Draft FIR Hydraulic Report

CDOT Project 23010
Replacement of Existing Structure F-20-J
US Highway 40

Prepared for:

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US Highway 40

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Introduction

Background

Region 4 of the Colorado Department of Transportation (CDOT) has proposed the replacement of the F-20-J timber bridge along U.S. Highway 40 (US 40E) over a tributary to East Bijou Creek. The existing F-20-J structure is about 56-feet in length with two rows of timber pile piers.

Site Location

The F-20-J bridge is located just north of the town of Deer trail, Colorado in Arapahoe county on US 40 at mile marker 349.6, upstream of the East Bijou Creek confluence. The legal location of the structure is the South East 1/4 of the North East 1/4 of Section 20, Township 5 South, Range 60 West of the Sixth Meridian. **Figure 1** shows the location of the bridge.

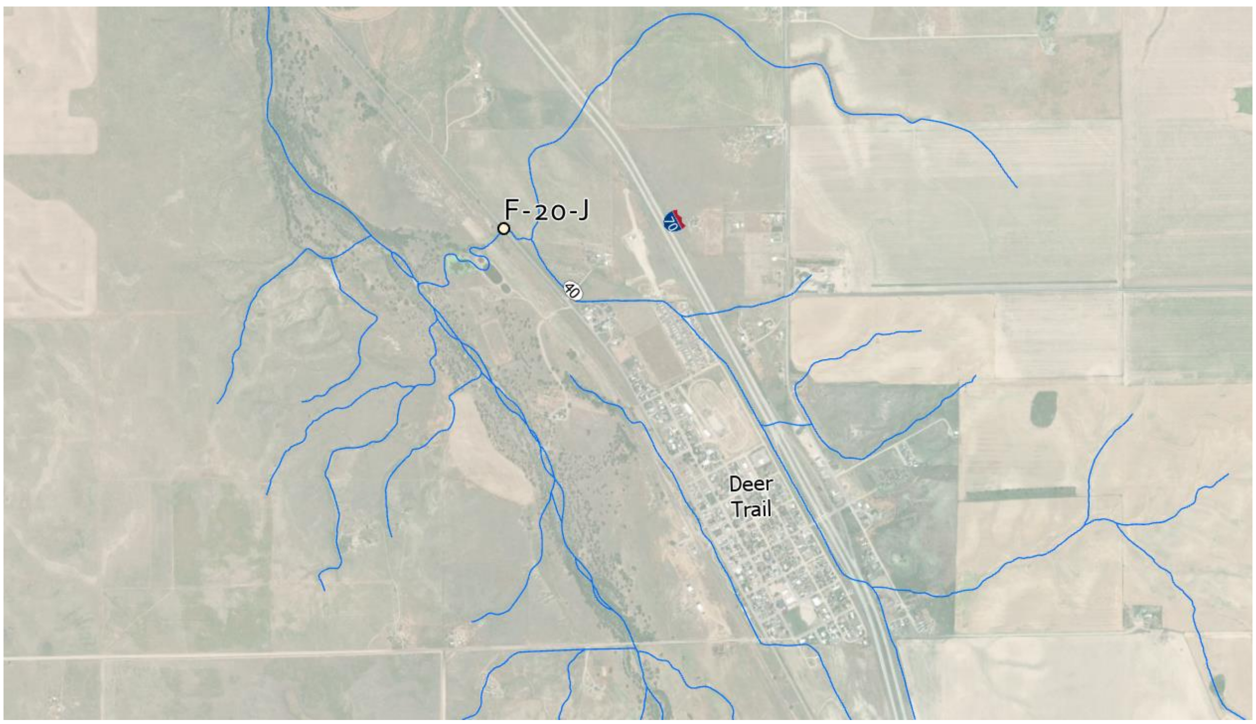

Figure 1: Project Location Map

Hydrology

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.

Regulatory Floodplain

The unnamed creek is located in FEMA Zone X, meaning that there are no effective flood elevations recognized by FEMA for this reach. The nearest floodplain to this reach is on the East Bijou Creek, which is about 2,000 feet downstream of the bridge location, and is mapped on FEMA community-panel number 08005C035K, effective on April 17th, 1989.

This unnamed creek has no designated floodway. CWCB regulations, based upon FEMA Regulation 44 CFR 60.3(c), for waterways with no designated floodway require that new development does not increase the water surface elevation from the Base Flood Elevation (BFE) by more than 0.5 feet at any location within the county, or by more than 0.00 feet at an insurable structure.

Design Flood Frequency

The CDOT Drainage Design Manual Table 7.2 indicates that the scour design flood for the F-20-J 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.

Due to bridge F-20-J being located on an unnamed tributary, no stream gage data exists and a HEC-HMS analysis was performed by CDOT using Aquaveo's WMS to obtain the model parameters. The hydrologic analysis used the SCS method, assuming a SCS type II, 24-hour storm of depths 4.24 inches, 4.68 inches, and 6.46 inches for the 50-, 100-, and 500-year storms, respectively. The SCS curve number was used to determine losses and the SCS unit hydrograph was used as the transform method. The reported peak flows are summarized in **Table 1.**

Table 1: HEC-HMS peak flow results at bridge F-20-J

Recurrence Interval	Flowrate (cfs)
10-Year	415
50-Year	855
100-Year	1,090
500-Year	1,726

East Bijou Creek was also modeled because F-20-J lies about 1,700 ft away from the confluence with it. East Bijou Creek lies on a zone A floodplain, so no available hydraulic and hydrologic data exists from the FIS report and there are no USGS stream gages. Therefore USGS's Streamstats was used to determine the peak flows, using regional regression equations. The report is included in **Appendix C** and the peak flow values are summarized in **Table 2.**

Table 2: Streamstats peak flow results of East Bijou Creek

Recurrence Interval	Flowrate (cfs)
10-Year	10,100
50-Year	27,300
100-Year	38,900
500-Year	76,800

Existing Structure

The existing F-20-J structure is an 89-year-old bridge constructed by the State Highway Department in 1931. The bridge has 3 spans of about 18 feet each supported by two rows of 1-foot diameter cross-braced timber pile bents and abutments. **Figure 2**, **Figure 3**, and **Figure 4** show the downstream face, upstream face, and roadway of the existing structure, respectively.

A field reconnaissance was performed by Ayres engineers on July 24, 2019, and the Olsson geomorphologists on July 30, 2019 to evaluate the existing bridge, document the study site, assess geomorphic conditions, obtain soil samples, identify hydraulically significant features, and estimate Manning roughness coefficients. This evaluation, which is indicated in Olson's geomorphic report indicates that the unnamed drainage is not highly dynamic over engineering time scales, and that no significant long-term degradation is expected. Conditions at the bridge suggest relatively infrequent flows with flows rarely exceeding 1.5 to 2 feet deep.

Figure 2: Upstream face of Bridge F-20-J

Figure 3: Downstream face of Bridge F-20-J

Figure 4: Upstream face of Bridge F-20-J, looking northwest along roadway

Survey and Topography

LiDAR and aerial imagery were acquired by the State of Colorado in May of 2018. Ground truthing and collection of photo identifiable points (PIDs) were used to calibrate the aerial data. Additional topographic ground survey was collected by 105West in August of 2019 at and around the F-20-J structure. The LiDAR and survey were merged into a comprehensive terrain dataset 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, U.S. Survey Feet) coordinates on the North American Vertical Datum of 1988 (NAVD-88) for the purposes of this analysis and design.

Structure Design Discussion

Replacement Considerations

The existing F-20-J bridge has a poor conditions rating. CDOT has determined that the bridge will be replaced.

Design Frequency and Floodplain Impacts

This bridge is designed to the 100-year 24-hour storm event per Table 7.2 CDOT criteria. The existing structure is not within a FEMA mapped floodplain. The comparison of the proposed bridge during 100-year event conditions and the existing bridge are included in the report.

Design Alternatives and Proposed F-20-J Structure

The roadway over the proposed structures consists of two travel lanes with shoulders in both the eastbound and westbound directions. The existing bridge opening will be expanded to the west and east for proposed conditions. The proposed bridge has one 68.5 ft span from centerline bearing to centerline bearing, with a clear span of 66 ft, or 5 ft to the outside of the existing abutments. The deck thickness will be 40.8 inches or 3.4 ft and the low chord elevation will be 5172.78 ft (NAVD-88) at the North abutment and 5170.83 ft (NAVD 88) at the South abutment. The roadway height and width of the proposed structure will increase compared to the existing structure by about 1.8 ft and 13 ft, respectively. The proposed bridge will have deep-foundation spill-through abutments with 2:1 (H:V) slopes. The terrain directly upstream of the bridge will be kept the same as the existing conditions.

Existing Conditions Hydraulic Modeling

Aquaveo's SMS version 13.0.5 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.

Existing Conditions Hydraulic Modeling Approach

The existing conditions model represents conditions prior to any changes as a result of the project. The model extends approximately 2500 ft downstream of the confluence with the unnamed drainage. The mesh consists of 119,961 predominantly triangular mesh elements with quadrilateral mesh elements representing the bridge and road approach sections on US 40E. Element size ranges from 3-foot elements at the bridge to 60-foot elements along mesh boundaries. The existing trestle bents on bridge F-

20-J are likely to catch debris. Therefore, they are modeled as continuous wall piers and were incorporated in the model as 1-foot wide, 34-foot long obstructions in the mesh.

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 n roughness values used for this design are presented in **Table 3.**

Table 3: Mannings n roughness values

Description	Manning's n value
Main channel	0.03
Grass	0.03
Pavement	0.013
Light Vegetation	0.03
Medium Vegetation	0.035
Dense Vegetation	0.04
Developed	0.05
Gravel	0.02
Riprap	0.04
Railroad	0.04
Ponds	0.001
Houses	0.1
Culvert	0.08

The inflow boundary conditions were developed using the HEC HMS model discussed previously for the unnamed channel and Streamstats for East Bijou Creek. The outflow boundaries were developed as normal depths downstream of the bridge due to a lack of available data. Two inflow boundaries were used for the unnamed channel and East Bijou Creek with three outflow boundaries (East, Middle, and West) downstream of bridge F-20-J for the 50-, 100-, and 500-year simulations as shown in **Table 4**.

Table 4: Boundary Conditions

Recurrence Interval	Peak Flows from HEC-HMS* Analysis and StreamStats**(cfs)		Downstream Boundary Water Surface Elevations (ft-NAVD88)		
	Unnamed Channel*	East Bijou Creek**	East	Middle	West
50-Year	855	10,100	5157.11	5155.00	n/a
100-Year	1,090	18,600	5158.59	5154.00	5152.00
500-Year	1,726	38,900	5160.55	5155.00	5151.55

F-20-J Existing Conditions Hydraulic Results

A summary of the existing condition hydraulic properties from two upstream cross-sections is shown in **Table 5** with locations illustrated in **Figure 5.** In **Figure 5**, section 1 is located just upstream of the zone of the bridge's local hydraulic influences and is useful for evaluation upstream hydraulics. Section 2 is located just upstream of the bridge and is useful for evaluating freeboard and local hydraulics

Table 5: Existing Conditions Model Results for F-20-J

Recurrence Interval	Flowrate (cfs)	Section 1 WSEL (ft-NAVD-88)	Section 2 WSEL (ft-NAVD-88)	Section 1 Max Velocity (ft/s)		Section 2 Max Velocity (ft/s)	
				Max	Avg	Max	Avg
50-year	855	5172.13	5171.18	2.89	1.03	4.51	3.66
100-Year	1,090	5172.51	5171.60	2.78	2.30	5.04	4.14
500-Year	1,726	5173.40	5172.48	2.49	2.13	5.21	4.24

The existing structure passes the 100-year event without pressure flow and the 500-year event with pressure flow through the bridge opening. During both the 100-year and 500-year events, the flow contracts into the banks of the unnamed creek about 240 ft upstream of bridge F-20-J where a drainage ditch causes recirculating flow. Also note that there is roadway overtopping of US40E during both the 100-year and 500-year events,1500 ft upstream of the bridge where the channel bends and is forced through a culvert. During the 500-year event there is additional roadway overtopping over US40E about 480 ft upstream of the bridge that is about 400 ft wide.

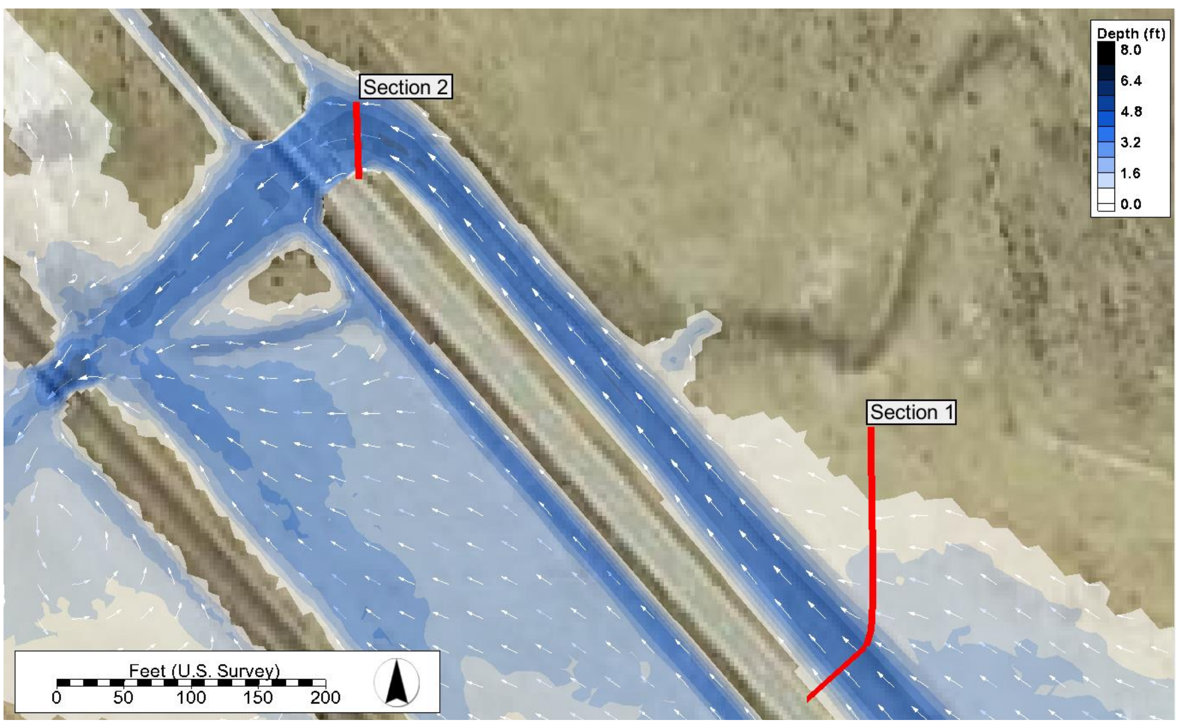

Figure 5: Existing structure F-20-J 100-year depth

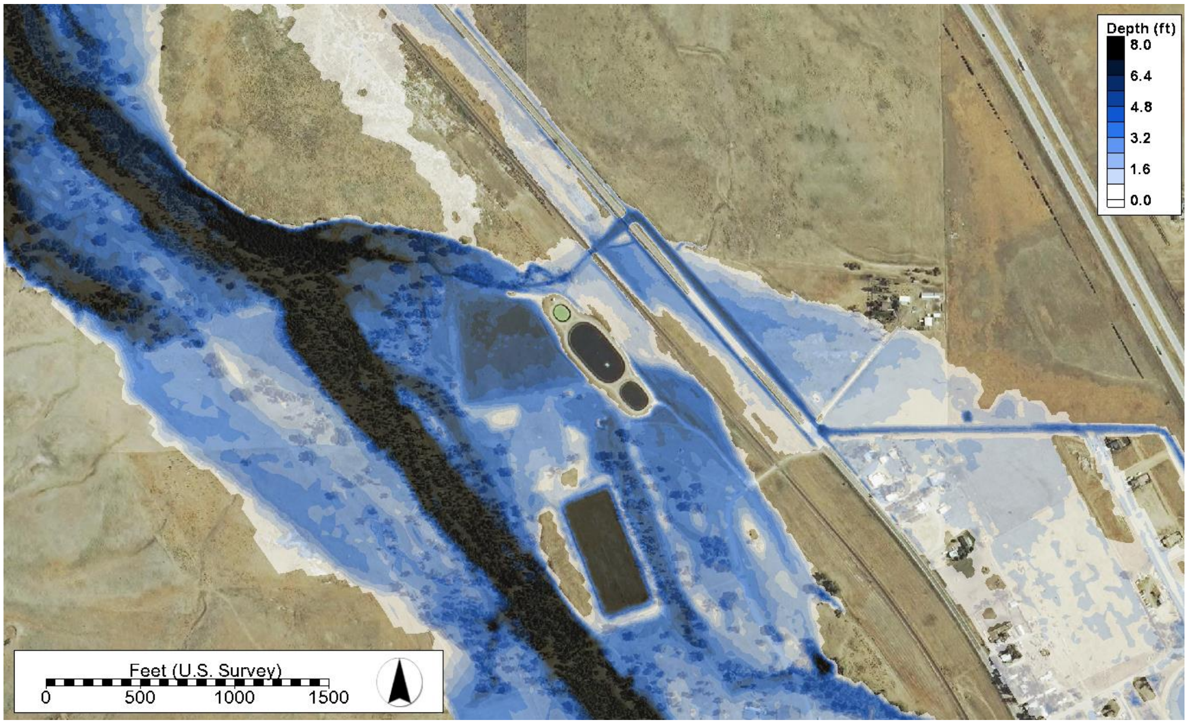

Figure 6: Existing structure F-20-J 500-year depth

Coincident Flows at the Stream Confluence

At a confluence there exists a coincidental probability of two hydrologic events occurring at the same time because the drainage areas have different sizes and time to peaks (Brown et al. 2009). For example, the 25-year event on the tributary and a 100-year event on the main channel and vice versa equate to a 100-year event near the confluence (Brown et al. 2009). It was first determined if East Bijou Creek influenced the hydraulics at F-20-J by performing a sensitivity analysis using a combination of flows. After comparing the 500-year event in the tributary (unnamed creek) and the 100-year event's velocity and depth in the main channel (East Bijou Creek) to the 500-year event in the tributary and the 50-year event in the main channel's velocity and depth, it was determined that the difference of the depth and velocity was less than 0.2 ft and ft/s (**Table 6**). To determine if East Bijou Creek had any effect on the bridge hydraulics during a lower flow event, the 50-year event on the tributary and the 50-year event on the main channel's velocity and depth were compared to the 50-year event on the tributary and 10-year event on the main channel's velocity and depth. It was determined that the difference between the two events' velocity and depth was less than 0.01 ft and ft/s (**Table 6)**.

