PRELIMINARY HYDRAULICS REPORT
STRUCTURE M-21-C REPLACEMENT

As a part of the
REGION TWO BRIDGE BUNDLE PACKAGE
OTERO COUNTY, COLORADO

A Part of Section 30, Township 26 South, Range 57 West of the 6th P.M.,
County of Otero, Colorado

February 5, 2021

Prepared for:

COLORADO
Department of Transportation

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1. INTRODUCTION

1.1 Background and Purpose

The objective of Colorado Department of Transportation (CDOT) Region 2 Bridge Bundle Design Build project is to replace nineteen (19) rural structures spread across highway corridors in southern and western Colorado. The structures are located on US 350, US 24, CO 9, and CO 239. The role of Stanley Consultants is to assist CDOT in the design build procurement, geotechnical engineering, environmental clearances, survey, utility location and coordination, hydrology and hydraulics, preliminary structural design and roadway design.

This design build project is partially funded by the USDOT FHWA Competitive Highway Bridge Program grant (14 structures, project number 23558) and funds from the Colorado Bridge Enterprise (5 additional structures, project number 23559). These projects are combined to form one design-build project.

The nineteen bridges identified to be included in the 'Region 2 Bridge Bundle' were selected based on similarities in the bridge conditions, risk factors, site characteristics, and probable replacement type, with the goal of achieving economy of scale. Seventeen of the bridges being replaced are at least 80 years old. Five of the bridges are Load Restricted limiting trucking routes through major sections of the US 24 and US 350 corridors. The bundle is comprised of nine timber bridges, four concrete box culverts, one corrugated metal pipe (CMP), four concrete I-beam bridges, and one I-beam bridge with corrugated metal deck.

1.2 Site Description

The purpose of this report is to document the preliminary hydraulic analysis and design for the replacement of Structure M-21-C as a part of the CDOT Region 2 Bridge Bundle Design Build. The project is located within Otero County at Mile Post 50.582 along US 350 between Trinidad and La Junta. Structure M-21-C crosses over the Hoe Ranch Arroyo. Figure 1 below illustrates the project location. The project is in Section 30, Township 26 South, Range 57 West of the 6th P.M., County of Otero, Colorado. Figure 1 shows the project limits.

The report will document preliminary hydrology, hydraulic, and scour analysis to support the proposed structure replacement design.

The Federal Emergency Management Agency (FEMA) has designated the project site as a FEMA Zone A, as determined by the Flood Insurance Rate Maps (FIRM) 0801320225B effective date August 19, 1985, as shown in Appendix A. FEMA Zone A is a special flood hazard area inundated by the 100-year flood, however base flood elevations are not determined in a Zone A designation. 44 Code of Federal Regulations (CFR) 60.3 (b) state that for Zone A floodplains, all cumulative impacts to the system from the time of the original study cannot result in a water surface elevation (WSE) increase of more than one foot. This report also reviews changes to the WSE due to the proposed alternatives. The goal for this preliminary analysis is to provide viable options for the design build contractor to achieve a no-rise condition for replacement structures within Zone A floodplains. The Otero County floodplain administrator has indicated that a no-rise certification will be necessary to obtain a floodplain development permit from the county. If a no-rise condition is not met, the contractor will be required to complete the Letter of Map Change (LOMC) process through FEMA.

Figure 1: Vicinity Map

2. HYDROLOGY

Preliminary hydrology for the watershed tributary to this structure was provided by CDOT. A memorandum provided by CDOT summarizes basin areas, runoff methodology and approximate flowrates determined by the preliminary analysis. Table 1 is a summary of the approximate flowrates provided by CDOT of structure M-21-C.

Table 1: Summary of Peak Discharge for Bridge M-21-C

River Location	Design Storm	100-year (cfs)	200-year (cfs)	500-year (cfs)
Upstream of Bridge	100-year	4,359	5,343	6,782

3. EXISTING CONDITIONS

3.1 Existing Structure

Existing structure is a three-span concrete deck, steel I beam girder, bridge built in 1937 to span Hoe Ranch Arroyo. The bridge is on a 60-degree skew. The existing bridge consist of three 40'-0" spans (bearing to bearing), with a total length of 126'-0" out to out of abutments. The width of the existing bridge is 30'-0" curb to curb, 33'-6" out to out of deck. The existing vertical clearance is approximately 15'-0".

Deck drains exist along the edges of the roadway that allow runoff to be collected on the bridge deck and fall directly onto the stream bed below. These drains are 3" pipes that are flush with top of the bridge and outlet at an angle through the girders. No utilities were found attached to the bridge.

It is located on US 350, southwest of La Junta, at milepost 50.582.

3.2 Watershed Overview

The Hoe Ranch Arroyo is a dry arroyo that flows from the southeast to the northwest toward Timpas Creek. The watershed tributary to Hoe Ranch Arroyo is approximately 21.8 square miles in area. The watershed generally slopes to the north. The stream bed does not have a base flow.

The stream flows at an angle to the current structure with an approximate angle of attack of 60 degrees. The area surrounding the bridge is rural with undeveloped land to both upstream and downstream sides of the bridge.

3.3 Site Investigation

A site investigation by Stanley Consultants in August 2020 was performed to gain an understanding of the key hydraulic and geomorphic features of the stream at the project site and of the overall watershed. This investigation found obvious scour damage to the base of the northwest abutment, and at the north pier columns as the footing of the abutment wall and pier columns are both exposed. Site photos are included in Appendix C.

4. HYDRAULIC ANALYSIS

A two-dimensional (2D) hydraulic model was developed using the Sediment and River Hydraulics 2D model (SRH-2D) software developed by the United States Bureau of Reclamation in 2008. A 2D model was chosen to represent this area due to the complexity of the stream and for the preliminary scour countermeasure design. The Surface Water Modeling System (SMS) was used to develop the inputs for the SRH-2D Version 13.0 model, as well as post-process the results. For this analysis, three models were developed:

- Existing Conditions
- Proposed Conditions: Box Culvert Replacement
- Proposed Conditions: Bridge Replacement

4.1 Debris Potential

The potential for debris production and delivery is estimated to be low (minimal) based on guidance from Federal Highway Administration (FHWA) Hydraulic Engineering Circular (HEC) No. 20. The flowchart for potential debris production is presented in Figure 2. The channel banks near the bridge are vegetated with tall grasses and shrubs, and no trees present, as confirmed with the site visit in August 2020. Aerial imagery of the watershed near the bridge is shown in Appendix B.

