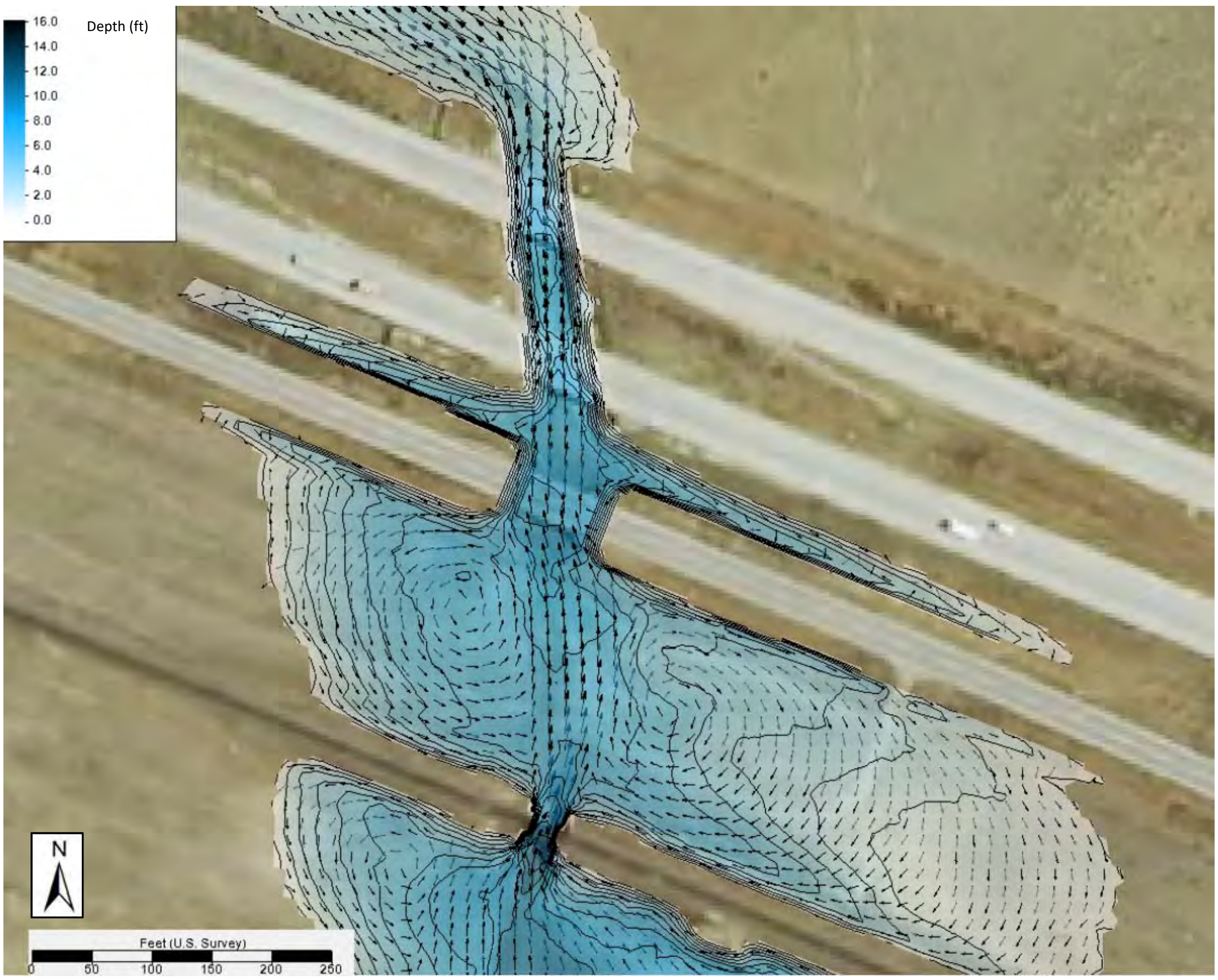


Figure 3. Proposed Conditions Depth, 100-Year

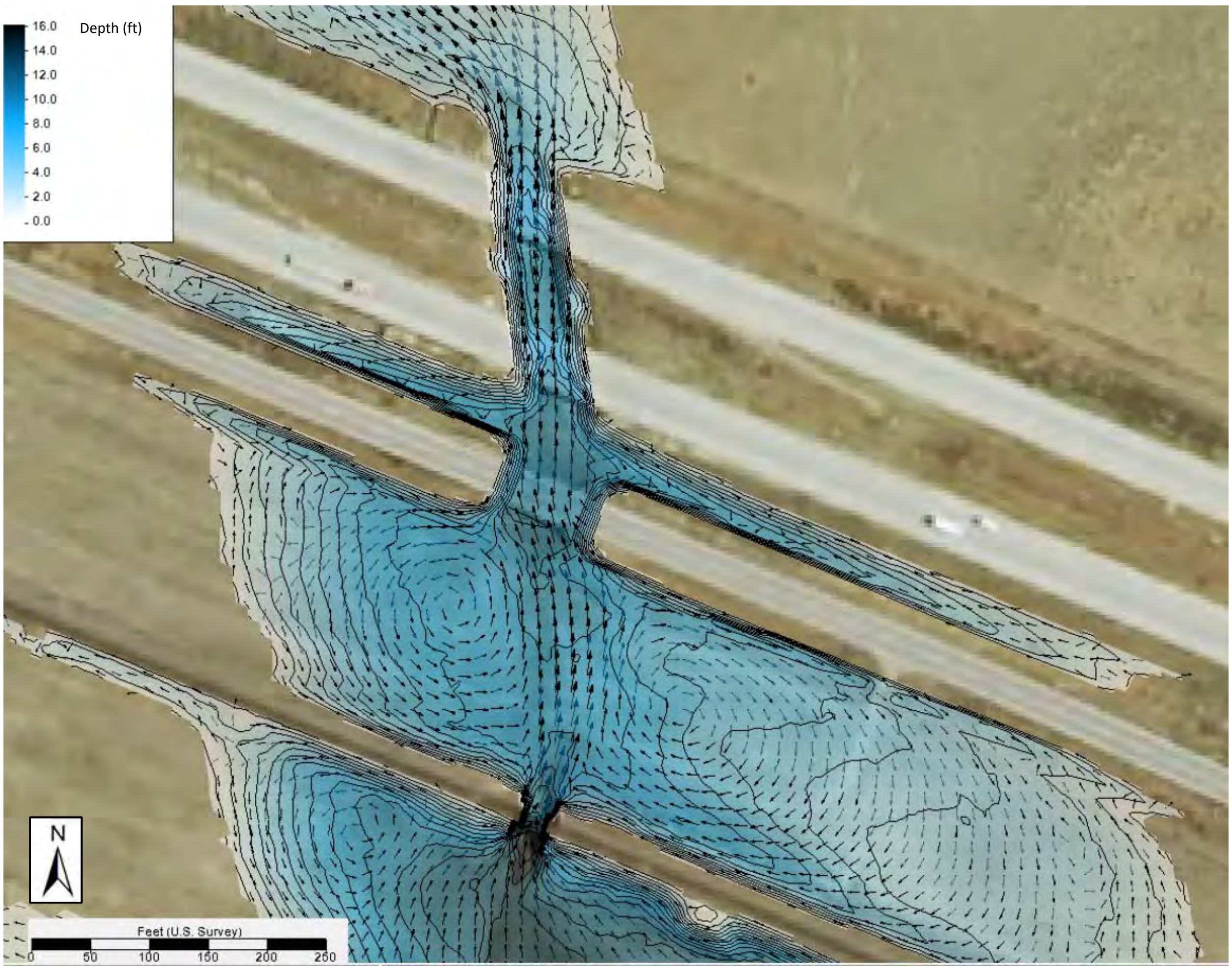


Figure 4. Proposed Conditions Depth, 500-Year

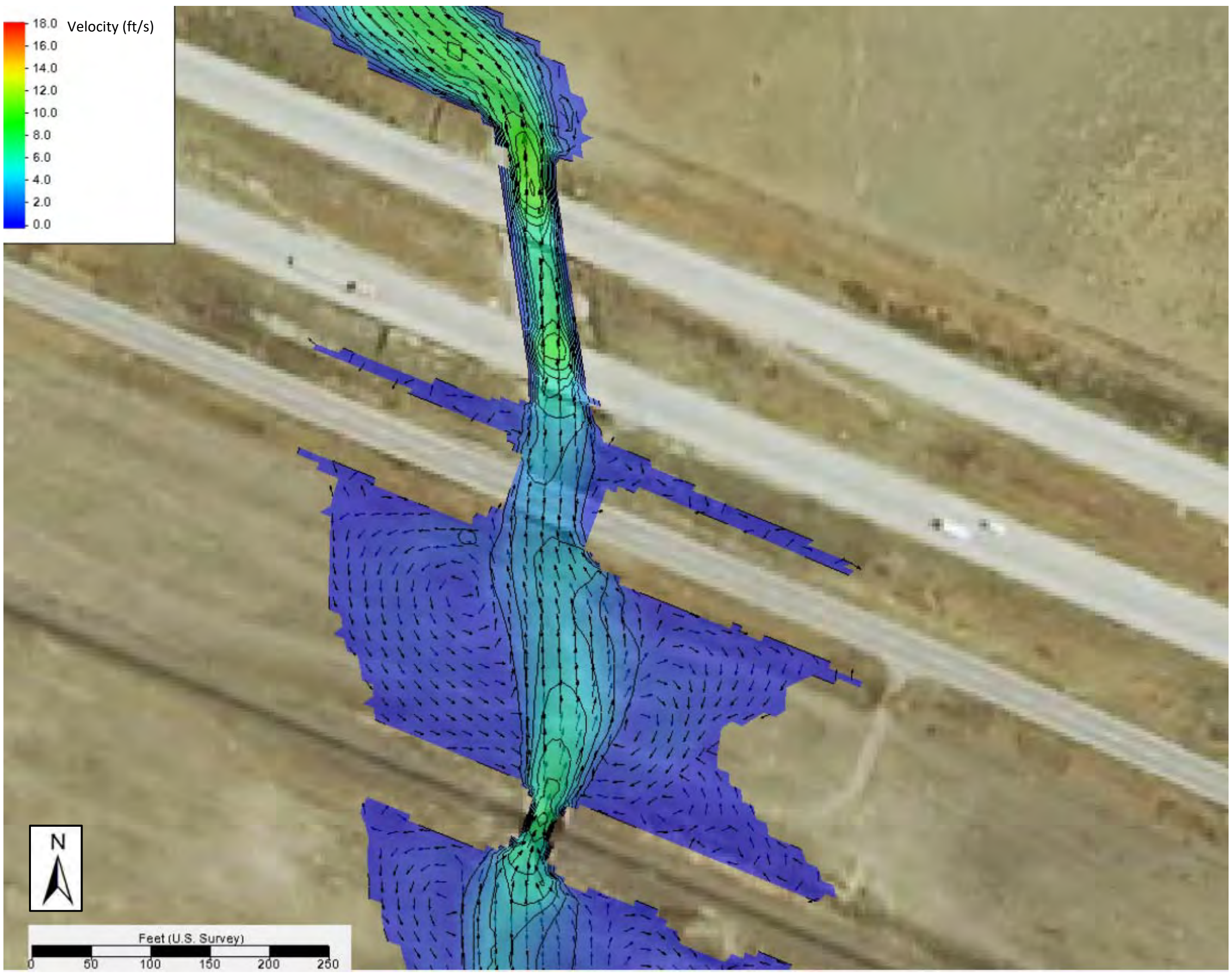


Figure 5. Proposed Conditions Velocity, 10-Year

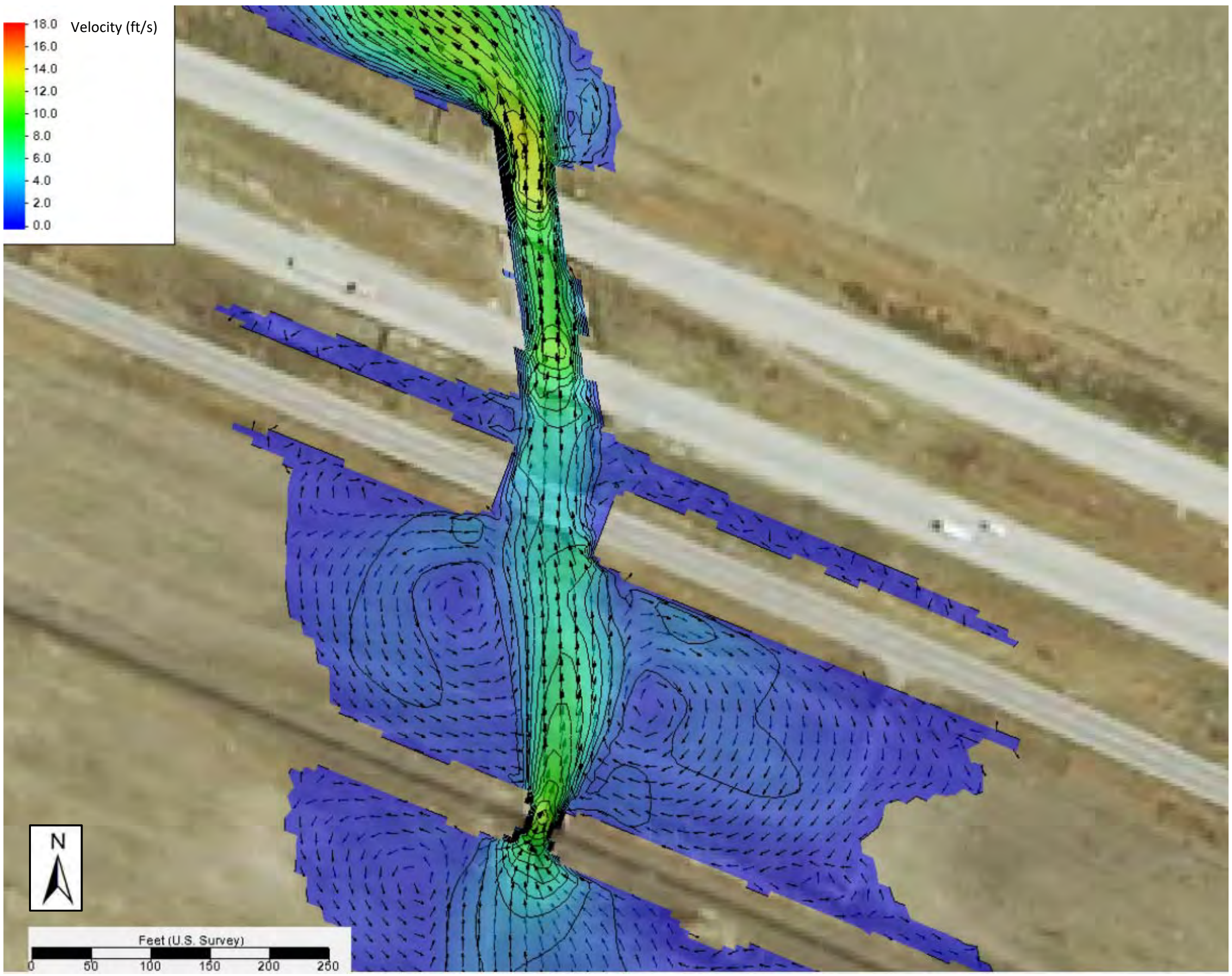


Figure 6. Proposed Conditions Velocity, 50-Year

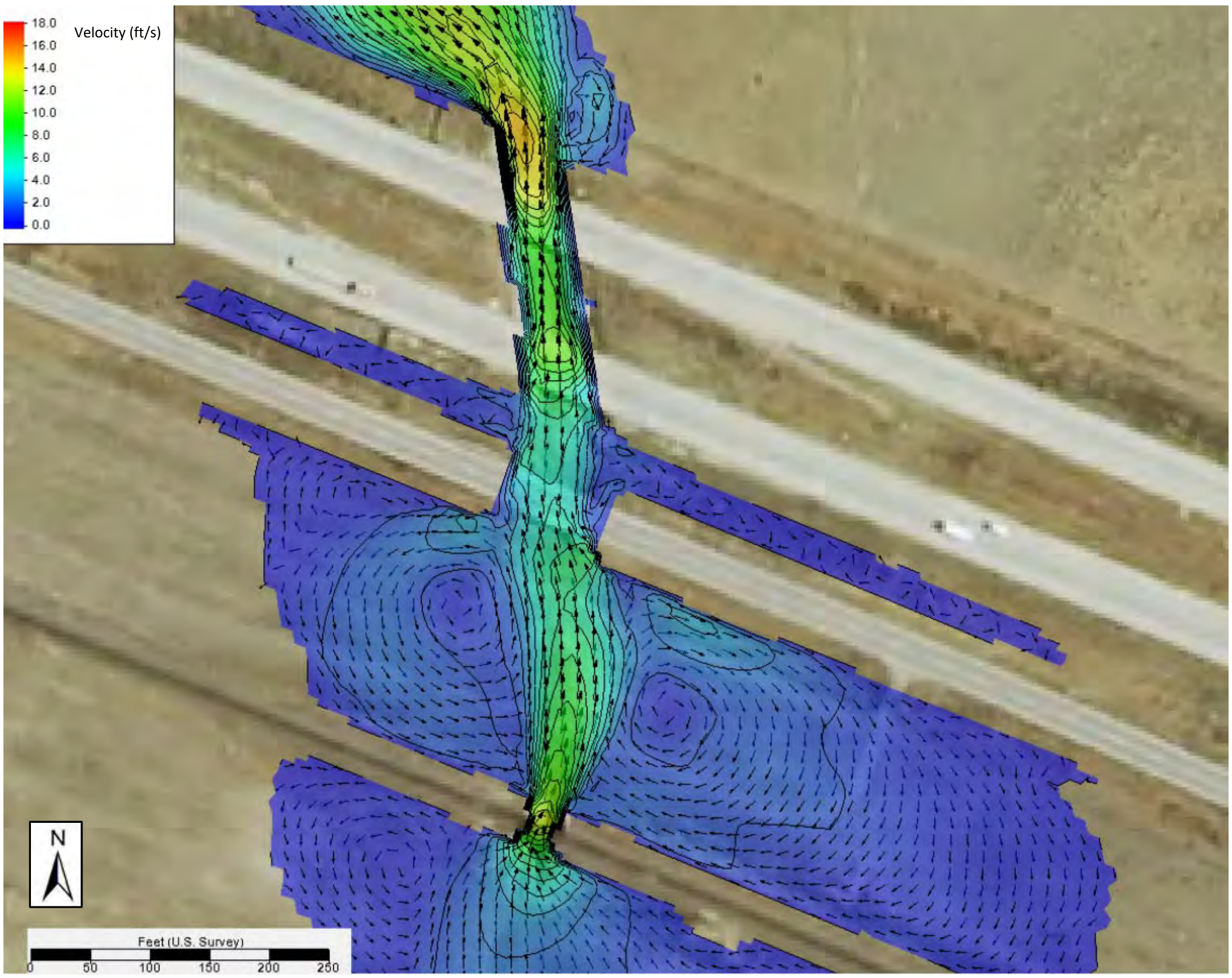


Figure 7. Proposed Conditions Velocity, 100-Year

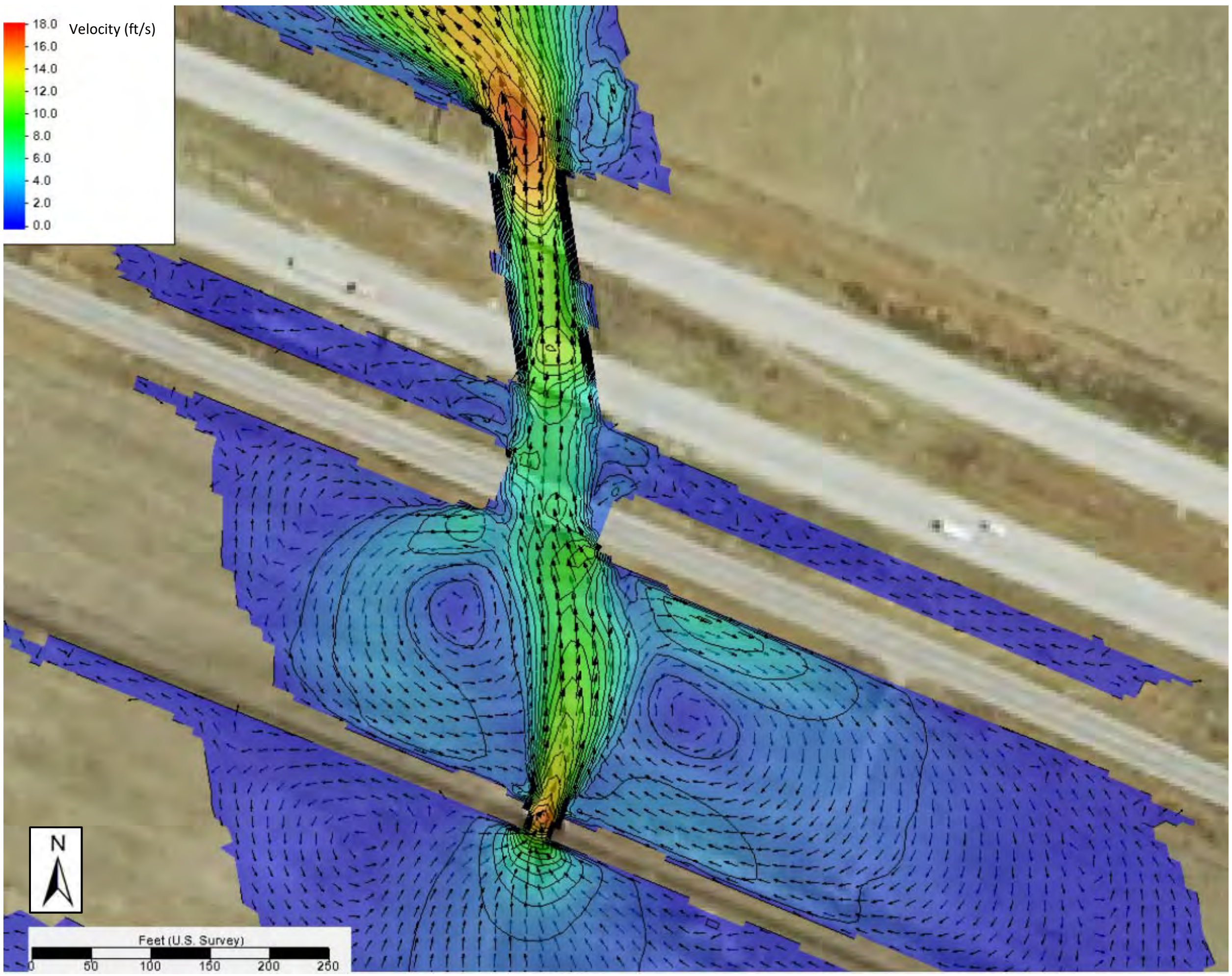


Figure 8. Proposed Conditions Velocity, 500-Year

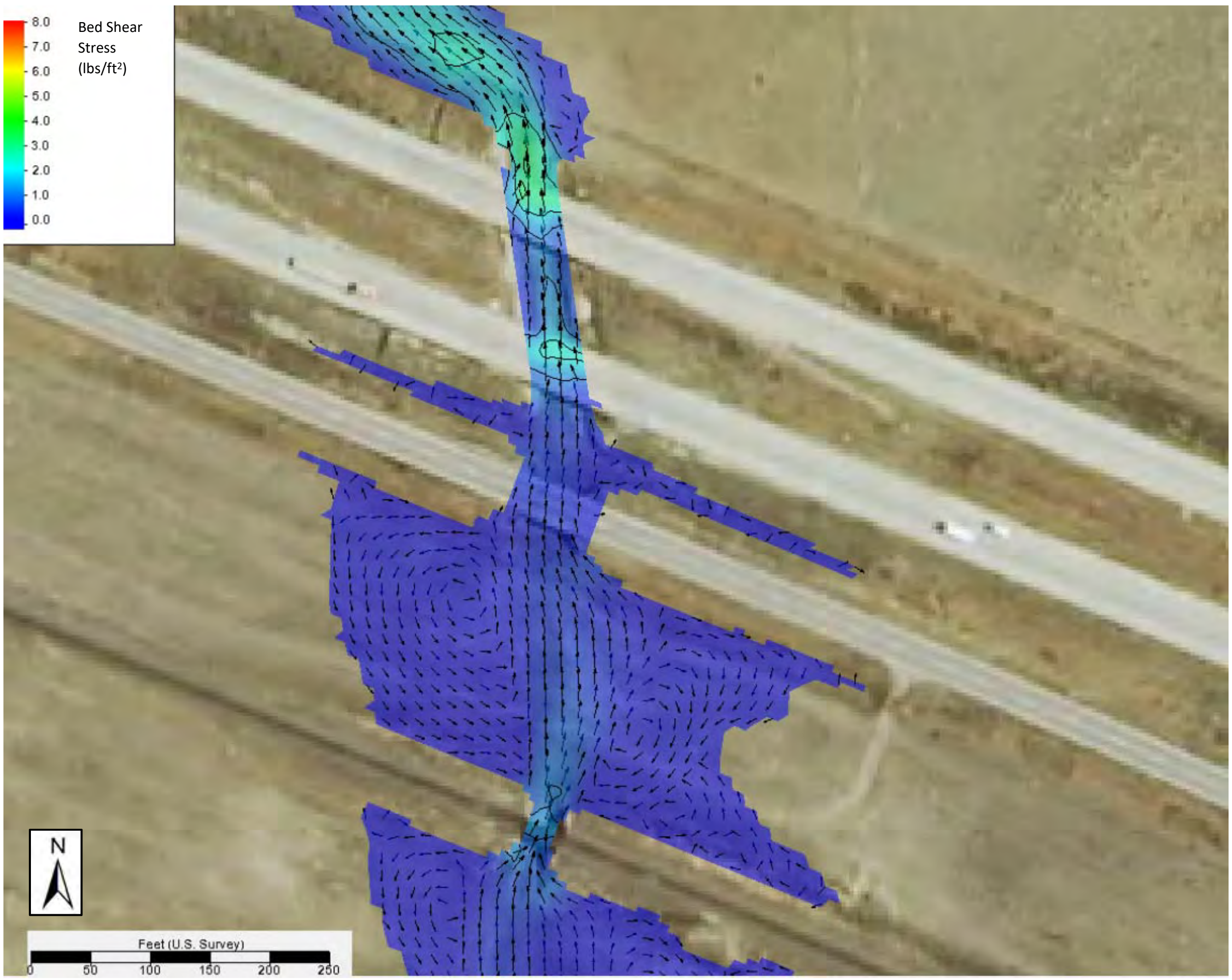


Figure 9. Proposed Conditions Bed Shear Stress, 10-Year

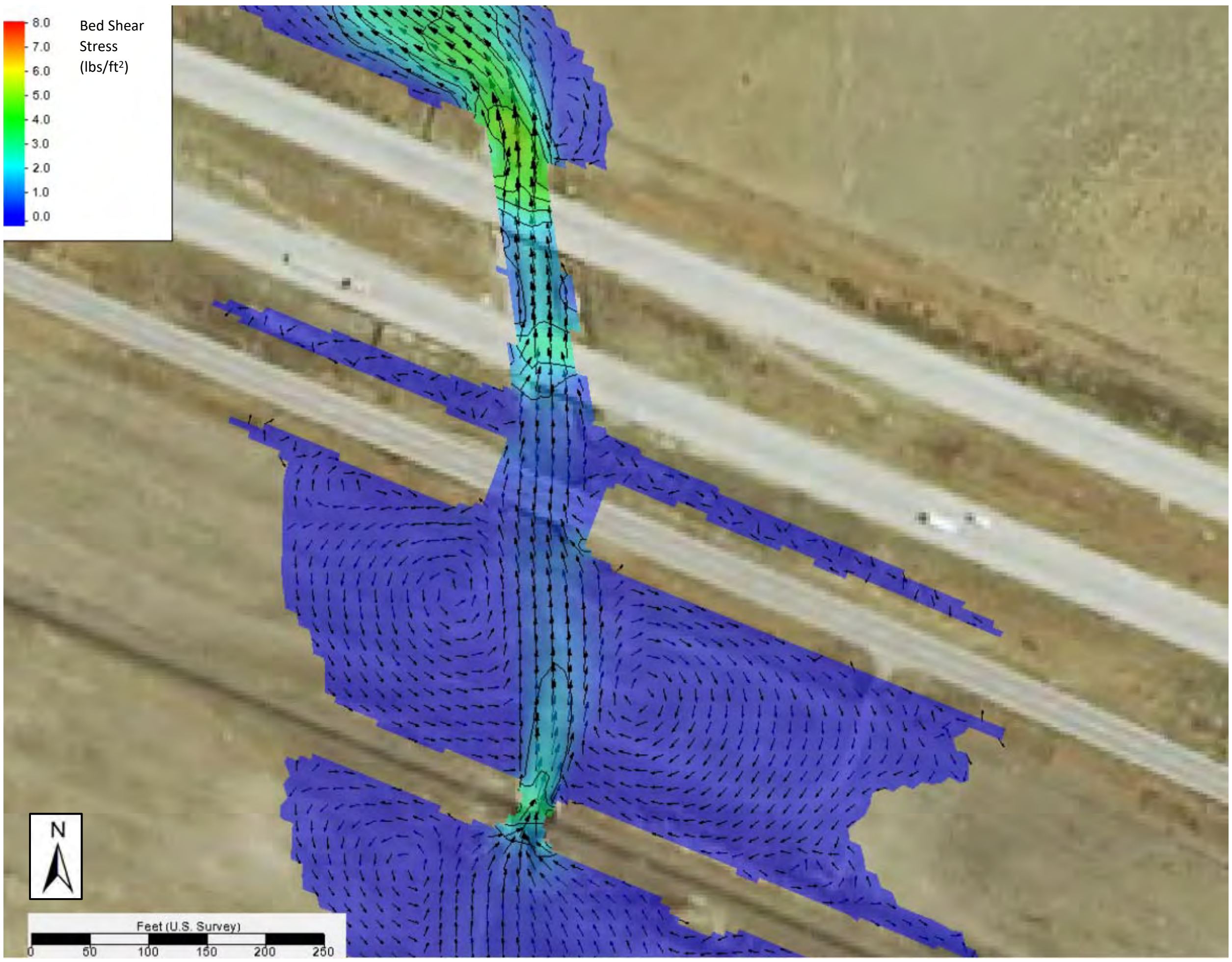


Figure 10. Proposed Conditions Bed Shear Stress, 50-Year

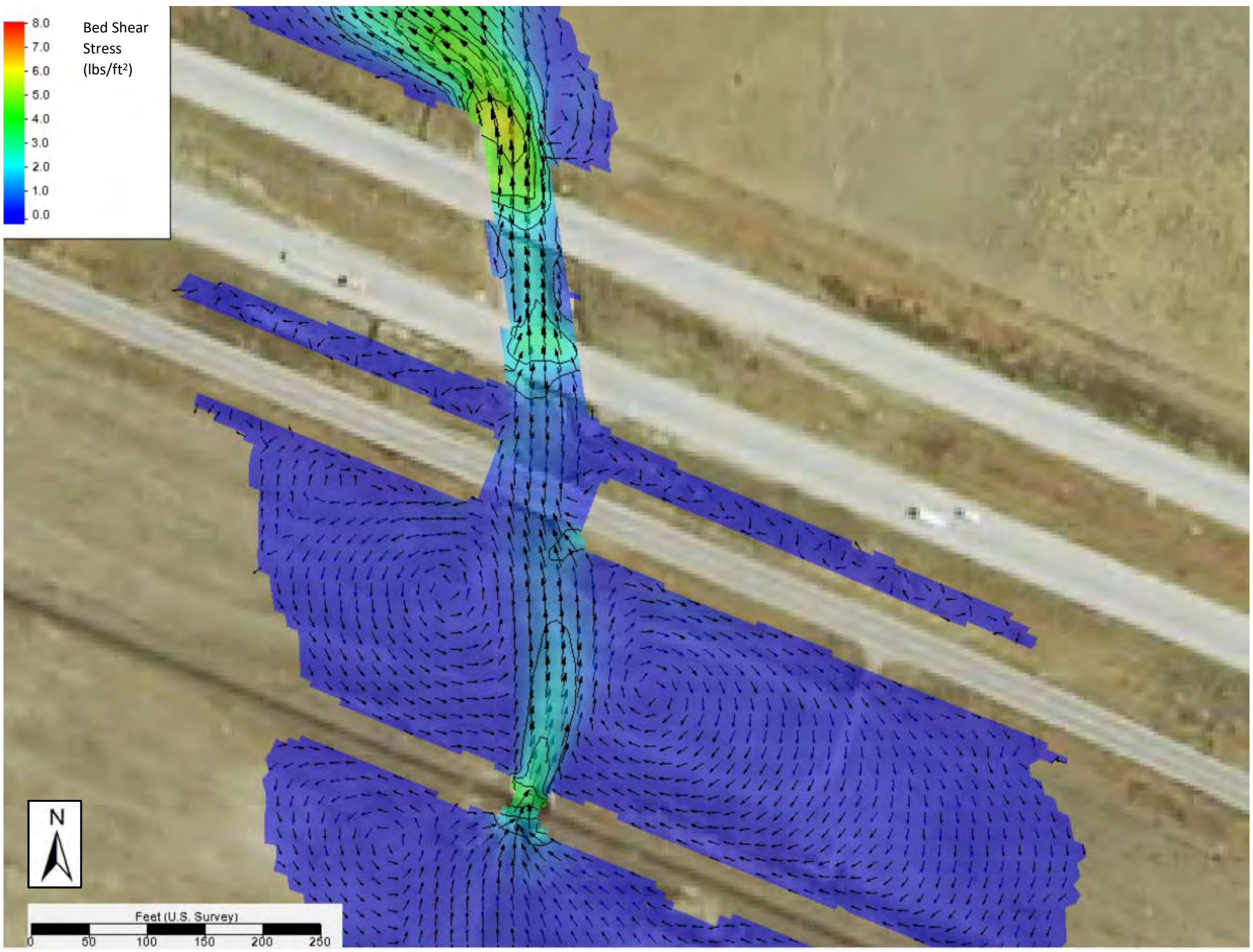


Figure 11. Proposed Conditions Bed Shear Stress, 100-Year

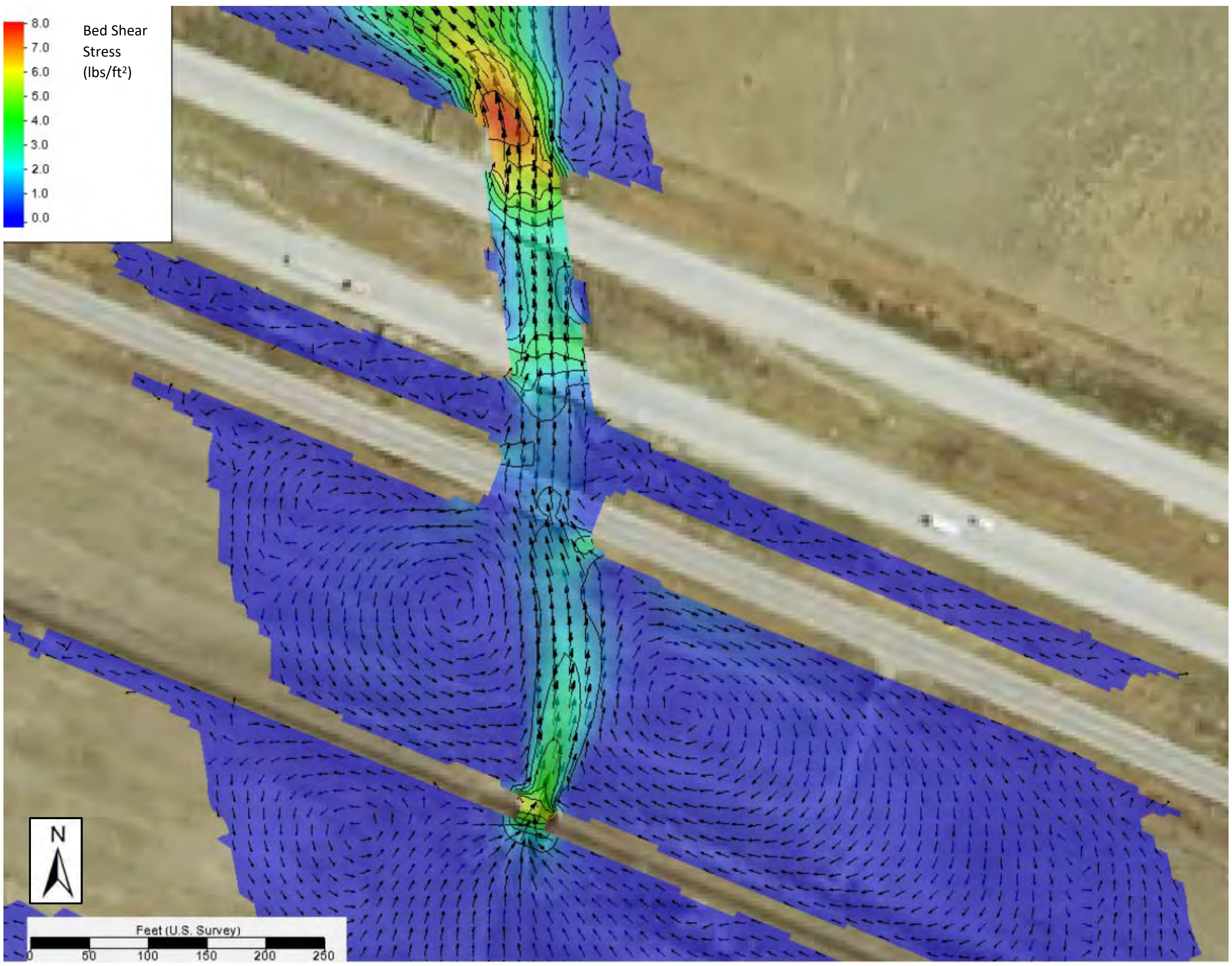


Figure 12. Proposed Conditions Bed Shear Stress, 500-Year

Appendix E

Geomorphic Assessment Memorandum

olsson

MEMO

TO:	Anthony Alvarado, PE (Ayres Associates)
FROM:	William Spitz, PG
RE:	BE Bridge F-20-L (MP 321.288) Geomorphic Assessment
DATE:	September 23, 2019
PROJECT #:	017-1690
CDOT PROJECT #:	20252
