Draft FIR Hydraulic Report

CDOT Project 23014
Replacement of Existing Structure G-21-A
US Highway 40

Prepared for:

Colorado Department of Transportation Region 4
Craig Schumacher and Scott Huson
Sterling Residency and Staff Bridge

February 10, 2020

Draft FIR Hydraulic Report Replacement of Existing Structure G-21-A

CDOT Project 23014
Replacement of Existing Structure G-21-A
US Highway 40

3665 JFK Parkway, Bldg. 2, Suite 100
Fort Collins, CO 80525-3152
970.223.5556
www.AyresAssociates.com

Ayres Associates Project No. 36-4648.22

Contents

List of Appendices

Appendix A Hydraulic Modeling

Appendix B Scour and Countermeasures Calculations

Appendix C Hydrology

Appendix D Geomorphic Assessment Memo

List of Figures

Page No.

List of Tables

Page No.

Introduction

Background

Region 4 of the Colorado Department of Transportation (CDOT) has proposed the replacement of the G-21-A timber bridge along U.S. Highway 40 (US 40E) over Agate Creek. The existing G-21-A structure is a 4-span bridge with a length of 94 feet and three rows of timber pile piers.

Site Location

The G-21-A bridge is located just north of the town of Agate, Colorado in Elbert county on US 40 at mile marker 364.53 upstream of the Middle Bijou Creek confluence. The legal location of the structure is the North West 1/4 of the South West 1/4 of Section 36, Township 6 South, Range 59 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

Agate Creek is located in FEMA Zone X, so there exist no effective flood elevations recognized by FEMA for this reach. The nearest floodplain to Agate Creek is on Middle Bijou Creek, which is over a mile downstream of the bridge location and is mapped on FEMA community-panel number 08039C0400C, effective on March 17th, 2011.

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 G-21-A 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.

No stream gage data exists near G-21-A 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 to perform calculations, assuming an SCS Type II, 24-hour storm distribution. The reported flows are summarized in **Table 1.**

Table 1: HEC-HMS peak flow results at bridge G-21-A

Recurrence Interval	Flowrate (cfs)
10-Year	510
50-Year	1,569
100-Year	2,254
500-Year	4,381

Existing Structure

The existing G-21-A structure is an 89-year-old bridge constructed by the State Highway Department in 1931. The bridge has 4 spans, each 23 feet in length, supported by three rows of 1-foot diameter cross-braced timber pile bents and concrete abutments. The existing bridge width is 26 feet, out to out. **Figure 2**, **Figure 3**, and **Figure 4** show the downstream face, upstream face, and roadway of the existing structure, respectively.

Field reconnaissance was performed by Ayres engineers on July 24, 2019, and the Olsson geomorphologist 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 outlined in Olsson's geomorphic report, indicates that Agate Creek is not highly dynamic over engineering time scales, and that no significant long-term degradation is expected. Conditions at and around the bridge suggest relatively infrequent flows.

Figure 2: Upstream face of Bridge G-21-A

Figure 3: Downstream face of Bridge G-21-A

Figure 4: Upstream face of Bridge G-21-A, looking south 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 September of 2019 at and around the G-21-A 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 G-21-A bridge has a condition rating of fair. CDOT has determined that the bridge will be replaced.

Design Frequency and Floodplain Impacts

The proposed structure 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.

Proposed Alternative

The following alternatives were analyzed with varying bridge spans:

- 83.5-foot span,
- 103.5-foot span,
- and 110-foot span.

All alternatives incorporated 2:1 sloped spill-through abutments, with the bridge centered at the existing bridge centerline. All alternatives met the no-rise and freeboard requirements of this project. Therefore, a bridge span of 83.5 feet is proposed for this project.

The roadway over the proposed structure consists of two travel lanes with shoulders in both the eastbound and westbound directions. The existing bridge opening will be contracted to the west and east by 5-feet inside of existing abutments for proposed conditions. The proposed bridge has one 83.5-foot span from centerline bearing to centerline bearing, with a clear span of 81-feet. The deck thickness will be 47.2 inches or 3.9 ft and the low chord elevation will be 5385.97 ft (NAVD-88) at the North abutment and 5386.48 ft (NAVD 88) at the South abutment. The road grade of the proposed structure will remain the same as the existing structure and the road width will be increased to 43 feet, out to out. 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 existing conditions.

Existing Conditions Hydraulic Modeling

Aquaveo's SMS version 13.0.10 was used to develop a Two-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 3,000 ft downstream of bridge G-21-A. The mesh consists of 38,174 predominantly triangular mesh elements with quadrilateral mesh elements representing the bridge and road approach sections on US 40E. Element size ranges from 1-foot elements at the bridge to 35-foot elements along mesh boundaries. The existing trestle bents on bridge G-21-A are likely to catch debris. Therefore, they are modeled as continuous wall piers and were incorporated in the model as 1-foot wide, 37-foot long holes 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 2.**

Table 2: Manning n roughness values

Description	Manning's n value
Main channel	0.03
Riprap	0.04
Pavement	0.013
Light Vegetation	0.035
Medium Vegetation	0.04
Railroad	0.04
Developed	0.05
Gravel	0.02

The inflow boundary conditions were developed using peak flood results from the HEC-HMS model discussed previously for Agate Creek. The outflow boundaries were developed as normal depths downstream of the bridge due to a lack of available data. A sensitivity analysis performed on the downstream boundary condition for the existing 500-year flood condition found that water surface effects do not carry more than 300 feet upstream of the outflow boundary and do not affect the area of interest. Boundary conditions for the 50-, 100-, and 500-year simulations are shown in **Table 3.**

Table 3: Boundary Conditions

Recurrence Interval	Upstream Peak Flows (cfs)	Downstream Water Surface Elevations (ft-NAVD88)
10-Year	510	5368.29
50-Year	1569	5369.31
100-Year	2254	5369.81
500-Year	4381	5371.07

G-21-A Existing Conditions Hydraulic Results

A summary of the existing condition hydraulic properties from two upstream cross-sections are shown in **Table 4,** with locations illustrated in **Figure 5.** In **Figure 5**, the two reference cross-sections are illustrated. Section 1 is located just upstream of the bridge and is useful for evaluating freeboard and local hydraulics. Section 2 is located just upstream of the zone of the bridge's local hydraulic influences and is useful for evaluating hydraulics further upstream of the bridge.

Table 4: Existing Conditions Model Results for G-21-A

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	1569	5384.33	5382.01	4.44	3.52	3.59	5.28
100-Year	2254	5384.97	5382.80	6.35	4.29	4.93	4.02
500-Year	4381	5386.44	5384.96	7.39	5.16	7.67	6.23

The existing structure passes all flows below the 500-year event without pressure flow occurring. At the bridge there exists a large scour hole around the two north-most piers. Flow hits the piers at a sharp 45 degree angle, with the majority of it concentrating to the north of the center pier as flow rounds the bend. Eddy flows occur downstream of each abutment, and upstream of the north abutment.

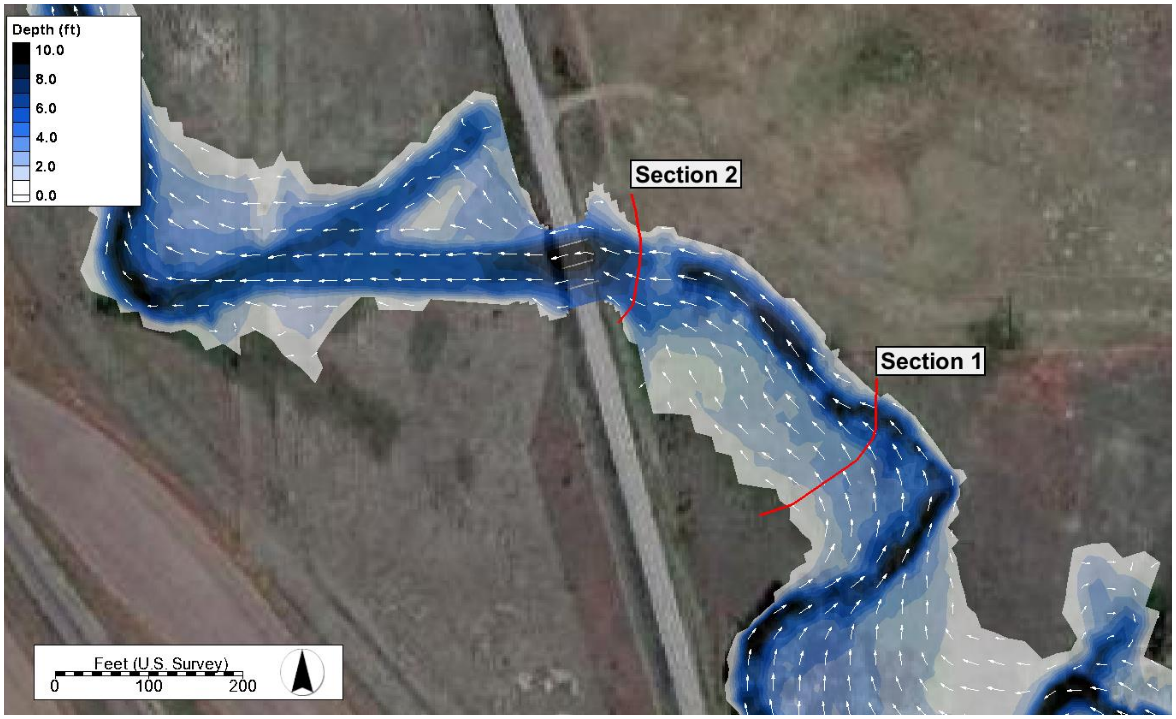


Figure 5: Existing structure G-21-A 100-year depth

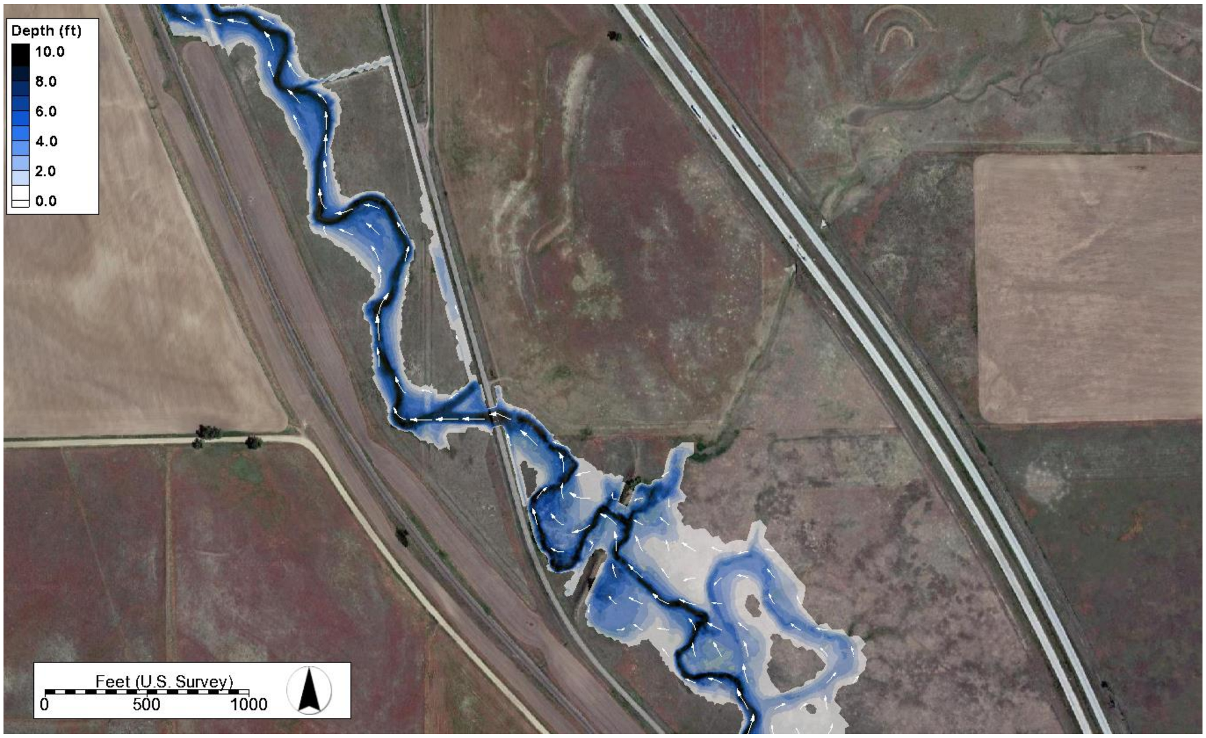


Figure 6: Existing structure G-21-A 500-year depth

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 held at its existing location, and no modifications to the numerical mesh configuration at the bridge were necessary. The holes in the mesh representing wall piers 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 G-21-A for proposed conditions is shown in **Table 5,** with locations illustrated in **Figure 5.** The sections are drawn perpendicular to flow.

Table 5: Proposed Conditions Model results for G-21-A

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	1569	5384.33	5382.07	4.59	3.47	5.68	3.72
100-Year	2254	5384.98	5382.90	5.22	3.97	4.5	6.73
500-Year	4381	5386.60	5385.44	6.62	4.80	7.52	5.12

The proposed condition passes all flows below the 500-year event with no pressure flow. The majority of flow concentrates at the toe of the north abutment as flow moves around the bend. During proposed conditions eddy flow occurs underneath the bridge at the south abutment, further constricting flow through the bridge. An eddy at this location was present for all alternatives modeled in this study. Eddy flow also occurs upstream and downstream of the north abutment. This is displayed in **Figure 7**, **Figure** 8, and **Figure** 9 where the velocity is lesser near the toe of the south abutment during proposed conditions due to the eddy, which did not occur during existing conditions due to the south-most pier. The removal of the piers and the constriction of the bridge opening during proposed conditions resulted in larger velocities, less than 4 ft/s upstream, downstream, and through the bridge.