It was evident that higher flow in the unnamed creek led to more severe hydraulics at the bridge after running the 10-year event in the unnamed channel and the 100-year event in East Bijou Creek and comparing the results to the 100-year event in the unnamed creek and the 25-year event in East Bijou Creek. Velocities and Depths were significantly higher during the 100-year event in the unnamed creek and 25-year event in East Bijou Creek (**Table 6**). Therefore, East Bijou Creek had a minimal impact on the hydraulics at F-20-J.

Table 6: Unnamed creek and East Bijou Creek's recurrence interval difference in WSEL and Velocity at Bridge F-20-J

Joint Recurrence Interval	Recurrence Interval on Unnamed Creek	Recurrence Interval on East Bijou Creek	Maximum Difference in WSEL at F-20-J (ft)*	Maximum Difference in Velocity at F-20-J (ft/s)*
50-year	50-year	50-year	0.007	-0.008
	50-year	10-year		
100-year	100-year	25-year	1.3	3.35
	10-year	100-year		
500-year	500-year	100-year	0.064	-0.14
	500-year	50-year		

*Note that a positive value indicates that the recurrence intervals in the left scenario has a greater value than the recurrence intervals in the right scenario. There are two scenarios for every joint recurrence interval.

Proposed Conditions Hydraulic Modeling

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 approximately held at its existing location, and few modifications to the numerical mesh configuration at the bridge were necessary. The wall pier obstructions at the existing bridge were removed. The materials coverage was also 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 and grading within the channel. Please note that because the exact roadway design and bridge layout are unknown, the results are preliminary. Another round of modeling will be conducted with the final design surface

Proposed Conditions Hydraulic Results

A summary of the hydraulic properties calculated from two approach sections upstream of Structure F-20-J for proposed conditions is shown in **Table 7** with locations illustrated in **Figure 5.** The sections are drawn perpendicular to flow.

Table 7: Proposed Conditions Model results for F-20-J

Recurrence Interval	Flowrate (cfs)	Section 1 WSEL (ft-NAVD-88)	Section 2 WSEL (ft-NAVD-88)	Section 1 Velocity (ft/s)		Section 2 Velocity (ft/s)	
				Max	Avg.	Max	Avg.
50-Year	855	5172.15	5171.14	2.87	2.20	4.61	3.63
100-Year	1,090	5172.64	5171.58	2.74	2.32	5.07	4.11
500-Year	1,726	5173.39	5172.42	2.47	1.93	5.32	4.44

Figure 7, **Figure 8**, and **Figure 9** display a comparison of existing to proposed velocity values during the 50-, 100-, and 500-year events. During all events the velocity increases through more than half of the

bridge, starting from the north abutment by less than 2 ft/s during proposed conditions. The flow must make a sharp bend, and the flow is guided less abruptly during existing conditions due to wingwalls protecting the abutments. The proposed condition abutments are protected by riprap, creating a more abrupt turn through the bridge, resulting in a larger diameter eddy on the inside of the bend (near the south abutment) than during existing conditions. This leads to lower velocities on the inside of the bend, but a narrower vena contracta creating larger velocities during proposed conditions, outside of the eddy.

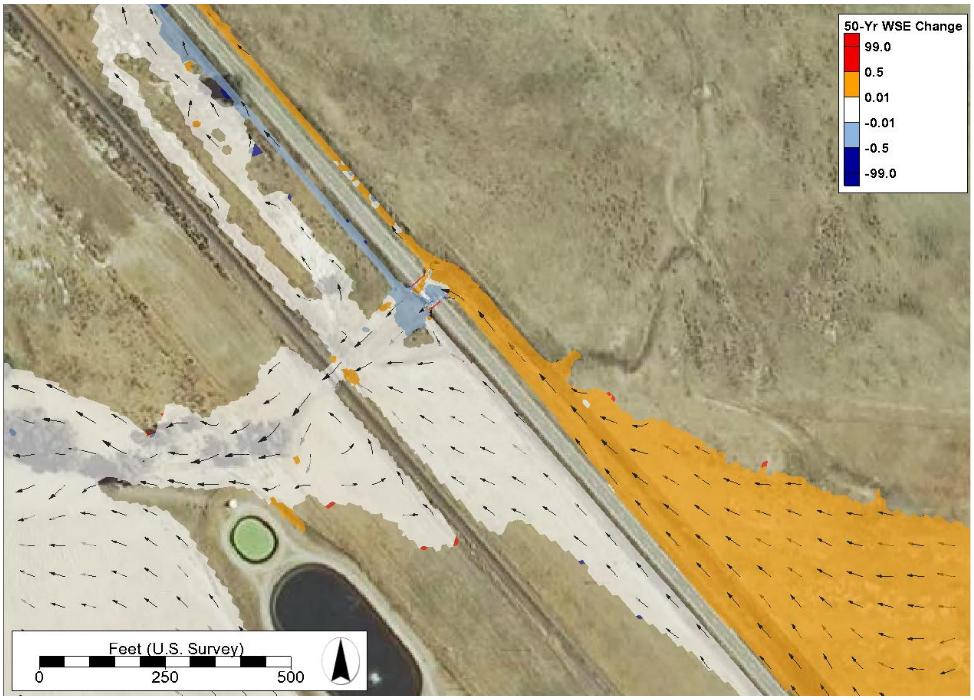

Figure 7: Proposed vs existing velocity change (orange and red indicate an increase in velocity) during the 50-year event

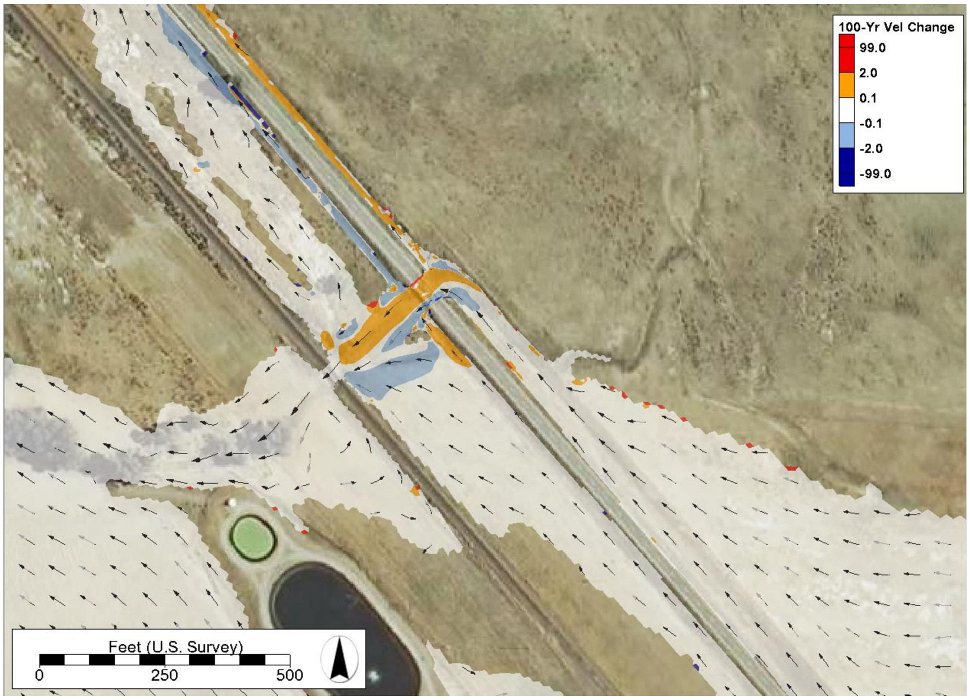

Figure 8: Proposed vs existing velocity change (orange and red indicate an increase in velocity) during the 100-year event

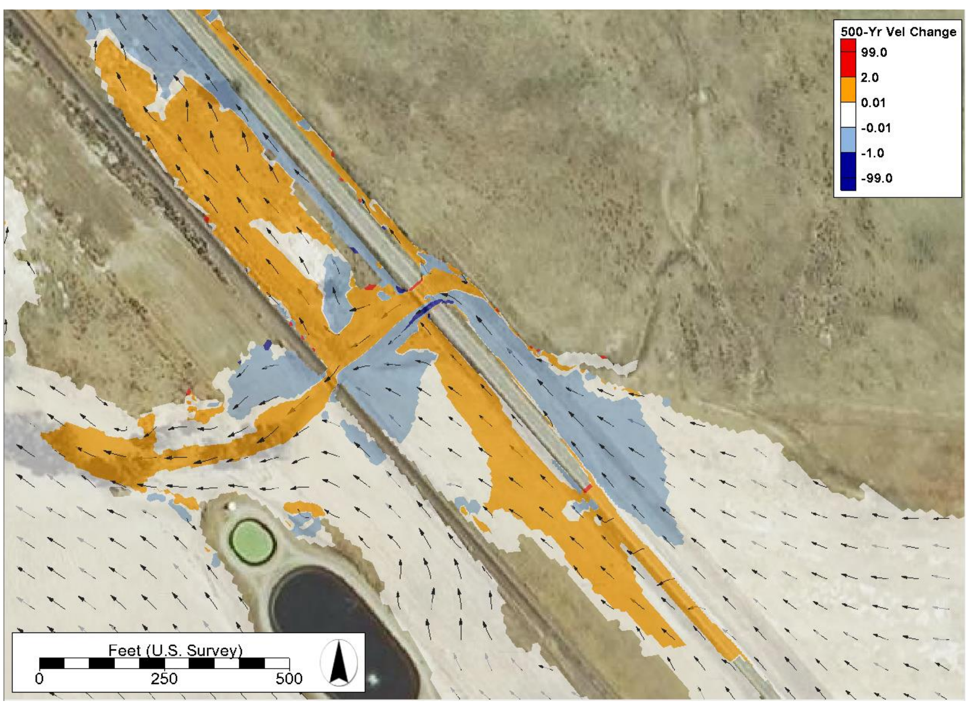

Figure 9: Proposed vs existing velocity change (orange and red indicate an increase in velocity) during the 500-year event

Figure 10, **Figure 11**, and **Figure 12** display a comparison of existing to proposed bed shear stress values during the 50-, 100-, and 500-year events. During all events, and similar to the velocity comparison above, the proposed conditions bed shear stress increases compared to the existing bed shear stress from the north abutment to more than half of the bridge span. This is due to the increase of velocity during the proposed conditions caused by the more abrupt change in flow direction than existing conditions, explained previously.

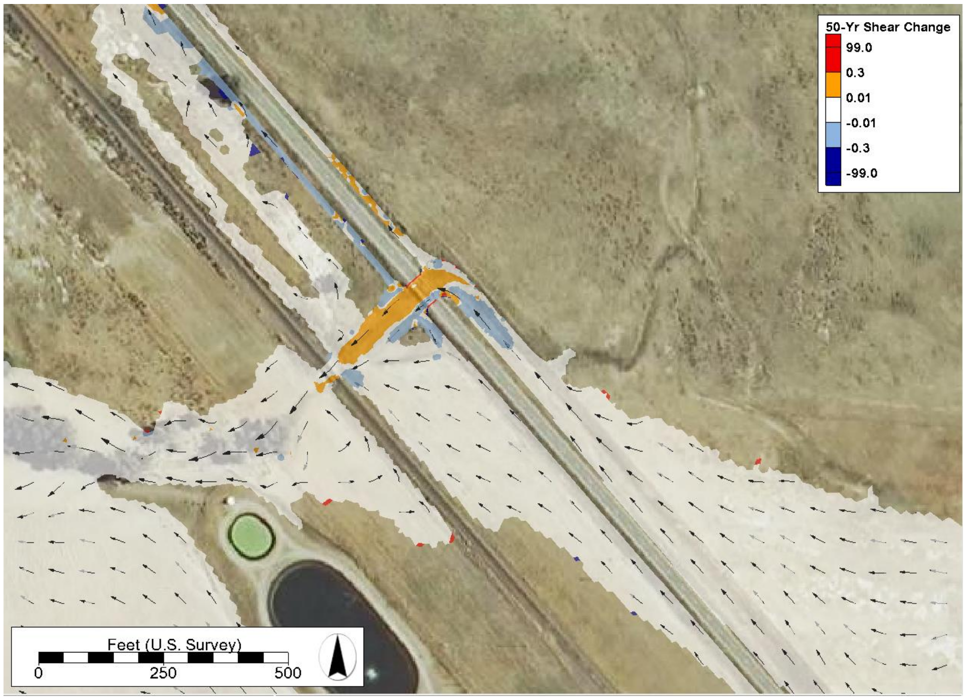

Figure 10: Proposed vs existing bed shear stress change (orange and red indicate an increase in shear stress) during the 50-year event

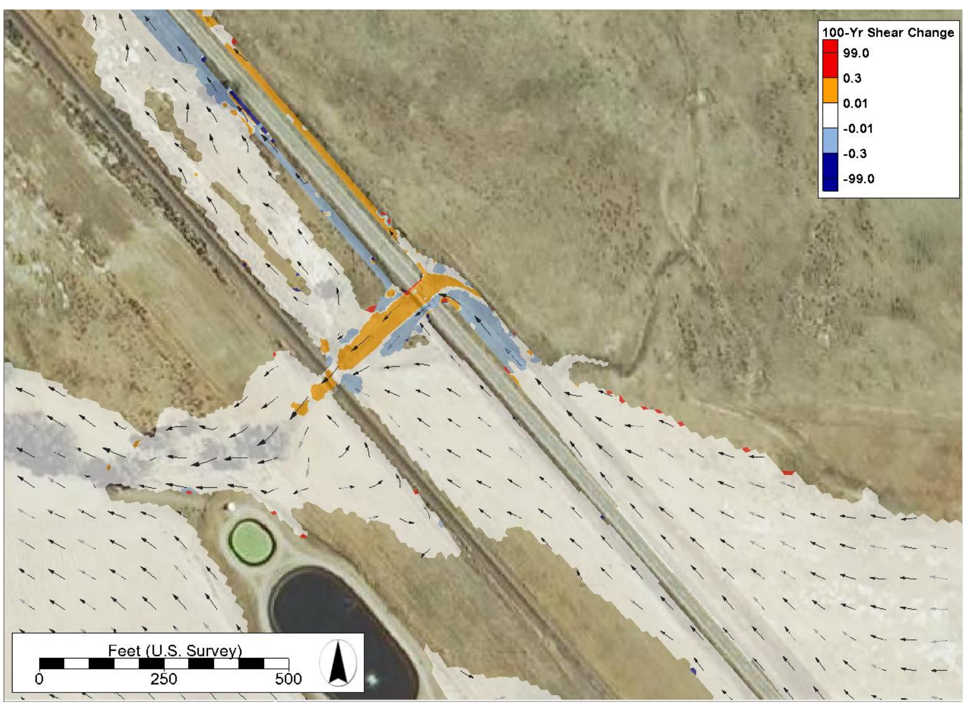

Figure 11: Proposed vs existing bed shear stress change (orange and red indicate an increase in shear stress) during the 100-year event

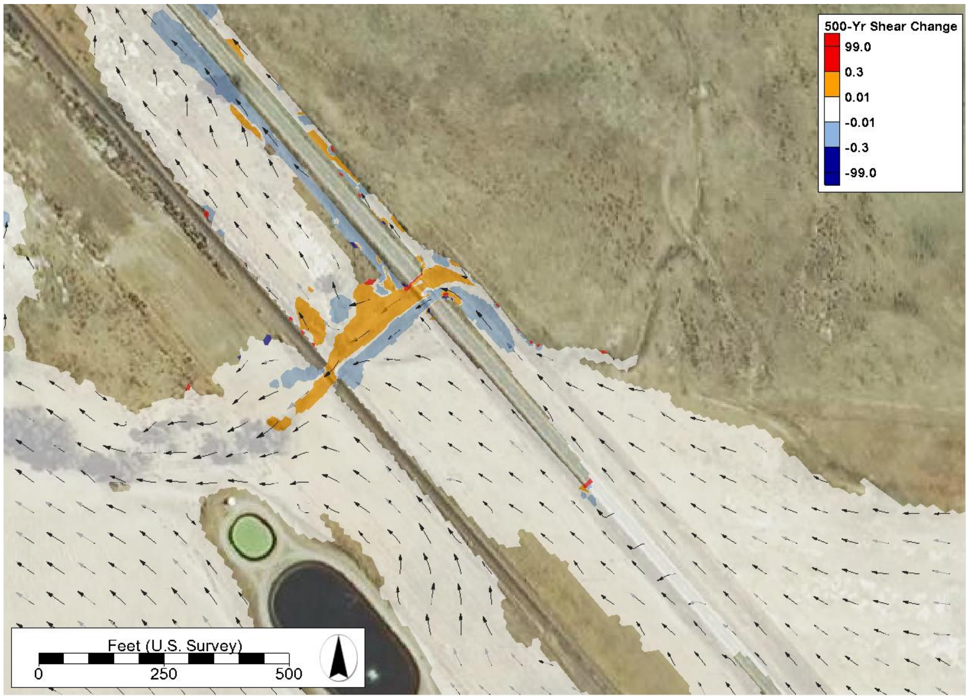

Figure 12: Proposed vs existing bed shear stress change (orange and red indicate an increase in shear stress) during the 500-year event

Proposed Comparison to Existing Water Surface Elevations

Analysis was performed using the Dataset Calculator within SMS to ensure no-adverse impact and improved hydraulic conditions of the proposed condition versus the existing condition. **Figure 13**, **Figure** 14**,** and **Figure** 15 show a comparison of the 50-, 100-, and 500-year event water surface elevations around the bridge. Outside of the immediate velocity of the proposed structure there is no WSEL rise greater than 0.5-foot, and no rises on insurable structures. In **Figure 14,** and **Figure** 15 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 a small area of increase upstream of the bridge. No insurable structures are impacted by WSEL changes associated with the proposed bridge design. Note that in **Figure 15,** which compares existing to proposed water surface elevations during the 500-year event**,** there is less impacted upstream of the bridge during proposed conditions due to the roadway overtopping.