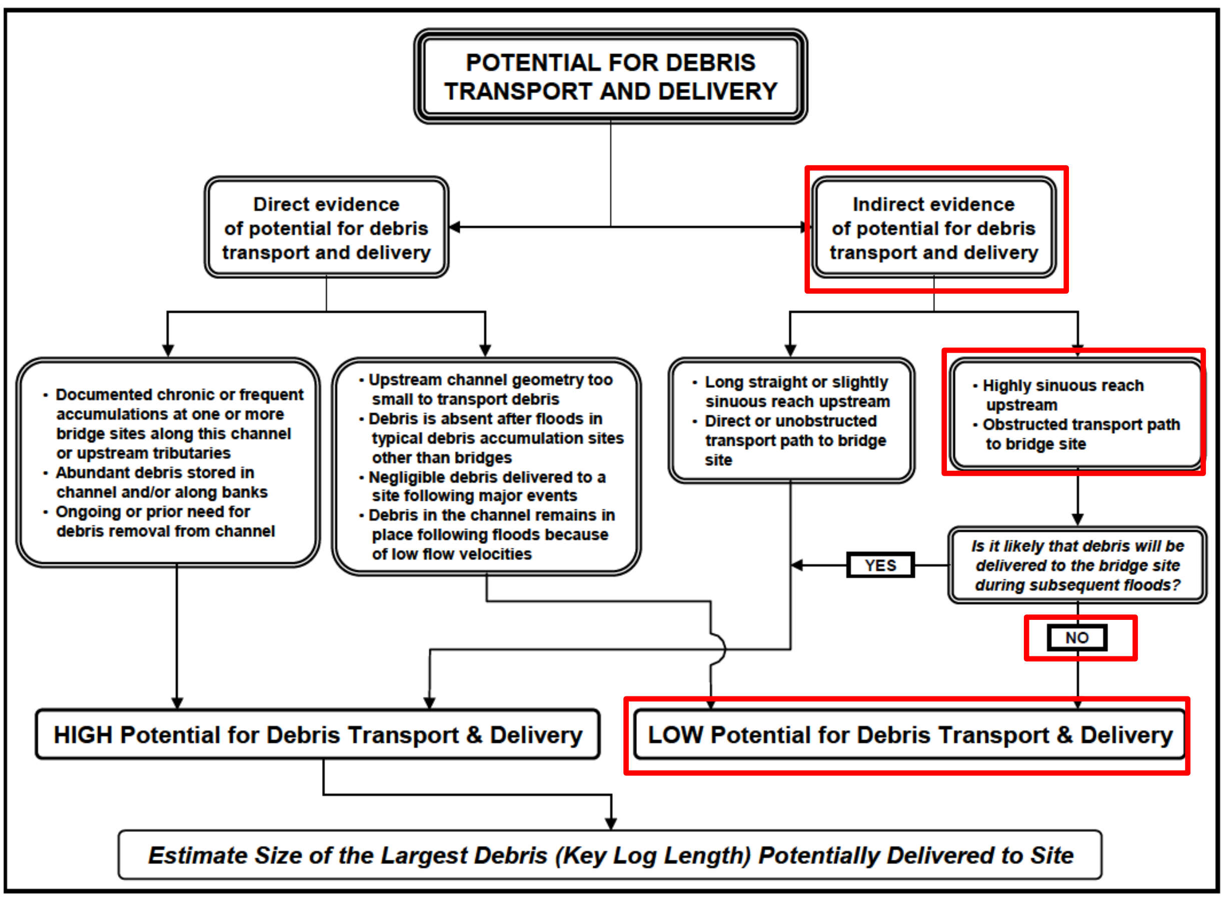


Figure 2: Flow Chart for Potential Debris Production (FHWA, HEC 20)

4.2 Freeboard

The CDOT Drainage Design Manual (2019) specifies freeboard requirements for all bridges. Freeboard is the minimum clearance between the design approach WSE and the low chord of the bridge. It is a factor of safety that acts as a buffer to account for unknown factors that could increase the height of the calculated WSE. Streams classified as high debris streams shall have a minimum of 4 feet of freeboard. Low-to-moderate streams CDOT highly encourages 2 feet be provided, where practical. The elevation of the water surface 50 to 100 feet upstream of the face of the bridge shall be the elevation to which the freeboard is added to get the bottom or low-girder elevation of the bridge.

The channel was not identified as having a high potential for debris production. Therefore, 2 feet of freeboard is required, if a bridge is selected for the proposed conveyance structure.

4.3 Modeling Parameters

4.3.1 Elevation Data

Existing conditions survey for the bridge and channel cross sections was performed by CDOT in June 2020. LiDAR was acquired by CDOT in June 2020. These two data sources were combined for the modeling elevation surface.

A local, custom projection was used for the data collection in the existing conditions survey. The survey was converted into NAD 1983 Colorado State Plane South US Survey Feet for the hydraulic modeling. All elevations are referenced to NAVD 88 (feet).

4.3.2 Computational Mesh

The computational mesh is an unstructured mesh, which allows for the use of triangles and quadrilaterals, with variable element sizes. Roadways and the channel were modelled with a patch mesh, which uses quadrilaterals. The faces of the quadrilaterals are lined up perpendicular to flow and allow for a more precise modelling of the conveyance structure. Triangles were typically used in the floodplain and the areas upstream and downstream of the highway crossing. The total number of mesh elements is 5,032 and the mesh extends approximately 1,350 feet upstream of the bridge and 630 feet downstream of the bridge.

4.3.3 Surface Roughness

Surface roughness, represented by the Manning's roughness coefficient, is presented in Table 2. A Manning's n-value was assigned to each land use based on aerial imagery, topography, a site visit in August 2020 and engineering judgment. Photos from the site visit used to confirm the n-values selected are shown in Appendix B. A map showing existing conditions materials coverages is shown in Appendix C.

Table 2: Manning's n-values

Land Use	n-value
Channel	0.035
Overbank	0.050
Railroad	0.025
Open Space	0.040
Paved Road	0.016

4.3.4 Boundary Conditions

The boundary conditions include a steady state inflow and a normal depth calculated outflow.

The peak flows developed in Table 1 were used to develop a steady-state inflow boundary condition. The inflow boundary condition extends the full length of the inundation boundary in the upstream portion of the project location. The model was set to a dry initial condition.

For the downstream boundary condition, the subcritical outflow option was selected. This outflow condition uses the inputs of anticipated flow, Manning's n-value, channel slope, and terrain data to determine the outflow constant water surface elevation. Table 3 presents the boundary condition values.

Table 3: Model Boundary Condition Inputs

Frequency Storm	Inflow (cfs)	Outflow Constant WSE (ft)
100-Year	4,359	4557.71

4.3.5 Hydraulic Structures

The modeled existing bridge geometry is based on the survey completed in August 2020. The survey data included shots detailing the bridge, including the existing pier locations. The high chord of the bridge is 4576.90 feet, at the grade center, while the low chord is 4573.55 feet. The bridge was modeled as overtopping which allows flow to overtop the bridge if the water surface elevation reaches an elevation greater than the high chord of the bridge.

The existing bridge piers were modeled as holes, across the width of the bridge, in the computational mesh, allowing flow to run around the piers which replicated true hydraulic conditions.

4.3.6 Simulation Control

The hydraulic simulations are run with a 1.0 second time step for 3 hours until a steady state solution is met. The parabolic turbulence method is used with a coefficient of 0.7.

4.4 Model Results

4.4.1 Existing Conditions

The range of depths experienced in the channel at the bridge during the 100-year event is from 3.7 feet to 9.5 feet. Figure 5 presents the depth for the entire floodplain and the bridge. The results demonstrate that the existing bridge does not overtop during the 100-year event. The results show that flows pond behind the embankment. The 100-year depth for the existing conditions are shown in Appendix C.

4.4.2 Alternatives Analysis

An alternatives analysis was completed in the preliminary design process to determine the most feasible options for the hydraulic conveyance structure. Both a reinforced concrete box culvert (RCBC) and bridge option were analyzed. Many factors were taken into consideration when determining the preferred alternative for this preliminary analysis. These factors include cost, constructability, effects on the stream hydraulics, environmental impacts, etc.