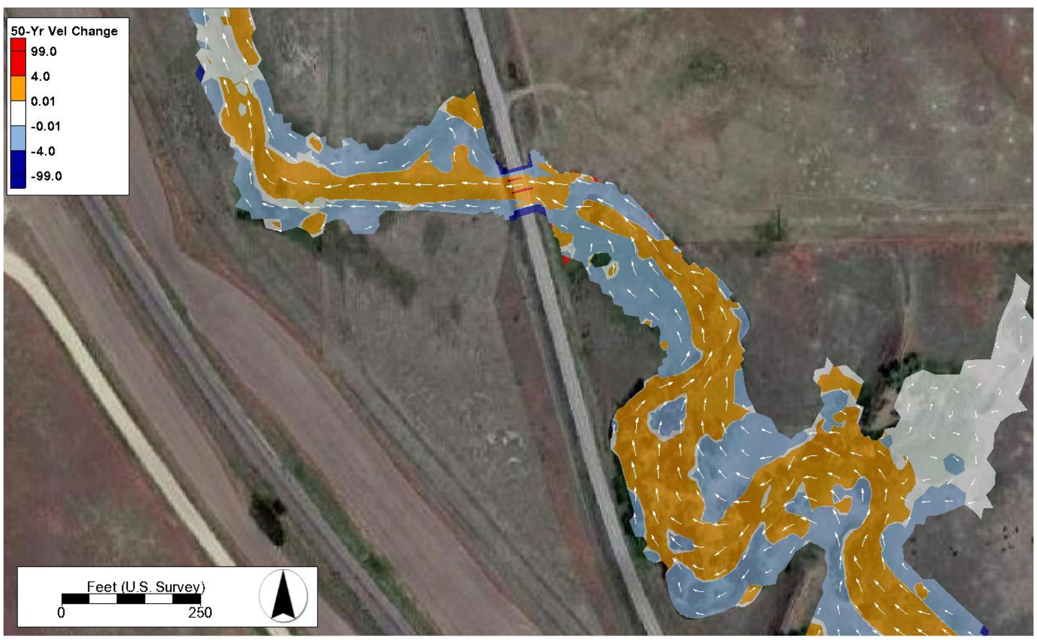


Figure 7: Proposed vs existing velocity change (orange and red indicate an increase in velocity) during the 50-year event

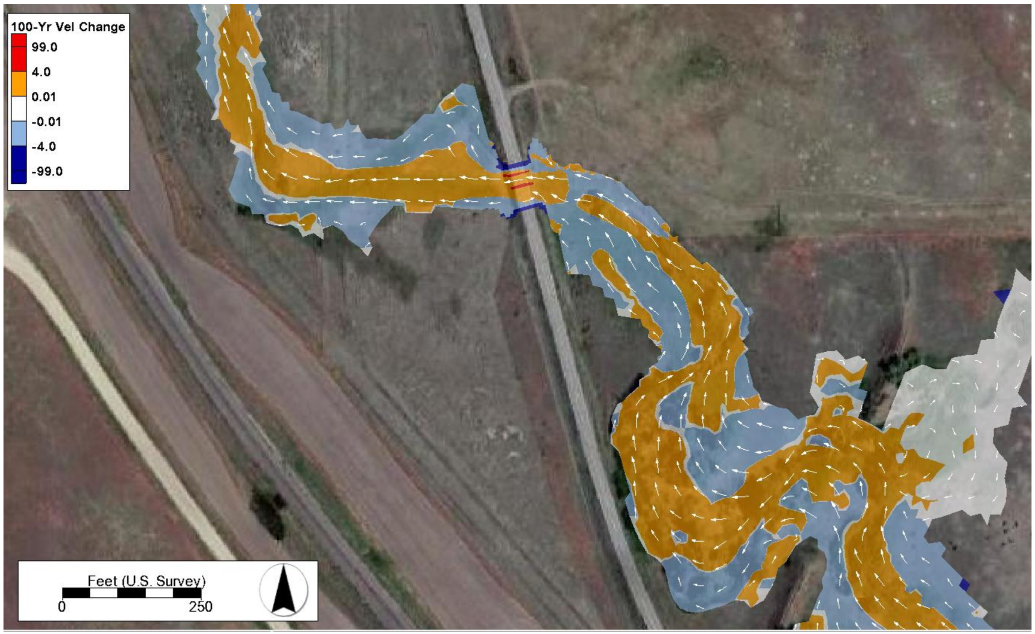


Figure 8: Proposed vs existing velocity change (orange and red indicate an increase in velocity) during the 100-year event

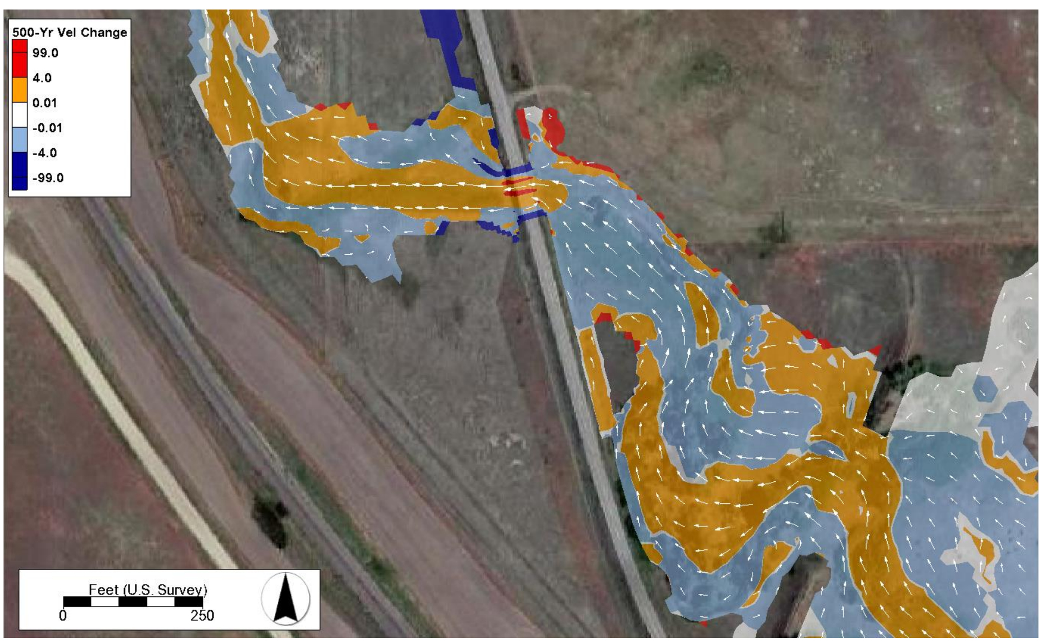


Figure 9: Proposed vs existing velocity change (orange and red indicate an increase in velocity) during the 500-year event

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, the shear stress increases upstream, downstream, and through the bridge. The removal of the piers and the constriction of the bridge opening during proposed conditions resulted in larger shear stresses, less than 2 psf, upstream, downstream, and through the bridge.

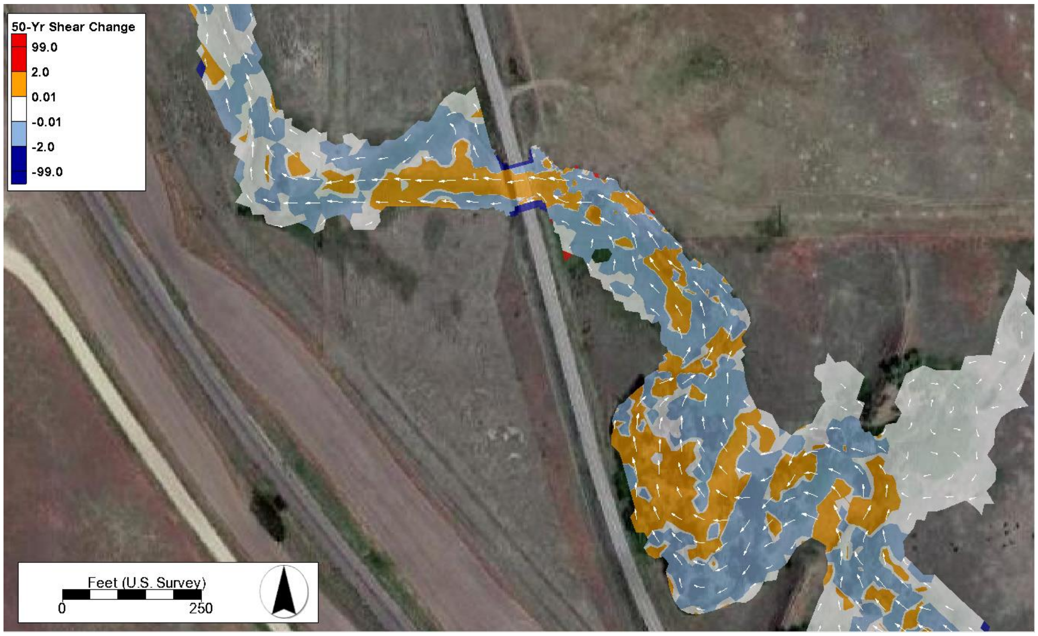


Figure 10: Proposed vs existing bed shear stress change (orange and red indicate an increase in shear stress) during the 50-year event

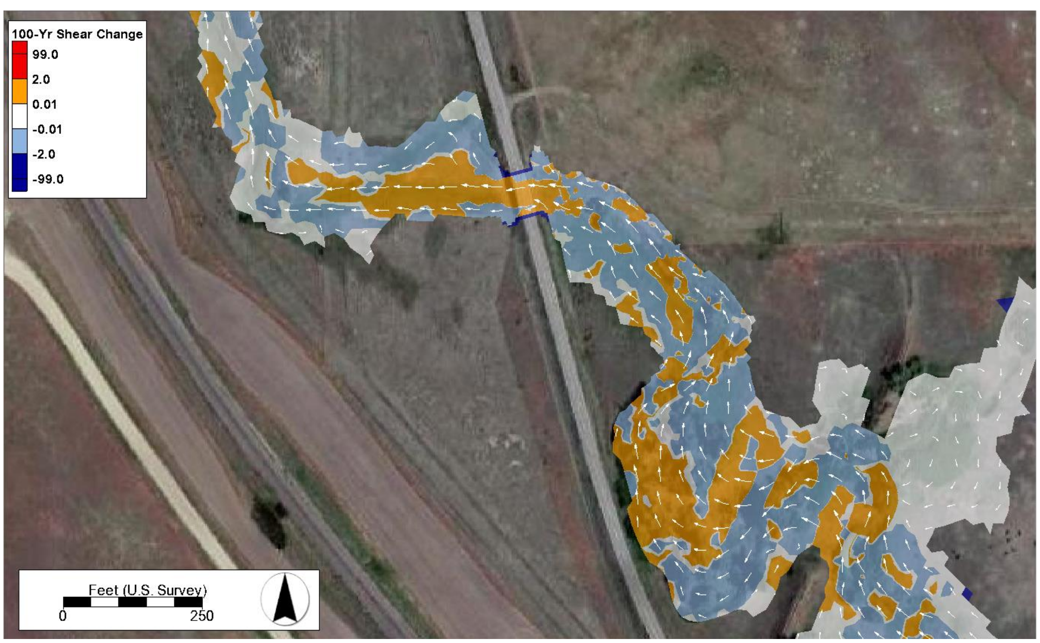


Figure 11: Proposed vs existing bed shear stress change (orange and red indicate an increase in shear stress) during the 100-year event

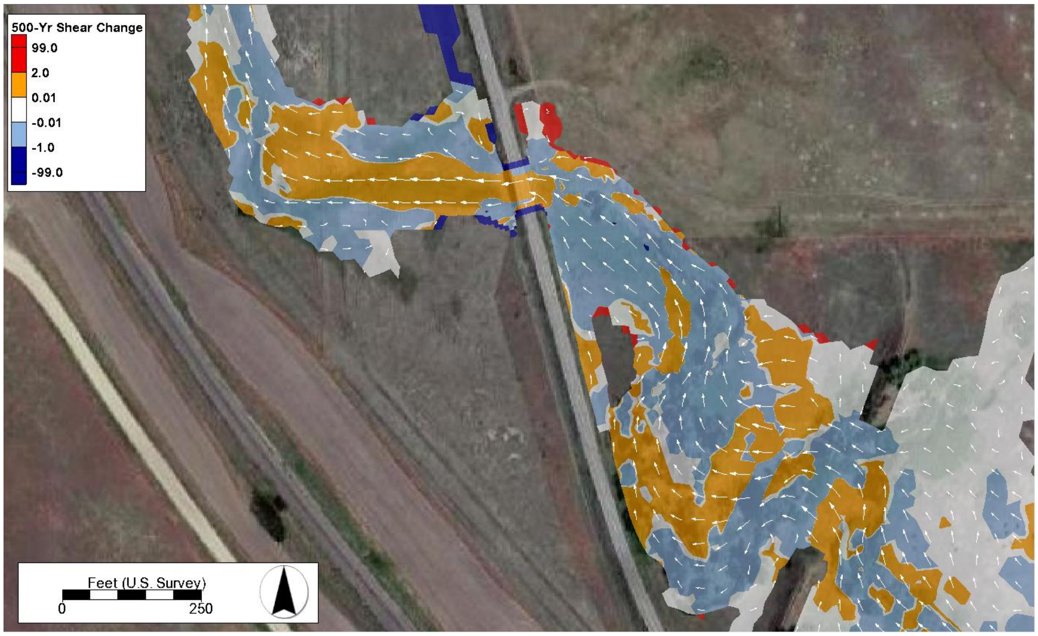


Figure 12: Proposed vs existing bed shear stress change (orange and red indicate an increase in shear stress) during the 500-year event

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 14** shows a comparison of the 100-year water surface elevations around the bridge. Outside of the immediate velocity of the proposed structure there are no WSEL rise greater than 0.5-foot, and no rises on insurable structures. In **Figure 14** 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 adverse hydraulic impacts of the proposed bridge are limited to the areas upstream of the bridge, with decreases in water surface elevations everywhere else. No insurable structures are impacted by WSEL changes associated with the proposed bridge design.

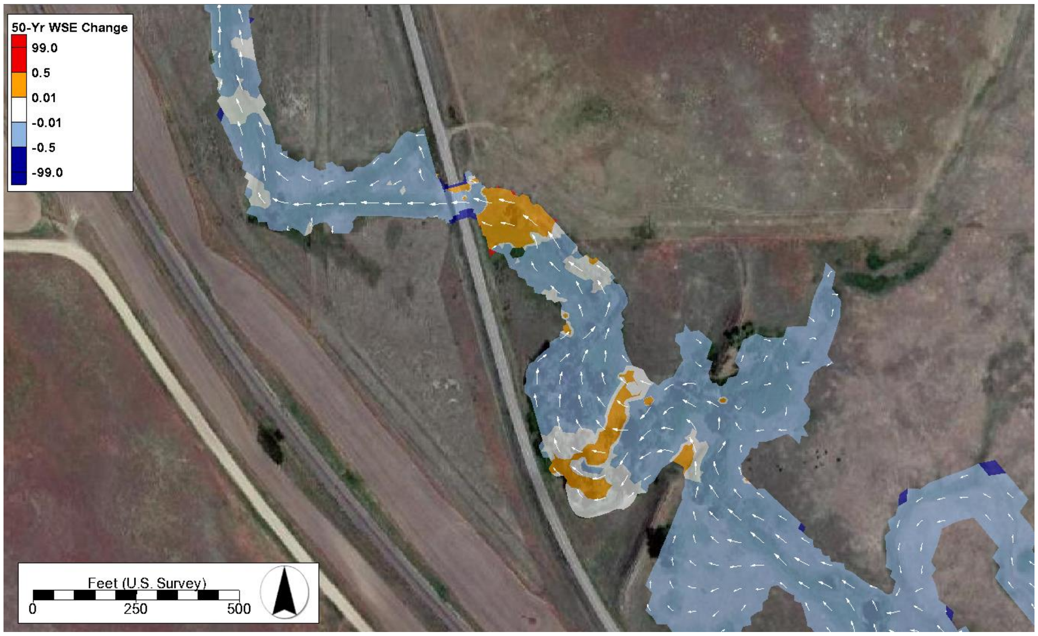


Figure 13: Proposed vs existing WSEL change (orange and red indicate an increase in WSEL) during the 50-year event

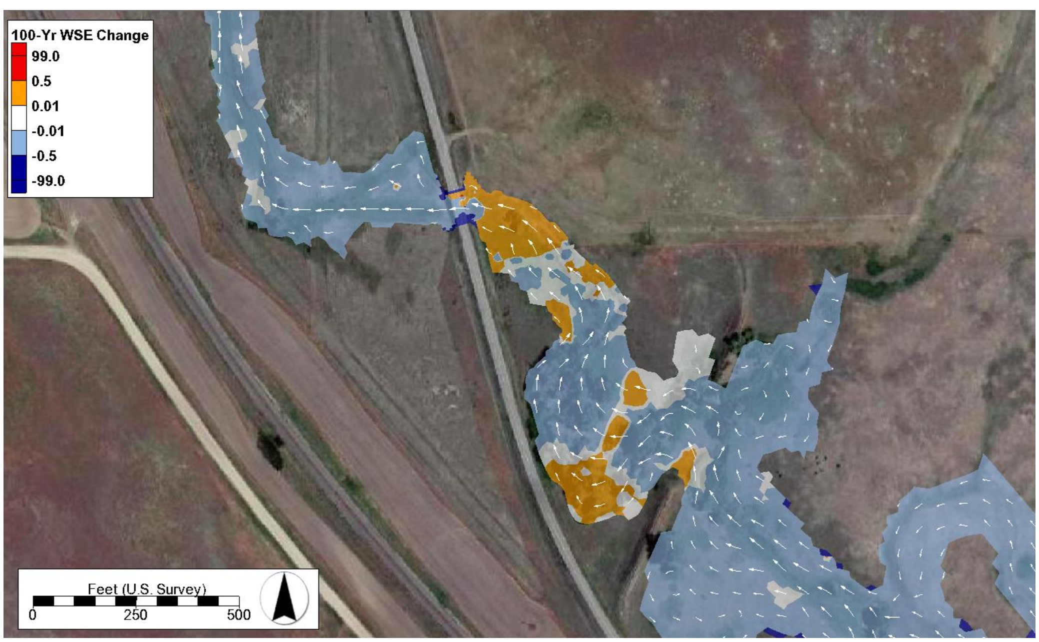


Figure 14: Proposed vs existing WSEL change (orange and red indicate an increase in WSEL) during the 100-year event