Figure 13: Proposed vs existing WSEL change (orange and red indicate an increase in WSEL) during the 50-year event

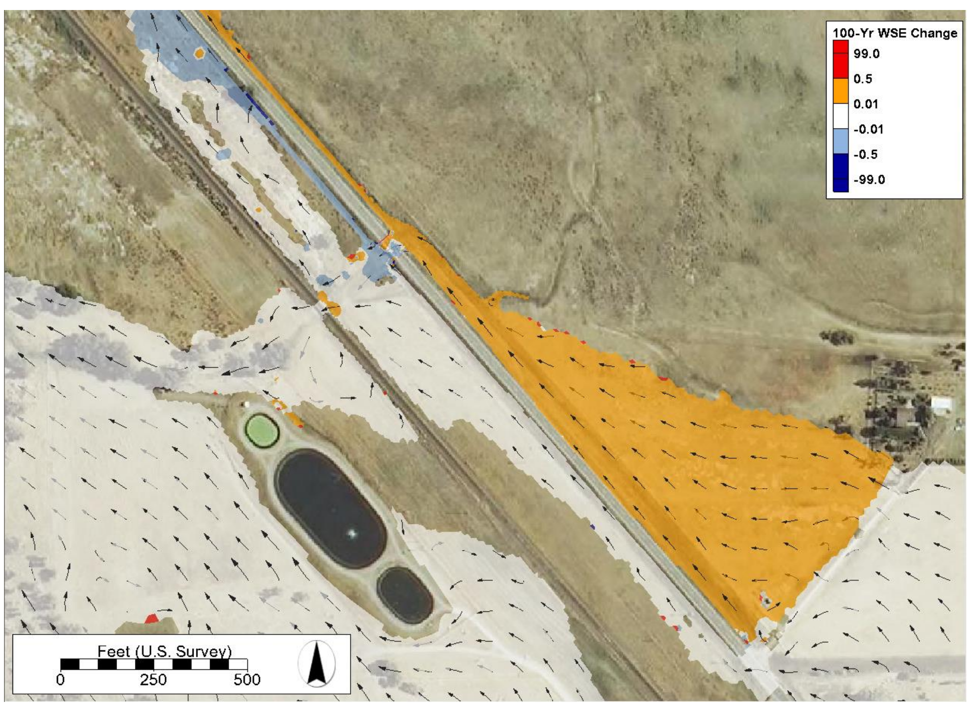

Figure 14: Proposed vs existing WSEL change (orange and red indicate an increase in WSEL) during the 100-year event

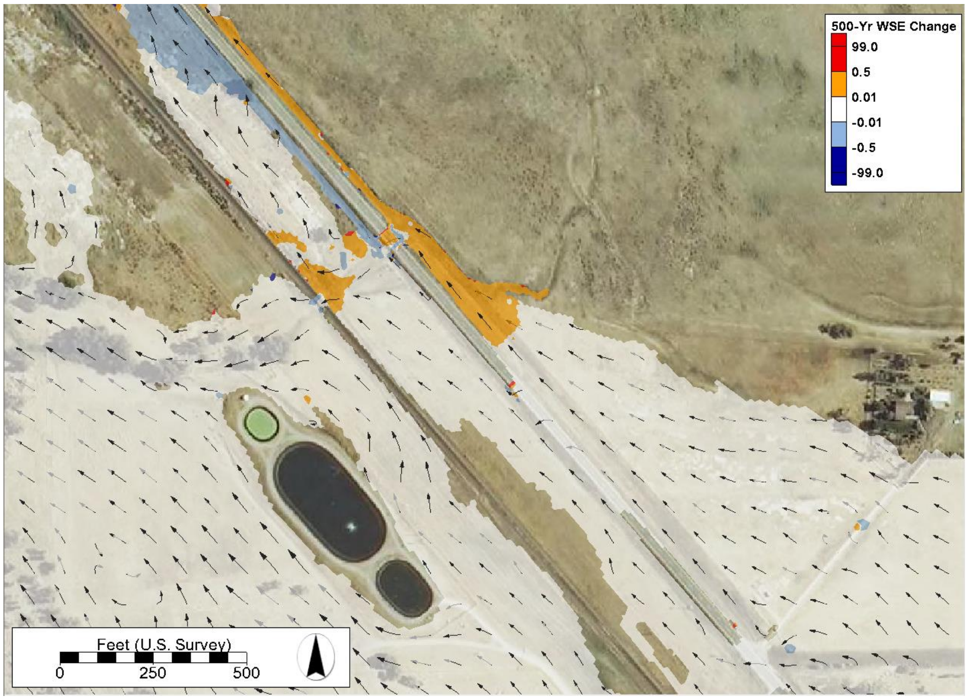

Figure 15: Proposed vs existing WSEL change (orange and red indicate an increase in WSEL) during the 500-year event

Freeboard Requirements

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

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

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 about 900 cfs through the bridge with an average velocity of 4.3 ft/s and a water surface elevation of 5171.78 (ft-NAVD-88). The result of these calculations is a low chord elevation no lower than 5172.69 (ft-NAVD-88). The Proposed design low chord is anticipated to be 5172.78 (ft-NAVD-88) at the location of two-thirds along the bridge length.

Scour and Countermeasures

Stream Stability

The unnamed creek flows intermittently through fat clay. Directly upstream of F-20-J the channel runs parallel to US 40 and is grass lined with a bankfull width of about 50 ft and a depth of about 5 ft.

Because the channel is grass lined and is constrained by the roadway embankment upstream of the bridge, it is likely that the channel will remain stable over engineering time scales, which is concluded in Olsson’s geomorphic report, attached in **Appendix D**. Downstream of F-20-J exists the railroad bridge, where flow is directed with manmade, vegetated banks. Therefore, it is unlikely the overall orientation and alignment of the larger channel will change significantly within the design life of the proposed structure.

Scour Potential

Scour potential at the proposed structure was analyzed for the 100-year design flood and the 500-year 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.

Scour Variables

Critical Velocity for Cohesive Soils

Clear-water scour equations assume sand-bed channels with no cohesive properties. The channel is composed of high plasticity loamy clay soils that is expected to exhibit cohesive properties, which results in a higher critical velocity than would be predicted using non-cohesive sediment equations. To account for this, the critical velocity was estimated based on properties of soil using Mirtskhoulava's simplified expression for cohesive sediments, shown below (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 (1997). Because these two parameters are relative unknowns for this soil, conservative estimates were made to estimate C_o. Bed material was assumed to be high-plasticity clay (liquidity index between 0.5-0.75) based on the sieve analysis and geomorphic report. A voids ratio of 1.05 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**. Note calculations were done in SI units.

Table 8. Depth-averaged critical velocity for cohesive soils (Mirtskhoulava 1988)

Description	Variable	100-year	500-year
Equilibrium depth of flow (m) POST-SCOUR.	h	2.3	2.9
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	1.0	1.0
Depth averaged critical velocity (m/sec).	U_c	0.60	0.61
Depth-averaged critical velocity (ft/sec)	$\mathbf{U_c}$	**1.97**	**2.01**

The D_{50} was adjusted so that the critical depth (Y_2) calculated using the time dependent contraction scour method matched the critical depth found using Mirskhoulava's critical depth value.

Time-Rate Contraction Scour

Because scour in cohesive soils occurs at a much slower rate than in sand-bed channels, it is reasonable to assume that the maximum potential scour would take multiple large storm events to produce. To account for this, clear-water scour calculations were adjusted to represent the scour depth that is likely to be reached over a single storm event for the 100-year and 500-year storms. 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. Storm duration was determined using flood hydrographs for the 100-year and 500-year events. Duration was measured as the amount of time that velocities were expected to be above the critical velocity of the soil. The 100-year hydrograph is shown in **Figure 16**. Time-dependent scour for the 100-year event is shown in **Figure 17.**

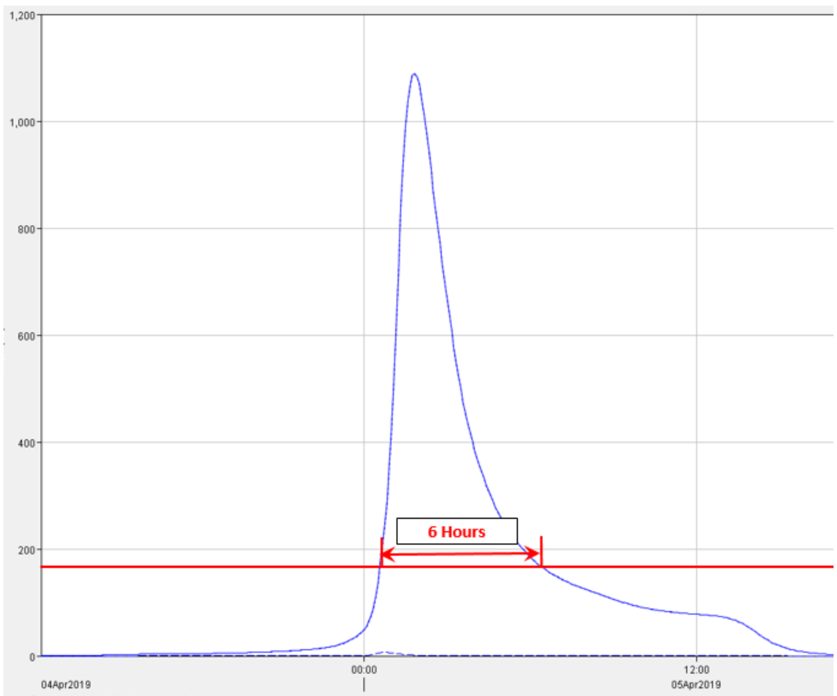

Figure 16: Flood hydrograph for the 100-year event

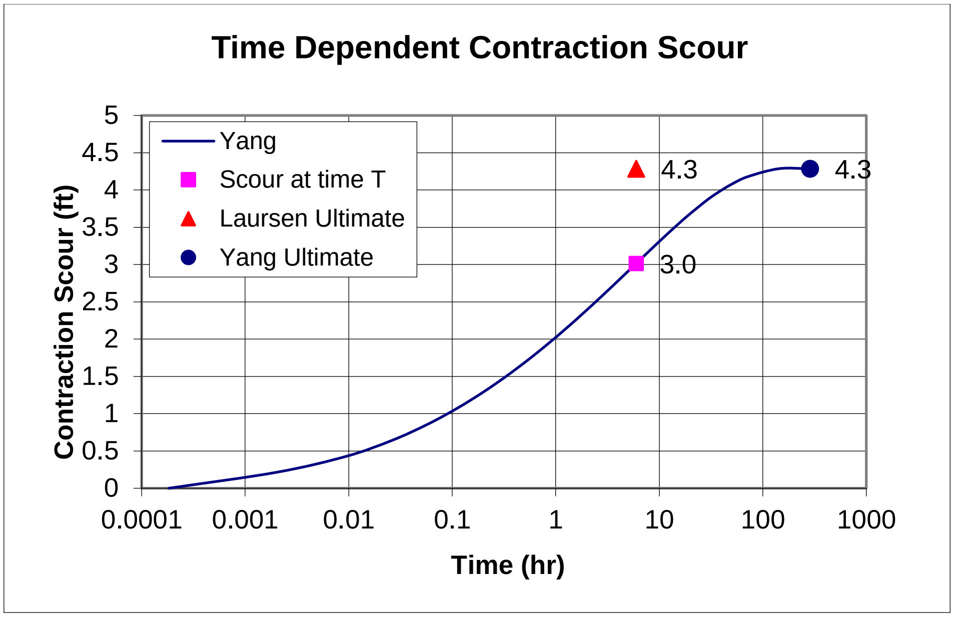

Figure 17: Time-dependent clear-water contraction scour for the 100-year event

Clear-Water Abutment Scour

Clear-water abutment scour applies an amplification factor to clear-water contraction scour based on abutment shape and the ratio between contracted unit discharge and upstream unit discharge. Amplification factors for the 100-year and 500-year events were determined using Figure 8.11 of the HEC-18 manual for spill-through abutments and clear-water conditions applied to the predicted average flow depth, post-scour.

Scour Summary

Scour calculations are included in **Appendix B** and summarized in **Table 9**. The soil samples indicated that the foundation soils are cohesive. Unit discharges for the 100-year event, as well as the location under the bridge where scour calculations were applied, is shown in **Figure 18**.

Table 9. Proposed Conditions Scour Summary

Recurrence Interval	Flowrate (cfs)	Critical Velocity (ft/s)	Storm Duration (hrs)	Contraction Scour (ft)	Abutment Scour (ft)	Contraction Scour Elevation (ft-NAVD)	Abutment Scour Elevation (ft-NAVD)
100-Year	957	1.97	6	3.0	6.2	5164.0	5160.8
500-Year	1,133	2.01	8	3.5	6.5	5163.5	5160.5

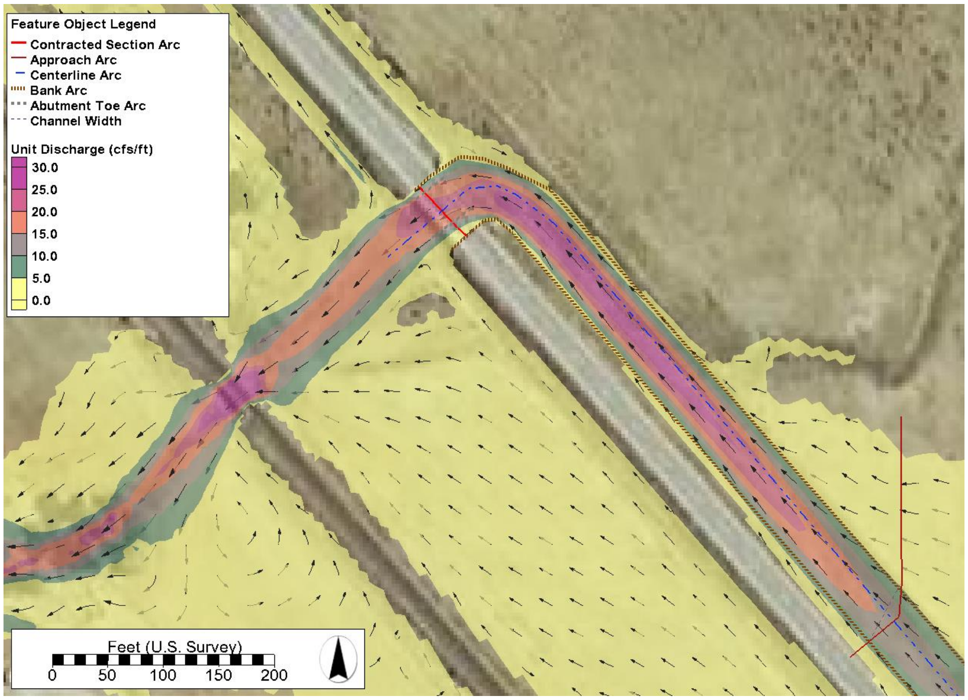

Figure 18: Approach arc with unit discharge map

Proposed Scour Countermeasures

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. In order to prevent erosion to the abutment slopes and mitigate scour, abutment rock riprap countermeasures were designed using the methods described in Hydraulic Engineering Circular Number 23 (HEC-23), Volume 2, *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance* 3rd edition engineering manual, published by the Federal Highway Administration.

The countermeasure calculations are included in **Appendix B** and indicate that a 2-foot thick section of 12-inch rock riprap, extending into the channel, with a width of $2Y_0$ or 8.3 ft and 1 ft below the thalweg will be required for 500-year event flow conditions (**Figure 19**). Please note that scour depths are measured downwards from the channel invert (thalweg) elevation. This practice is appropriate for several reasons: first because the low flow channel is expected to migrate laterally over time, secondly because the formulation of the contraction scour equations is based upon the assumption of uniform scour across the cross-section, and lastly because of the uncertainty associated with specific scour locations. The riprap will extend down an additional 0.4 ft to 500-year contraction scour.

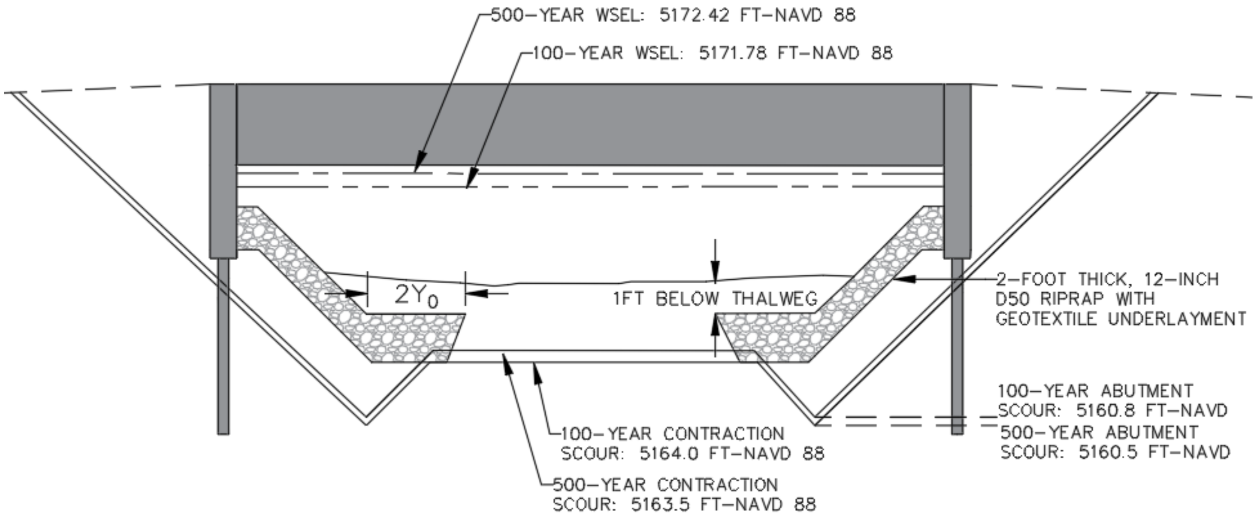

Figure 19: Proposed bridge cross-section, including scour countermeasures. Please note that the vertical scale is exaggerated

Conclusion

The existing F-20-J structure along US 40 over the unnamed creek will be replaced by CDOT. This preliminary hydraulic analysis has concluded that a 66-foot clear span bridge will be a suitable replacement, causing no WSEL increase in excess of the maximum 0.5' allowed in Zone X floodplains nor have any adverse impact during the 100-year flood event. A scour analysis has been performed for the 100 and 500-year floods producing scour elevations of 5161.38 and 5160.12 (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 the spill-through abutment slopes and recommends a 2-foot thick section of 12-inch rock riprap. Please note that the countermeasure design, as configured, will not protect roadway embankment fill against the expected 100-year event scour. Also, once the finalized roadway and bridge configuration are completed, the 2-D model should be updated with the finalized surface and reran, as the bridge hydraulics may change.