Proposed RCBC

This option was modeled using the same SRH-2D model as was used for the existing conditions. Modifications to the model included adjusting the mesh for the culvert and grading upstream and downstream to allow for the conveyance of flow. To model the culvert, holes in the mesh were used to model the outer and inner walls and the skew. This allows for it to be modelled as a 2D culvert, since the boundary conditions and materials coverage are unaffected. This method was chosen due to the limitations of modelling a culvert in SRH-2D with HY-8. HY-8 assumes the culvert crossing is perpendicular and outlets the flow perpendicular to the boundary condition. Since, the existing road and arroyo intersect at a 60-degree skew, it was determined HY-8 would not correctly model this area. Upstream and downstream grading was done in the mesh to aid in conveyance and the fitting of a culvert in the arroyo. The proposed model has 4,971 mesh elements.

Due to the bridge existing in a floodplain, a similar opening size was used for the box culvert to keep the WSEs the same or lower than existing conditions. The preliminary model shows the roadway embankment sloping at 2:1, and the proposed culvert being 87 feet in length. The RCBC option for this structure required a 5 cell 20-foot wide by 10-foot tall structure. The culvert is assumed to be buried two feet deep to allow for the passage of natural wildlife and to minimize the impact to the ecosystem. This structure size was determined to allow zero rise in the WSEs of the channel. Due to the large size required for a box culvert alternative, this option was not further analyzed in the Structures Selection Report.

Depths and velocity grids for the proposed RCBC show depths from 7 to 8.4 feet and velocities from 5.5 to 9.3 ft/s. See Appendix D for 100-year depths and velocities graphics for this option.

Proposed Bridge

This option was modeled using the same SRH-2D model as was used for the existing conditions. Modifications to the model included adjusting the mesh for a two-span bridge and lengthening the span of the proposed bridge length. The proposed model has 5,051 mesh elements. The proposed model has a two-span concrete deck with a set of piers in the middle. The bridge will match the existing skew and lay on the same grade. The spans are 60’ long from bearing to bearing, with a total length of 121.5 centerline to centerline of the abutments. The low chord of the bridge is at 4574.30' elevation, and the high chord didn't change from the existing condition. Roadway embankments were graded at 2:1.

Depths and velocity grids for the proposed bridge show depths from 4 to 10 feet and velocities from 6.7 to 13.6 ft/s. See Appendix E for 100-year depths and velocities graphics for this option.

5. FEMA FLOODPLAIN ANALYSIS

FEMA has designated the project site as a Zone A, as determined by the FIRM 0801320225B effective date August 19, 1985, as shown in Appendix A.

FEMA Zone A is a special flood hazard area inundated by the 100-year flood; however, base flood elevations are not determined in a Zone A designation. 44 CFR 60.3 (b) states that for Zone A floodplains, all cumulative impacts to the system from the time of the original study cannot result in a WSE increase of more than one foot. A Floodplain Development Permit will be submitted to Otero County during the next phase of design. For this preliminary design, the goal is to demonstrate a no-rise condition, so that a CLOMR is not needed.

Proposed RCBC

Based on modeling results, the proposed RCBC will not increase the WSE by more than 1 foot. Because the opening of the proposed RCBC is about the same as the existing opening, no change in WSE is expected, with a decrease seen immediately upstream and downstream of the RCBC opening.

To perform a comparison between the existing and proposed WSE, nine cross sections were cut across the 2D hydraulic model results upstream and downstream of the proposed bridge. The average WSE was determined for both existing and the proposed RCBC option, as shown in Appendix F.

For the proposed culvert option, upstream of Bridge M-21-C (Cross Sections 1-4), the WSE decreases between 0.37 and 1.39 feet between existing and proposed. Downstream of Bridge M-21-C (Cross Sections 5,6 & 8), the WSE decreases between 0.14 and 0.51 feet between existing and proposed. Also downstream of Bridge M-21-C (Cross Section 8), the WSE increases a maximum of 0.06 feet between existing and proposed. The WSE comparison at these sections is shown in Table 4.

Table 4: WSE Comparison for RCBC Option

Cross Section	Location Relative to Proposed Culvert	Existing WSE (ft)	Proposed WSE (ft)	Proposed vs Existing
1	Upstream	4567.55	4566.58	-0.97
2	Upstream	4567.76	4566.70	-1.06
3	Upstream	4567.56	4566.16	-1.39
4	Upstream	4566.30	4565.93	-0.37
5	Downstream	4566.21	4565.70	-0.51
6	Downstream	4565.19	4565.05	-0.14
7	Downstream	4563.77	4563.83	0.06
8	Downstream	4562.70	4562.32	-0.38

Proposed Bridge

Similarly, the model for the proposed bridge will not increase the WSE by more than 1 foot. The bridge opening for this option is very similar to the existing structure. Therefore, no change in WSE is expected.

For the proposed bridge option, upstream of Bridge M-21-C (Cross Sections 1-4), the WSE decreases between 0.06 and 0.10 feet between existing and proposed. Downstream of Bridge M-21-C (Cross Sections 5 & 8), the WSE decreases between 0.11 and 0.37 between existing and proposed. Also downstream of Bridge M-21-C (Cross Sections 6 & 7), the WSE increases a maximum of 0.06 feet between existing and proposed.

Appendix G shows the cross sections used for the proposed bridge option as well as the floodplain limit changes between existing and proposed for this scenario. Table 5 shows a WSE comparison at each section for the proposed bridge option.

Table 5: WSE Comparison for Bridge Option

Cross Section	Location Relative to Proposed Bridge	Existing WSE (ft)	Proposed WSE (ft)	Proposed vs Existing
1	Upstream	4567.55	4567.49	-0.06
2	Upstream	4567.76	4567.68	-0.08
3	Upstream	4567.56	4567.46	-0.10
4	Upstream	4566.30	4566.23	-0.07
5	Downstream	4566.21	4566.10	-0.11
6	Downstream	4565.19	4565.20	0.01
7	Downstream	4563.77	4563.83	0.06
8	Downstream	4562.70	4562.33	-0.37

6. BRIDGE SCOUR ANALYSIS

6.1 Scour Overview

For the proposed bridge option, as determined in the alternatives analysis, a scour analysis was performed for Hoe Ranch Arroyo at the bridge. The scour analysis is intended to inform the structural design of the crossing and countermeasure design. The FHWA recommends that bridges with complex flow characteristics use a 2D model to represent hydraulic conditions.

For the scour analysis, the FHWA Hydraulic Toolbox Version 5.0 software program was used. The Hydraulic Toolbox program uses equations presented in the FHWA Hydraulic Engineering Circular No. 18 Evaluation of Scour at Bridges (HEC-18) and the National Cooperative Highway Research Program (NCHRP) 24-20. SRH-2D was used as the hydraulic model platform and it has the capability to extract the data needed for these calculations directly from the model.

Based on Table 2.1 from HEC-18 and the conditions of the bridge, the 100-year event is used as the hydraulic design flood frequency, the 200-year event results are used as the scour design flood frequency, and the 500-year results are used as the scour design check flood frequency. However, only 100-year flows are readily available. Therefore, scour was calculated for only the 100-year event for this preliminary analysis. 200-, and 500-year scour analysis and design will be completed in a later phase of the design.