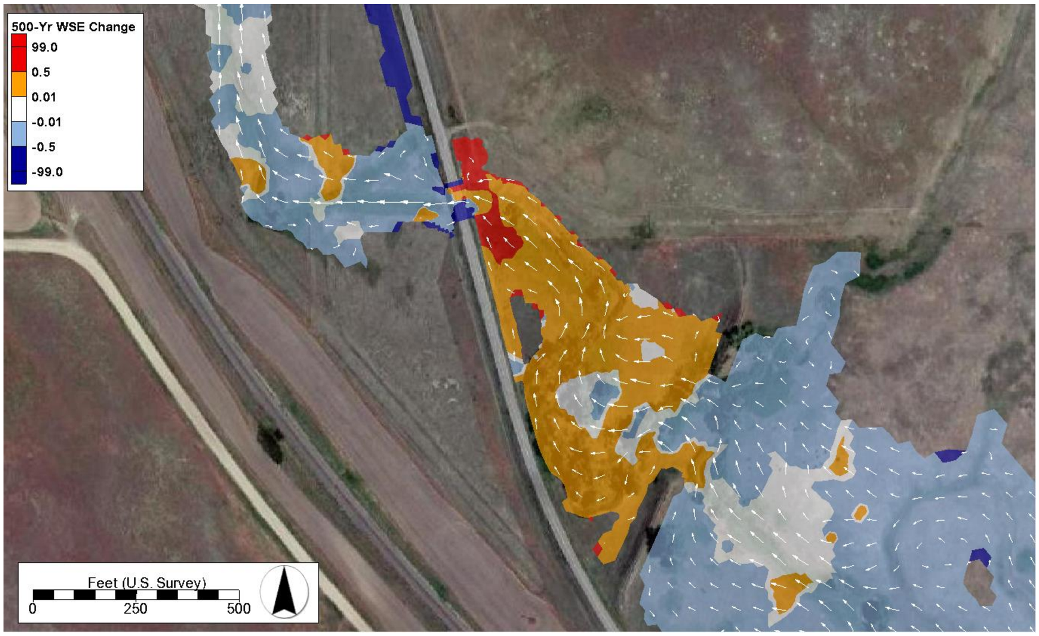


Figure 15: Proposed vs existing WSEL change (orange and red indicate an increase in WSEL) during the 500-year event

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 2,254 cfs through the bridge, with an average velocity of 5.25 ft/s and a water surface elevation of 5382.89 (ft-NAVD-88). The result of these calculations is a low chord elevation no lower than 5,384.12 feet. The proposed design low chord is anticipated to be 5,386.48 feet.

Scour and Countermeasures

Stream Stability

Agate Creek flows intermittently through lean clay. The vegetated channel follows a meandering path along US 40 with a bankfull width of about 50 feet and a depth of about 5 feet.

The channel is heavily vegetated upstream of G-21-A. Upstream and downstream of G-21-A has remained relatively stable since 1953, with the trees along the bank still in place after an aerial photo analysis was performed in Olsson's geomorphic report, attached in **Appendix D**. Therefore, it is unlikely that 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. Due to the small size of channel sediment and the presence of vegetation in the channel, scour is expected to be clear-water contraction scour.

Scour Variables

Critical Velocity for Cohesive Soils

Clear-water scour equations assume sand-bed channels with no cohesive properties. The channel is composed of low plasticity loamy clay soils that is expected to exhibit some 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 low-plasticity loamy clay (liquidity index between 0-0.25) based on the sieve analysis and geomorphic report. A voids ratio of 0.95 was assumed to give slightly more conservative results. Post-scour equilibrium depth of flow was determined iteratively to determine a final estimation of depth-averaged critical velocity. Parameters and results of this analysis are given in **Table 6**. Note calculations were done in SI units.

Table 6. Depth-averaged critical velocity for cohesive soils (Mirtskhoulava 1988)

Description	Variable	100-year	500-year
Equilibrium depth of flow (m) POST-SCOUR.	h	4.7	8.1
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	0.7	0.7
Depth averaged critical velocity (m/sec).	U_c	0.65	0.69
Depth-averaged critical velocity (ft/sec)	**U_c**	**2.13**	**2.25**

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.**

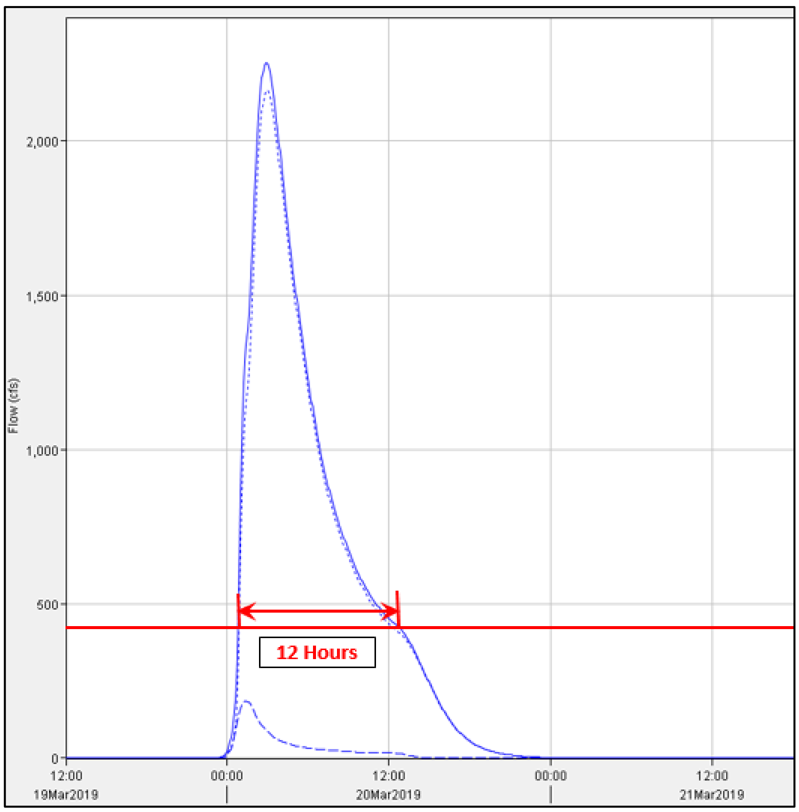


Figure 16. Flood Hydrograph for the 100-year event

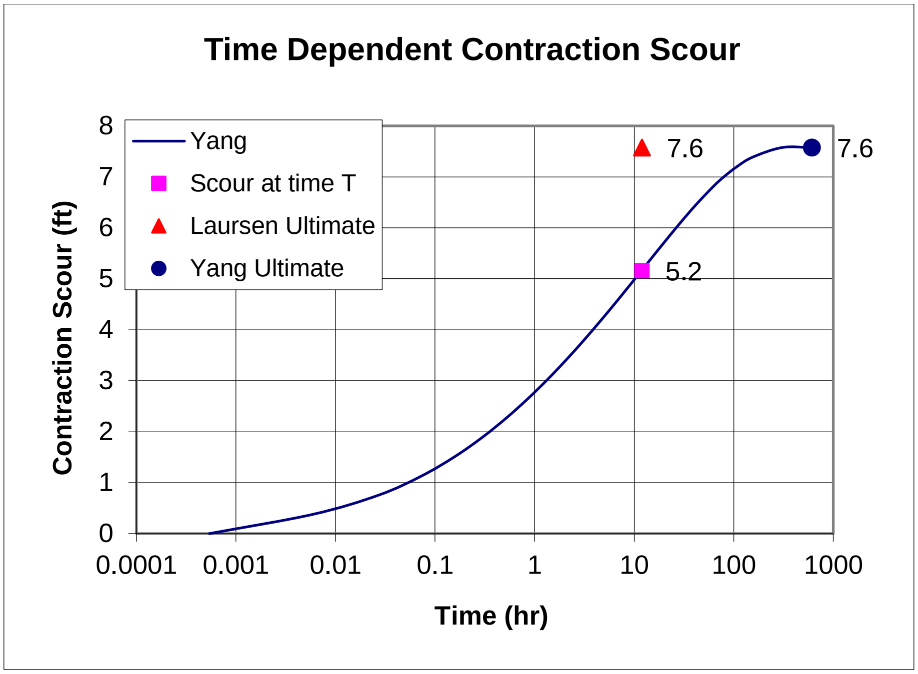


Figure 17. Time-dependent clear-water contraction scour for the 100-year event

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 7**. 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 7. 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-88)	Abutment Scour Elevation (ft NAVD-88)
100-Year	2,254	2.13	12	5.3	15.8	5367.4	5356.9
500-Year	4,381	2.25	14	9.6	19.0	5363.1	5353.7

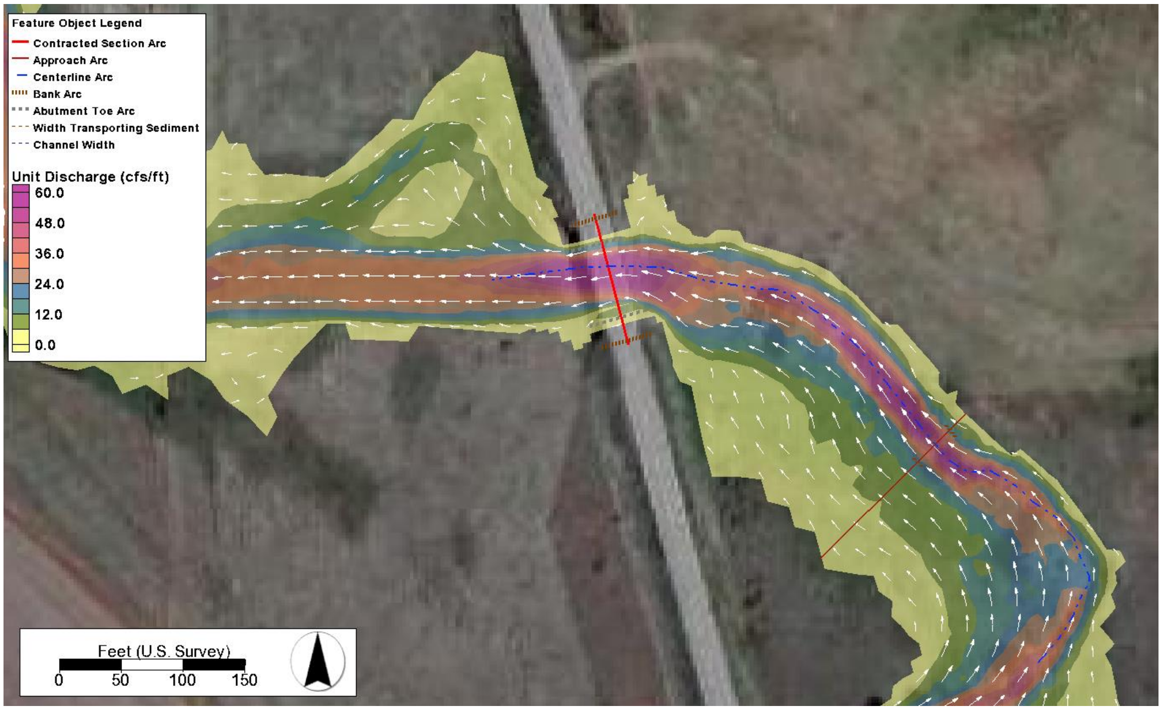


Figure 18. Approach arc with 100-year unit discharge map

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, included in **Appendix B**, indicate that a median rock size of greater than 2 feet, extending across the channel (based on $2Y_0$), will be required for 500-year event flow conditions**.** Therefore, it is recommended to use a 2 foot thick layer of matrix riprap to reduce the rock size necessary to 12 inches for protection. The riprap should be keyed in at 4 ft slightly downstream of the bridge. Please note that scour depths are measured downwards from the channel invert (thalweg) elevation. This 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.

A proposed bridge cross section, including scour and countermeasures, is shown in **Figure 19.**

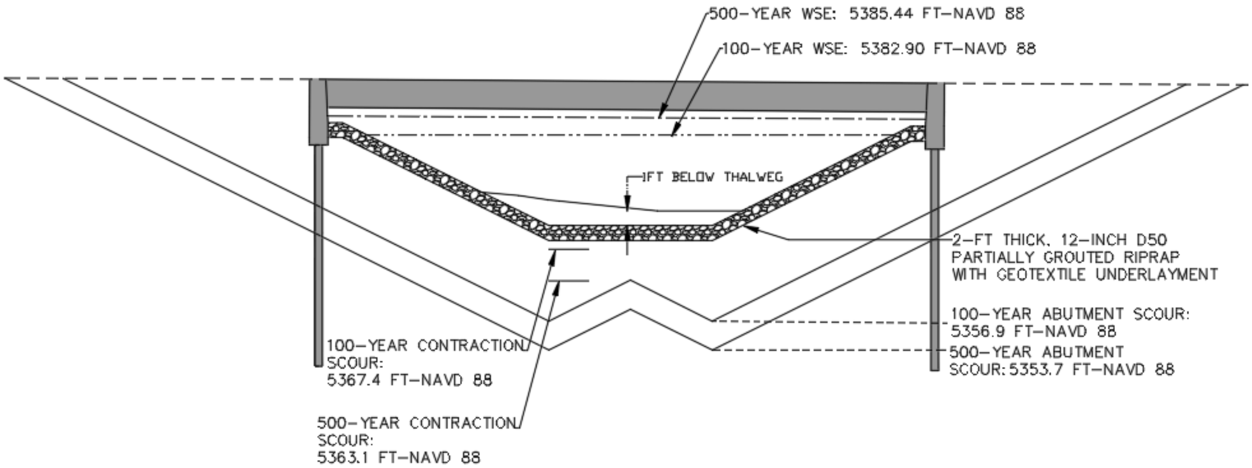


Figure 19: Proposed bridge cross-section, including scour countermeasures

Conclusion

The existing G-21-A structure along US 40 over Agate Creek will be replaced by CDOT. This preliminary hydraulic analysis has concluded that an 81-foot clear span bridge will be a suitable replacement, causing no WSEL increase in excess of the 0.5 foot allowed in Zone X floodplains, nor have any adverse impact to structures during the 100-year flood event. A scour analysis has been performed for the 100-year and 500-year floods, producing scour elevations of 5357.2 and 5353.9 feet (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 partially-grouted 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.

References

Arcement, George J., Schneider, Verne R., USGS 1989. “Guide for selecting Manning's roughness coefficients for natural channels and flood plains.” USGS Publications Warehouse.

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.

Barnes, Harry H., USGS 1967. “Roughness Characteristics of Natural Channels.” U.S. Geological Survey Water Supply Paper 1849. United States Government Printing Office, Washington: 1967.

Brown, S.A., Schall, J.D., Morris, J.L., Doherty, C.L., Stein, S.M., Warner, J.C., 2009. “Urban Drainage Design Manual,” Federal Highways Administration Hydraulic Engineering Circular 22, Third Edition, Washington, D.C.

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

Hoffmans, G,J,C,M. and Verheij, H.J. “Scour Manual.” Delft Hydraulics: 1997

“Hydraulic Design Report, C-21-BE SH52 Over South Platte River”, Project No. 20256. RESPEC Consulting & Services, April 2016.