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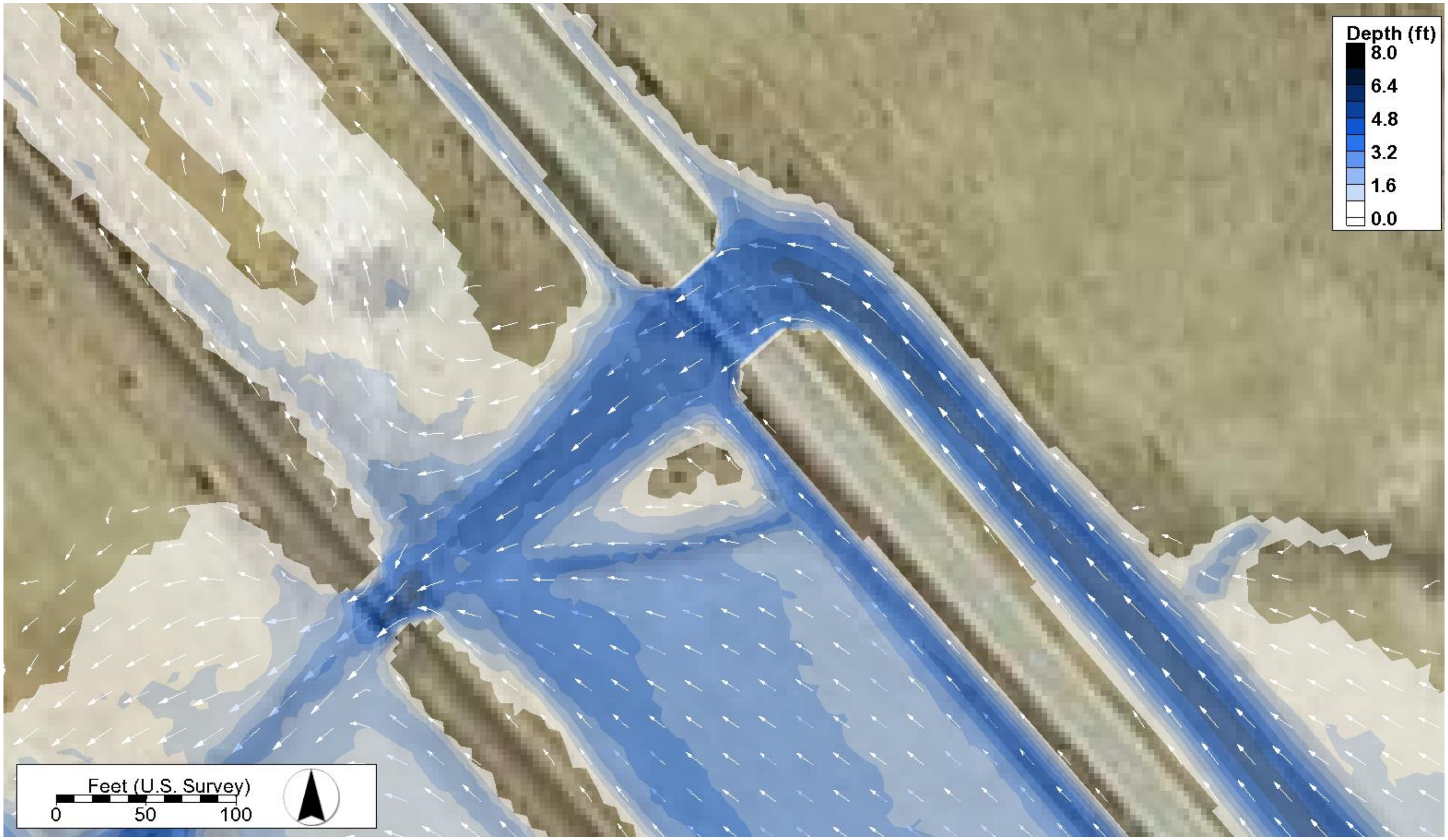
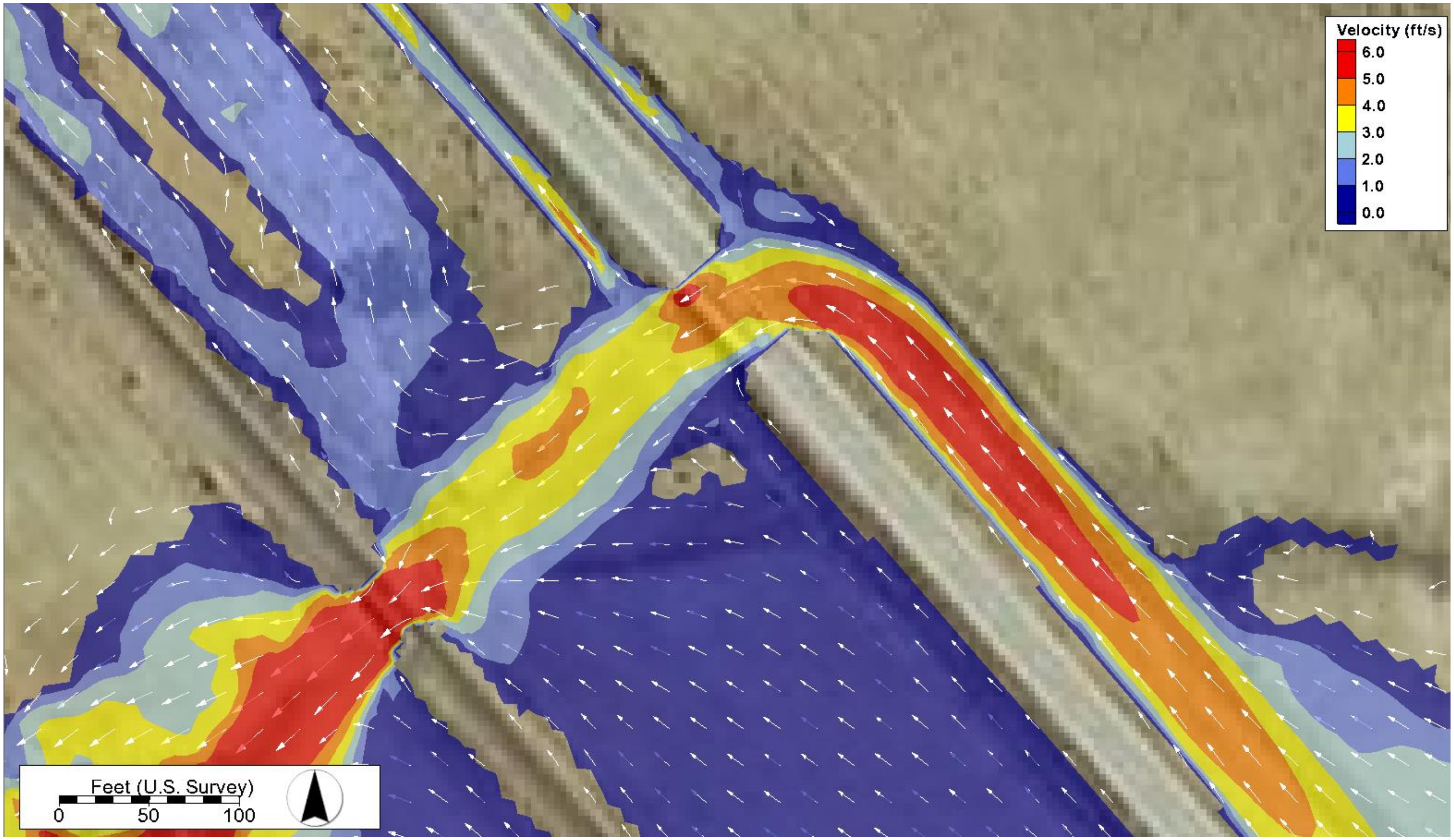
Appendix A

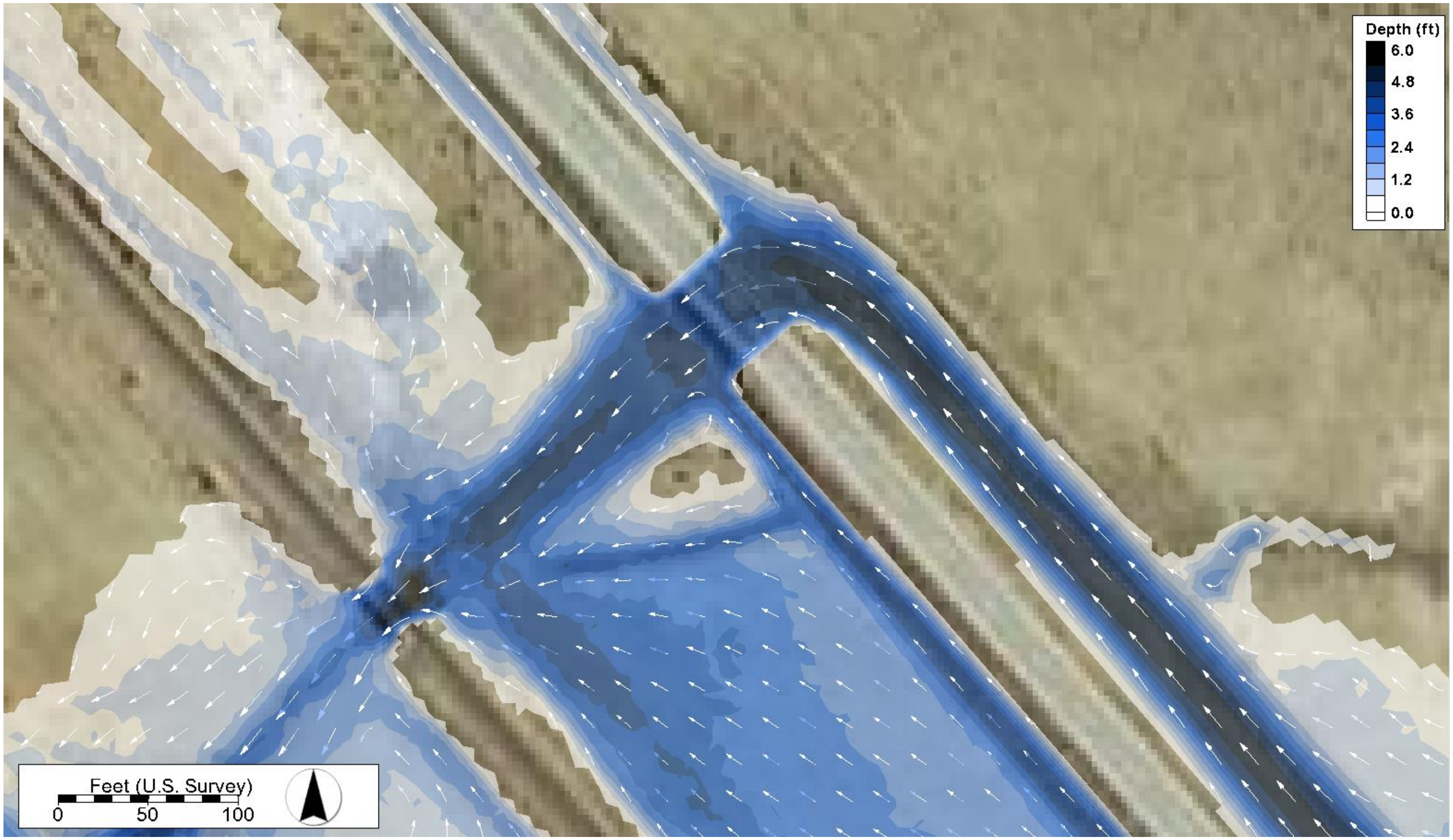
Hydraulic Modeling



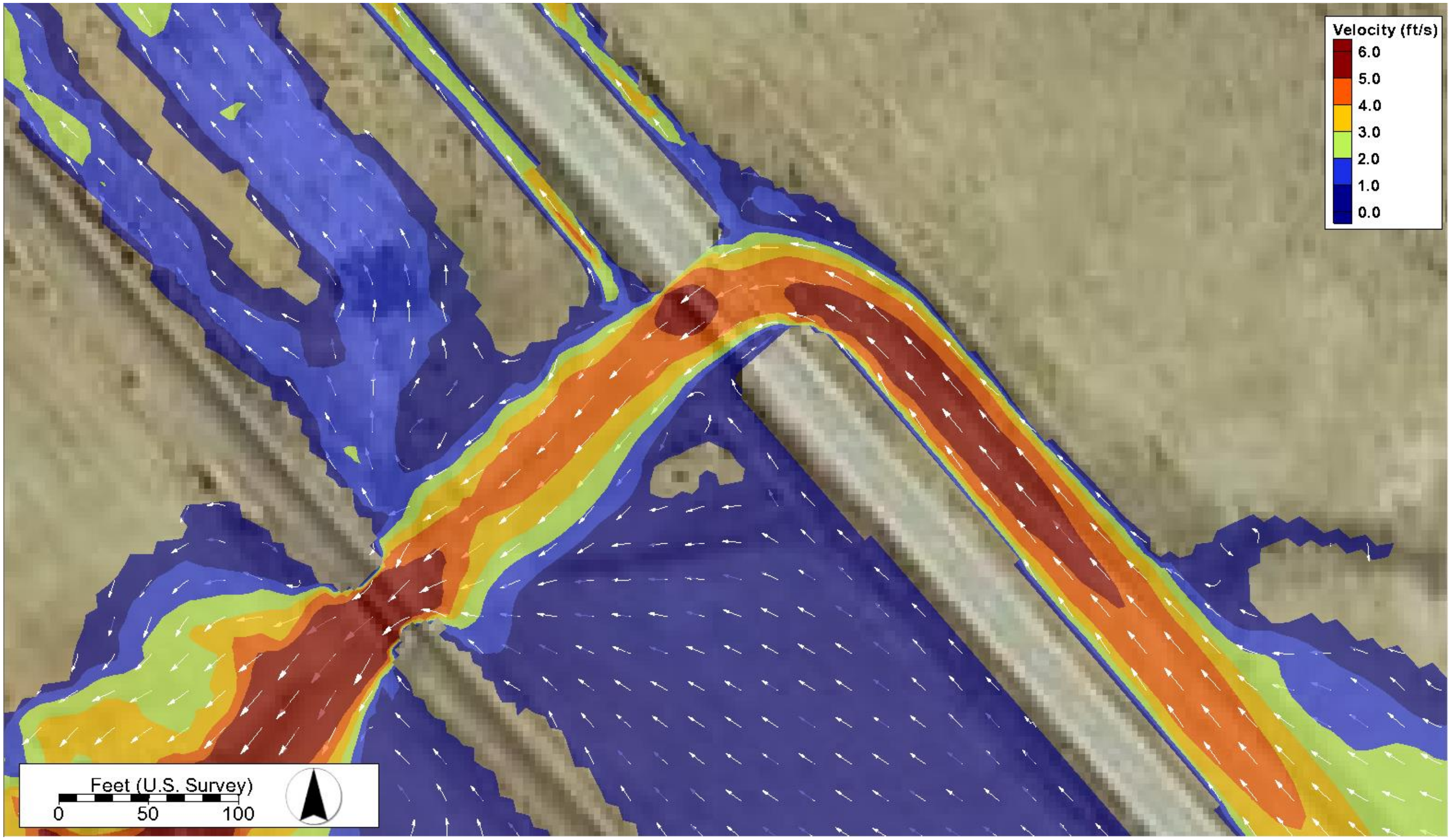
Existing conditions, 100-year, depth



Existing conditions, 100-year, velocity



Proposed conditions, 100-year, depth



Proposed conditions, 100-year, velocity

Appendix B

Scour and Countermeasures Calculations

Hydraulic Data from I:\36\36-464822_Eastern Plains BE Bridges\Hydraulics\G-21-A v13.0.10\G-21-A.sms

	100-yr. Flood Data				500-yr. Flood Data			
Hydraulic:	Y_o	=	3.35	ft. adjacent	Y_o	=	4.13	ft. adjacent
	Left Y_1	=		ft. adjacent	Left Y_1	=		ft. adjacent
	Right Y_1	=		ft. adjacent	Right Y_1	=		ft. adjacent
	Q_1	=	957	cfs	Q_1	=	1,295	cfs
	Channel Q_2	=	902	cfs	Channel Q_2	=	1,133	cfs
	Left Q_2	=		cfs	Left Q_2	=		cfs
	Right Q_2	=		cfs	Right Q_2	=		cfs
	A_1	=	391.48	$ft.^2$	A_1	=	566.55	$ft.^2$
	A_2	=	201.11	$ft.^2$	A_2	=	248.05	$ft.^2$
	W_1	=	195.30	ft.	W_1	=	238.05	ft.
	Channel W_2	=	60.00	ft.	Channel W_2	=	60.00	ft.
	Left W_2	=		ft.	Left W_2	=		ft.
	Right W_2	=		ft.	Right W_2	=		ft.
	D_{50}		0.200	mm	D_{50}		0.200	mm
	Energy Slope	=	2.200E-03	ft/ft	Energy Slope	=	2.000E-03	ft/ft
	Gravity Acceleration	=	32.2	ft/sec^2	Gravity Acceleration	=	32.2	ft/sec2
	Velocity @ Abutment Toe	=	3.32	ft/sec			6.39	ft/sec
	Depth @ Abutment Toe	=	7.10	ft			9.16	ft
	Fall Vel., ω	=	0.0906646	fps	Fall Vel., ω	=	0.0906646	fps
	V_m	=	2.44	fps	V_m	=	2.29	fps
	D_{50}	=	0.000656	ft.	D_{50}	=	0.000656	ft.