At the project site, the following scour components were calculated:

- Contraction Scour
- Pier Scour
- Abutment Scour
- Long-Term Degradation

All scour calculations can be found in Appendix G.

6.2 Site Geology/Geotechnical Information and Impact to Scour Depths

A geotechnical analysis was completed by Yeh and Associates for the project. Gradation of the stream bed was provided in this investigation and used for this preliminary scour analysis. Only one sample was taken from the channel, therefore this sample will be applied to contraction, pier (local), abutment (local) and long-term degradation scour. Results from the geotechnical investigation are provided in Appendix H.

Borings at each abutment and one at each bridge approach, were also conducted as part of the field exploration. These were used to better understand subsurface conditions at the bridge crossing. Soils information from borings were not used in the scour analysis because boring samples at the abutments were assumed to not be as representative of channel bed conditions as the channel sample discussed above.

Because exact bedrock elevations are not known, no adjustment was made to the scour depths shown below.

6.3 Scour Results

Below, Table 6 summarizes the preliminary results for scour at the bridge over the Hoe Ranch Arroyo.

Table 6: Scour Analysis Results

Scour Type (ft)						
Storm Event	**Contraction**	**Long-Term Degradation**	**Abutment (Local)**	**Pier (Local)**	**Total Abutment Scour***	**Total Pier Scour***
100-Year	0	0.8	7.2	7.5	8.0	8.3
500-Year	0	1.4	11.0	7.6	12.4	9.0

*Contraction Scour is not included in the Total Scour when computing the NCHRP methodology.

6.4 Scour Countermeasures

The proposed bridge foundations will be designed to withstand the effects of scour up to and including the 500-year Scour Design Check Flood Frequency. Scour countermeasures will be designed to protect the approach roadway and bridge embankments from the effects of scour for the 100-year Hydraulic Design Flood Frequency.

This reach of the river has a deep incised main channel with steep, near vertical banks and highly erosive soils. The deep nature of the main channel directly conveys most of the flood flow. There is a tributary downstream of the bridge forming a confluence of the main channel immediately downstream of the bridge. These conditions indicate a significant scour potential at this bridge crossing. Vertical wall abutments with wing walls and riprap are recommended as scour countermeasures. The FHWA Hydraulic Toolbox Version 5.0 (FHWA, 2018) was used to size riprap along the abutments of the proposed structure for the 100-year scour design event. The Hydraulic Toolbox applies methodology outlined in the FHWA Hydraulic Engineering Circular No. 23 Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance (HEC-23) for sizing riprap at abutments based on abutment type, set-back ratio, Froude number, specific gravity of rock riprap, and a characteristic average velocity in the channel. Vertical wall abutments were specified for the abutment type in order to ensure a conservative design.

Results of the Hydraulic Toolbox analysis are provided in Appendix G, and final design values summarized in Table 7. A riprap with D50 of 18-inches (in) (Class 5 per HEC-23) is recommended with a thickness of 2.0 D50 or D100. The resulting recommended thickness is 36-in based on HEC-23 D50 for Class 5. Please refer to Table 506-2 of CDOT's Division 500 Structures Specifications for the recommended gradation of an 18-in riprap.

Riprap should also be placed over a Class 1, non-woven geotextile filter material. According to CDOT's Division 700 Materials Details, geotextile materials should be selected from the New York Department of Transportation's Approved Products List of Geosynthetic materials that meet the National Transportation Product Evaluation Program (NTPEP) and AASHTO M-288 testing requirements. Class 1 geotextiles is the only class approved for applications related to slope protection.

The riprap apron at each abutment should extend from elevation 4568.2 feet (2 feet above the 100-year water surface elevation) down the maximum 2:1 side slope to the channel bottom. 2-ft of freeboard is being proposed for this design, between the 100-year water surface elevation and low chord of the bridge.

The top of the apron should be flush with the existing grade of the channel. Toeing-in the apron down to the total scour depth (elevation 4557.7 feet) is suggested to prevent channel scour undercutting. The upstream and downstream coverage should extend back from the abutment (e.g. perpendicular to the channel) 25 feet to protect the approach embankment.

The riprap slope protection at each wing wall should extend 25' from the end of the wing walls along the roadway embankment and configured with the data shown in Table 5. Riprap placed below existing grade shall be constructed with a maximum 2:1 side slope. Riprap above grade will be placed at the roadway embankment slope and no steeper than 2:1.

Table 7: Riprap Apron Countermeasure Summary

D_{50} (in)	**18**
Recommended Thickness (in)	36
Side Slopes	2:1
Toe Down Depth (ft)	8
Bottom Ref. Elevation (ft)	4551.7
Top Ref. Elevation (ft)	4568.2

7. CONCLUSIONS

This report presents preliminary analysis and results from the hydrologic and hydraulic study for the Region 2 Bridge Bundle Design Build – Bridge M-21-C. This report documents preliminary analysis in determining costs for proposed structure replacement at this location. It also includes preliminary FEMA floodplain analysis and scour analysis.

A two-dimensional model was developed to analyze the flows through the existing bridge and compare the WSEs and velocities to the proposed design. This model was utilized to optimize the proposed solution to replacement of the existing bridge.

Based on the hydraulic analysis, the proposed replacement for this bridge is a 2-span 120-foot bridge. The proposed freeboard is 2 feet and the proposed WSE 100 feet upstream of the proposed bridge is 4566.23 feet, giving a final recommended bridge low chord of 4568.23 feet. The proposed low chord is 4574.3 feet, which exceeds the 2 feet of freeboard that is required.