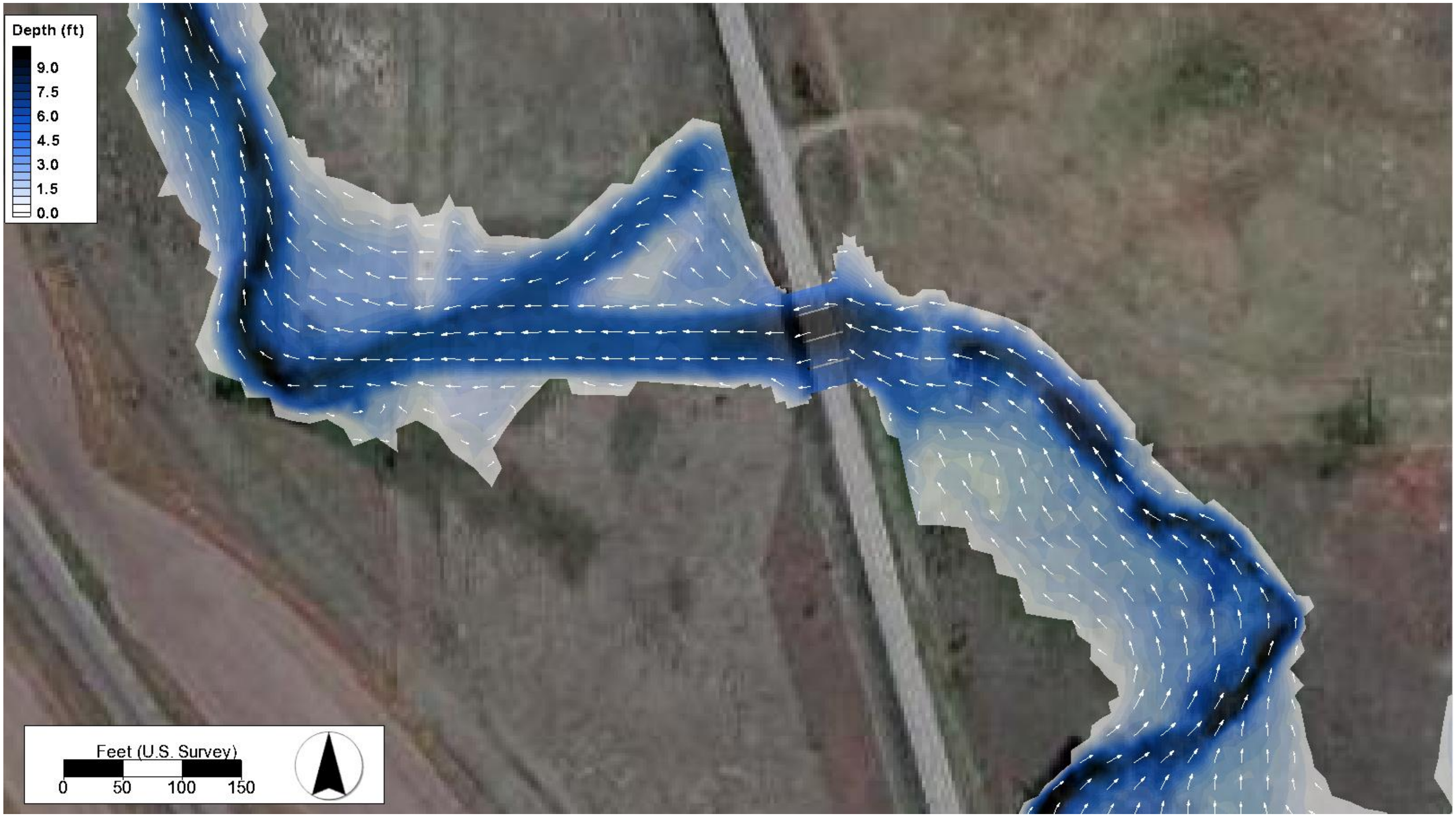
Lagasse, P.F., Clopper, P.E., Pagan-Ortiz, J.E., Zevenbergen, L.W., Arneson, L.A., Schall, J.D., and Girard, L.G., 2009. “Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance,” Federal Highways Administration Hydraulic Engineering Circular 23 Volumes 1 and 2, Third Edition, Washington, D.C.

Zevenbergen, L.W., Arneson, L.A, Hunt, J.H., Miller, A.C. 2012. “Hydraulic Design of Safe Bridges,” Federal Highways Administration Hydraulic Engineering Circular 7, Washington D.C.

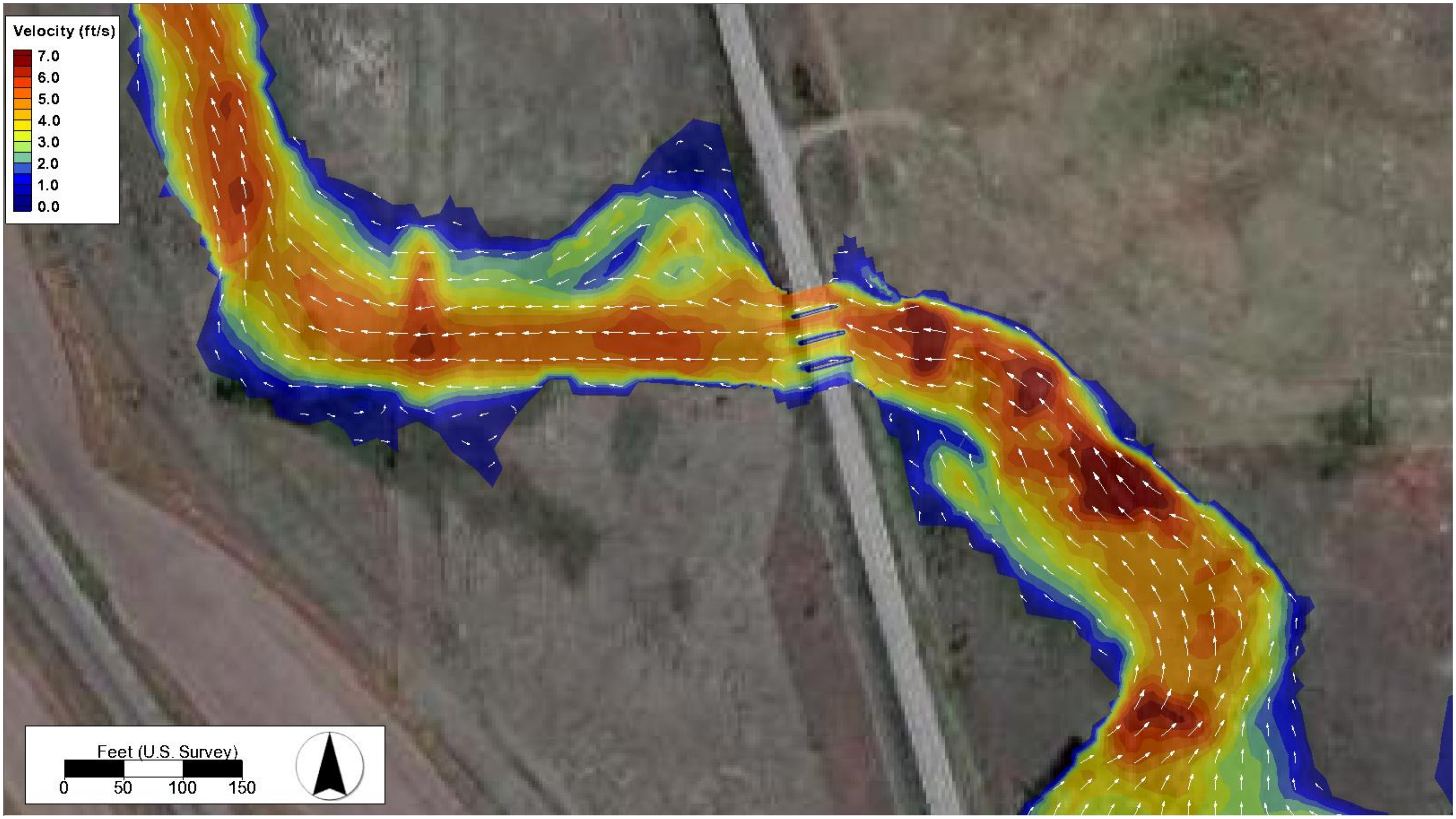
“Geomorphology Assessment of Stream Stability, Non-BE Bridge G-21-A (US36 MP361.530) over Agate Creek near Agate, CO”, Project No. 017-1690. Olsson. September 23, 2019.

Appendix A

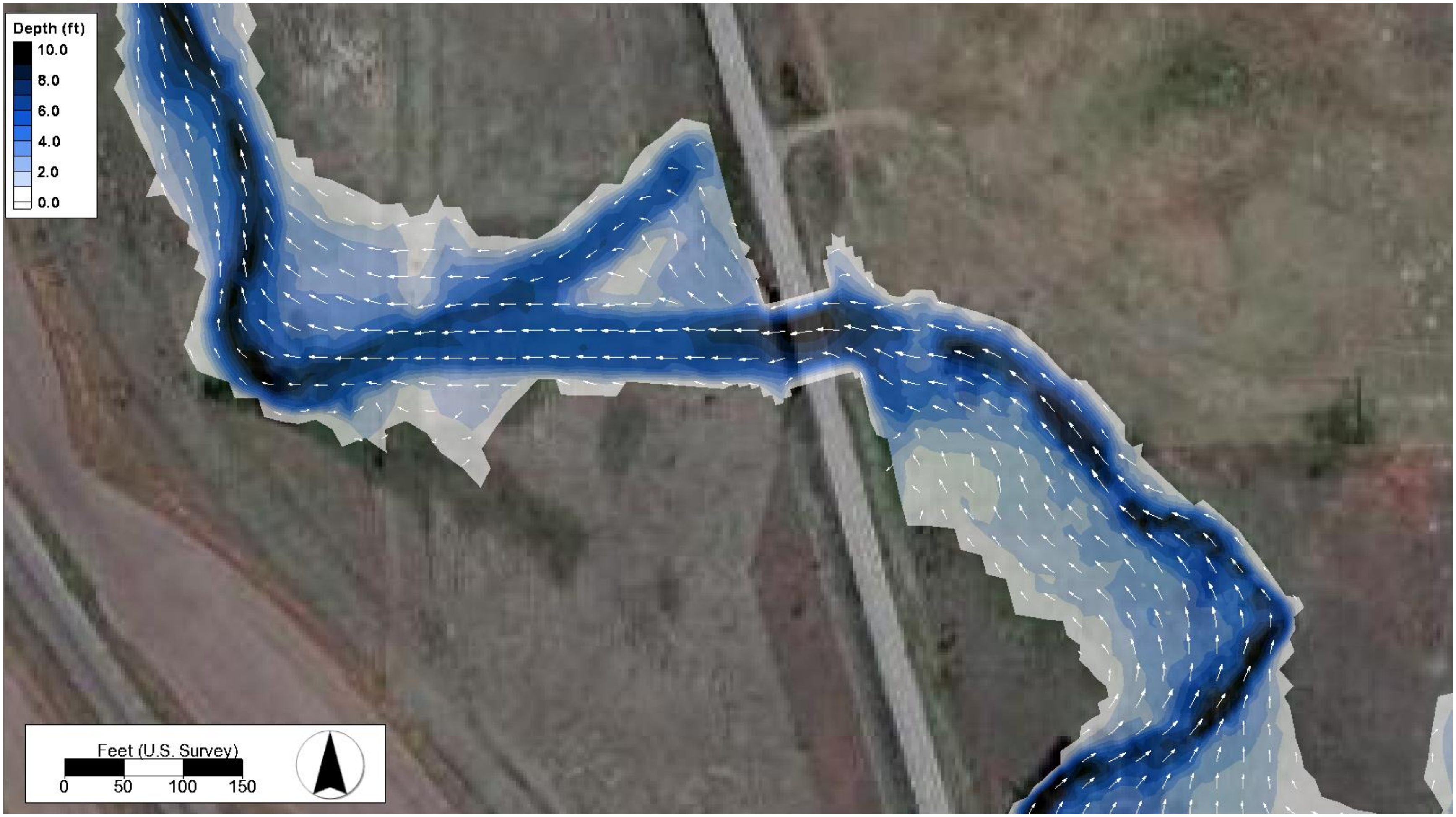
Hydraulic Modeling



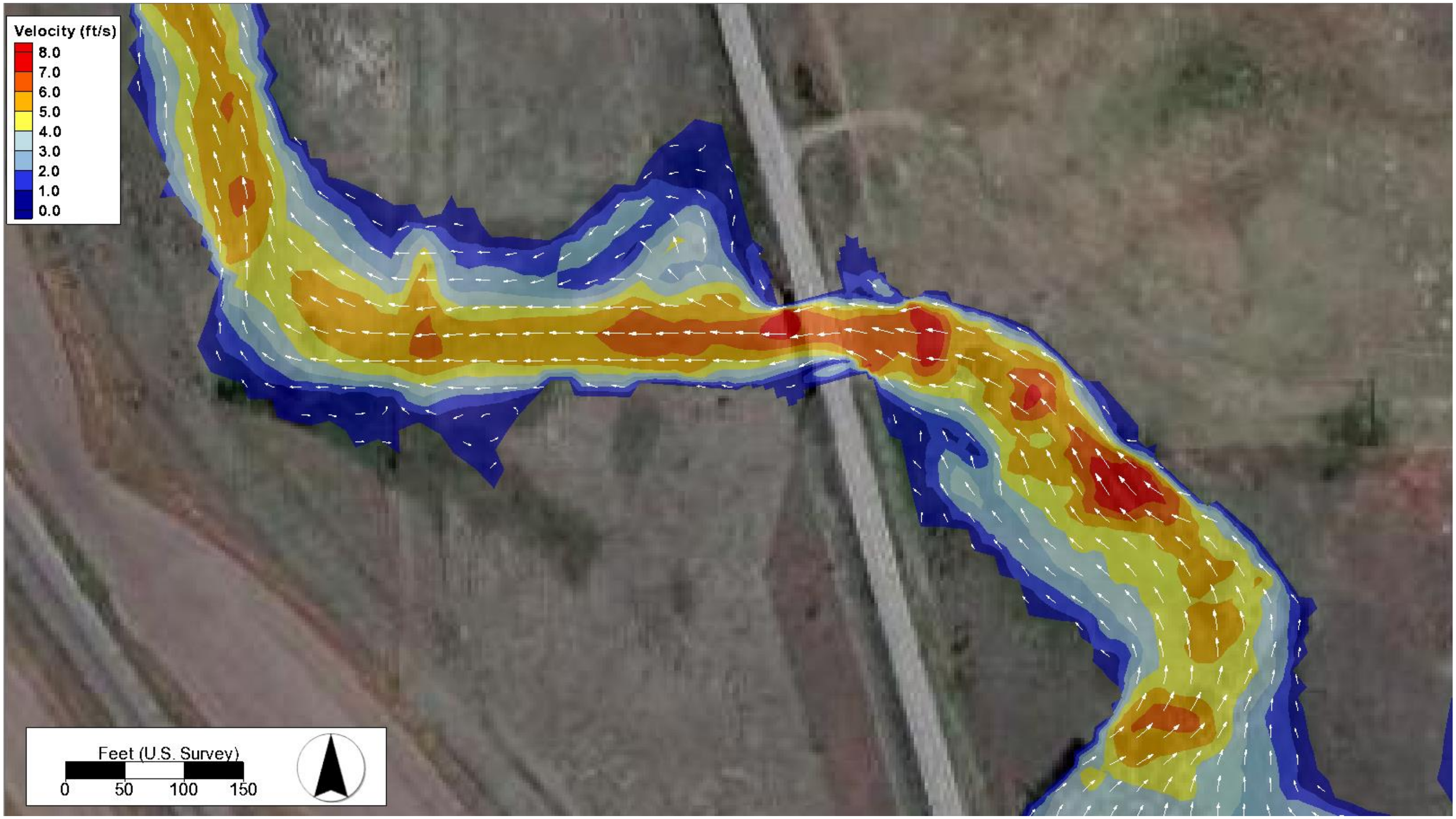
Existing conditions, 100-year, depth



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	=	6.65	ft. adjacent	Y_o	=	8.11	ft. adjacent
	Y_1	=	2.97	ft. adjacent	Y_1	=	3.83	ft. adjacent
	Left Y_0	=		ft. adjacent	Left Y_0	=		ft. adjacent
	Right Y_0	=		ft. adjacent	Right Y_0	=		ft. adjacent
	Q_1	=	2,268	cfs	Q_1	=	4,471	cfs
	Channel Q_2	=	2,252	cfs	Channel Q_2	=	4,350	cfs
	Left Q_2	=		cfs	Left Q_2	=		cfs
	Right Q_2	=		cfs	Right Q_2	=		cfs
	A_1	=	469.49	$ft.^2$	A_1	=	841.19	$ft.^2$
	A_2	=	456.42	$ft.^2$	A_2	=	592.01	$ft.^2$
	W_1	=	157.86	ft.	W_1	=	219.85	ft.
	Channel W_2	=	68.65	ft.	Channel W_2	=	73.04	ft.
	Left W_2	=		ft.	Left W_2	=		ft.
	Right W_2	=		ft.	Right W_2	=		ft.
	D_{50}		0.074	mm	D_{50}		0.074	mm
	Energy Slope	=	4.218E-03	ft/ft	Energy Slope	=	4.339E-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	=	0.0184779	fps	Fall Vel., ω	=	0.0184779	fps
	V_m	=	4.83	fps	V_m	=	5.32	fps
	D_{50}	=	0.000243	ft.	D_{50}	=	0.000243	ft.