*D98 = 0.074 mm. D_{50} not reported by sieve analysis.

BY: **MWG**

DATE: **13-Feb-20**

100-YEAR SCOUR SUMMARY
FOR STATE HWY 40 OVER UNNAMED CREEK
BRIDGE F-20-J
ARAPAHOE COUNTY, COLORADO

FEBRUARY 2020

Pier/Bent	Groundline Elevation (ft-NAVD88)	Initial Embedment (ft)	Contraction Scour (ft)	Local Scour (ft)	Total Scour (ft)	Scour Elevation (ft-NAVD88)
1	5167.0	--	3.0	6.2	6.2	5160.8
2	5167.0	--	3.0	6.2	6.2	5160.8

500-YEAR SCOUR SUMMARY
FOR STATE HWY 40 OVER UNNAMED CREEK
BRIDGE F-20-J
ARAPAHOE COUNTY, COLORADO

FEBRUARY 2020

Pier/Bent	Groundline Elevation (ft-NAVD88)	Initial Embedment (ft)	Contraction Scour (ft)	Local Scour (ft)	Total Scour (ft)	Scour Elevation (ft-NAVD88)
1	5167.0	--	3.6	6.5	6.5	5160.5
2	5167.0	--	3.6	6.5	6.5	5160.5

NOTES:

If a soil horizon exists beneath the bridge which is resistant to scour, the predicted scour depths could be reduced to reflect the competence of the material. This reduction would require examination and approval by a qualified geotechnical engineer with knowledge of the properties of the material. The analysis considered both cohesive soil effects and the limited duration of scouring flows at the site.

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SCOUR MODE COMPUTATION
FOR STATE HWY 40 OVER UNNAMED CREEK
BRIDGE F-20-J
ARAPAHOE COUNTY, COLORADO

February 2020

The following computations are made using Laursen's Equation (Equation 4.2 in HEC-18):

$$V_c = Ku \times Y_1^{1/6} \times D_{50}^{1/3}$$

100-YEAR DISCHARGE
MAIN CHANNEL SCOUR MODE

APPROACH SECTION MAIN CHANNEL AREA (ft^2), A_1	=	391
APPROACH SECTION MAIN CHANNEL WIDTH (ft), W_1	=	195
APPROACH SECTION AVERAGE CHANNEL DEPTH (ft), $Y_1 = A_1/W_1$	=	2.00
MEDIAN GRAIN SIZE (ft), D_{50}	=	0.000656
Ku		11.17
BED TRANSPORT CRITICAL VELOCITY (fps), V_c	=	1.09
DISCHARGE IN APPROACH CHANNEL (cfs), Q_1	=	957
MEAN VELOCITY IN APPROACH CHANNEL (fps), V_m	=	2.44
MAIN CHANNEL SCOUR MODE	=	~~LIVE-BED~~

*Assumed CLEAR-WATER because channel is vegetated and V is approximate to V_c for vegetation

500-YEAR DISCHARGE
MAIN CHANNEL SCOUR MODE

APPROACH SECTION MAIN CHANNEL AREA (ft^2), A_1	=	567
APPROACH SECTION MAIN CHANNEL WIDTH (ft), W_1	=	238
APPROACH SECTION AVERAGE CHANNEL DEPTH (ft), $Y_1 = A_1/W_1$	=	2.38
MEDIAN GRAIN SIZE (ft), D_{50}	=	0.000656
Ku		11.17
BED TRANSPORT CRITICAL VELOCITY (fps), V_c	=	1.12
DISCHARGE IN APPROACH CHANNEL (cfs), Q_1	=	1,295
MEAN VELOCITY IN APPROACH CHANNEL (fps), V_m	=	2.29
MAIN CHANNEL SCOUR MODE	=	~~LIVE-BED~~

*Assumed CLEAR-WATER because channel is vegetated and V is approximate to V_c for vegetation

Calc. By:	MWG	Date:	2/13/2020
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CRITICAL AVERAGE VELOCITY CALCULATION
FOR STATE HWY 40 OVER UNNAMED CREEK
BRIDGE F-20-J
ARAPAHOE COUNTY, COLORADO

February 2020

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

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

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 fat clay (high plasticity) with a liquidity index between 0.5 and 0.75 based on the geomorphic report. A void ratio of 1.05 was assumed for a slightly more conservative result [Table 2.5, Hoffman, 1997]

SOIL PROPERTIES

Porosity	n	=	0.58
voids ratio	e	=	1.381
Specific Gravity	G	=	2.65
Saturated Water Content	w_s	=	0.521
Liquid Limit	ll	=	0.57
Plastic Limit	pl	=	0.23
Plasticity Index	pi	=	0.34
Liquidity Index	li	=	0.86
Soil Type (USCS)		=	C and D
Cohesion of Clay (Pa) [From Table 2.5, Hoffman, 1997].	C_o	=	28.4

COHESIVE SOIL CRITICAL VELOCITY

EVENT			**100-year**	**500-year**
Computed depth of contracted section (ft), Y_2	y_2	=	7.64	9.38
Equilibrium depth of flow (m) Post Scour.	h	=	2.3	2.9
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	=	1.0	1.0
Depth averaged critical velocity (m/sec).	U_c	=	0.60	0.61
Depth averaged critical velocity (ft/sec)	U_c	=	1.97	2.01
Applied Depth Averaged Critical Velocity (ft/sec)	**U_c**	**=**	**1.97**	**2.01**

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CONTRACTION SCOUR COMPUTATIONS
FOR STATE HWY 40 OVER UNNAMED CREEK
BRIDGE F-20-J
ARAPAHOE COUNTY, COLORADO

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:

$Y_2 = V_0 Y_0 / V_c$

and :

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

100-YEAR AND 500-YEAR DISCCHARGE
MAIN CHANNEL
CLEAR-WATER CONTRACTION SCOUR CALCULATIONS

		100-yr Condition	500-yr Condition
AVERAGE VELOCITY IN CONTRACTED SECTION (fps), V_0	=	4.49	4.57
AVERAGE WATER DEPTH (ft), Y_0	=	3.35	4.13
CRITICAL VELOCITY (fps), V_c	=	1.97	2.01
COMPUTED DEPTH OF CONTRACTED SECTION (ft), Y_2	=	7.64 ✓	9.37
DEPTH OF CONTRACTION SCOUR (ft), Y_s	**=**	**4.29** ✓	**5.24**

Calc. By:	MWG	Date:	2/13/2020
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100-YEAR TIME SCOUR SUMMARY
FOR STATE HWY 40 OVER UNNAMED CREEK
BRIDGE F-20-J
ARAPAHOE COUNTY, COLORADO

FEBRUARY 2020

	Approach	Bridge (t=0)	Live-Bed Bridge (ultimate)	Clear-Water Bridge (ultimate)
Manning n	0.03			
Q (cfs)	957	902		
W (ft)	195.30	60.00		
Y (ft)	2.00	3.35	4.45	7.64
A (ft^2)	391	201.1	267	458
V (ft/s)	2.44	4.49	3.38	1.97
Sf	0.000963	0.001635	0.000636	0.000105

D50mm	0.454	*Iterate D_{50} until Y_2 (clear water) below matches cohesive clear-water Y_2 CD.
D50ft	0.001490	
Vcrit (ft/s)	4.90	HARD_WIRED, assumed Clear-Water ✓
Sediment Sg	2.65	
Fall vel. (ft/s)	0.231	
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)	0.249	0.420	0.302	0.161
Re*	34.42	57.98	41.66	22.18
Vcr/omega	2.35	2.13	2.26	2.60
log(C)	2.523	3.178	2.536	1.121
C ppm-wt	333.6	1507.3	343.6	13.2
Qs(cfs)	0.120	0.513	0.117	0.004

Clear Water Controls

Y2 (clear water)	7.64
Y2 (ultimate live-bed)	4.45
Qs Ratio	0.971
	Run Goal Seek
Laursen Ys(ultimate)	4.3
Yang Ys(ultimate)	4.3

Scour Time T (hr)	6	C/Report Narrative
Scour at Time T (ft)	**3.0**	

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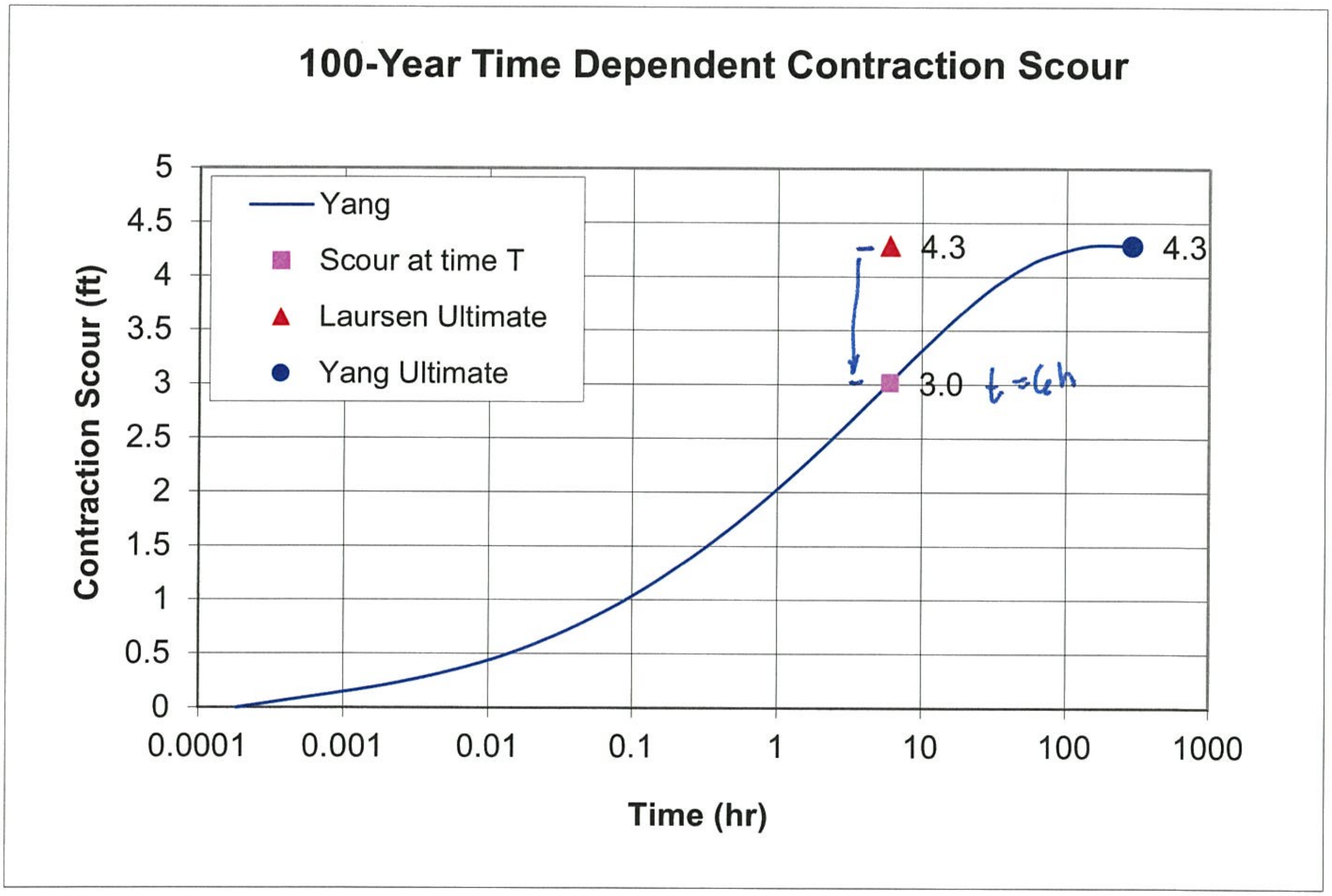

100-Year Time Dependent Contraction Scour
Contraction Scour (ft)
Time (hr)
Yang
Scour at time T
Laursen Ultimate
Yang Ultimate
4.3
4.3
3.0
t = 6h
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
0.0001
0.001
0.01
0.1
1
10
100
1000

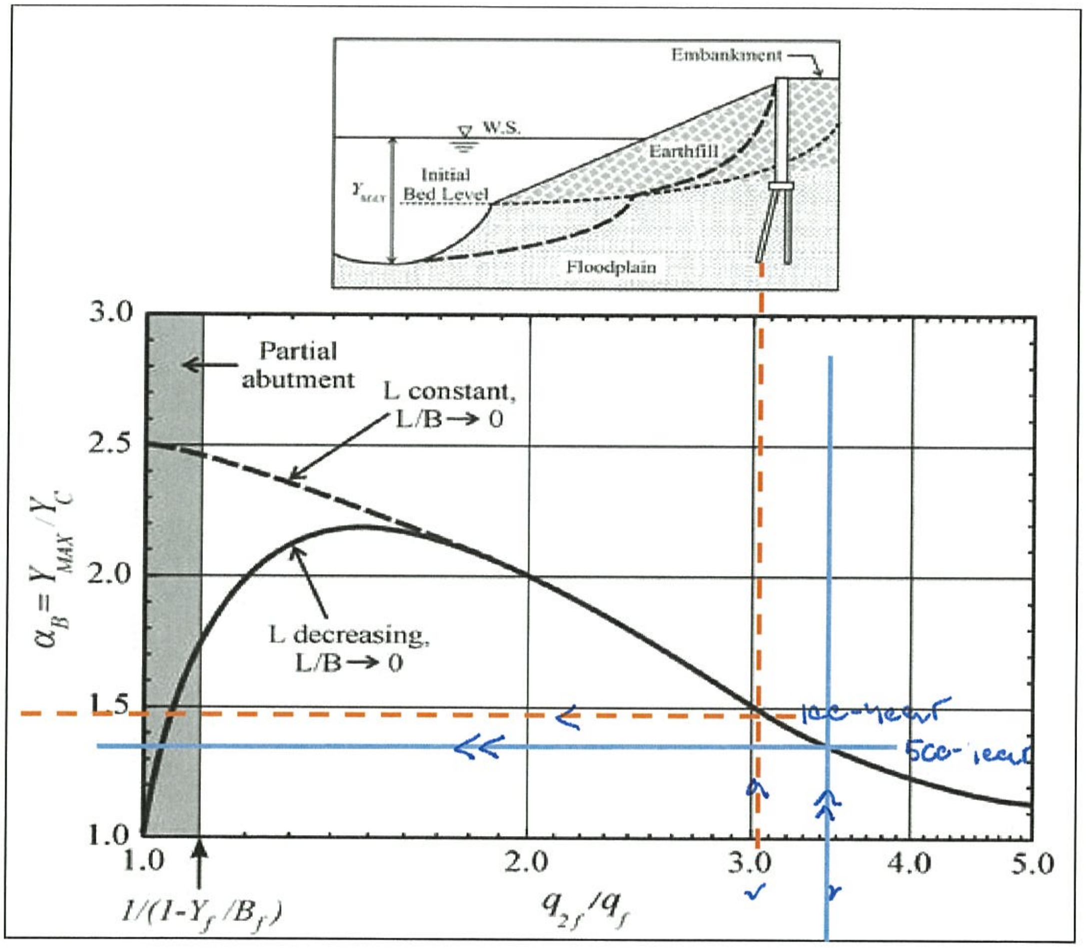
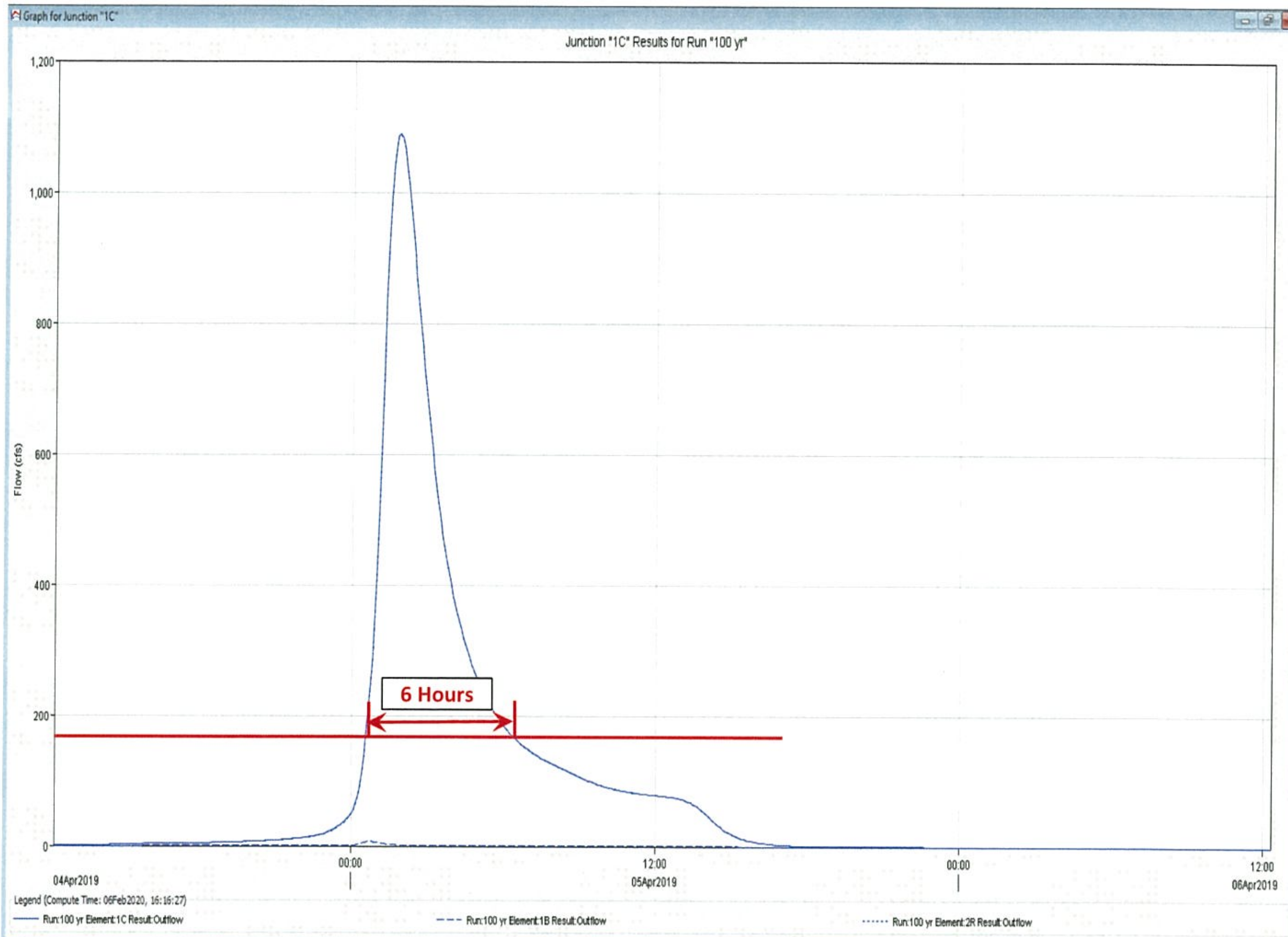


Figure 8.11. Scour amplification factor for spill-through abutments and clear-water conditions (NCHRP 2010b).