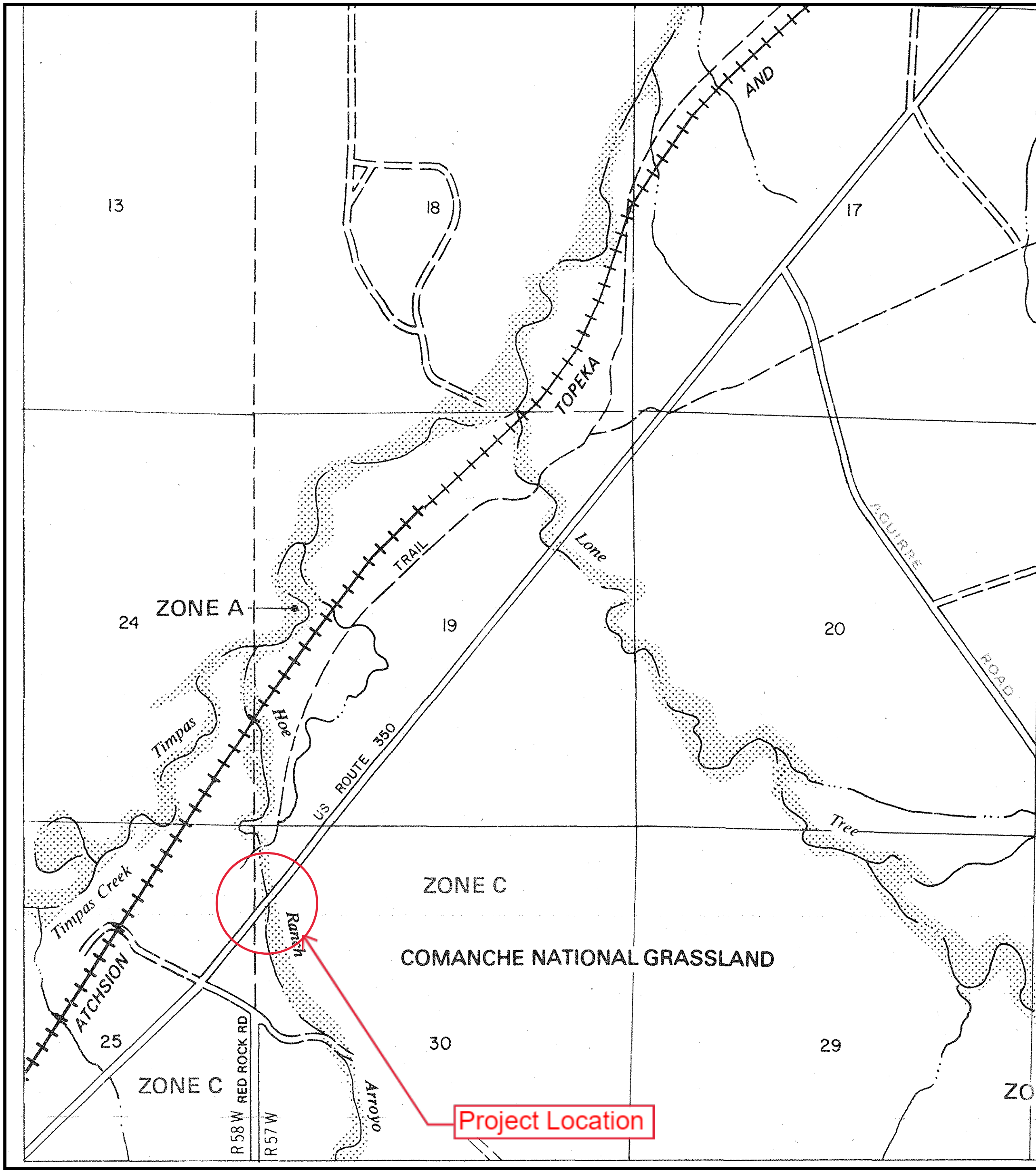
Floodplain analysis demonstrates that the proposed bridge opening will not cause a rise in flood levels during the 100-year design event. This meets guidelines in CFR Sections 60.3 (b). A floodplain development permit is required to be approved through the Otero County floodplain administrator during the final design phase of this Design Build project.

Total design scour for the bridge abutments was determined to be 8 feet at the 100-year design event. This accounts for the long-term degradation impacts that could potentially affect the proposed bridge abutments and pier. A riprap apron was designed in order to protect the proposed abutments.

8. REFERENCES

1. "Colorado Department of Transportation Drainage Design Manual", Colorado Department of Transportation, 2019.
2. Mile High Flood District, Urban Storm Drainage Criteria Manual (USDCM), Volumes I, II, and III, August 2018.
3. "Hydraulic Engineering Circular No. 18 – Evaluating Scour At Bridges Fifth Edition". U.S. Department of Transportation Federal Highway Administration, April 2012.
4. "Hydraulic Engineering Circular No. 20 – Stream Stability at Highway Structures". U.S. Department of Transportation Federal Highway Administration, April 2012.
5. "Hydraulic Engineering Circular No. 23 – Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance – Third Edition," U.S. Department of Transportation, Federal Highway Administration, September 2009.
6. CDOT Region 2 2D Quick Check Hydrology Summary Report and Matrix, Colorado Department of Transportation, 2020.

APPENDIX A FEMA FIRM



APPROXIMATE SCALE IN FEET

2000 0 2000

NATIONAL FLOOD INSURANCE PROGRAM

FIRM
FLOOD INSURANCE RATE MAP

OTERO COUNTY, COLORADO
(UNINCORPORATED AREAS)

PANEL 225 OF 325
(SEE MAP INDEX FOR PANELS NOT PRINTED)