*D_{69} = 0.074 mm. D_{50} not reported by sieve analysis.

BY: **BLC**

DATE: **13-Feb-20**

100-YEAR SCOUR SUMMARY
FOR STATE HWY 40 OVER AGATE CREEK
BRIDGE G-21-A
ELBERT 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	5372.7	--	5.3	15.8	15.8	5356.9
2	5372.7	--	5.3	15.8	15.8	5356.9

500-YEAR SCOUR SUMMARY
FOR STATE HWY 40 OVER AGATE CREEK
BRIDGE G-21-A
ELBERT 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	5372.7	--	9.6	19.0	19.0	5353.7
2	5372.7	--	9.6	19.0	19.0	5353.7

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. This analysis considered both cohesive soil effects and the limited duration of scouring flows at the site.

Calc. By:	BLC	Date:	2/13/2020
Check By:	[illegible]	Date:	14 Feb 2020

SCOUR MODE COMPUTATION
FOR STATE HWY 40 OVER AGATE CREEK
BRIDGE G-21-A
ELBERT 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	=	469
APPROACH SECTION MAIN CHANNEL WIDTH (ft), W_1	=	158
APPROACH SECTION AVERAGE CHANNEL DEPTH (ft), $Y_1 = A_1/W_1$	=	2.97
MEDIAN GRAIN SIZE (ft), D_{50}	=	0.000656
Ku		11.17
BED TRANSPORT CRITICAL VELOCITY (fps), V_c	=	1.16
DISCHARGE IN APPROACH CHANNEL (cfs), Q_1	=	2,268
MEAN VELOCITY IN APPROACH CHANNEL (fps), V_m	=	4.83
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	=	841
APPROACH SECTION MAIN CHANNEL WIDTH (ft), W_1	=	220
APPROACH SECTION AVERAGE CHANNEL DEPTH (ft), $Y_1 = A_1/W_1$	=	3.83
MEDIAN GRAIN SIZE (ft), D_{50}	=	0.000656
Ku		11.17
BED TRANSPORT CRITICAL VELOCITY (fps), V_c	=	1.21
DISCHARGE IN APPROACH CHANNEL (cfs), Q_1	=	4,471
MEAN VELOCITY IN APPROACH CHANNEL (fps), V_m	=	5.32
MAIN CHANNEL SCOUR MODE	=	~~LIVE-BED~~

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

Calc. By:	BLC	Date:	2/13/2020
Check By:	[signature]	Date:	2/14/2020

CRITICAL AVERAGE VELOCITY CALCULATION
FOR STATE HWY 40 OVER AGATE CREEK
BRIDGE G-21-A
ELBERT COUNTY, COLORADO

FEBRUARY 2020

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

$$U_c = \log\left(\frac{8.8h}{d_a}\right)\sqrt{\frac{0.4}{\rho}\left[(\rho_s - \rho)gd_a + 0.6C_f\right]}$$ *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 Loamy Clay (Low Plasticity) with a liquidity index between 0 and 0.25 based on the geomorphic report. A void ratio of 0.95 was assumed for a slightly more conservative result [Table 2.5, Hoffman, 1997]

SOIL PROPERTIES

Porosity	n	=	
voids ratio	e	=	
Specific Gravity	G	=	2.65
Saturated Water Content	w_s	=	0.000
Liquid Limit	ll	=	0.39
Plastic Limit	pl	=	0.18
Plasticity Index	pi	=	0.21
Liquidity Index	li	=	
Soil Type (USCS)		=	C and D
Cohesion of Clay (Pa) [From Table 2.5, Hoffman, 1997].	C_o	=	18.6

COHESIVE SOIL CRITICAL VELOCITY

EVENT			100-year	500-year
Computed depth of contracted section (ft), Y_2	y_2	=	15.43	26.47
Equilibrium depth of flow (m) POST-SCOUR.	h	=	4.7	8.1
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	=	0.7	0.7
Depth averaged critical velocity (m/sec).	U_c	=	0.65	0.69
Depth averaged critical velocity (ft/sec)	U_c	=	2.13	2.25
Applied Depth Averaged Critical Velocity (ft/sec)	**U_c**	**=**	**2.13**	**2.25**

Calc. By:	BLC	Date:	2/13/2020
Check By:	[illegible]	Date:	14-Feb-2020

CONTRACTION SCOUR COMPUTATIONS
FOR STATE HWY 40 OVER AGATE CREEK
BRIDGE G-21-A
ELBERT 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 DISCHARGE
MAIN CHANNEL
CLEAR-WATER CONTRACTION SCOUR CALCULATIONS

AVERAGE VELOCITY IN CONTRACTED SECTION (fps), V_0	=	4.93
AVERAGE WATER DEPTH (ft), Y_0	=	6.65
CRITICAL VELOCITY (fps), V_c	=	2.13
COMPUTED DEPTH OF CONTRACTED SECTION (ft), Y_2	=	15.43
DEPTH OF CONTRACTION SCOUR (ft), Y_s	**=**	**8.78**

Calc. By:	BLC	Date:	2/13/2020
Check By:	[signature]	Date:	14 Feb 2020

CONTRACTION SCOUR COMPUTATIONS
FOR STATE HWY 40 OVER AGATE CREEK
BRIDGE G-21-A
ELBERT 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.

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

AVERAGE VELOCITY IN CONTRACTED SECTION (fps), V_0	=	7.35
AVERAGE WATER DEPTH (ft), Y_0	=	8.11
CRITICAL VELOCITY (fps), V_c	=	2.25
COMPUTED DEPTH OF CONTRACTED SECTION (ft), Y_2	=	26.47
DEPTH OF CONTRACTION SCOUR (ft), Y_s	**=**	**18.36**

Calc. By:	BLC	Date:	2/13/2020
Check By:	[illegible]	Date:	14 Feb 2020

100-YEAR TIME SCOUR SUMMARY
FOR STATE HWY 40 OVER AGATE CREEK
BRIDGE G-21-A
ELBERT COUNTY, COLORADO

FEBRUARY 2020

		Approach	Bridge (t=0)	Live-Bed Bridge (ultimate)	Clear-Water Bridge (ultimate)
Manning n	=	0.03			
Q (cfs)	=	2,268	2,252		
W (ft)	=	157.86	68.65		
Y (ft)	=	2.97	6.65	5.28	15.43
A (ft^2)	=	469	456.4	363	1059
V (ft/s)	=	4.83	4.93	6.21	2.13
Sf	=	0.002224	0.000794	0.001710	0.000048

D50mm	=	0.403	*Iterate D_{50} until Y_2 (clear water) below matches cohesive clear-water Y_2 ✓
D50ft	=	0.001321	
Vcrit (ft/s)	=	4.90	HARD_WIRED, assumed Clear-Water
Sediment Sg	=	2.65	
Fall vel. (ft/s)	=	0.205	
Sediment Porosity	=	0.58	
Scour hole side slope	=	2.5	

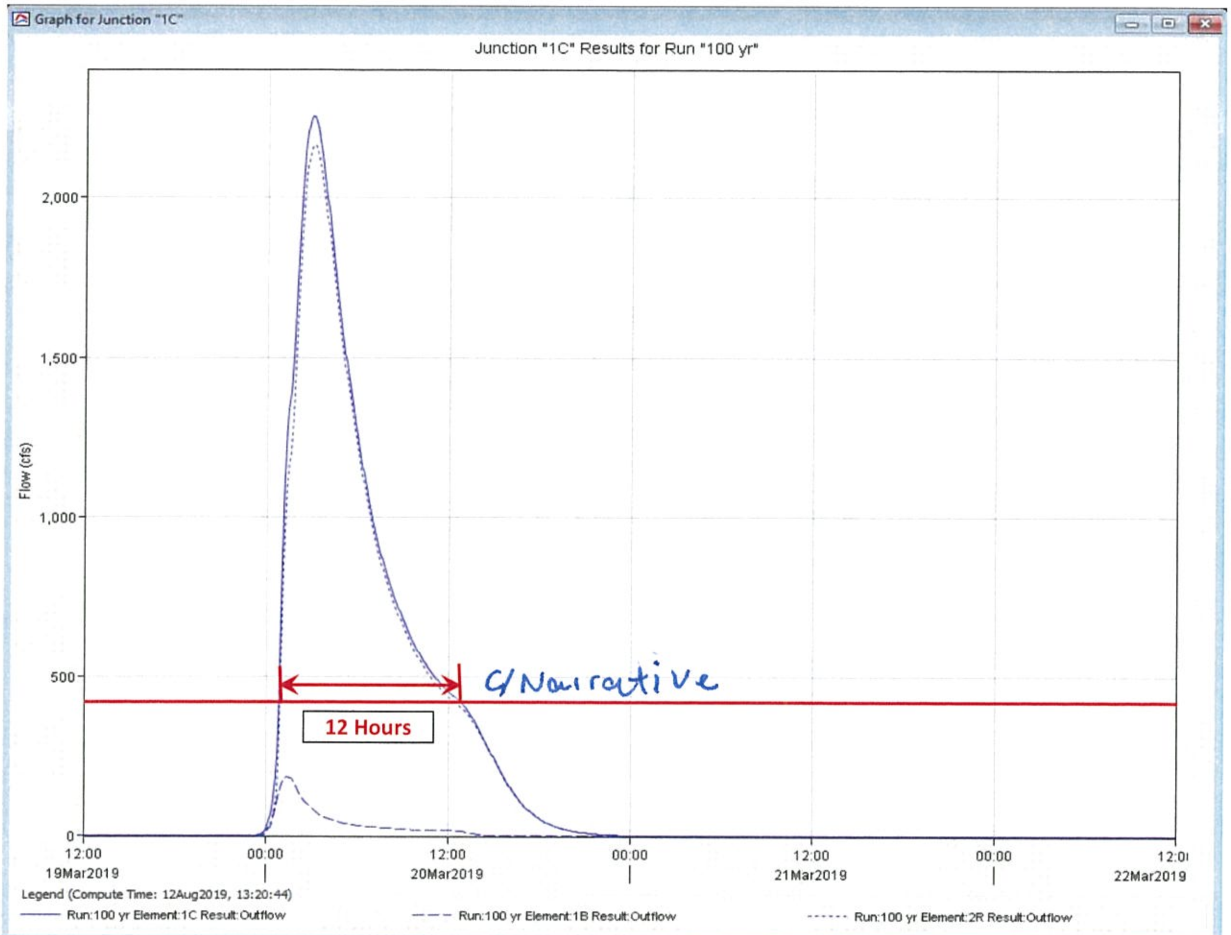
		Approach	Bridge (t=0)	Live-Bed Bridge (ultimate)	Clear-Water Bridge (ultimate)
nu (ft^2/s)	=	1.08E-05			
g (ft/s^2)	=	32.2			
water wt (lb/ft^3)	=	62.4			
U* (ft/s)	=	0.462	0.412	0.539	0.154
Re*	=	56.48	50.45	66.00	18.89
Vcr/omega	=	2.14	2.18	2.08	2.72
log(C)	=	3.398	2.905	3.402	0.744
C ppm-wt	=	2502.7	802.8	2521.0	5.6
Qs(cfs)	=	2.142	0.682	2.142	0.005

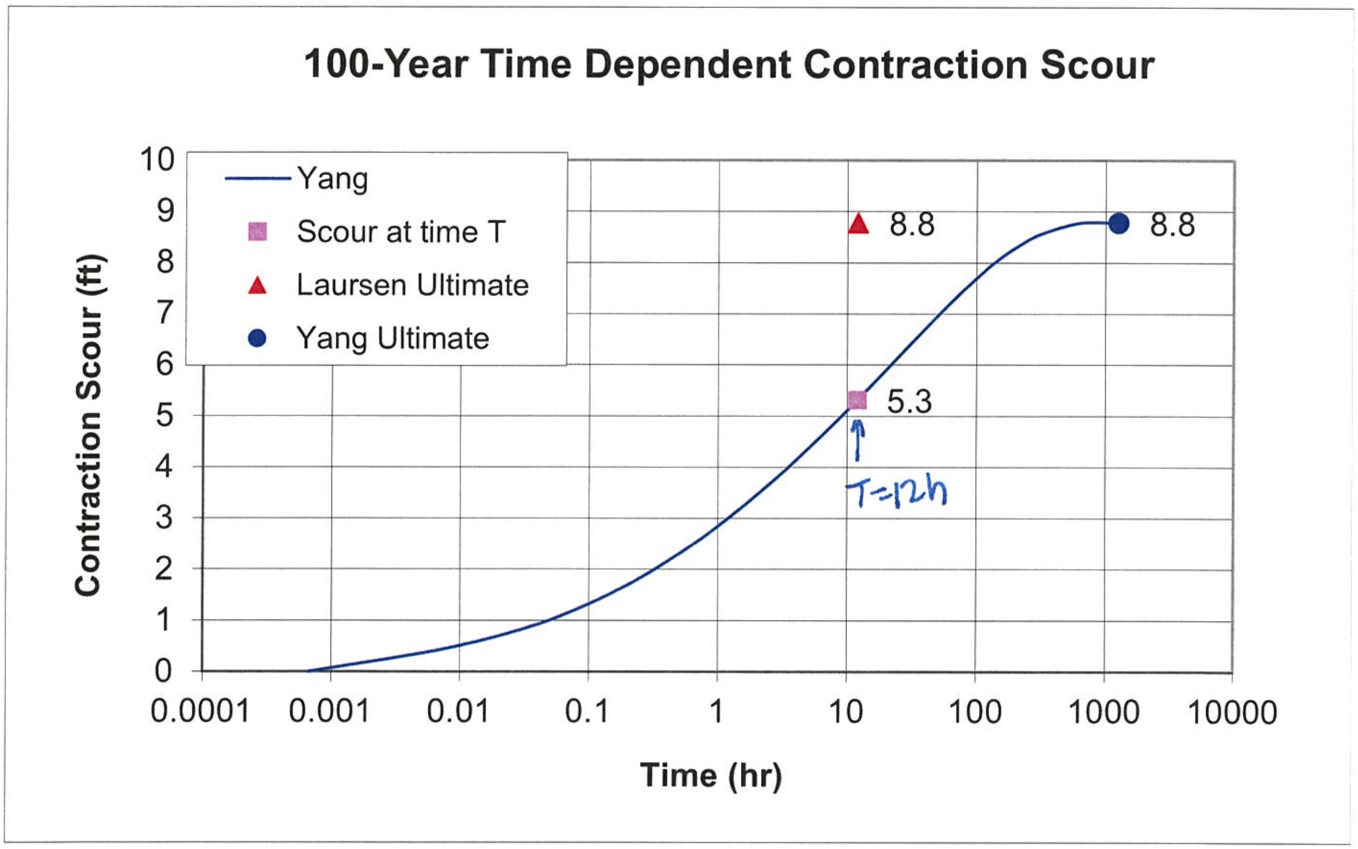
Clear Water Controls

Y2 (clear water)	15.43
Y2 (ultimate live-bed)	5.28
Qs Ratio	1.000
	Converged
Laursen Ys(ultimate)	8.8
Yang Ys(ultimate)	8.8

Scour Time T (hr)	12
Scour at Time T (ft)	**5.3**

Calc. By:	BLC	Date:	2/13/2020
Check By:	MWD	Date:	2/14/2020


Graph for Junction "1C"
Junction "1C" Results for Run "100 yr"
Flow (cfs)
2,000
1,500
1,000
500
0
12:00
19Mar2019
00:00
12:00
20Mar2019
00:00
12:00
21Mar2019
00:00
12:00
22Mar2019
12 Hours
c/Narrative
Legend (Compute Time: 12Aug2019, 13:20:44)
Run:100 yr Element:1C Result:Outflow
Run:100 yr Element:1B Result:Outflow
Run:100 yr Element:2R Result:Outflow