Graph for Junction "1C"
Junction "1C" Results for Run "100 yr"
Flow (cfs)
1,200
1,000
800
600
400
200
0
6 Hours
00:00
12:00
00:00
12:00
04Apr2019
05Apr2019
06Apr2019
Legend (Compute Time: 06Feb2020, 16:16:27)
Run:100 yr Element:1C Result:Outflow
Run:100 yr Element:1B Result:Outflow
Run:100 yr Element:2R Result:Outflow

500-YEAR TIME SCOUR SUMMARY
FOR STATE HWY 40 OVER UNNAMED CREEK
BRIDGE F-20-J
ARAPAHOE COUNTY, COLORADO

FEBRUARY 2020

	Approach	Bridge (t=0)	Live-Bed Bridge (ultimate)	Clear-Water Bridge (ultimate)
Manning n	0.03			
Q (cfs)	1,295	1,133		
W (ft)	238.05	60.00		
Y (ft)	5.81	4.13	15.97	9.37
A (ft^2)	1384	248.1	958	562
V (ft/s)	0.94	4.57	1.18	2.02
Sf	0.000034	0.001281	0.000014	0.000084

D50mm	0.440	*Iterate D_{50} until Y_2 (clear water) below matches cohesive clear-water Y_2
D50ft	0.001444	
Vcrit (ft/s)	4.90	HARD_WIRED, assumed Clear-Water
Sediment Sg	2.65	
Fall vel. (ft/s)	0.224	
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)	0.080	0.413	0.085	0.159
Re*	10.70	55.26	11.42	21.27
Vcr/omega	3.24	2.15	3.17	2.63
log(C)	-0.788	3.077	-0.799	1.016
C ppm-wt	0.2	1195.3	0.2	10.4
Qs(cfs)	0.000080	0.511	0.000068	0.004

Clear Water Controls

Y2 (clear water)	9.37
Y2 (ultimate live-bed)	15.97
Qs Ratio	0.852
	Run Goal Seek
Laursen Ys(ultimate)	5.2
Yang Ys(ultimate)	5.2

Scour Time T (hr)	8
Scour at Time T (ft)	**3.6**

Calc. By:	MWG	Date:	2/13/2020
Check By:	[signature]	Date:	19 FEB 2020

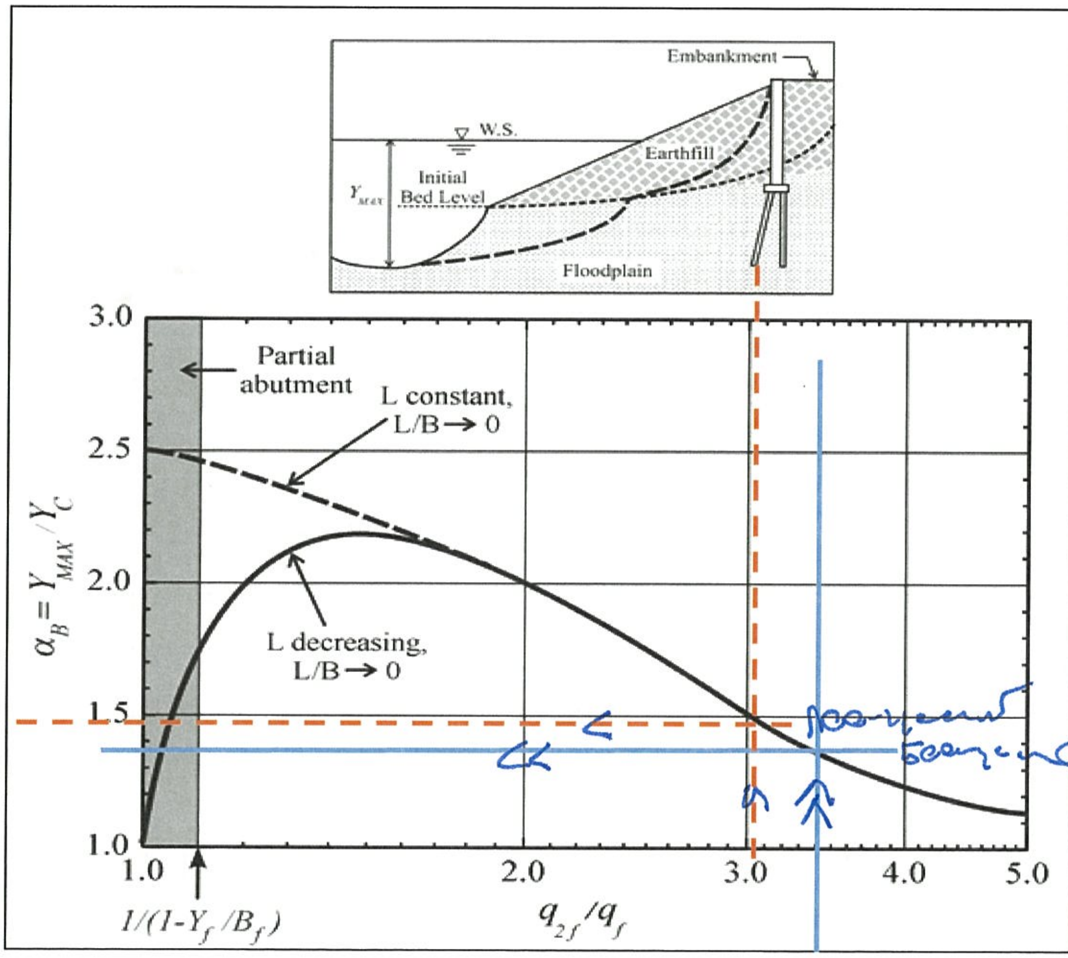
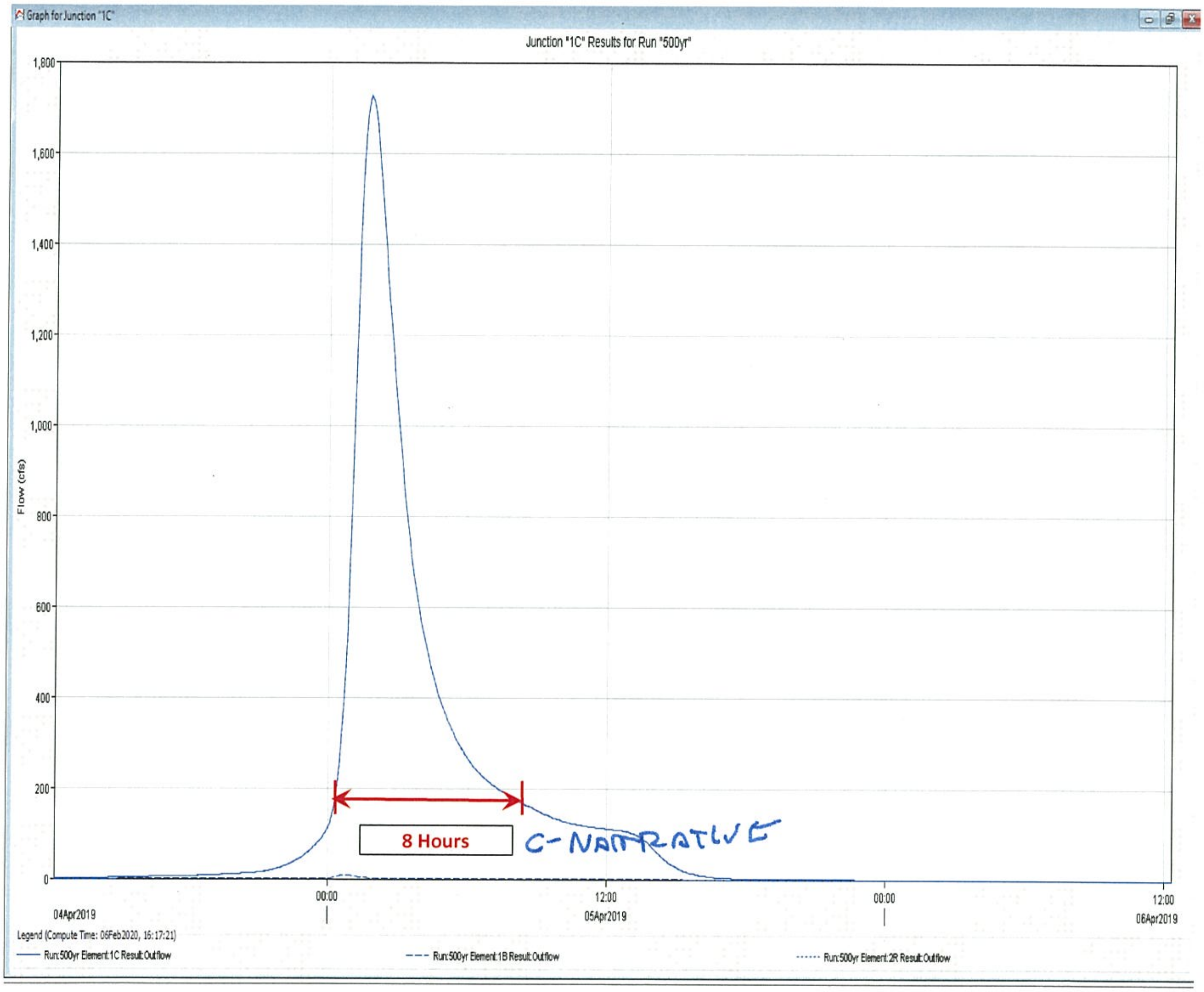
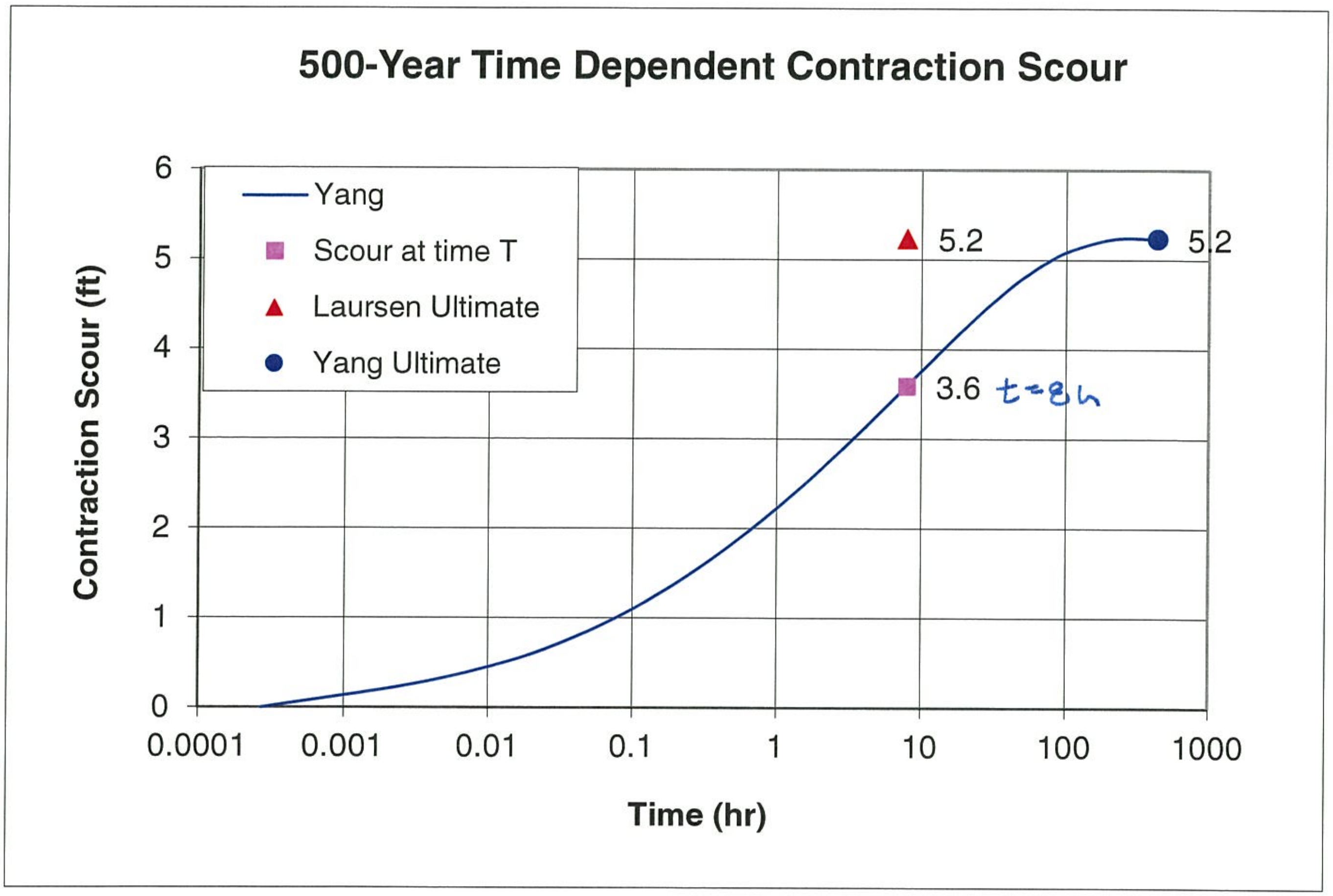


Figure 8.11. Scour amplification factor for spill-through abutments and clear-water conditions (NCHRP 2010b).


Graph for Junction "1C"
Junction "1C" Results for Run "500yr"
Flow (cfs)
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
8 Hours
C-NARRATIVE
00:00
12:00
05Apr2019
00:00
12:00
06Apr2019
04Apr2019
Legend (Compute Time: 06Feb2020, 16:17:21)
Run:500yr Element:1C Result:Outflow
Run:500yr Element:1B Result:Outflow
Run:500yr Element:2R Result:Outflow


500-Year Time Dependent Contraction Scour
Contraction Scour (ft)
Time (hr)
0
1
2
3
4
5
6
0.0001
0.001
0.01
0.1
1
10
100
1000
Yang
Scour at time T
Laursen Ultimate
Yang Ultimate
5.2
3.6
t=8h
5.2

TIME-DEPENDENT ABUTMENT SCOUR COMPUTATIONS FOR COHESIVE SOILS
FOR STATE HWY 40 OVER UNNAMED CREEK
BRIDGE F-20-J
ARAPAHOE COUNTY, COLORADO

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 (CHANNEL) UNIT DISCHARGE (ft^2/s), q_f	=	4.90	5.44
CONTRACTED SECTION UNIT DISCHARGE (ft^2/s), q_{2f}	=	15.03	18.88
DEPTH-AVERAGED CRITICAL VELOCITY (ft/s), V_c	=	1.97	2.01
COMPUTED DEPTH OF CONTRACTED SECTION (ft), Y_2	=	7.64	9.37
STORM DURATION (hrs), t	=	6.0	8.0
TIME RATE FLOW DEPTH (ft), Y_{2t}	=	6.37	7.72
DISCHARGE RATIO, q_{2f}/q_f	=	3.07	3.47
AMPLIFICATION FACTOR LIVE-BED CONDITIONS, α_B	=	1.50	1.38
COMPUTED WATER DEPTH AT ABUTMENT (ft), Y_{max}	=	9.55	10.66
AVERAGE WATER DEPTH AT BRIDGE (ft), Y_0	=	3.35	4.13
AVERAGE ABUTMENT SCOUR DEPTH, Y_s	=	**6.2**	**6.5**

Calc. By:	MWG	Date:	2/13/2020
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Abutment Riprap Sizing Calculation

Date: 1/28/2020
Project: CDOT F-20-J
Project #: 36-4648.22

$$D_{50} = \frac{yk}{(S_s - 1)}\left(\frac{V^2}{gy}\right) \; if \; Fr \le 0.80$$

$$D_{50} = \frac{yk}{(S_s - 1)}\left(\frac{V^2}{gy}\right)^{0.14} \; if \; Fr \ge 0.80$$

HEC 23, Volume 2 Edition 3, Equations 14.1 and 14.2

Variables		100yr	500yr
Velocity in contracted section	V (ft/s)	5.70	6.50
Depth of flow in the contracted bridge opening	y (ft)	4.00	5.00
Specific Gravity of rock riprap	S_s	2.55	2.55
Gravitational acceleration	g (ft/s^2)	32.20	32.20
Froude Number	Fr	0.50	0.51
Abutment shape coefficient	K	0.89	0.89