CDOT TO #:	22

Geomorphic Assessment of Stream Stability
BE Bridge F-20-L (US36 MP321.288) over Unnamed Creek near Peoria, 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-20-L, which is located in Arapahoe County about 3/4 mile northwest of Peoria, Colorado at Mile Post 321.288 (**Figure 1**). The unnamed drainage channel flows from the south to the north and is tributary to Rattlesnake Creek about ¾ miles north of the bridge. The bridge site and US36 are surrounded by low rolling hills and bound by railroad tracks/embankment about 240 feet to the south and by eastbound I-70 about 75 feet to the north.

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 the watershed is the upper Cretaceous Laramie Formation, which consists of shale, claystone, sandstone, and major coal beds. The upper part of the watershed may also be partially underlain by the Denver Formation which consists of arkosic sandstone, shale, mudstone, conglomerate, and local coal beds. Overlying the Laramie 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-20-L over an unnamed drainage channel near Peoria, 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 Nunn-Bresser-Ascalon complex, 0 to 3 percent slopes (approx. 37% of the area), Bresser and Truckton soil, 3 to 9 percent slopes, eroded (approx. 31% of the area), and Bresser-Truckton sandy loams, 3 to 5 percent slopes (approx. 16% of the area).

The Nunn-Bresser-Ascalon complex, 0 to 3 percent slopes are considered prime farmland soils if irrigated. The Nunn, Bresser, and Ascalon components make up 40%, 25%, and 20% of the complex, respectively, and are located on playas, stream terraces, and streams. The parent material of the Nunn component is eolian deposits. The parent material of the Bresser component is noncalcareous sandy alluvium and/or noncalcareous sandy eolian deposits. The parent material of the Ascalon component is outwash reworked by wind. The unit is well drained, is in the low runoff class, with the Nunn component belonging to the Hydrologic Soil Group C and the Bresser and Ascalon components 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.

Bed Sediment

A bulk sediment sample of sediment was collected on the upstream side of the bridge during the site visit. The grain size distribution from the dry sieve analysis indicates that the sampled material is primarily 21% silt and clay (< 0.074 mm), 31% sand (≤ 2.0 mm), and 48% gravel (> 2.0 mm). The Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI) of 35, 19, and 16, respectively, indicating that the sample is a clayey gravel with sand (GC). Given that there is no well defined channel upstream and no indication of sediment transport from upstream, it is likely that the coarse fraction of this sample was either likely artificially derived as part of the highway construction and maintenance or was excavated from the subsurface during formation of the scour hole under the bridge. Therefore, the true bed material upstream and downstream of the bridge, although covered with dense vegetation, is likely predominately silt and clay.

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 is approximately 3.93 mi^2 (**Figure 2**). Mean annual precipitation is about 15.77 inches. The maximum 6-hour, 2-year recurrence precipitation is estimated to be 1.94 inches and the maximum 24-hr, 100-year recurrence precipitation is estimated to be 4.9 inches. The estimated peak flow statistics for the unnamed channel at Bridge F-20-L are provided in **Table 1**.

Table 1. Estimated Peak Flows for unnamed channel at Bridge F-20-L.