100-Year Time Dependent Contraction Scour
Yang
Scour at time T
Laursen Ultimate
Yang Ultimate
8.8
8.8
5.3
T=12h
Contraction Scour (ft)
10
9
8
7
6
5
4
3
2
1
0
0.0001
0.001
0.01
0.1
1
10
100
1000
10000
Time (hr)

500-YEAR TIME SCOUR SUMMARY
FOR STATE HWY 40 OVER AGATE CREEK
BRIDGE G-21-A
ELBERT COUNTY, COLORADO

FEBRUARY 2020

		Approach	Bridge (t=0)	Live-Bed Bridge (ultimate)	Clear-Water Bridge (ultimate)
Manning n		0.03			
Q (cfs)	=	4,471	4,350		
W (ft)	=	219.85	73.04		
Y (ft)	=	3.83	8.11	8.03	26.47
A (ft^2)	=	841	592.0	587	1933
V (ft/s)	=	5.32	7.35	7.41	2.25
Sf	=	0.001924	0.001352	0.001393	0.000026

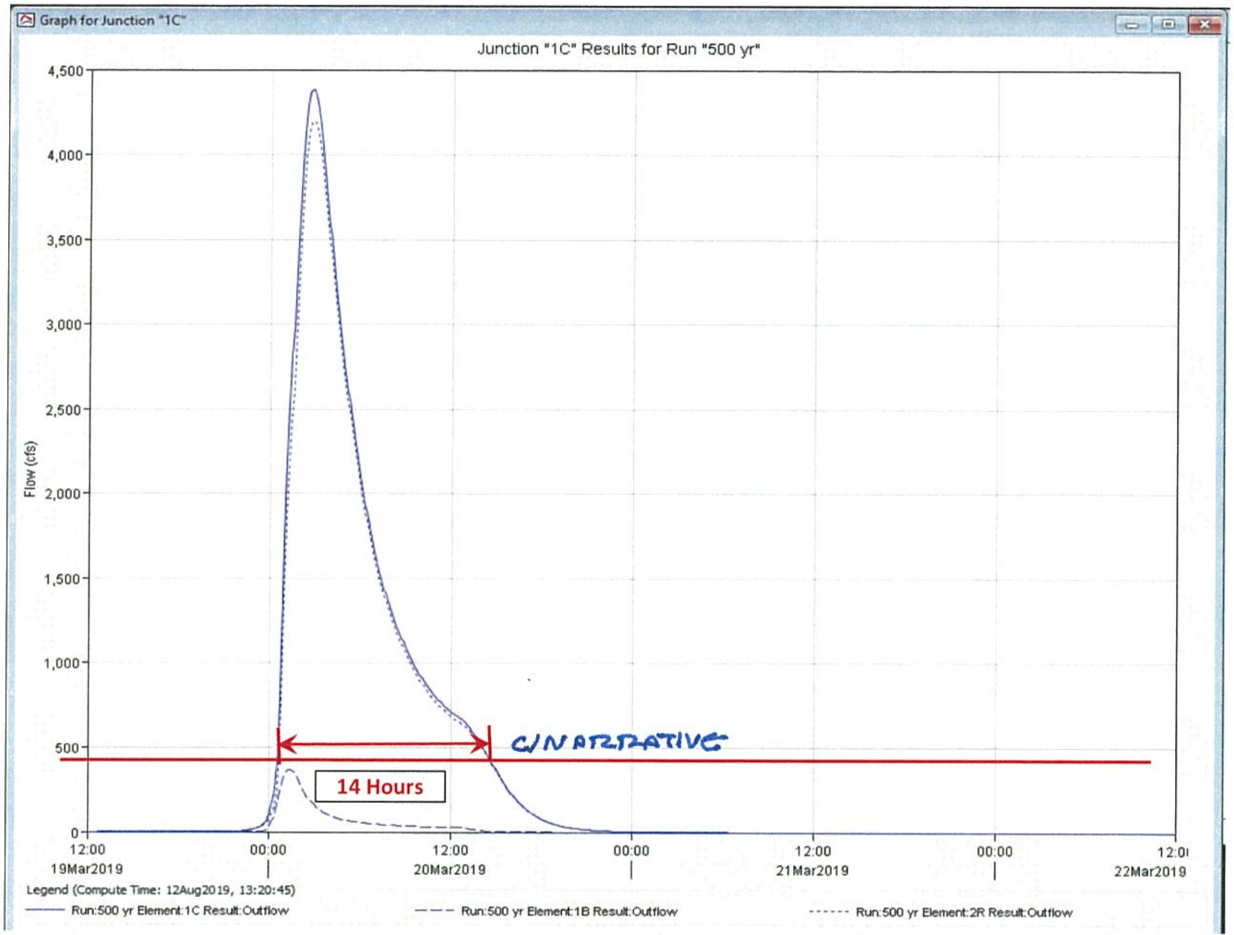
D50mm	=	0.365	*Iterate D_{50} until Y_2 (clear water) below matches cohesive clear-water Y_2		
D50ft	=	0.001196			
Vcrit (ft/s)	=	6.00	HARD_WIRED, assumed Clear-Water		
Sediment Sg	=	2.65			
Fall vel. (ft/s)	=	0.185			
Sediment Porosity	=	0.58			
Scour hole side slope	=	2.5			
	=				
nu (ft^2/s)	=	1.08E-05			
g (ft/s^2)	=	32.2			
water wt (lb/ft^3)	=	62.4			
U* (ft/s)	=	0.487	0.594	0.600	0.149
Re*	=	53.96	65.83	66.52	16.55
Vcr/omega	=	2.16	2.08	2.08	2.82
log(C)	=	3.408	3.401	3.420	0.438
C ppm-wt	=	2558.9	2519.1	2629.6	2.7
Qs(cfs)	=	4.317	4.136	4.317	0.004

Clear Water Controls

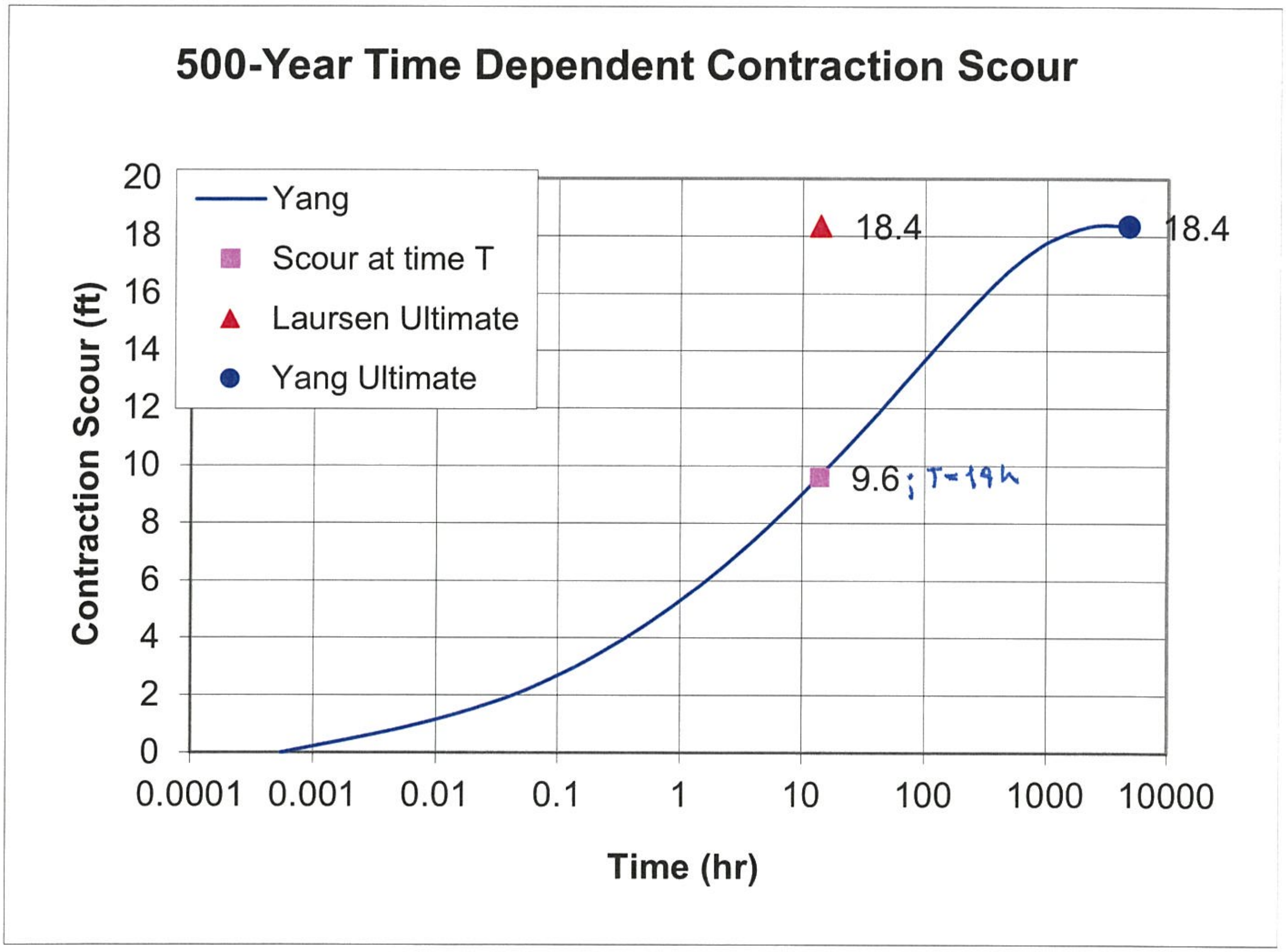
Y2 (clear water)	26.47
Y2 (ultimate live-bed)	8.03
Qs Ratio	1.000
	Converged
Laursen Ys(ultimate)	18.4
Yang Ys(ultimate)	18.4

Scour Time T (hr)	14
Scour at Time T (ft)	**9.6**

Calc. By:	BLC	Date:	2/13/2020
Check By:	Am-ND	Date:	14 Feb 2020



Graph for Junction "1C"
Junction "1C" Results for Run "500 yr"
4,500
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
Flow (cfs)
C/NARRATIVE
14 Hours
12:00
19Mar2019
00:00
12:00
20Mar2019
00:00
12:00
21Mar2019
00:00
12:00
22Mar2019
Legend (Compute Time: 12Aug2019, 13:20:45)
Run:500 yr Element:1C Result:Outflow
Run:500 yr Element:1B Result:Outflow
Run:500 yr Element:2R Result:Outflow


500-Year Time Dependent Contraction Scour
Yang
Scour at time T
Laursen Ultimate
Yang Ultimate
18.4
18.4
9.6; T=14h
Contraction Scour (ft)
0
2
4
6
8
10
12
14
16
18
20
0.0001
0.001
0.01
0.1
1
10
100
1000
10000
Time (hr)

TIME-DEPENDENT ABUTMENT SCOUR COMPUTATIONS FOR COHESIVE SOILS
FOR STATE HWY 40 OVER AGATE CREEK
BRIDGE G-21-A
ELBERT 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 UNIT DISCHARGE (ft^2/s), q_f	=	14.37	20.34
CONTRACTED SECTION UNIT DISCHARGE (ft^2/s), q_{2f}	=	32.81	59.56
Ku, CLEAR WATER CONTRACTION SCOUR COEFFICIENT	=	1.486	1.486
DEPTH-AVERAGED CRITICAL VELOCITY (ft/s), V_c	=	2.13	2.25
COMPUTED DEPTH OF CONTRACTED SECTION (ft), Y_2	=	15.43	26.47
STORM DURATION (hrs), t		12.0	14.0
TIME RATE FLOW DEPTH (ft), Y_{2t}	=	11.96	17.71
DISCHARGE RATIO, q_{2f}/q_f	=	2.28	2.93
AMPLIFICATION FACTOR CLEAR-WATER CONDITIONS, α_B	=	1.88 ✓	1.53
COMPUTED WATER DEPTH AT ABUTMENT (ft), Y_{max}	=	22.49	27.09
AVERAGE WATER DEPTH AT BRIDGE (ft), Y_0	=	6.65	8.11
AVERAGE ABUTMENT SCOUR DEPTH, Y_s	=	**15.8**	**19.0**

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

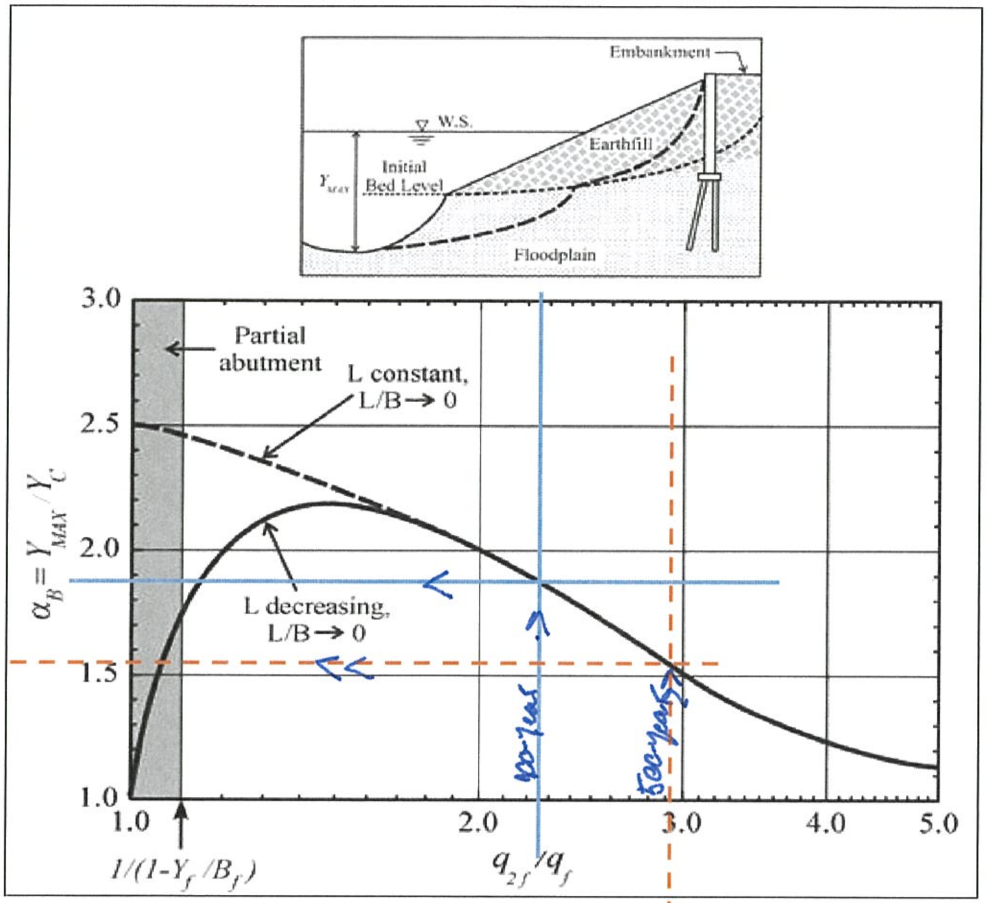


Figure 8.11. Scour amplification factor for spill-through abutments and clear-water conditions (NCHRP 2010b).