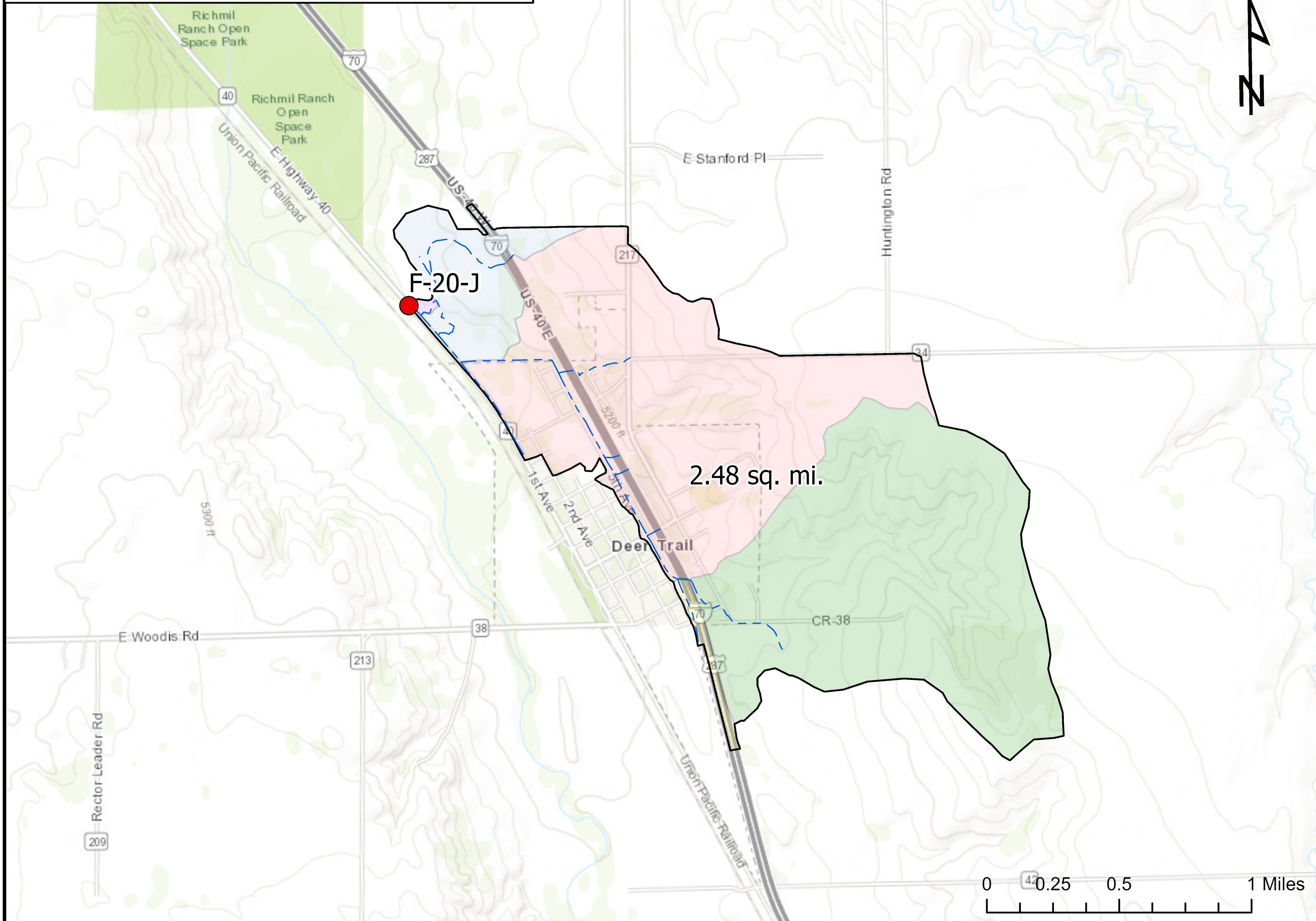
Calculated riprap size	D_{50} (ft)	0.58	0.75
Nominal riprap size	D_{50} (ft)	12-Inch	12-Inch

By: MWG Date: 1/28/2020
Checked By: MDH Date: 2/14/2020

Appendix C

Hydrology

F-20-J Drainage Basin Map



F-20-J

Global Summary

10-year Global Summary				
Hydrologic Element	Drainage Area (sq mi)	Peak Discharge (CFS)	Time of Peak	Volume (in)
5B	1.1454	216.9	05Apr2019, 01:19	0.88
4B	0.0256	7.9	05Apr2019, 02:14	2.17
3B	1.0931	164.3	05Apr2019, 02:14	1.07
2B	0.207	82.4	05Apr2019, 01:09	1.53
1B	0.0048	3.6	05Apr2019, 00:44	2.02
5C	1.1454	216.9	05Apr2019, 01:19	0.88
4C	1.171	222.2	05Apr2019, 01:24	0.91
3C	2.2641	373.2	05Apr2019, 01:59	0.98
2C	2.4711	414.5	05Apr2019, 01:59	1.03
1C	2.4759	414.8	05Apr2019, 01:59	1.03
5R	1.1454	216.4	05Apr2019, 01:19	0.88
4R	1.171	215.7	05Apr2019, 01:54	0.9
3R	2.2641	372.3	05Apr2019, 02:04	0.98
2R	2.4711	414.1	05Apr2019, 01:59	1.03

50-year Global Summary				
Hydrologic Element	Drainage Area (sq mi)	Peak Discharge (CFS)	Time of Peak	Volume (in)
5B	1.1454	474.7	05Apr2019, 01:19	1.78
4B	0.0256	12.4	05Apr2019, 02:14	3.39
3B	1.0931	327.5	05Apr2019, 02:09	2.01
2B	0.207	144.3	05Apr2019, 01:09	2.66
1B	0.0048	5.7	05Apr2019, 00:44	3.25
5C	1.1454	474.7	05Apr2019, 01:19	1.78
4C	1.171	482.9	05Apr2019, 01:19	1.81
3C	2.2641	775.3	05Apr2019, 01:49	1.9
2C	2.4711	855.1	05Apr2019, 01:49	1.97
1C	2.4759	855	05Apr2019, 01:49	1.97
5R	1.1454	474.4	05Apr2019, 01:19	1.78
4R	1.171	469.1	05Apr2019, 01:44	1.81
3R	2.2641	773.1	05Apr2019, 01:54	1.9
2R	2.4711	853.7	05Apr2019, 01:49	1.97

F-20-J

Summary of Methods

Basin Input	
Basin Name	F-20-J
Subbasin	
Area (Mi^2)	2.48
Canopy Method	None
Surface Method	None
Loss Method	SCS Curve Number
Transform Method	SCS Unit Hydrograph
Baseflow Method	None
Transform	
Graph Type	Standard (PRF 484)

Precipitation Input	
Basin Name	F-20-J
Design Storm	
Storm Hyetograph	SCS Type 2
Storm Duration	24-hour
Precipitation (Inches)	
10-year	2.95
50-year	4.24
100-year	4.86
500-year	6.46

Reach Input	
Basin Name	F-20-J
Reach	
Routing Method	Muskingum-Cunge
Loss/Gain Method	None
Routing	
Time Step Method	Automatic Fixed Interval
Shape	Trapezoid

Control Specifications	
Basin Name	F-20-J
Start Date	4-Apr-19
Start Time	12:19 PM
End Date	6-Apr-19
End Time	12:19 PM
Time Interval (Min)	5.0

F-20-J

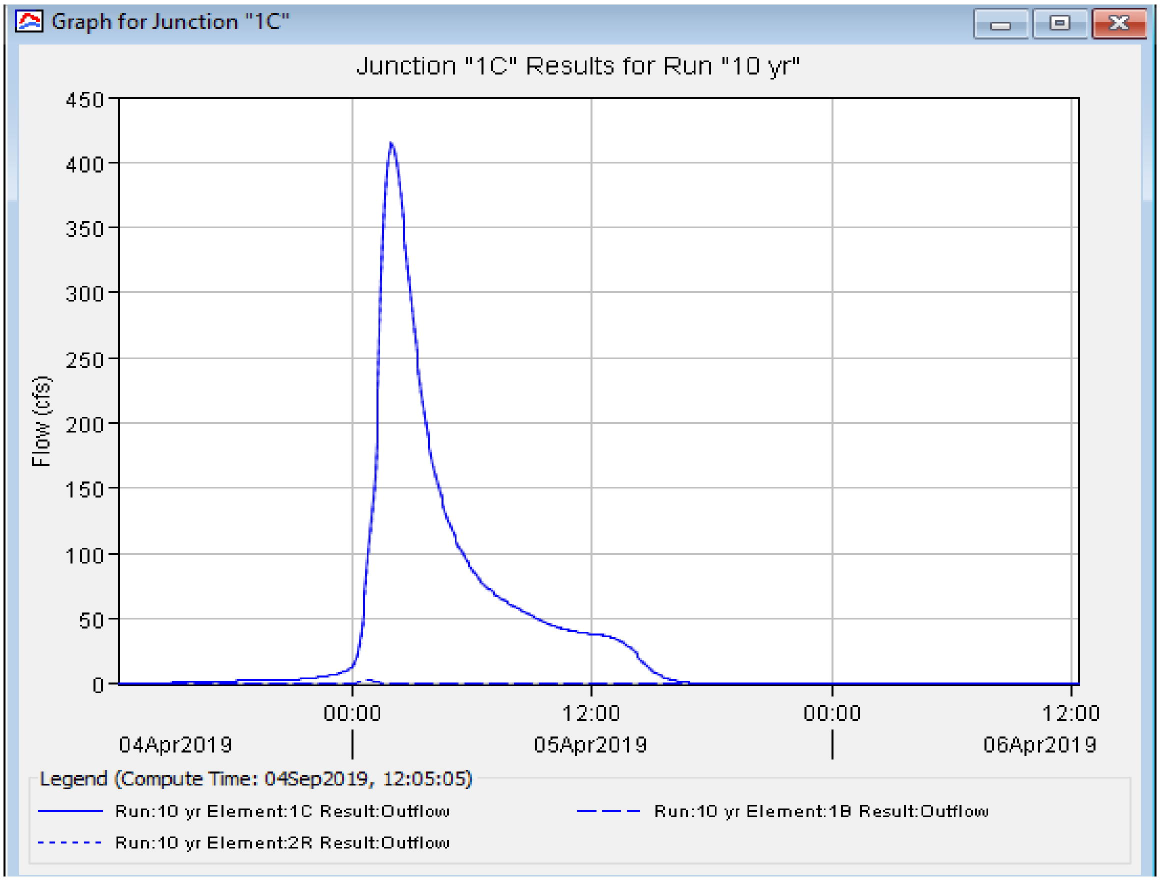
Global Summary

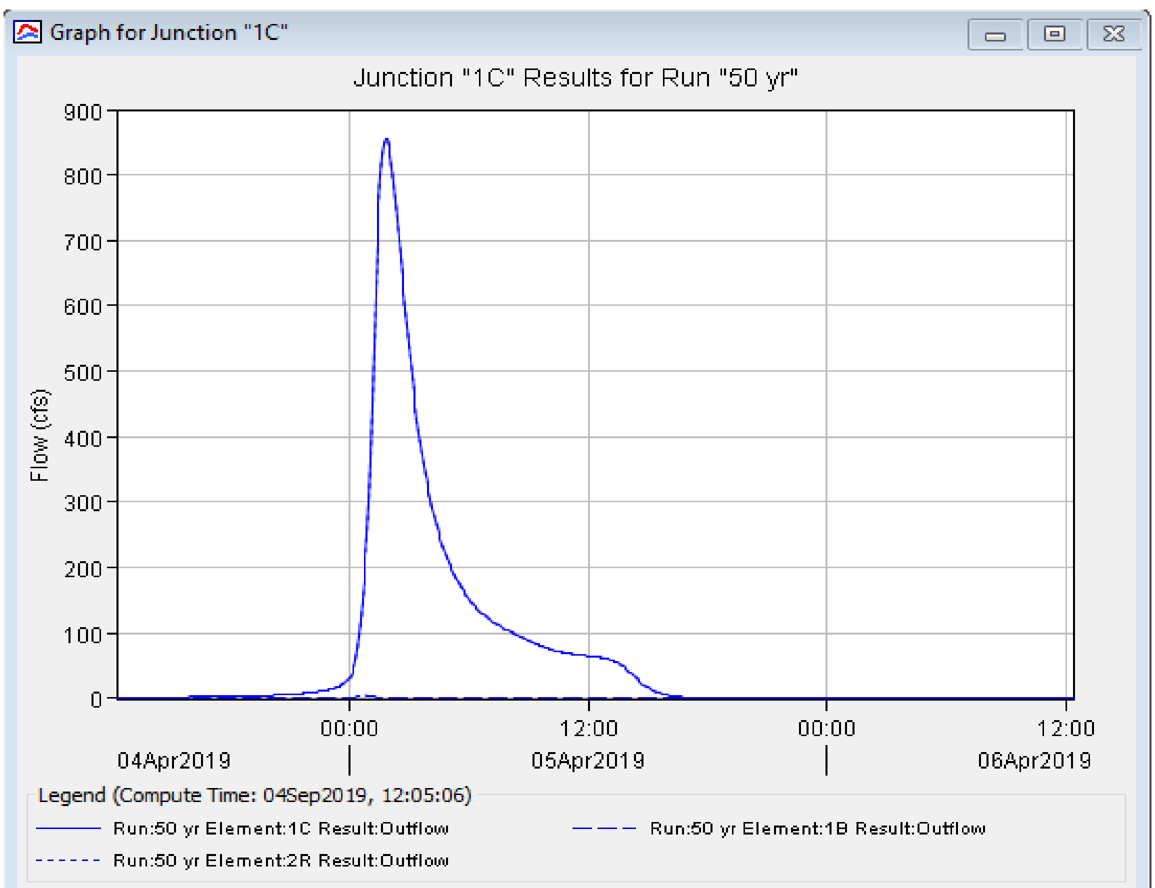
100-year Global Summary				
Hydrologic Element	Drainage Area (sq mi)	Peak Discharge (CFS)	Time of Peak	Volume (in)
5B	1.1454	611.5	05Apr2019, 01:19	2.25
4B	0.0256	14.6	05Apr2019, 02:14	3.99
3B	1.0931	413.9	05Apr2019, 02:09	2.5
2B	0.207	174.9	05Apr2019, 01:09	3.22
1B	0.0048	6.7	05Apr2019, 00:44	3.85
5C	1.1454	611.5	05Apr2019, 01:19	2.25
4C	1.171	621.5	05Apr2019, 01:19	2.29
3C	2.2641	987.8	05Apr2019, 01:49	2.39
2C	2.4711	1088.9	05Apr2019, 01:49	2.46
1C	2.4759	1089.9	05Apr2019, 01:49	2.46
5R	1.1454	611.4	05Apr2019, 01:19	2.25
4R	1.171	603	05Apr2019, 01:39	2.29
3R	2.2641	985.5	05Apr2019, 01:54	2.39
2R	2.4711	1088.3	05Apr2019, 01:49	2.46

500-year Global Summary				
Hydrologic Element	Drainage Area (sq mi)	Peak Discharge (CFS)	Time of Peak	Volume (in)
5B	1.1454	988.3	05Apr2019, 01:14	3.58
4B	0.0256	20.2	05Apr2019, 02:14	5.55
3B	1.0931	649.7	05Apr2019, 02:09	3.85
2B	0.207	254.7	05Apr2019, 01:09	4.72
1B	0.0048	9.3	05Apr2019, 00:44	5.41
5C	1.1454	988.3	05Apr2019, 01:14	3.58
4C	1.171	1000.4	05Apr2019, 01:19	3.62
3C	2.2641	1570.6	05Apr2019, 01:44	3.73
2C	2.4711	1724.6	05Apr2019, 01:44	3.81
1C	2.4759	1726	05Apr2019, 01:44	3.81
5R	1.1454	987	05Apr2019, 01:14	3.58
4R	1.171	974.5	05Apr2019, 01:39	3.61
3R	2.2641	1564	05Apr2019, 01:49	3.73
2R	2.4711	1723.7	05Apr2019, 01:44	3.81