Recurrence	StreamStats Peak Flow (cfs)	CDOT Peak Flows (cfs)
2-yr	85	---
5-yr	283	---
10-yr	518	545
25-yr	964	---
50-yr	1,430	1,170
100-yr	2,050	1,505
500-yr	4,090	2,420

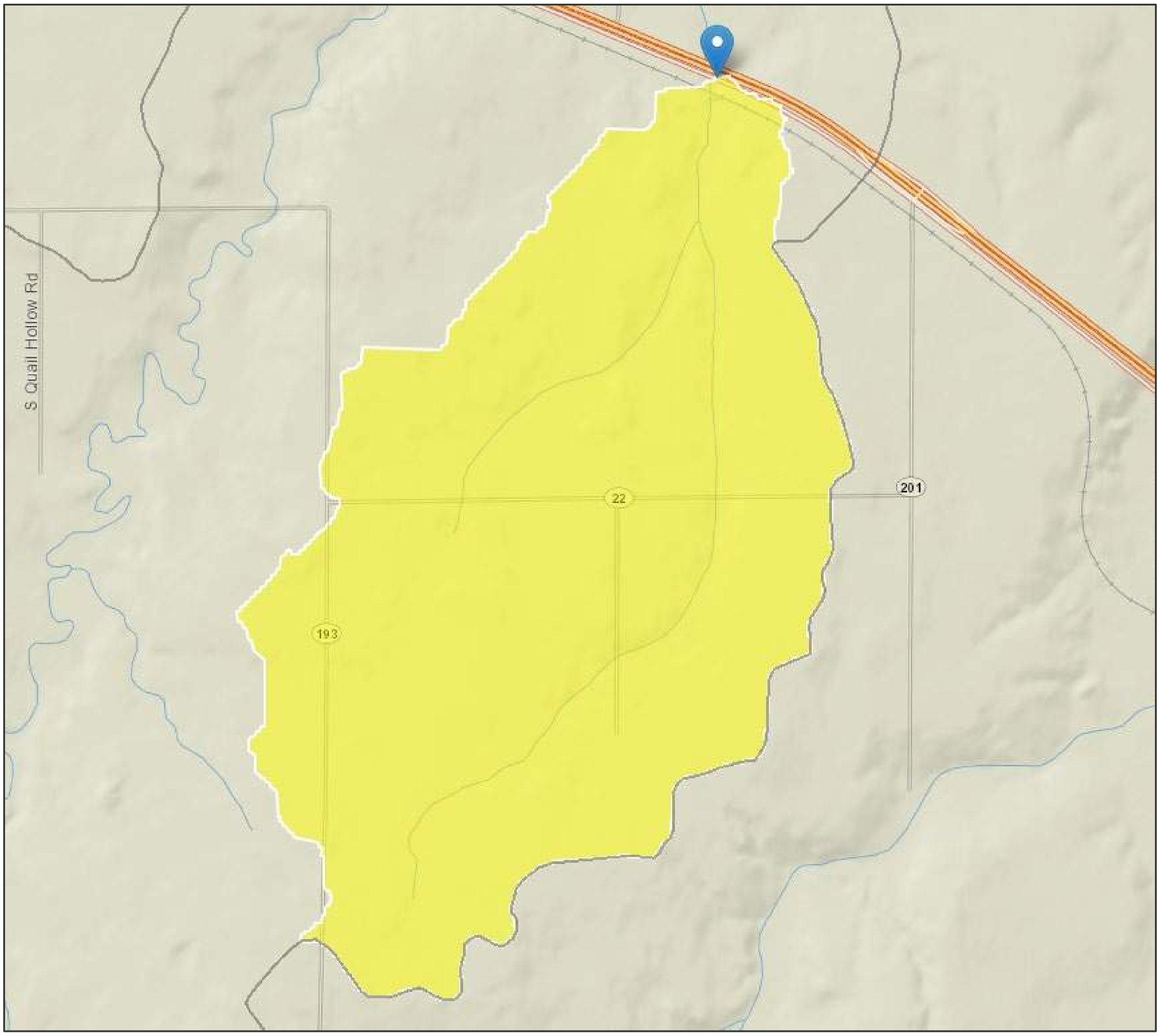


Figure 2. Drainage area for unnamed channel at US36 Bridge F-20-L near Peoria, CO.

CDOT also developed hydrology for the bridge site using HEC-HMS. The drainage area above the bridge site as delineated by CDOT is approximately 5.1 mi^2, or about 23% larger than that delineated by StreamStats. Although the drainage area delineated by CDOT is larger, the peak discharges estimated by CDOT, which are also shown in Table 1, are substantially smaller than those estimated by StreamStats.

Aerial Photo Analysis

Relatively good resolution aerial photo of the area taken in 1953 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 (south) and downstream (north) and drains predominately grazed prairie lands. The only changes noted since 1953 was the construction of I-70 and changes in land use, with some areas upstream of the bridge being cultivated in 1953.

Site Visit and Assessment

Bridge F-20-L, which is about 38 feet wide, consists of vertical timber retaining wall abutments with timber wingwalls. The bridge sits on 1 pile bent which contains 7 wood piles that are 12 inches in diameter. Wood cross braces are present on the pile bent. The pile bents are skewed about 10° relative to the upstream channel and the bridge is offset from the upstream railroad bridge by about 80 feet centerline to centerline (see Figure 2). **Figure 3** shows the configuration of the bridge.

Figure 3. View looking upstream at US36 Bridge F-20-L near Peoria, CO.

The channel upstream and downstream of the bridge contains a dense growth of grasses and weeds. Although there appears to be about 1 to 2 feet of general erosion or scour under the bridge between the middle pile bent and the left abutment (**Figure 4**), conditions at the bridge and upstream and downstream suggest general channel stability and relatively infrequent flows. High water marks/stains under the bridge indicate that flows rarely are more than 1-1.5 feet deep.

The I-70 bridges just downstream of Bridge F-20-L are also offset, but the concrete wall piers and spill-through abutments of the I-70 bridges are aligned with the opening of Bridge F-20-L. The I-70 abutment slopes are protected by large rock riprap with sheet pile walls at the toe.

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. Conditions at the bridge indicate that scour conditions are minimal although the alignment of the upstream railroad bridge and the skew of flow to the bridge results in flow and scour being directed primarily at the left bridge abutment.

Figure 4. View looking upstream at shallow scour hole under US36 Bridge F-20-L.

Appendix F

Scour and Countermeasure Calculations

Project Name	East Timber Bridges – Bridge F-20-L	Project No.	17-030.22
Design Calculation	Scour Analysis	Version	2
Originator	NLN	Date:	March 4, 2020
Checker	ALR	Date:	March 5, 2020

PURPOSE:

The Colorado Department of Transportation has identified the need to replace the existing US 40 Bridge (Structure No. F-20-L) over an unnamed drainage channel at Mile Post 321.288 in Arapahoe County and approximately 0.75 miles northwest of Peoria, 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, I-70 Service Road Bridge Northwest of the Town of Peoria.* January 17, 2019.

Olsson Associates, Inc., *Geomorphic Assessment of Stream Stability BE Bridge F-20-L (US36 MP321.88) Over Unnamed Creek near Peoria, CO.* September 23, 2019.

BRIDGE SCOUR ANALYSIS:

The proposed bridge design at structure No. F-20-L 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, 100-foot-span replacement bridge was computed for the 100- and 500-year events. FHWA's Hydraulic Engineering Circular No. 18 (HEC-18) and hydraulic data taken from the SMS model (SMS 13.0.8) were utilized for the scour analysis. Clips containing the approach and contracted sections are provided below.

Site Geology

Structure F-20-L is situated along a channel with a low-plasticity clayey sand 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 21% silt and clay (< 0.074 mm), 31% sand (≤ 2.0 mm), and 48% gravel (>2.0mm). The Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI) of 35, 19, and 16, respectively. The D_{50} particle size of the channel material is 1.8 mm and has been utilized to determine the critical velocity for scour calculations, see ***Figure 1*** for a summary of the bulk sediment sample. HEC-18 equations have been used to determine the channel's critical velocity and results qualify the stream as a live-bed. See attached calculations.

As reported in Olsson Associate's geomorphic assessment, there appears to be 1 to 2 feet of erosion or scour under the bridge between the middle pile bent and the left abutment. This is likely due to the alignment of the upstream railroad bridge and skew of flow towards the Bridge. Ultimately, due to the dense growth of grasses and weeds and infrequent flows, Olsson Associate's 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.

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-20-L	2019-4150	None	MP 321.288		35	19	16	1.6		A-2-6(0)	

Gradations:

mm	75	25	19	9.5	#4	#10	#40	#200
in	3	1	3/4	3/8				
%Pass	100	99	95	90	82	52	30	21
As Run				95	86	55	32	22

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 165-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), it's flow vectors are minimally skewed, and its width avoids capturing any return flow due to the large eddying that is happening on the left and right overbanks of the floodplain. The contraction section was cut through the centerline of the proposed bridge to capture the bridge geometry (Section 2). Average velocities, flow depths, and flow rates were taken at these sections for scour calculations.

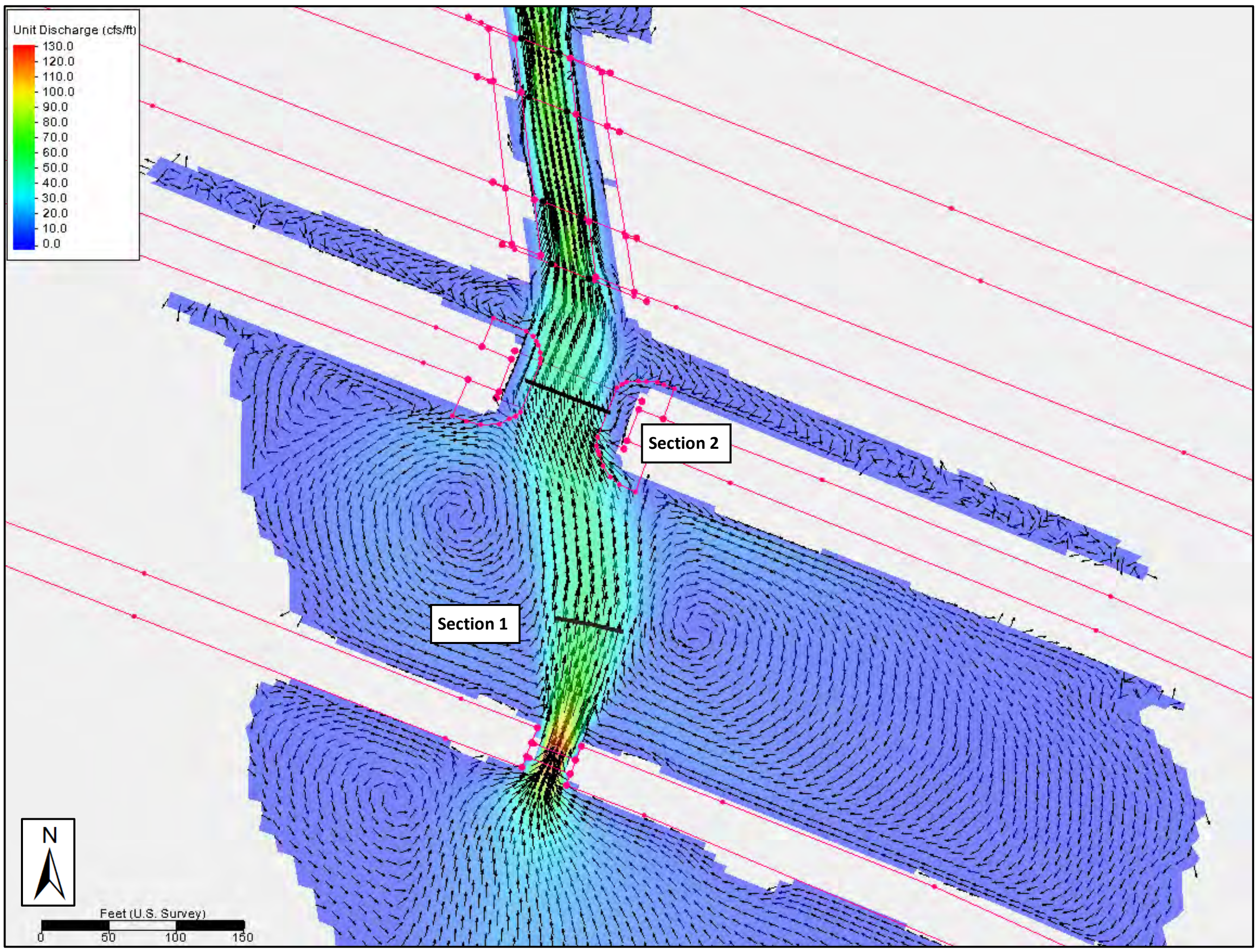


Figure 2. *Approach Sections Considered (Plan View)*

Contraction Scour

The modified version of Laursen's 1960 equation for live-bed scour was utilized to determine contraction scour at the Bridge. The mode of transport material for the 100-year and 500-year scour events was determined to be mostly suspended bed material. The results of both storm events revealed a negative scour depth result, indicating that no scour hole is expected to form in the main channel at the bridge. Since the restriction of the upstream railroad bridge is much smaller—approximately 25-ft—than the constriction at the US 40 Bridge, water expands rapidly on the downstream end of the railroad bridge and begins to form shallow eddies along the left and right overbanks of the main channel. Due to the recirculation of these flows, contraction at the bridge is not as severe as would be expected if the channel were more incised and conveying all of the flows uniformly and perpendicularly towards the Bridge.