COMMUNITY-PANEL NUMBER
080132 0225 B

EFFECTIVE DATE:
AUGUST 19, 1985

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

APPENDIX B AERIAL IMAGERY AND PHOTOS

AERIAL IMAGERY
STRUCTURE M-21-C
FIGURE 3

PHOTO 1: BRIDGE STRUCTURE SIGN

STRUCTURE M-21-C

APPENDIX A

PHOTO 2: BRIDGE M-21-C EXISTING STRUCTURE

STRUCTURE M-21-C

APPENDIX A

PHOTO 3: LOOKING SOUTH UPSTREAM OF BRIDGE
STRUCTURE M-21-C
APPENDIX A

PHOTO 4: DITCH UNDER THE BRIDGE LOOKING SOUTH

STRUCTURE M-21-C

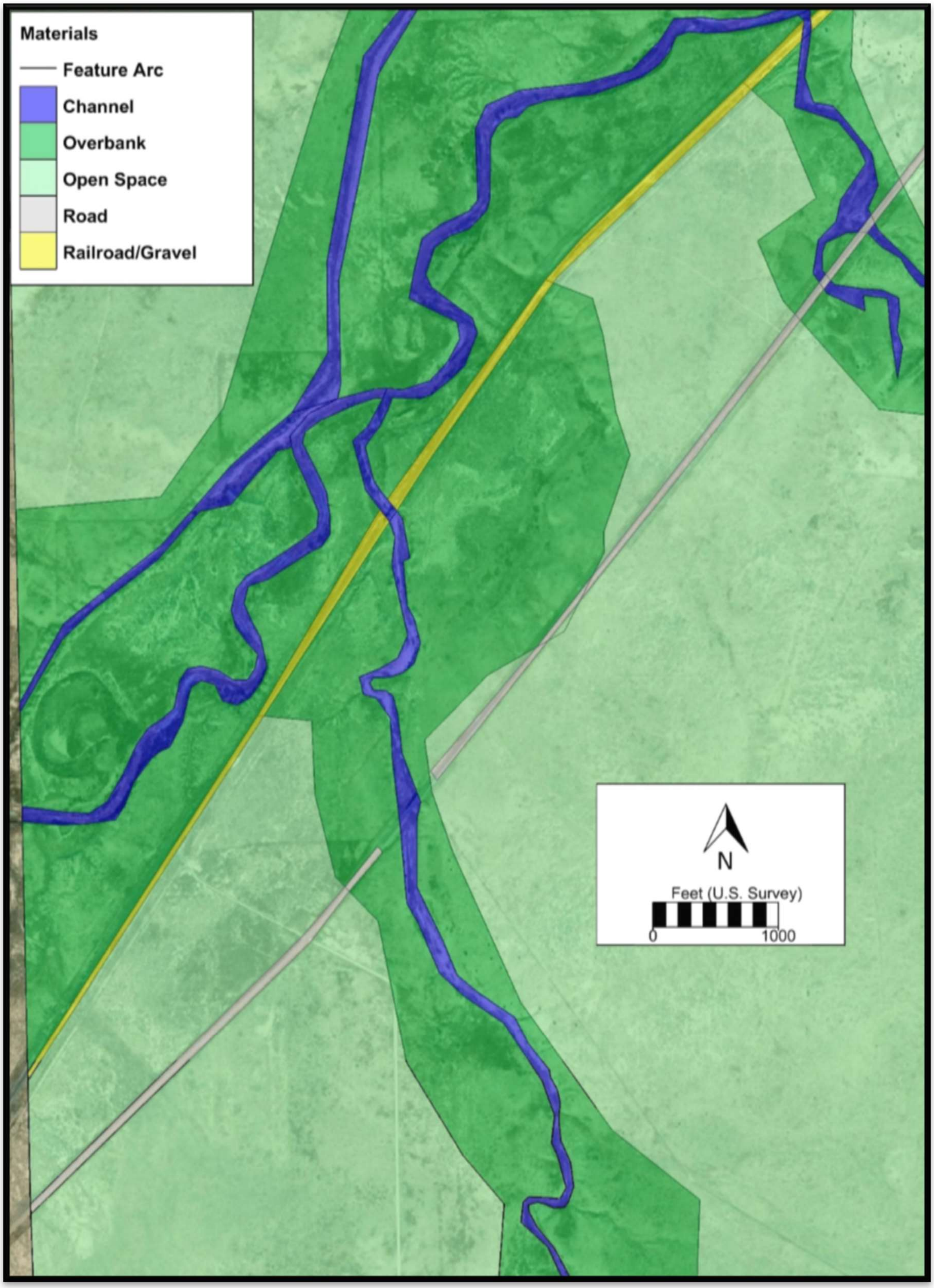
APPENDIX A

PHOTO 5: DOWNSTREAM OF BRIDGE LOOKING NORTH

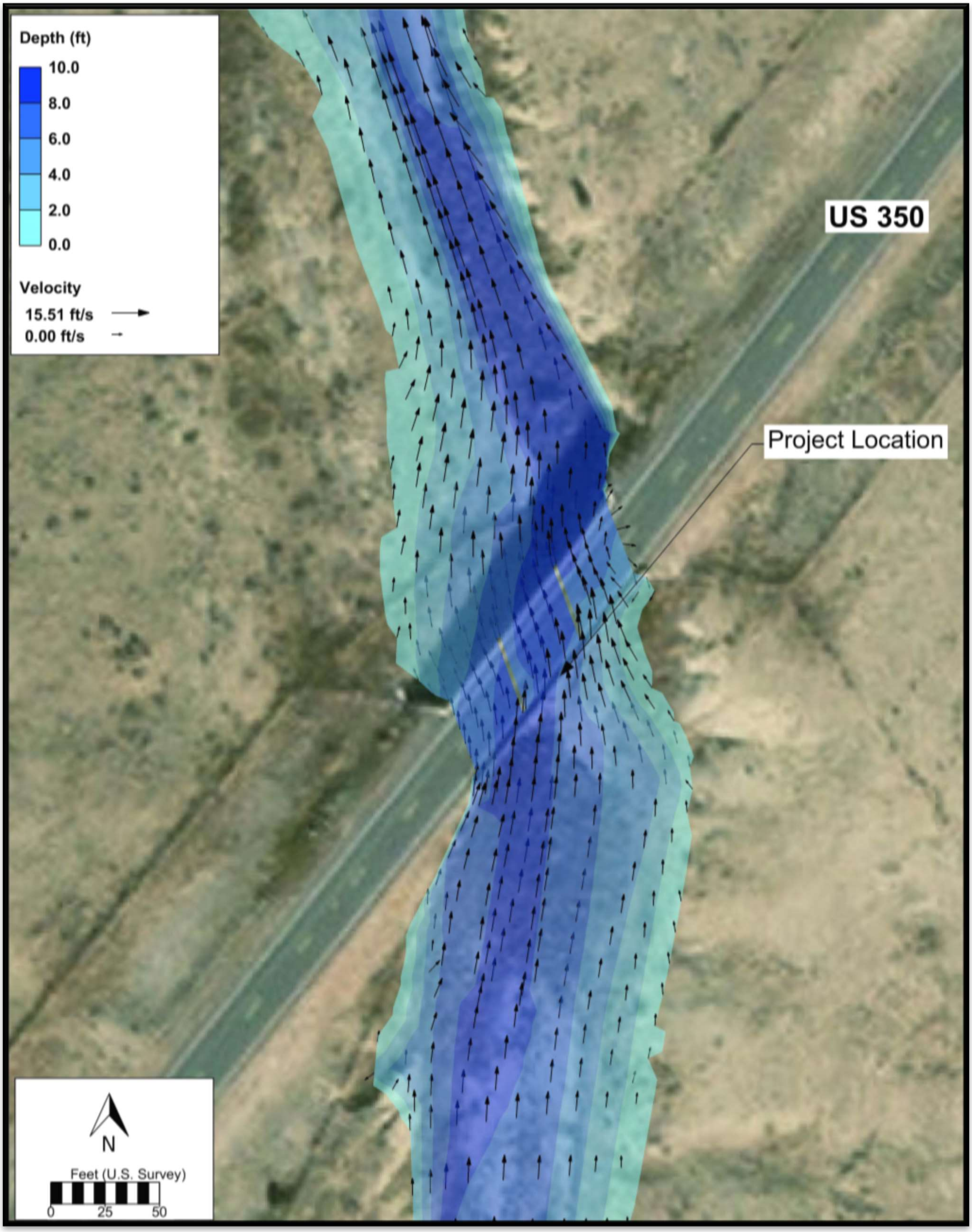
STRUCTURE M-21-C

APPENDIX A

APPENDIX C EXISTING CONDITIONS MODEL GRAPHICS



MATERIALS COVERAGE
STRUCTURE M-21-C
FIGURE 4

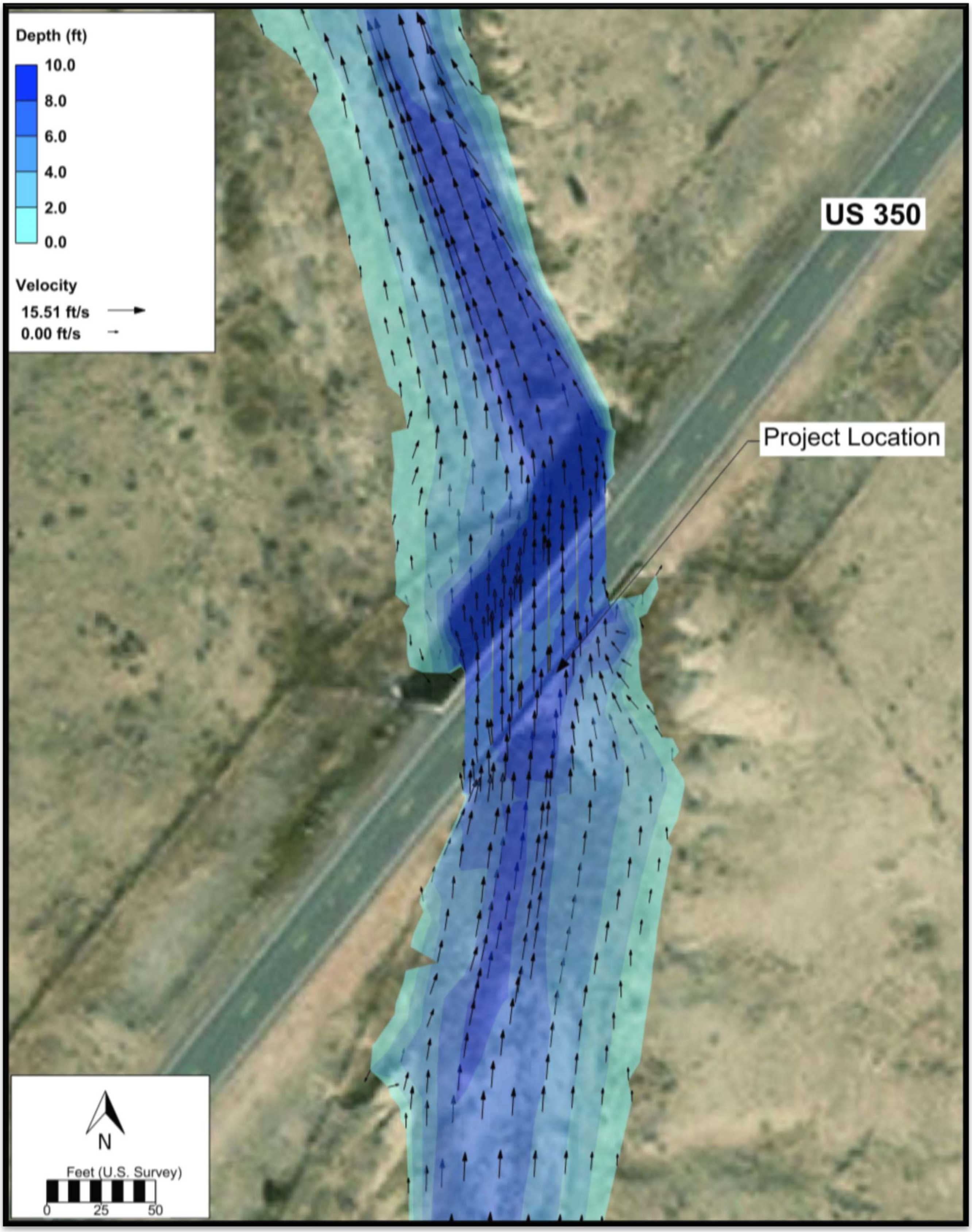


EXISTING CONDITIONS 100-YEAR DEPTH RESULTS

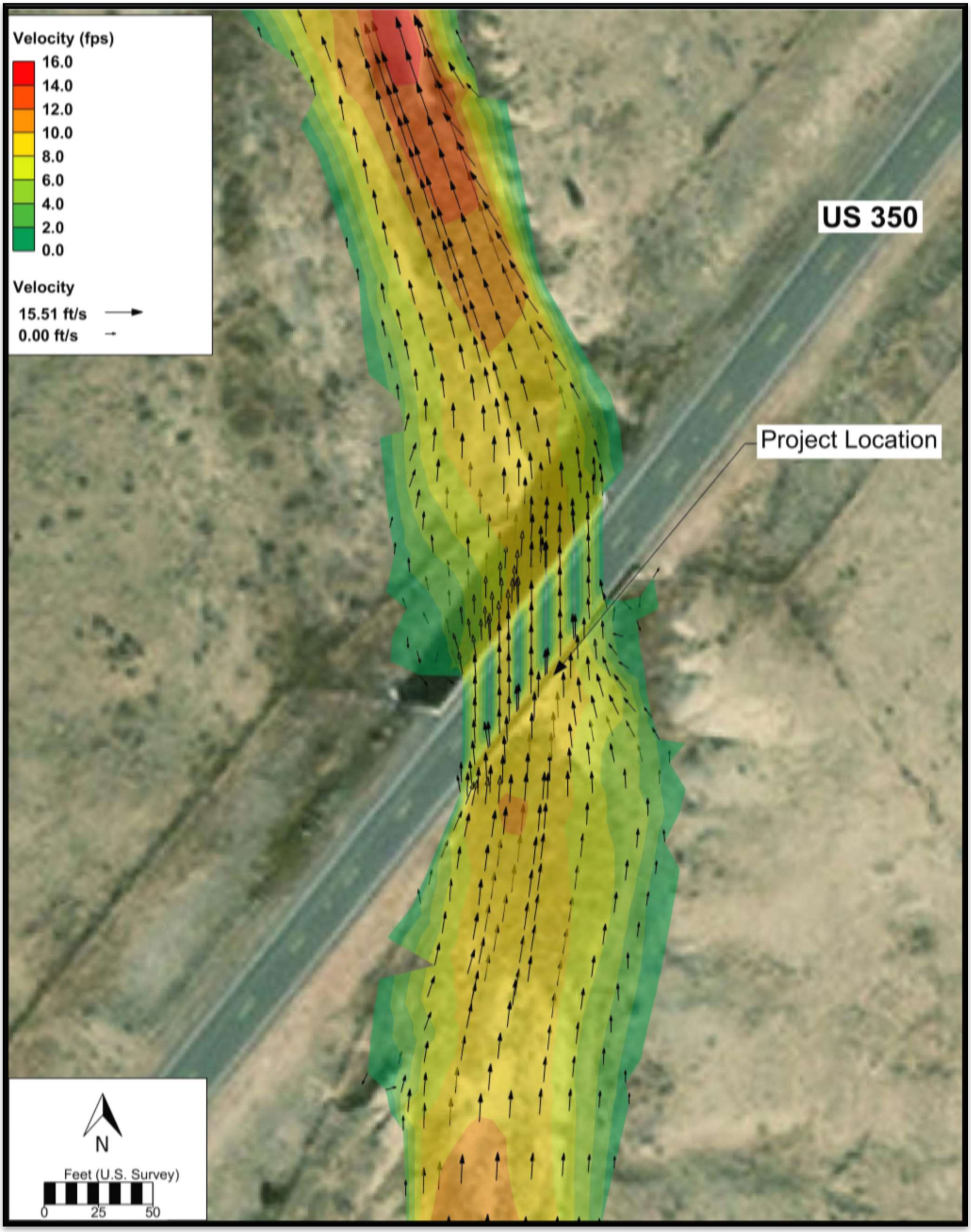
STRUCTURE M-21-C