F-20-J

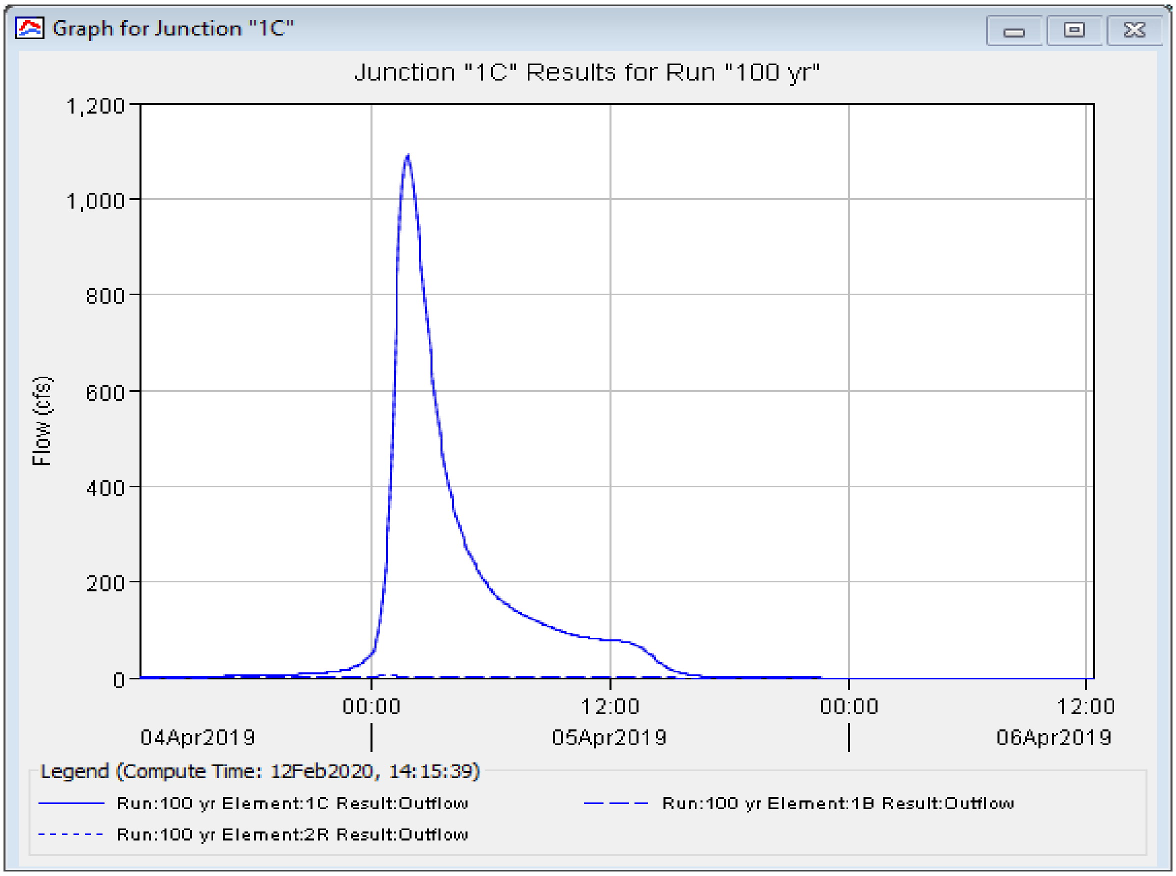
Flood Hydrographs

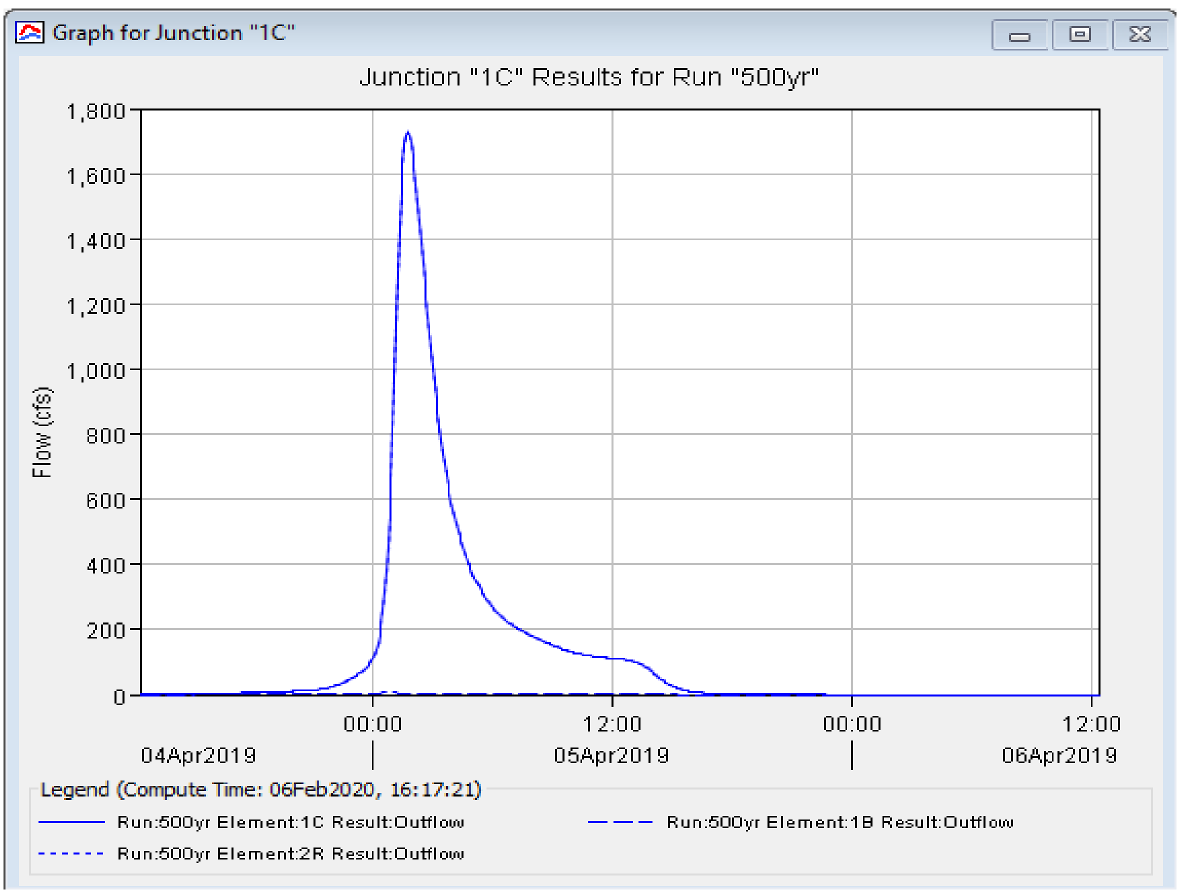




F-20-J

Flood Hydrographs





Appendix D

Geomorphic Assessment Memo

MEMO

TO:	Anthony Alvarado, PE (Ayres Associates)
FROM:	William Spitz, PG
RE:	BE Bridge F-20-J (MP 349.672) Geomorphic Assessment
DATE:	September 23, 2019
PROJECT #:	017-1690
CDOT PROJECT #:	20252
CDOT TO #:	22

Geomorphic Assessment of Stream Stability
BE Bridge F-20-J (US36 MP349.672) over Unnamed Creek near Deer Trail, 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-J, which is located in Arapahoe County about a quarter mile north of Deer Trail, Colorado at Mile Post 349.672 (**Figure 1**). The unnamed drainage channel flows from the southeast to the northwest and is tributary to East Bijou Creek, which is located to the west of the highway. The confluence of the channel with East Bijou Creek is located about 1,700 feet west of the bridge site. The bridge site and US36 are bound by hills to the east and railroad tracks/embankment about 225 feet to the west. Upstream of the bridge, the channel is located on the east side of US36 and parallels the highway for about 1,500 feet where it splits into 2 different stormwater drainage channels. At the bridge, the upstream channel makes a 90° turn to the southwest as it passes under the bridge. Downstream of the bridge the channel is straight and is bound by US36 and the railroad tracks.

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.

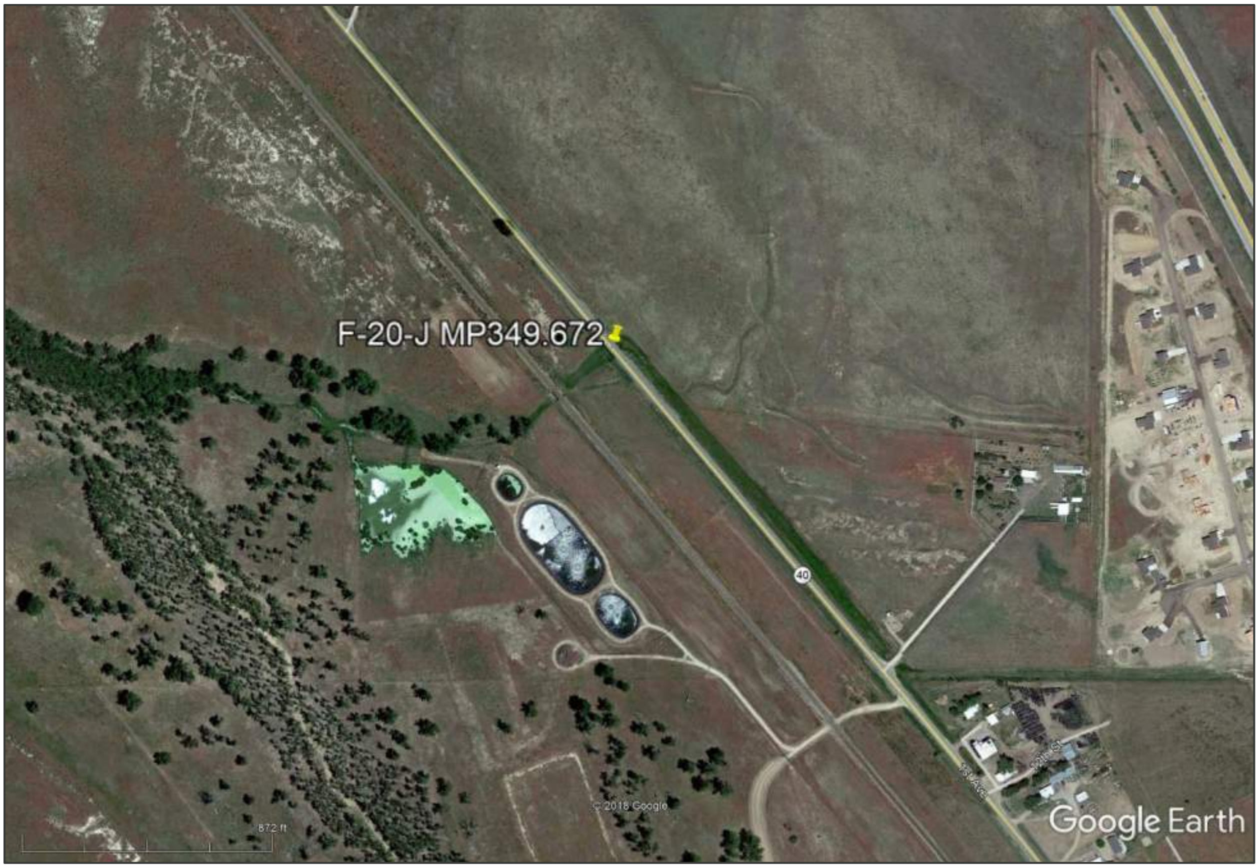


Figure 1. Location of US36 Bridge F-20-J over an unnamed drainage channel near Deer Trail, Colorado.

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. Overlying the Laramie Formation in the area are eolian (windblown) deposits as noted in the NRCS Soils Report for Arapahoe County.

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. 17% of the area), Weld-Deertrail silt loams, 0 to 3 percent slopes (approx. 16% of the area), Renohill-Buick loams, 3 to 9 percent slopes (approx. 15.5% of the area), Weld silt loam, 0 to 3 percent slopes (approx. 9.5% of the area), and the Samsil-Renohill clay loams, 3 to 20 percent slopes (approx. 7.5% of the area).

The Adena-Colby silt loams, 1 to 5 percent slopes are considered farmland soils of statewide importance. This unit is located on drainageways and hills. The parent material of the Adena component, which makes up about 65% of the unit, is eolian deposits. The parent material of the Colby component, which makes up about 25% of the unit, 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.

The Weld-Deertrail silt loams are not prime farmland. They are found in depressions and on low slopes. The unit is well drained, is in the lo runoff class, and belongs to the Hydrologic Soil Group C.

The Renohill-Buick loams, 3 to 9 percent slopes are not prime farmland. The Renohill component, which makes up about 65% of the unit, is located in drainageways and the parent material is loamy silty and clayey alluvium. The Buick component, which makes up about 25% of the unit, is located on ridges and the parent material is alluvium or eolian deposits. The unit is well drained and is in the medium runoff class. The Renohill component belongs to the Hydrologic Soil Group D and the Buick component belongs to the Hydrologic Soil Group C.

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.

Group D soils have a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. Group D soils have a very slow rate of water transmission.

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 98% silt and clay (< 0.074 mm) and 2% sand (≤ 2.0 mm). The Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI) of 57, 23, and 34, respectively, indicating that the sample is a fat clay (CH).

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 as delineated by StreamStats, which may be slightly incorrect, is approximately 3.17 mi^2 (**Figure 2**). Mean annual precipitation is about 15.8 inches. He maximum 6-hour, 2-year recurrence precipitation is estimated to be 1.42 inches and the maximum 24-hr, 100-year recurrence precipitation is estimated to be 4.85 inches. The estimated peak flows for the unnamed channel at Bridge F-20-J are provided in **Table 1**.

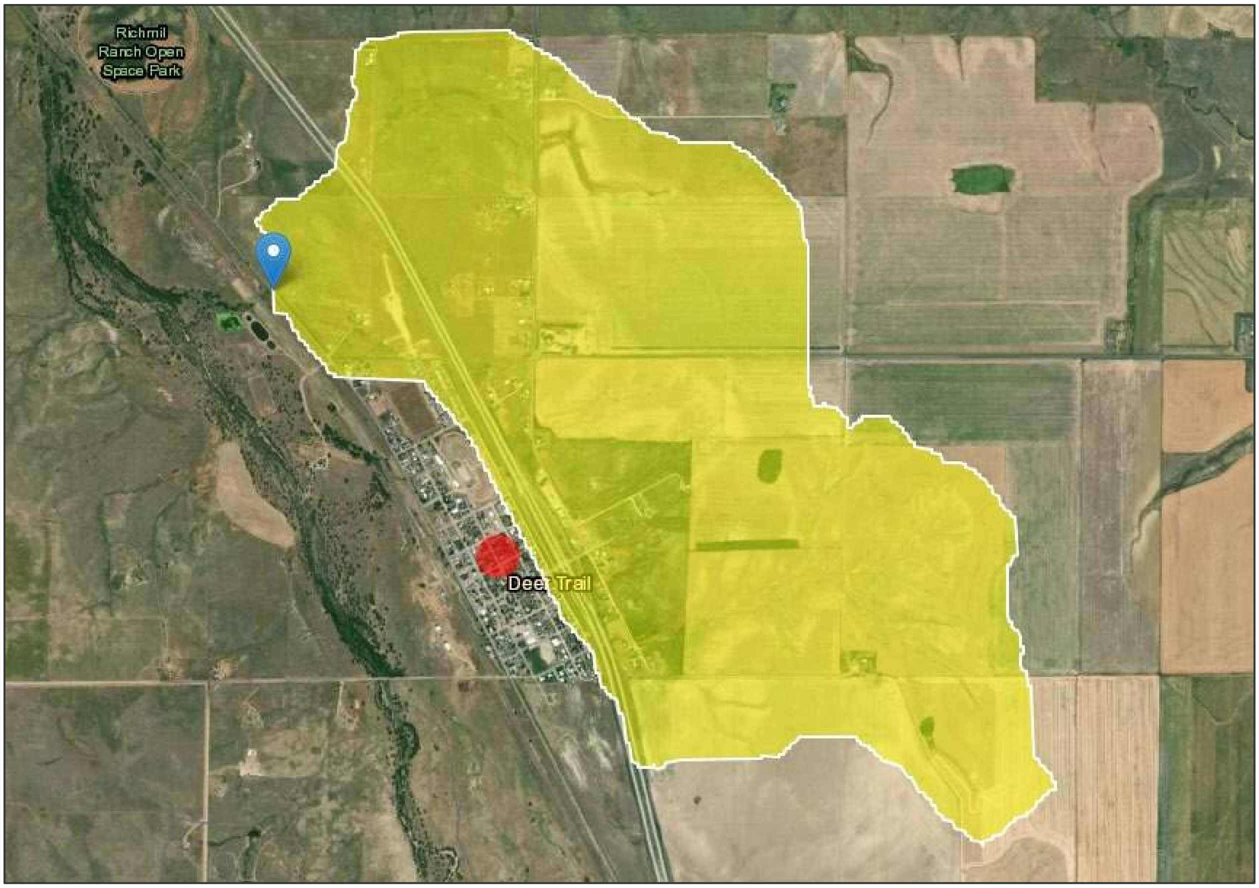


Figure 2. Drainage area for unnamed channel at US36 Bridge F-20-J near Deer Trail, CO.

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

Recurrence	StreamStats Peak Flow (cfs)	CDOT Peak Flows (cfs)
2-yr	106	---
5-yr	372	---
10-yr	695	300
25-yr	1,310	---
50-yr	1,950	695
100-yr	2,810	910
500-yr	5,700	1,510

As noted above, the drainage area and the peak flow values may be slightly incorrect since the drainage area delineated by StreamStats does not include a part of the north end of the town of Deer Trail. However, an examination of the current aerial photography shows a 1,450 foot long stormwater drainage channel that is present along the east side of US36 that appears to collect stormwater from the northwest end of town and connects up with the main channel about 1,500 feet southeast of the bridge (**Figure 3**).

Figure 3. Google Earth Street View of stormwater channel along US36 and confluence with main unnamed channel.

CDOT also developed hydrology for the bridge site using HEC-HMS. The drainage area above the bridge site as delineated by CDOT is approximately 2.5 mi^2, which is about 21% smaller than that delineated by StreamStats. The peak discharges estimated by CDOT, which are also shown in Table 1, reflect the smaller drainage area delineated by CDOT. One reason for the smaller drainage area may be that a portion of the basin to the north that is captured by StreamStats may not have been included in the CDOT delineation as the drainage in that area appears poorly defined.

Aerial Photo Analysis
Relatively good resolution aerial photos of the area taken in 1953 and 1968 were 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 predominately a stormwater drainage channel for the town of Deer Trail. The 1953 and 1968 aerial photos show a small channel to the northeast of the bridge that contains several small dams and associated stock ponds. The channel appears to be a relic of an old drainage and does not appear to contribute any significant flow to the main channel. The confluence of this small tributary with the current channel is located about 260 feet southeast of the bridge. The current drainage channel is about 30 feet wide. The straight channel reach upstream (southeast) of the bridge provides stormwater drainage for the town of Deer Trail.

Site Visit and Assessment
Bridge F-20-J, which is 60 feet wide, consists of vertical timber retaining wall abutments with timber wingwalls. The bridge sits on 3 pile bents each of which contains 7 wood piles that are 12 inches in diameter. Wood cross braces are present on each pier bent. The pier bents are not skewed but the channel just upstream makes a 90° turn into the bridge opening (**Figure 4**). **Figure 5** shows the configuration of the bridge.

Figure 4. View looking upstream (south) toward Deer Trail along the unnamed channel southeast of US36 Bridge F-20-J.

The channel upstream and downstream of the bridge contains a dense growth of grasses and weeds that appear to be mowed regularly. Conditions at the bridge suggest general channel stability and relatively infrequent flows. High water marks/stains under the bridge indicate that flows rarely are more than 1.5 to 2 feet deep.

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, the sharp turn into the bridge opening could be problematic during flood flows with regard to upstream backwater effects and downstream flow acceleration. A roadside drainage channel along the west side of US36 that drains the area to the south of the bridge, intersects the main channel between the Bridge F-20-J and the railroad bridge. In addition, the downstream railroad bridge opening is only about 40 feet compared to the US36 bridge opening which is 60 feet. Thus, the addition of stormwater flows from the southwest side of US36 combined with flow in the main channel and the contraction at the railroad bridge may add to the backwater at the US36 bridge during major flood flows. If significant backwater upstream of the bridge occurs during flood flows, potential overtopping of the US36 roadway to the southeast is possible.

Figure 5. View looking downstream at US36 Bridge F-20-J over unnamed drainage channel near Deer Trail.