Pressure Scour

The bridge is under pressure flow, but not overtopping, during the 500-year event and as such pressure scour calculations were carried out. Pressure scour is equal to the sum of the separation zone thickness (t) and average depth in the contracted section (y_2) less the vertical size of the bridge opening prior to scour (h_b). The bridge opening was determined by cutting a cross-section at the upstream face of the bridge and subtracting the invert elevation from the low-chord elevation. **Figure 3** below, taken from HEC-18, illustrates where each of the hydraulic variables mentioned above are located in relation to the bridge. It should be noted that $h_{b,}$ in a non-overtopping condition, is equal to the effective upstream channel flow depth for live-bed (h_{ue}), or as Equation 6.2 in HEC-18 refers to it—y_1. The proposed scour depth calculated at Bridge F-20-L utilizing the ratio of the upstream main channel bottom width to the contracted section's channel bottom width (W_1/W_2) resulted in a negative scour depth (- 0.05-ft); to test sensitivity and produce a more conservative pressure scour depth, the calculation was carried out assuming a ratio of 1 for W_1/W_2, which resulted in a scour depth of 0.85-ft. Additional details and the equation utilized to determine zone thickness can be found in the attachments.

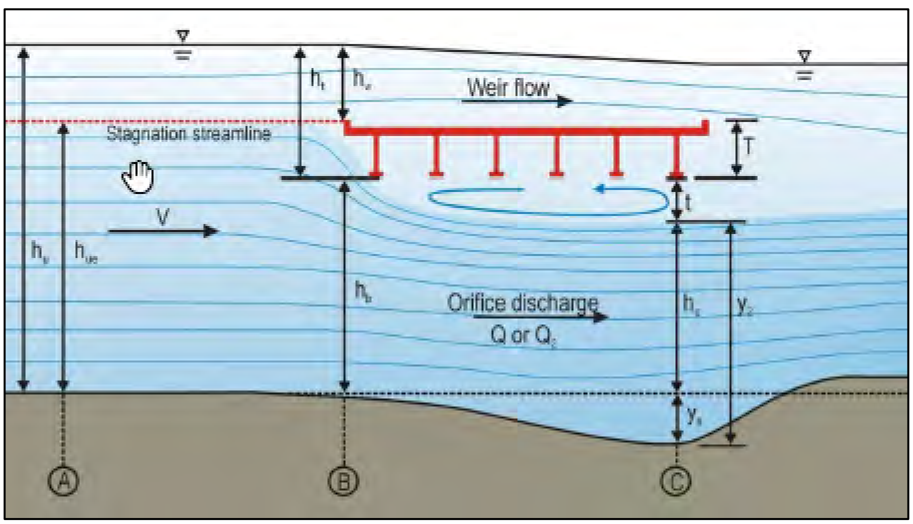


Figure 3. *Pressure Scour: Hydraulic Variables*

Live-Bed Abutment Scour

Initially, the NCHRP abutment scour approach for live-bed scour was used to determine abutment scour; the results of the NCHRP abutment calculations were negative, indicating that no abutment scour would be experienced. Due to the high flows during the 100- and 500-year events, skewed flow vectors approaching the bridge, and the 1-2-ft depth scour analyzed during site reconnaissance, these results did not seem realistic.

Froehlich's abutment scour equation was used to calculate an alternate abutment scour depth that is more realistic. These calculations resulted in 11.5-ft and 11.1-ft of scour for both the left and right abutments during the 100-year storm event. The 500-year event abutment scour depths are 14.8-ft and

14.3-ft for left and right abutments, respectively. See attachments for calculation details and **Table 1** below for detailed scour results.

Scour Results

Estimated scour results for Structure F-20-L are provided in the table below.

Table 1. *Proposed Condition Scour Results*

Recurrence Interval	Contraction Scour (ft)	Left Abutment Scour (ft)	Right Abutment Scour (ft)	Pressure Scour (ft)
100-Year	0	11.45	11.10	-----
500-Year	0	14.77	14.33	0.85*

*Pressure scour assumed $W_1/W_2 = 1$

There is no contraction scour expected for the proposed bridge, pressure scour is minimal, and as expected the left abutment will experience slightly larger scour depths than the right due to the direction of flow through the bridge.

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.*

While the 100-year riprap sizes for protection against scour has a D_{50} of 5.07 inches, the 500-year size for protection against scour was calculated as a D_{50} of 10-½ inches. However, to conform with standard riprap sizing it is recommended that 12-inch riprap be installed at a depth of 2-feet. Two different options are available for the protection. One option is riprap installed 2-ft below the channel invert, which will allow for the granular channel bed to naturally scour during the rising stage of a runoff event and fill during the falling stage, without restricting flow through the bridge during the falling stages. A second option of 12-inch riprap extended down the abutments at 2:1 to a depth of 11.5-ft was also considered. HEC-23 suggests that the riprap extend parallel to the channel bed to a width of $2Y_0$ or 16-ft, but this extension from left and right abutment would intersect at a distance less than 16-ft (approximately 13-ft). Because abutment scour is so deep, it may be more cost effective and easier to install riprap all the way across the channel then for the full 11.5-ft depth. However, since both options appear to be feasible, they are provided below in **Figure 4** and **Figure 5**. The maximum scour depth elevation realized at the left abutment for the 100-year and 500-year events is 5142.50 and 5139.18.

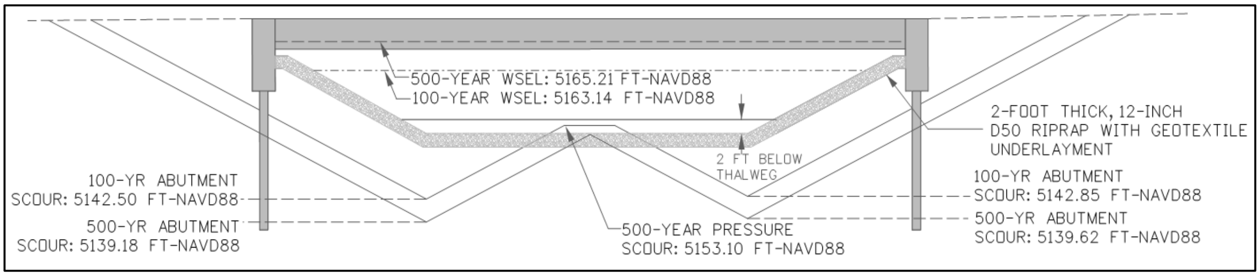


Figure 4. Riprap Armoring for Scour: Option 1

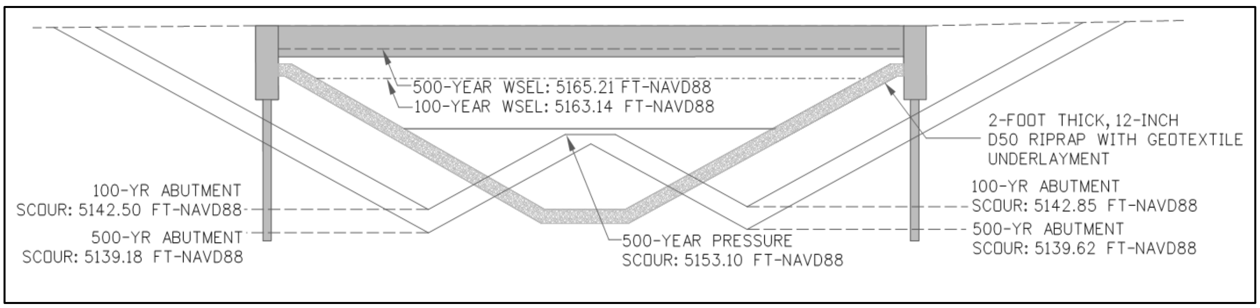


Figure 5. Riprap Armoring for Scour: Option 2

100-Yr HEC-18 CRITICAL VELOCITY (EQN 6.1)		
Inputs:		
y	6.67	Avg depth of flow upstream of bridge (ft)
D_{50}	0.071	Median bed particle size (ft)
K_u	11.17	English Units
V_{avg}	6.86	Average Upstream Channel velocity (ft/s)
Outputs:		
V_c	6.34	Critical Velocity at which D_{50} will be transported
Mode:	Live-Bed	

Only use the Live-Bed equation sheets from here on. Pier Scour accounts for scour mode in the equation, so there is only one sheet for it.

$$V_c = K_u\, y^{1/6}\, D^{1/3} \qquad (6.1)$$

where:

V_c	=	Critical velocity above which bed material of size D and smaller will be transported, ft/s (m/s)
y	=	Average depth of flow upstream of the bridge, ft (m)
D	=	Particle size for V_c, ft (m)
D_{50}	=	Particle size in a mixture of which 50 percent are smaller, ft (m)
K_u	=	6.19 SI units
K_u	=	11.17 English units

500-Yr HEC-18 CRITICAL VELOCITY (EQN 6.1)

Inputs:		
y	8.67	Avg depth of flow upstream of bridge (ft)
D_{50}	0.071	Median bed particle size (ft)
K_u	11.17	English Units
V_{avg}	9.14	Average Upstream Channel velocity (ft/s)
Outputs:		
V_c	6.63	Critical Velocity at which D_{50} will be transported
Mode:	Live-Bed	

Only use the Live-Bed equation sheets from here on. Pier Scour accounts for scour mode in the equation, so there is only one sheet for it.

$$V_c = K_u\, y^{1/6}\, D^{1/3} \tag{6.1}$$

where:

V_c	=	Critical velocity above which bed material of size D and smaller will be transported, ft/s (m/s)
y	=	Average depth of flow upstream of the bridge, ft (m)
D	=	Particle size for V_c, ft (m)
D_{50}	=	Particle size in a mixture of which 50 percent are smaller, ft (m)
K_u	=	6.19 SI units
K_u	=	11.17 English units