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

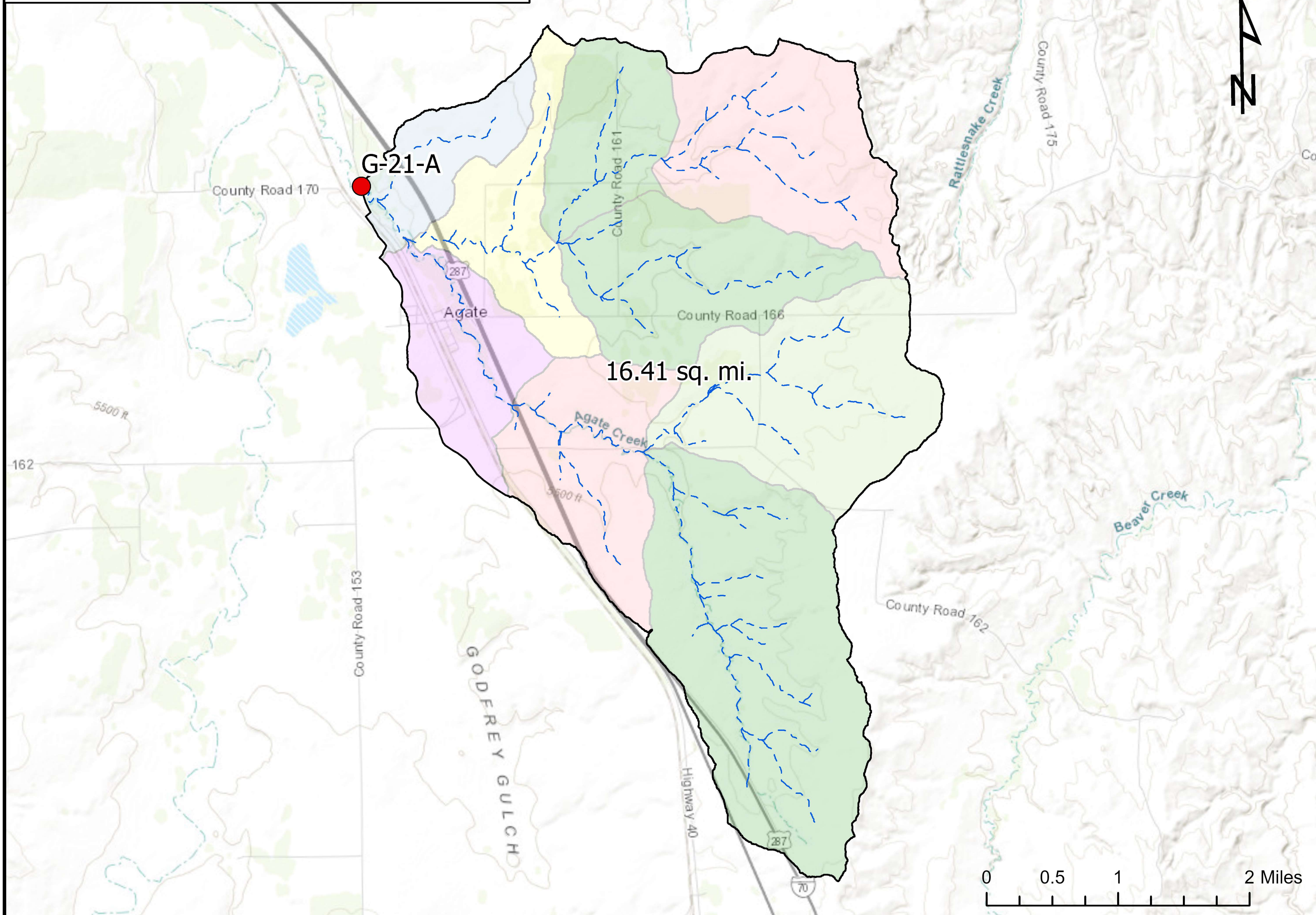
Calculated riprap size	D_{50} (ft)	0.58	0.75
Nominal riprap size	D_{50} (ft)	12-Inch	12-Inch

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

Appendix C

Hydrology

G-21-A Drainage Basin Map



G-21-A

Summary of Methods

Basin Input	
Basin Name	G-21-A
Subbasin	
Area (Mi^2)	16.41
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	G-21-A
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	G-21-A
Reach	
Routing Method	Muskingum-Cunge
Loss/Gain Method	None
Routing	
Time Step Method	Automatic Fixed Interval
Shape	Trapezoid

Control Specifications	
Basin Name	G-21-A
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

G-21-A
Global Summary

10-year Global Summary				
Hydrologic Element	Drainage Area (sq mi)	Peak Discharge (CFS)	Time of Peak	Volume (in)
7B	2.451	71.1	20Mar2019, 02:45	0.31
8B	3.8421	136	20Mar2019, 04:10	0.45
6B	1.6706	41.8	20Mar2019, 02:00	0.24
4B	2.1113	137.6	20Mar2019, 02:00	0.52
5B	1.9311	74	20Mar2019, 02:45	0.39
10B	1.116	216	20Mar2019, 00:45	0.74
2B	1.2249	57.2	20Mar2019, 02:30	0.44
3B	1.2093	26.3	20Mar2019, 02:30	0.23
1B	0.8332	35.8	20Mar2019, 01:40	0.34
7C	6.2931	194.3	20Mar2019, 03:40	0.4
6C	7.9637	220.7	20Mar2019, 04:00	0.36
4C	2.1113	137.6	20Mar2019, 02:00	0.52
10C	5.1584	266.2	20Mar2019, 02:20	0.52
2C	15.5563	492.8	20Mar2019, 03:40	0.41
1C	16.3895	510.4	20Mar2019, 03:45	0.41
7R	6.2931	194.1	20Mar2019, 04:10	0.4
6R	7.9637	220.4	20Mar2019, 04:50	0.36
4R	2.1113	137.3	20Mar2019, 02:30	0.52
10R	5.1584	265.1	20Mar2019, 02:55	0.52
2R	15.5563	492.2	20Mar2019, 03:50	0.41

50-year Global Summary				
Hydrologic Element	Drainage Area (sq mi)	Peak Discharge (CFS)	Time of Peak	Volume (in)
7B	2.451	247.9	20Mar2019, 02:20	0.87
8B	3.8421	376.9	20Mar2019, 03:45	1.12
6B	1.6706	175.6	20Mar2019, 01:40	0.75
4B	2.1113	375.6	20Mar2019, 01:55	1.23
5B	1.9311	228.3	20Mar2019, 02:25	1.01
10B	1.116	509.7	20Mar2019, 00:40	1.58
2B	1.2249	166.9	20Mar2019, 02:15	1.1
3B	1.2093	109.4	20Mar2019, 02:00	0.73
1B	0.8332	125.5	20Mar2019, 01:25	0.92
7C	6.2931	572.9	20Mar2019, 03:00	1.02
6C	7.9637	668.6	20Mar2019, 03:15	0.96
4C	2.1113	375.6	20Mar2019, 01:55	1.23
10C	5.1584	725.5	20Mar2019, 02:05	1.22
2C	15.5563	1510.7	20Mar2019, 03:00	1.04
1C	16.3895	1568.7	20Mar2019, 03:05	1.04
7R	6.2931	572.1	20Mar2019, 03:25	1.02
6R	7.9637	666.6	20Mar2019, 03:55	0.96
4R	2.1113	374.5	20Mar2019, 02:15	1.23
10R	5.1584	723.3	20Mar2019, 02:35	1.22
2R	15.5563	1508.9	20Mar2019, 03:05	1.04

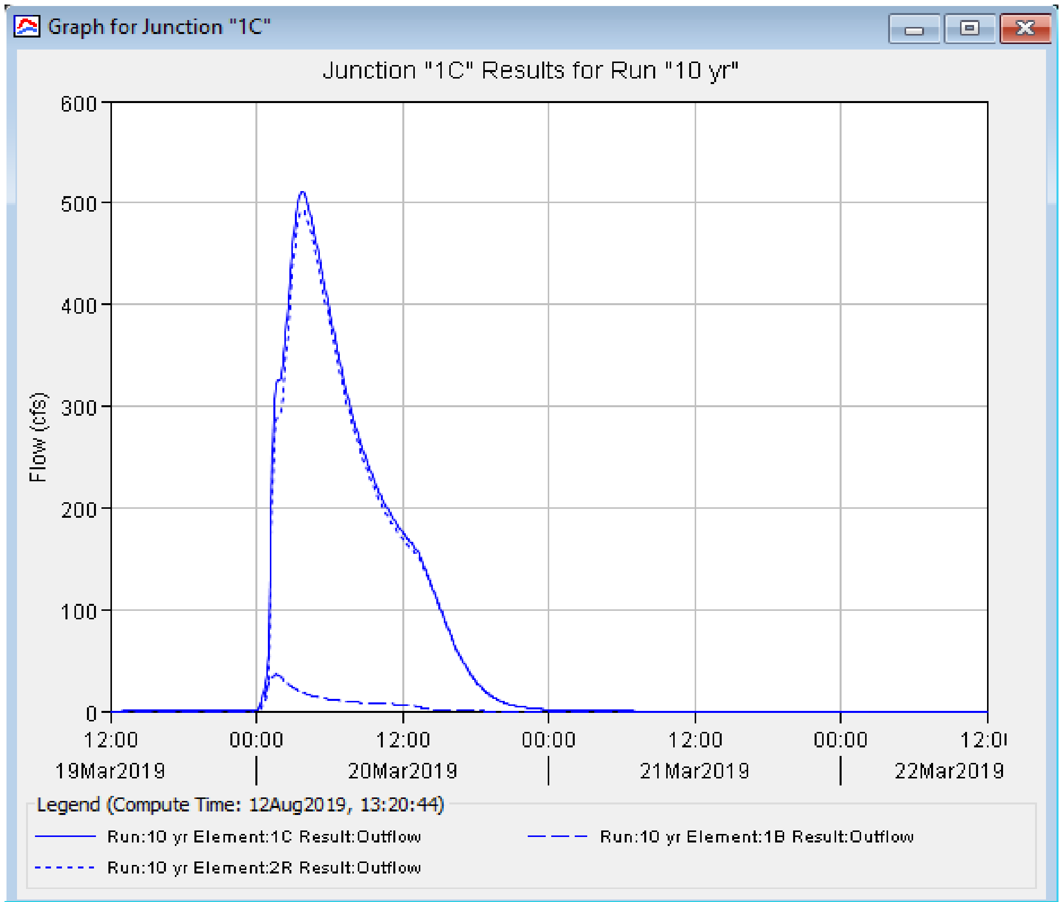
G-21-A

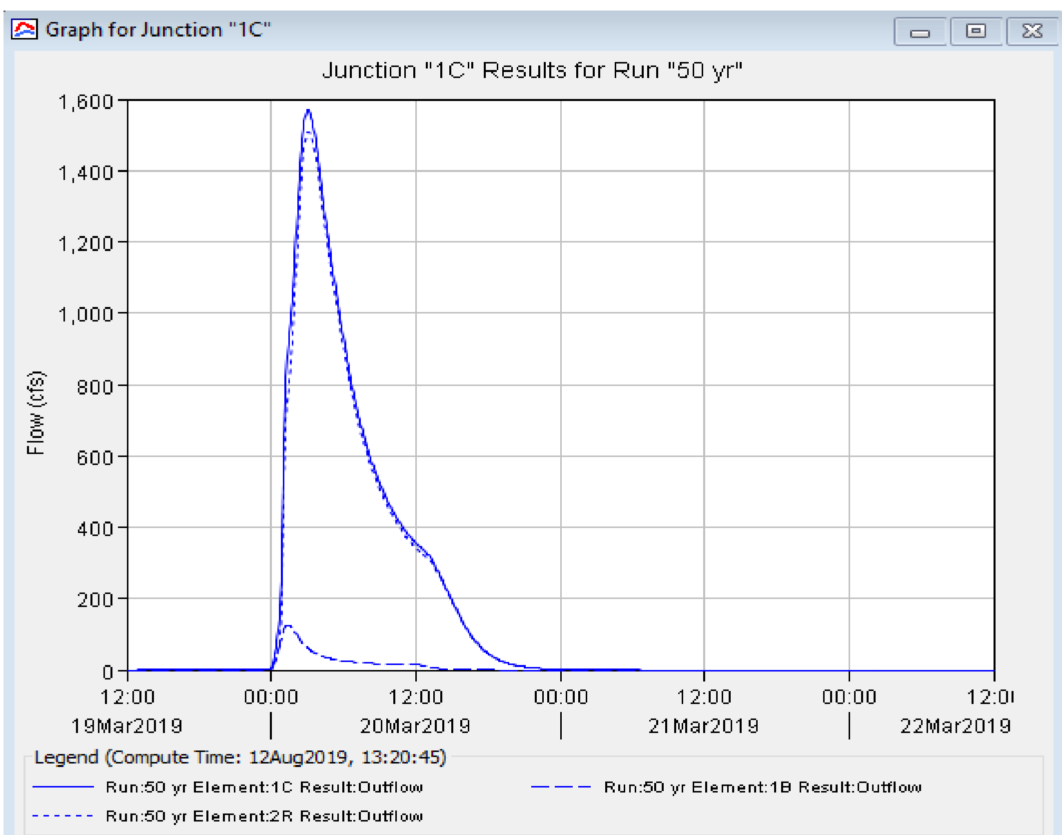
Global Summary

100-year Global Summary				
Hydrologic Element	**Drainage Area (sq mi)**	**Peak Discharge (CFS)**	**Time of Peak**	**Volume (in)**
7B	2.451	365.5	20Mar2019, 02:15	1.21
8B	3.8421	525.3	20Mar2019, 03:45	1.51
6B	1.6706	269.3	20Mar2019, 01:35	1.07
4B	2.1113	519.6	20Mar2019, 01:50	1.64
5B	1.9311	326.5	20Mar2019, 02:25	1.38
10B	1.116	674.3	20Mar2019, 00:40	2.04
2B	1.2249	235.4	20Mar2019, 02:15	1.49
3B	1.2093	167.8	20Mar2019, 01:55	1.05
1B	0.8332	184.6	20Mar2019, 01:25	1.28
7C	6.2931	814.8	20Mar2019, 03:00	1.4
6C	7.9637	956.4	20Mar2019, 03:10	1.33
4C	2.1113	519.6	20Mar2019, 01:50	1.64
10C	5.1584	1005.4	20Mar2019, 02:05	1.63
2C	15.5563	2167.5	20Mar2019, 02:50	1.42
1C	16.3895	2253.5	20Mar2019, 02:55	1.41
7R	6.2931	813.6	20Mar2019, 03:20	1.4
6R	7.9637	953.9	20Mar2019, 03:40	1.33
4R	2.1113	517.5	20Mar2019, 02:10	1.64
10R	5.1584	1002.3	20Mar2019, 02:30	1.63
2R	15.5563	2164.5	20Mar2019, 02:55	1.42

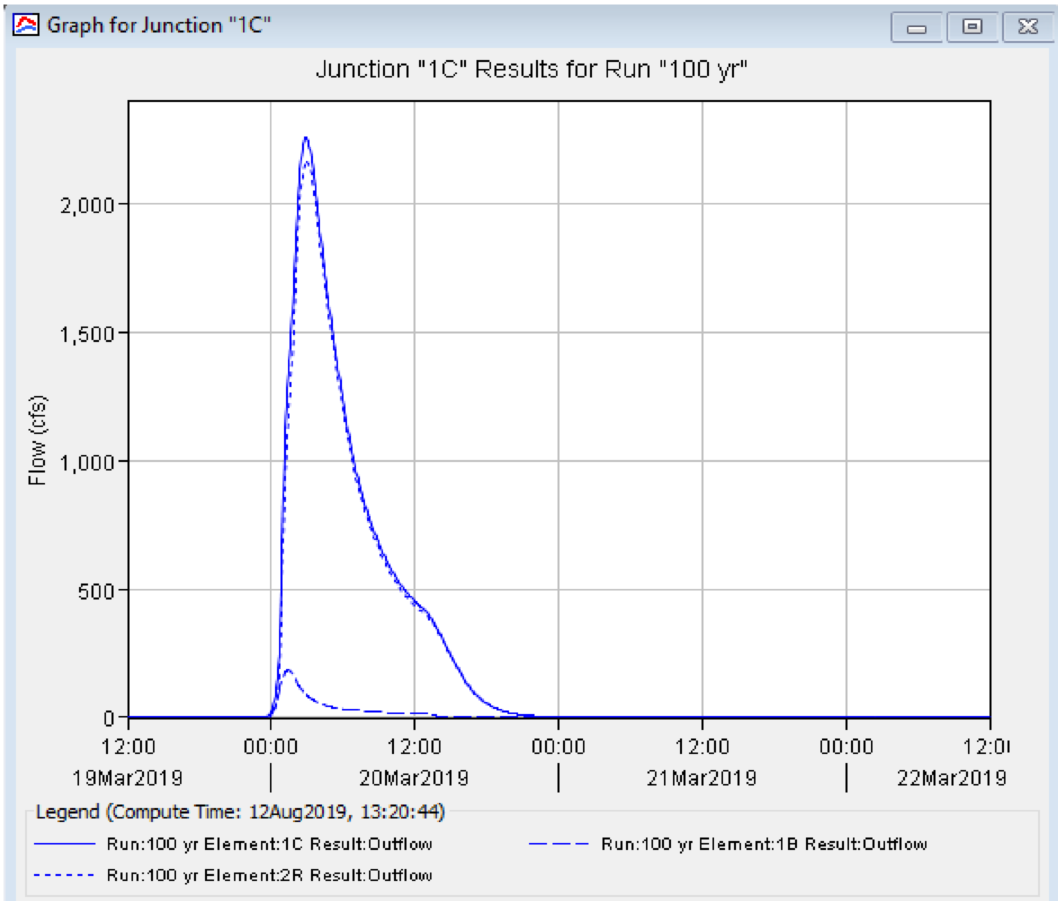
500-year Global Summary				
Hydrologic Element	**Drainage Area (sq mi)**	**Peak Discharge (CFS)**	**Time of Peak**	**Volume (in)**
7B	2.451	734.8	20Mar2019, 02:10	2.26
8B	3.8421	970.1	20Mar2019, 03:35	2.67
6B	1.6706	573.3	20Mar2019, 01:35	2.05
4B	2.1113	941.2	20Mar2019, 01:50	2.84
5B	1.9311	626.5	20Mar2019, 02:20	2.49
10B	1.116	1135.1	20Mar2019, 00:40	3.36
2B	1.2249	440.4	20Mar2019, 02:10	2.64
3B	1.2093	357.9	20Mar2019, 01:50	2.02
1B	0.8332	367.7	20Mar2019, 01:20	2.35
7C	6.2931	1556.3	20Mar2019, 02:50	2.51
6C	7.9637	1849.1	20Mar2019, 02:55	2.41
4C	2.1113	941.2	20Mar2019, 01:50	2.84
10C	5.1584	1836.5	20Mar2019, 01:55	2.82
2C	15.5563	4203.7	20Mar2019, 02:35	2.53
1C	16.3895	4380.8	20Mar2019, 02:40	2.52
7R	6.2931	1553.5	20Mar2019, 03:10	2.51
6R	7.9637	1845.5	20Mar2019, 03:20	2.41
4R	2.1113	938.1	20Mar2019, 02:05	2.84
10R	5.1584	1830.6	20Mar2019, 02:20	2.82
2R	15.5563	4196.9	20Mar2019, 02:40	2.53