100-YR HEC-18 NCHRP 24-20 LIVE-BED ABUTMENT AND CONTRACTION SCOUR (EQNS 8.3, 8.4, AND 8.5)		
Inputs:		
y_o	8.54	Flow depth prior to scour (ft)
y_1	6.86	Upstream flow depth (ft)
q_1	45.74	Upstream unit discharge (ft^2/s)
q_{2c}	35.96	Contracted section unit discharge (ft^2/s)
q_2/q_1	0.79	**Assume 1**
α_A	1.2	From appropriate figure
Outputs:		
y_c	5.58	Flow depth including live-bed contraction scour (ft)
y_{max}	6.70	Maximum flow depth resulting from abutment scour (ft)
y_s	-1.84	Abutment scour depth (ft) **also includes contraction scour**

$$y_{max} = \alpha_A y_c \text{ or } y_{max} = \alpha_B y_c \quad (8.3)$$

$$y_s = y_{max} - y_0 \quad (8.4)$$

where:

y_{max}	=	Maximum flow depth resulting from abutment scour, ft (m)
y_c	=	Flow depth including live-bed or clear-water contraction scour, ft (m)
α_A	=	Amplification factor for live-bed conditions
α_B	=	Amplification factor for clear-water conditions
y_s	=	Abutment scour depth, ft (m)
y_0	=	Flow depth prior to scour, ft (m)

$$y_c = y_1\left(\frac{q_{2c}}{q_1}\right)^{6/7} \quad (8.5)$$

where:

y_c	=	Flow depth including live-bed contraction scour, ft (m)
y_1	=	Upstream flow depth, ft (m)
q_1	=	Upstream unit discharge, ft^2/s (m^2/s)
q_{2c}	=	Unit discharge in the constricted opening accounting for non-uniform flow distribution, ft^2/s (m^2/s)

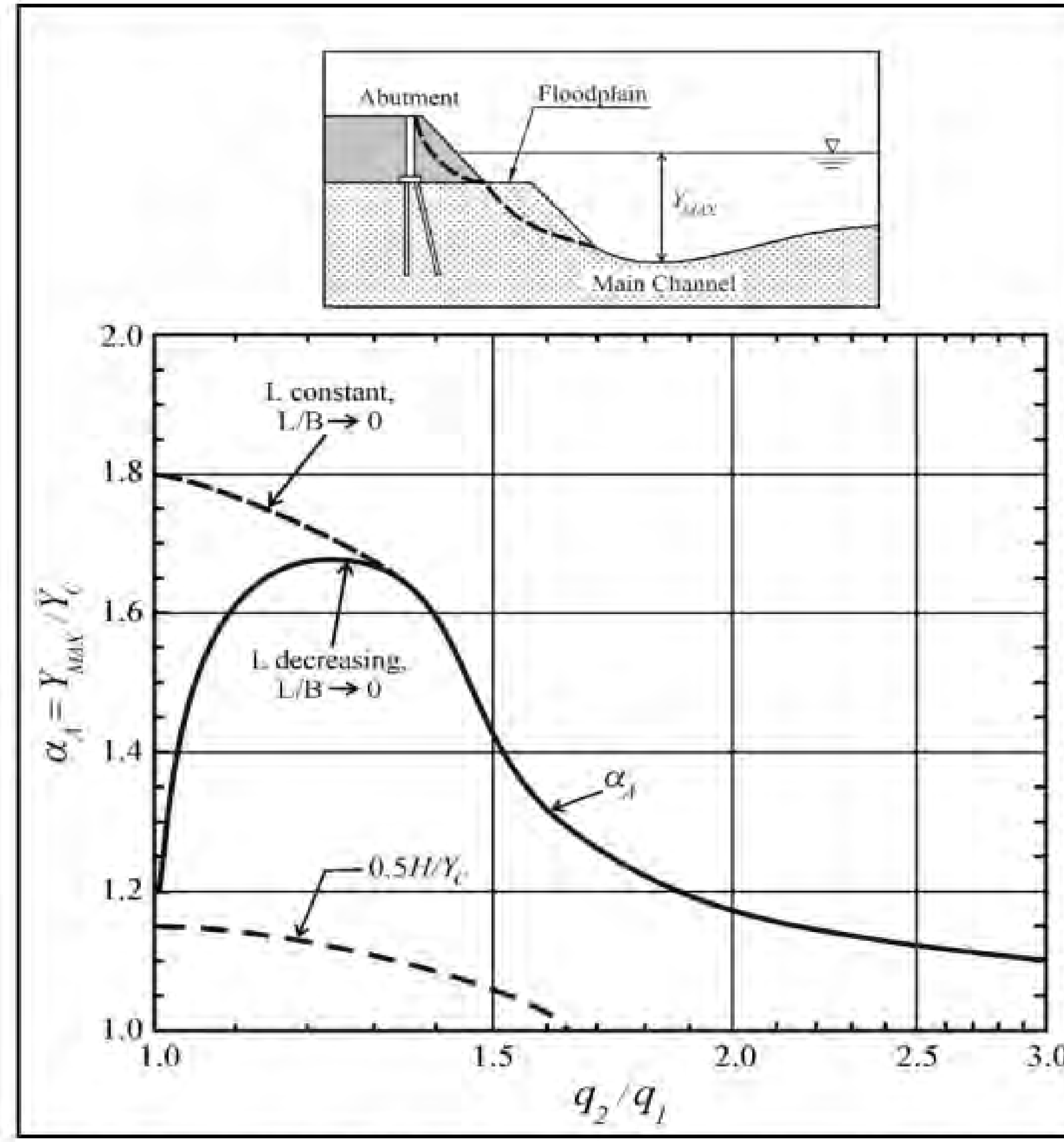


Figure 8.9. Scour amplification factor for spill-through abutments and live-bed conditions (NCHRP 2010b).

500-YR HEC-18 NCHRP 24-20 LIVE-BED ABUTMENT AND CONTRACTION SCOUR (EQNS 8.3, 8.4, AND 8.5)

Inputs:		
y_o	9.60	Flow depth prior to scour (ft)
y_1	8.67	Upstream flow depth (ft)
q_1	79.22	Upstream unit discharge (ft^2/s)
q_{2c}	53.46	Contracted section unit discharge (ft^2/s)
q_2/q_1	0.67	**Assume 1**
α_A	1.2	From appropriate figure
Outputs:		
y_c	6.19	Flow depth including live-bed contraction scour (ft)
y_{max}	7.42	Maximum flow depth resulting from abutment scour (ft)
y_s	-2.18	Abutment scour depth (ft) **also includes contraction scour**

$$y_{max} = \alpha_A\, y_c \text{ or } y_{max} = \alpha_B\, y_c \quad (8.3)$$

$$y_s = y_{max} - y_0 \quad (8.4)$$

where:

y_{max}	=	Maximum flow depth resulting from abutment scour, ft (m)
y_c	=	Flow depth including live-bed or clear-water contraction scour, ft (m)
α_A	=	Amplification factor for live-bed conditions
α_B	=	Amplification factor for clear-water conditions
y_s	=	Abutment scour depth, ft (m)
y_0	=	Flow depth prior to scour, ft (m)

$$y_c = y_1\left(\frac{q_{2c}}{q_1}\right)^{6/7} \quad (8.5)$$

where:

y_c	=	Flow depth including live-bed contraction scour, ft (m)
y_1	=	Upstream flow depth, ft (m)
q_1	=	Upstream unit discharge, ft^2/s (m^2/s)
q_{2c}	=	Unit discharge in the constricted opening accounting for non-uniform flow distribution, ft^2/s (m^2/s)

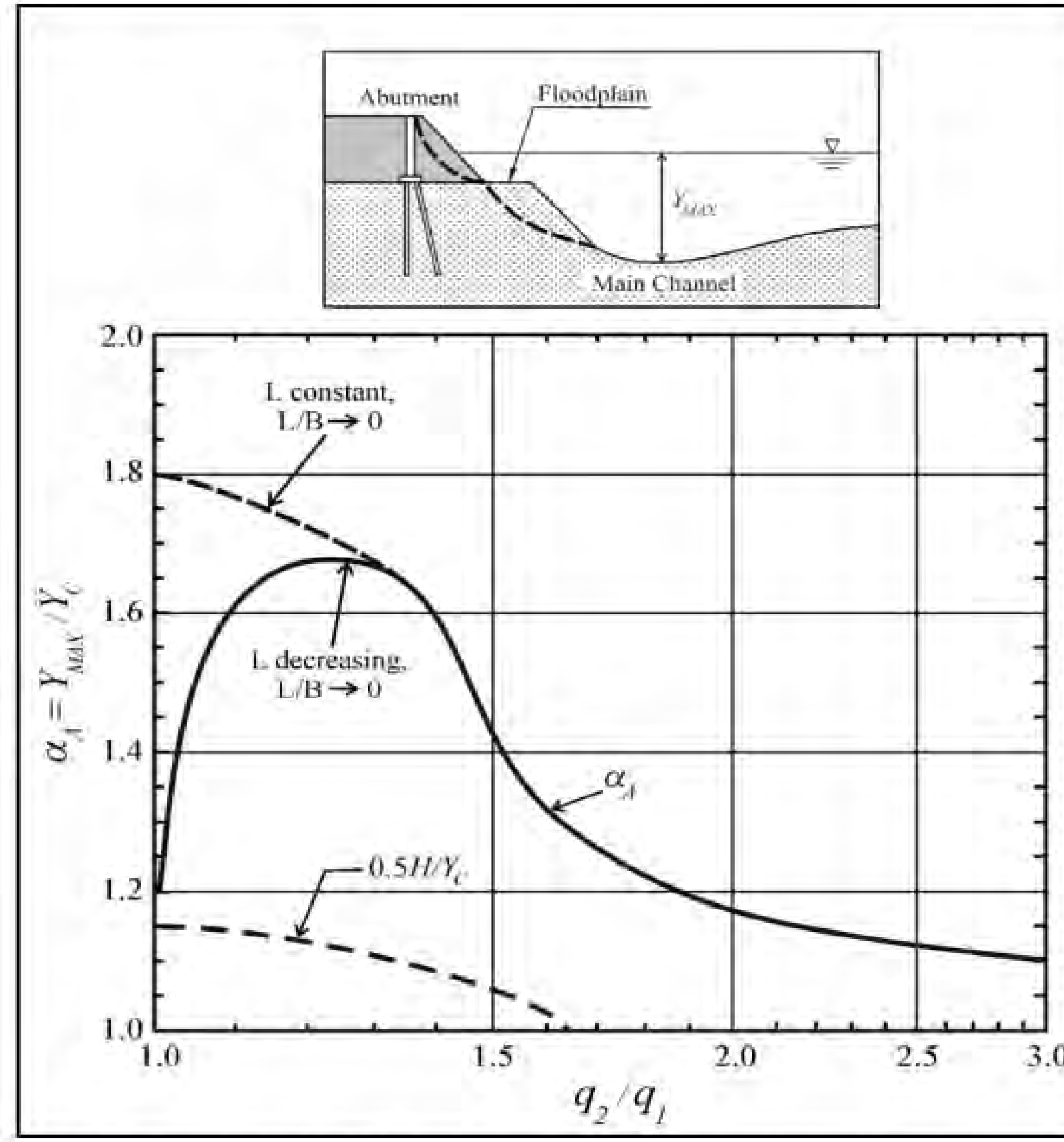


Figure 8.9. Scour amplification factor for spill-through abutments and live-bed conditions (NCHRP 2010b).