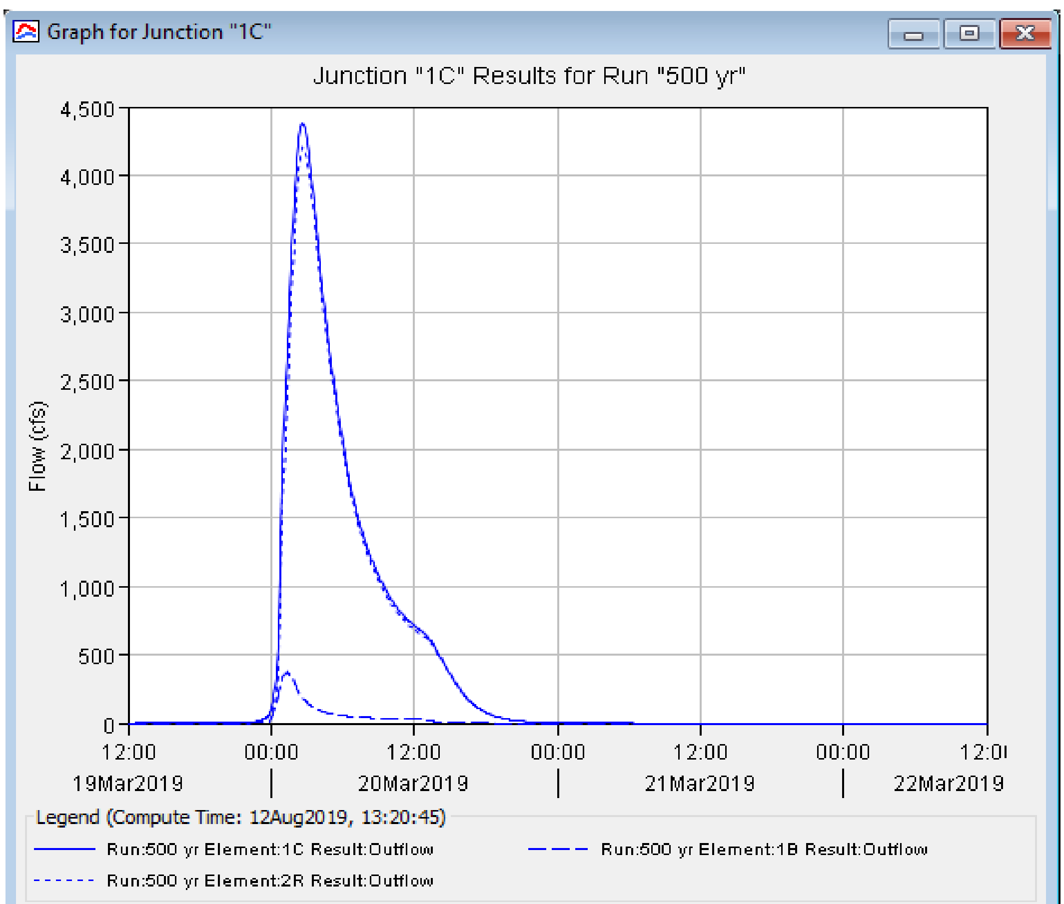
G-21-A
Flood Hydrographs





G-21-A
Flood Hydrographs





Appendix D

Geomorphic Assessment Memo

MEMO

TO:	Anthony Alvarado, PE (Ayres Associates)
FROM:	William Spitz, PG
RE:	Non-BE Bridge G-21-A (MP 361.530) Geomorphic Assessment
DATE:	August 30, 2019
PROJECT #:	017-1690
CDOT PROJECT #:	20252
CDOT TO #:	23

Geomorphic Assessment of Stream Stability
Non-BE Bridge G-21-A (US36 MP361.530) over Agate Creek near Agate, CO

The following memo describes the geomorphic assessment of the stability of Agate Creek and the US36 (SH40) crossing of the creek by the non-Bridge Enterprise (BE) Bridge G-21-A, which is located in Arapahoe County about 1 mile north of Agate, Colorado at Mile Post 361.530 (**Figure 1**). Agate Creek 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 Agate Creek with East Bijou Creek is located about 1 mile north of the bridge site. The bridge site and US36 are bound by I-70 to the east and railroad tracks/embankment to the west. Upstream of the bridge, Agate Creek is bound by I-70 and US36, and downstream of the bridge the creek 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 stream. The local geology and soils provide insight into local topographic controls and the characteristics and caliber of sediment delivered to and transported by the creek.

1880 Fall River Drive, Suite 200 / Loveland, CO 80538
O 970.461.7733 / **olsson.com**

Figure 1. Location of US36 Bridge G-21-A over Agate Creek near Agate, 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 Agate Creek 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 Baca loam (approx. 35% of the area), Weld loam (approx. 35% of the area), and the Renohill complex (approx. 11% of the area).

The Baca loam, 5 to 15 percent slopes, comprises about one third of the watershed soils. Areas with this soil unit are not prime farmland. It is located on hills and the parent material is loess. The unit is well drained, is in the medium runoff class, and belongs to the Hydrologic Soil Group C.

The Weld loam, 1 to 3 percent slopes and 3 to 5 percent slopes, also comprise about one third of the watershed soils. These soils are prime farmland when irrigated. They are found in interfluves and the parent material is calcareous loess. The units are well drained, are in the medium runoff class, and belong to the Hydrologic Soil Group D.

The Renohill complex, 3 to 15 percent slopes, eroded, comprise about 10% of the watershed soils. Areas with this soil unit are not prime farmland. It is located on hills and the parent material is residuum weathered from shale. The unit is well drained, is in the medium runoff class, and belongs to the Hydrologic Soil Group C.

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 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 C and 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 69% silt and clay (< 0.074 mm) and 25% sand (≤ 2.0 mm). The Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI) of 39, 18, and 21, respectively, indicating that the sample is a lean clay (CL).

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 16.4 mi^2 (Figure 2). The mean annual precipitation is about 16.2 inches. He maximum 6-hour, 2-year recurrence precipitation is estimated to be 1.47 inches and the maximum 24-hr, 100-year recurrence precipitation is estimated to be 4.78 inches. The estimated peak flow statistics for Agate Creek at Bridge G-21-A are provided in **Table 1**.

CDOT also developed hydrology for the bridge site using HEC-HMS. The drainage area above the bridge site as delineated by CDOT is also the same as StreamStats at approximately 16.4 mi^2. The peak discharge estimated by CDOT, which are also shown in Table 1, are less than half of those estimated by StreamStats.

Table 1. Estimated Peak Flows for Agate Creek at Bridge G-21-A.

Recurrence	StreamStats Peak Flow (cfs)	CDOT Peak Flows (cfs)
2-yr	198	---
5-yr	655	---
10-yr	1,190	530
25-yr	2,240	---
50-yr	3,350	1,550
100-yr	4,830	2,205
500-yr	9,960	4,220

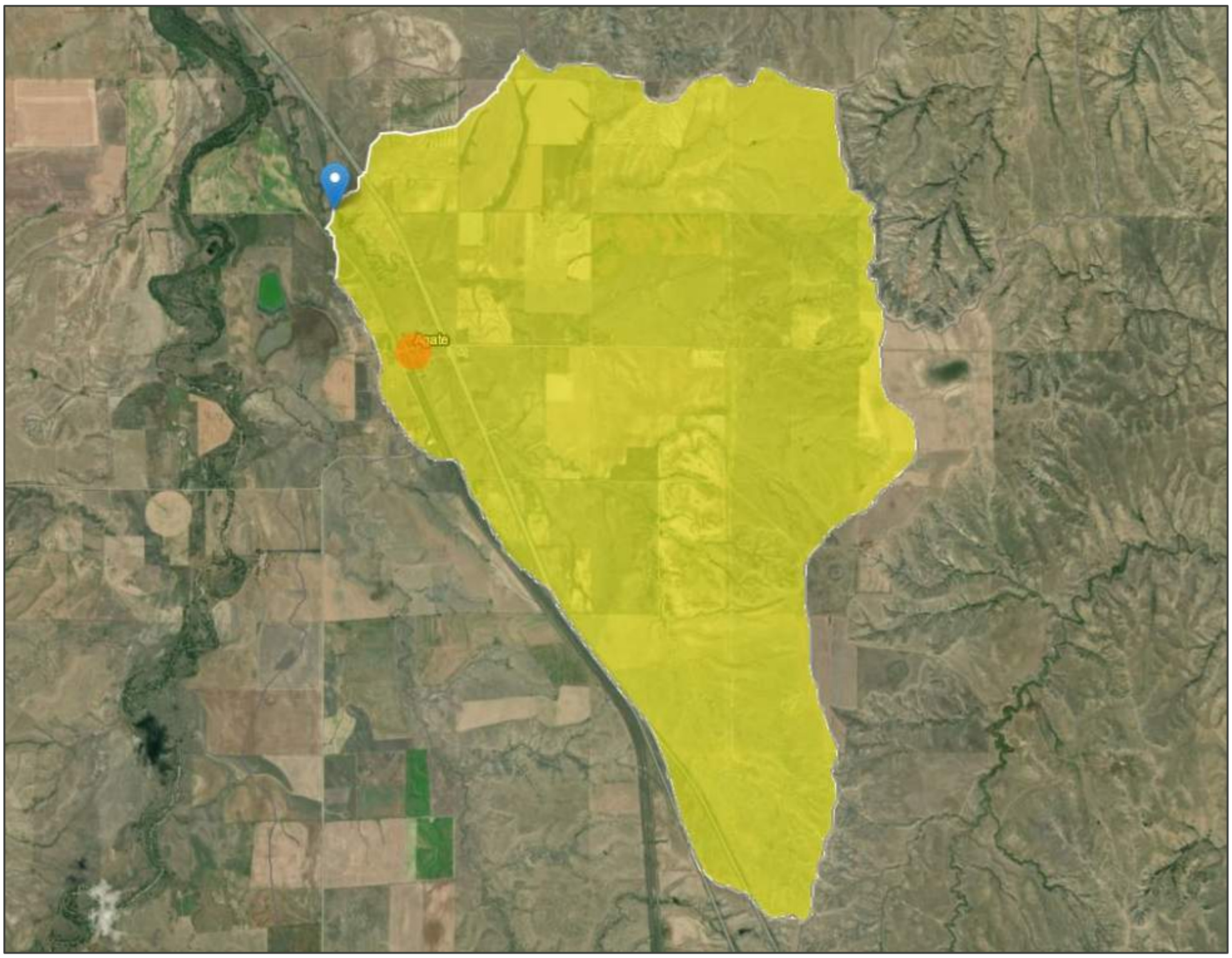


Figure 2. Drainage area for Agate Creek at US36 Bridge G-21-A.

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 the highly sinuous channel segment upstream and the generally straighter channel segment downstream of the bridge have changed relatively little since 1953. Almost all of the mature trees seen along the banks of the upstream channel segment in the 1953 aerial photo have remained in place during this time period. One feature within the channel near the bridge that has remained intact since 1953 is an earthen berm across the channel about 60 feet upstream of the bridge. The berm appears to have been constructed to impound flows in order to create a livestock pond in the upstream channel.

Downstream of the bridge about a half mile to the north, the old highway embankment cut off a series of meander bends as seen in the 1953 aerial. However, these cutoffs do not appear to have affected the vertical stability of the creek between the cut off bends and Bridge G-21-A. The old highway embankment is still visible on the east side of the railroad tracks on the west side of the tight meander bend just downstream of Bridge G-21-A.

Site Visit and Assessment

Bridge G-21-A, which is 95 feet wide, consists of 2 steeply sloping concrete spill-through abutments with concrete wingwalls. The bridge sits on 3 pier bents each of which contains 6 wood piers sitting on a single wood cross beam resting on square concrete footers located under each pier. Wood cross braces are present on each pier bent. The pier bents are skewed about 10° to the channel alignment. **Figure 3** shows the configuration of the bridge.

Figure 3. View looking downstream at US36 Bridge G-21-A over Agate Creek.

Besides the impounded area upstream of the bridge, the area under the bridge also appears to be used by livestock for shade and to transit between the upstream and downstream areas. Thus, the area under the bridge has become well trampled, forming a wide, shallow depression (see Figure 3) that has the appearance of a scour hole, which it is not. High water stains on the left abutment and the concrete pier footings suggest that water levels rarely get above the top of the footings.

A roadside ditch along the east side of the highway upstream (south) of the bridge could potentially be a conduit for flood flows that break out of Agate Creek where it comes in close proximity to the highway embankment about 500 feet to the south (**Figure 4**). The ditch could potentially accommodate flood flows and develop into an avulsion if flood flows are sufficiently significant. If this were to occur, the left bridge abutment could be threatened by scour and undermining. Survey data and hydraulic modeling should be able to determine if this is possible.

Figure 4. View of US34 bridge G-21-A across Agate Creek and upstream channel and ditch.

A short, discontinuous channel on the north side of the bridge that intersects the main channel about 200 feet downstream of the bridge appears to be a relic of the original channel of Agate Creek prior to the construction of the current US36 alignment. This relic channel may be the downstream remnant of a pair of small meander bends that might have been located in the path of the current US36 alignment. The bends were likely cut off and filled in when the current embankment for US36 was constructed and the current channel alignment was probably constructed as part of the meander cutoffs for a better alignment with the downstream channel. The original highway embankment is still visible west of the current highway and bridge as seen in Figure 4.

The channel upstream and downstream of the bridge is well vegetated with grasses and weeds, indicating that flows in the creek are relatively infrequent. Mature trees lining the banks of the creek and growing along the margins of the channel bottom would also suggest general channel stability and relatively infrequent flows.

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. The potential for flood flows occupying the upstream ditch and potentially resulting in an avulsion is the only potential threat to the bridge that was identified. However, without further analysis, it is unknown if this condition is possible.

The berm across the channel immediately upstream of the bridge is not currently a threat and the loss of the berm, either through erosion or mechanical removal, is not likely to be a direct hazard to the bridge. However, if the loss of the berm were to induce upstream degradation, the potential failure of some of the trees along the upstream channel could result in the delivery of some woody debris to the bridge.