100-Yr Contraction Scour: Modified Laursen's for Live-Bed Stream

D_{50}	1.8 mm
D_{50}	0.0708 ft
Fall Vel., T	0.666 ft/s
Acceleration of Gravity, g	32.2 ft/s^2
Slope of Energy Grade Line,S_1	0.01 ft/ft
Shear Velocity,V* (Calculated)	1.47 ft/s
Flow in Upstream Channel,Q_1	2,501 cfs
Flow in Contracted Channel, Q_2	2,485 cfs
Average depth in upstream main channel, y_1	6.67 ft
V*/T	2.20
Mode of Bed Exponent,K_1^*	0.69

W_1/W_2	0.84
Q_2/Q_1	0.99
Y_2	5.89
Scour Hole	**-2.64**

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \qquad (6.2)$$

$y_s = y_2 - y_o$ = (average contraction scour depth) (6.3)

where:

y_1 = Average depth in the upstream main channel, ft (m)
y_2 = Average depth in the contracted section, ft (m)
y_o = Existing depth in the contracted section before scour, ft (m) (see Note 7)
Q_1 = Flow in the upstream channel transporting sediment, ft^3/s (m^3/s)
Q_2 = Flow in the contracted channel, ft^3/s (m^3/s)
W_1 = Bottom width of the upstream main channel that is transporting bed material, ft (m)
W_2 = Bottom width of main channel in contracted section less pier width(s), ft (m)
k_1 = Exponent determined below

V*/T	k_1	Mode of Bed Material Transport
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

V_* = $(\tau_o/\rho)^{1/2} = (g y_1 S_1)^{1/2}$, shear velocity in the upstream section, ft/s (m/s)
T = Fall velocity of bed material based on the D_{50}, m/s (Figure 6.8)
For fall velocity in English units (ft/s) multiply T in m/s by 3.28
g = Acceleration of gravity (32.2 ft/s^2) (9.81 m/s^2)
S_1 = Slope of energy grade line of main channel, ft/ft (m/m)
τ_o = Shear stress on the bed, (lb/ft^2) (Pa (N/m^2))
ρ = Density of water (1.94 $slugs/ft^3$) (1000 kg/m^3)

500-Yr Contraction Scour: Modified Laursen's for Live-Bed Stream

D_{50}	1.8 mm
D_{50}	0.071 ft
Fall Vel., T	0.66557 ft/s
Acceleration of Gravity, g	32.2 ft/s^2
Slope of Energy Grade Line, S_1	0.01 ft/ft
Shear Velocity (Calculated), V*	1.52 ft/s
Flow in Upstream Channel, Q_1	4,031 cfs
Flow in Contracted Channel, Q_2	4,010 cfs
Average depth in upstream main channel, y_1	8.67 ft
V*/T	2.28
Mode of Bed Exponent, K_1^*	0.69

W_1/W_2	0.85
Q_2/Q_1	0.99
Y_2	7.72
Scour Hole	**-0.95**

Alternate y_2 Calculation Assuming W1/W2=1

W_1/W_2	1.00
Q_2/Q_1	0.99
Y_2	8.63
Scour Hole	**0.04**

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \qquad (6.2)$$

$$y_s = y_2 - y_o = \text{(average contraction scour depth)} \qquad (6.3)$$

where:

y_1 = Average depth in the upstream main channel, ft (m)
y_2 = Average depth in the contracted section, ft (m)
y_o = Existing depth in the contracted section before scour, ft (m) (see Note 7)
Q_1 = Flow in the upstream channel transporting sediment, ft^3/s (m^3/s)
Q_2 = Flow in the contracted channel, ft^3/s (m^3/s)
W_1 = Bottom width of the upstream main channel that is transporting bed material, ft (m)
W_2 = Bottom width of main channel in contracted section less pier width(s), ft (m)
k_1 = Exponent determined below

V_*/T	k_1	Mode of Bed Material Transport
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

V_* = $(\tau_o/\rho)^{1/2} = (gy_1 S_1)^{1/2}$, shear velocity in the upstream section, ft/s (m/s)
T = Fall velocity of bed material based on the D_{50}, m/s (Figure 6.8)
For fall velocity in English units (ft/s) multiply T in m/s by 3.28
g = Acceleration of gravity (32.2 ft/s^2) (9.81 m/s^2)
S_1 = Slope of energy grade line of main channel, ft/ft (m/m)
τ_o = Shear stress on the bed, (lb/ft^2) (Pa (N/m^2))
ρ = Density of water (1.94 slugs/ft^3) (1000 kg/m^3)

Pressure Scour for Live-Bed Stream (No Overtopping)	
Top of bridge superstructure elevation:	5168.90 ft
Bottom of bridge superstructure elevation:	5164.48 ft
Height of the bridge obstruction,T	4.42 ft
Vertical size of bridge opening prior to scour,h_b	10.3 ft
Distance from the water surface to lower face of bridge girders,h_t	0.21 ft
Flow in Upsream Channel,Q_1	4,031 cfs
Flow in Contracted Channel, Q_2	4,010 cfs
Bottom Width of Upstream Main Channel ,W_1	50 ft
Bottom Width of Main Channel in Contracted Section,W_2	58 ft
W_1/W_2	0.85
Average depth in the contracted section,y_2	7.72 ft
Seperation zone thickness,t:	2.52 ft

The separation zone thickness, t, is calculated using Equation 6.16:

$$\frac{t}{h_b} = 0.5\left(\frac{h_b \cdot h_t}{h_u^2}\right)^{0.2}\left(1-\frac{h_w}{h_t}\right)^{-0.1} \qquad (6.16)$$

where:

h_b = Vertical size of the bridge opening prior to scour, ft (m)

h_t = Distance from the water surface to the lower face of the bridge girders, equals $h_u - h_b$, ft (m)

h_w = Weir flow height = h_t - T for h_t > T, h_w = 0 for $h_t \leq$ T

Pressure Scour Depth,y_s	-0.05 ft
Pressure Scour assuming W_1/W_2 = 1	**0.85 ft**

$$y_s = y_2 + t - h_b$$

100-Yr Abutment Scour : Froelich's Live-Bed Stream

Left Abutment

Coefficient for Abutment Shape, K_1	0.55
Coefficient for Angle of Embankment to Flow, K_2	1.03
Length of Active Flow Obstrubted by Embankment, L'	5
Average Depth of Flow on the Floodplain, y_a	6.67
Froude Number of Approach Section, F_r	0.47
	11.45 ft

Right Abutment

Coefficient for Abutment Shape, K_1	0.55
Coefficient for Angle of Embankment to Flow, K_2	0.96
Length of Active Flow Obstrubted by Embankment, L'	5
Average Depth of Flow on the Floodplain, y_a	6.67
Froude Number of Approach Section, F_r	0.47
	11.10 ft

$$\frac{y_s}{y_a} = 2.27\, K_1\, K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1 \qquad (8.1)$$

where:

- K_1 = Coefficient for abutment shape (Table 8.1)
- K_2 = Coefficient for angle of embankment to flow
- K_2 = $(\theta/90)^{0.13}$ (see Figure 8.5 for definition of θ)
  - θ<90° if embankment points downstream
  - θ>90° if embankment points upstream
- L' = Length of active flow obstructed by the embankment, ft (m)
- A_e = Flow area of the approach cross section obstructed by the embankment, ft^2 (m^2)
- Fr = Froude Number of approach flow upstream of the abutment = $V_e/(gy_a)^{1/2}$
- V_e = Q_e/A_e, ft/s (m/s)
- Q_e = Flow obstructed by the abutment and approach embankment, ft^3/s (m^3/s)
- y_a = Average depth of flow on the floodplain (A_e/L), ft (m)
- L = Length of embankment projected normal to the flow, ft (m)
- y_s = Scour depth, ft (m)

500-Yr Abutment Scour : Froelich's Live-Bed Stream

Left Abutment

Coefficient for Abutment Shape,K_1	0.55
Coefficient for Angle of Embankment to Flow,K_2	1.03
Length of Active Flow Obstrubted by Embankment, L'	5
Average Depth of Flow on the Floodplain,y_a	8.67
Froude Number of Approach Section,F_r	0.55
	14.77 ft

Right Abutment

Coefficient for Abutment Shape,K_1	0.55
Coefficient for Angle of Embankment to Flow,K_2	0.96
Length of Active Flow Obstrubted by Embankment, L'	5
Average Depth of Flow on the Floodplain,y_a	8.67
Froude Number of Approach Section,F_r	0.55
	14.33 ft

$$\frac{y_s}{y_a} = 2.27\, K_1\, K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1 \qquad (8.1)$$

where:

K_1 = Coefficient for abutment shape (Table 8.1)
K_2 = Coefficient for angle of embankment to flow
K_2 = $(\theta/90)^{0.13}$ (see Figure 8.5 for definition of θ)
θ<90° if embankment points downstream
θ>90° if embankment points upstream
L´ = Length of active flow obstructed by the embankment, ft (m)
A_e = Flow area of the approach cross section obstructed by the embankment, ft^2 (m^2)
Fr = Froude Number of approach flow upstream of the abutment = $V_e/(gy_a)^{1/2}$
V_e = Q_e/A_e, ft/s (m/s)
Q_e = Flow obstructed by the abutment and approach embankment, ft^3/s (m^3/s)
y_a = Average depth of flow on the floodplain (A_e/L), ft (m)
L = Length of embankment projected normal to the flow, ft (m)
y_s = Scour depth, ft (m)

Abutment Riprap Sizing Calculation
HEC-23, Volume 2 Edition 3, Equation 14.1

100-yr Riprap Sizing			
	Froude	**Max Velocity (ft/s)**	**Max Water Depth (ft)**
Max	**0.29**	5.02	9.09

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

K	0.89	
Ss	2.65	
V	5.02	ft/s
g	32.20	ft^3/s
y	9.09	
D50	5.07	in

500-yr Riprap Sizing			
	Froude	**Max Velocity (ft/s)**	**Max Water Depth (ft)**
Max	**0.39**	7.25	10.51

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

K	0.89	
Ss	2.65	
V	7.25	ft/s
g	32.20	ft^3/s
y	10.51	
D50	10.57	in