FIGURE 5

APPENDIX D PROPOSED RCBC ALTERNATIVE MODEL GRAPHICS



PROPOSED 100-YEAR DEPTH RESULTS – RCBC OPTION
STRUCTURE M-21-C
FIGURE 6

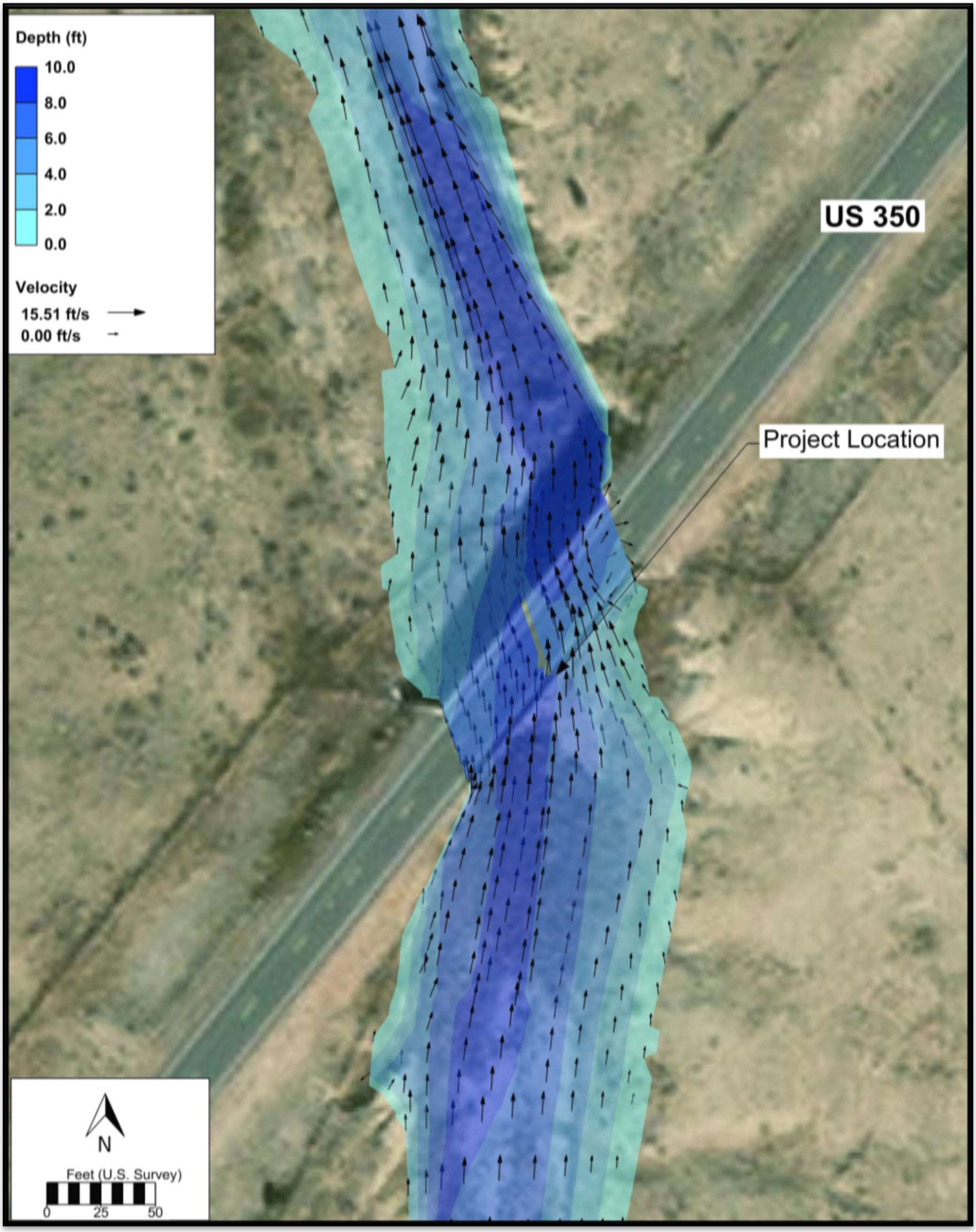


PROPOSED 100-YEAR VELOCITY RESULTS – RCBC OPTION

STRUCTURE M-21-C

FIGURE 7

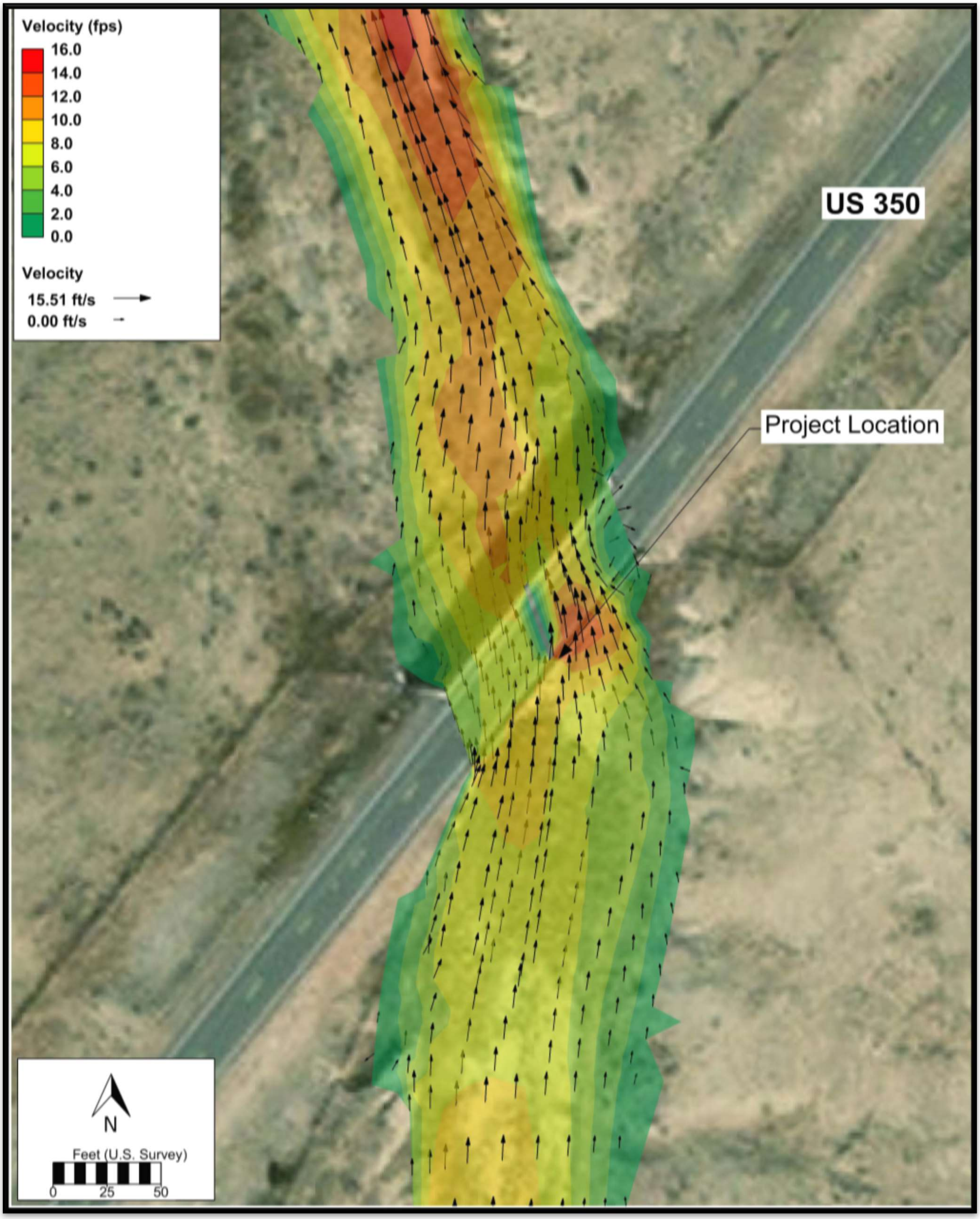
APPENDIX E PROPOSED BRIDGE ALTERNATIVE MODEL GRAPHICS



PROPOSED 100-YEAR DEPTH RESULTS – BRIDGE OPTION

STRUCTURE M-21-C

FIGURE 8

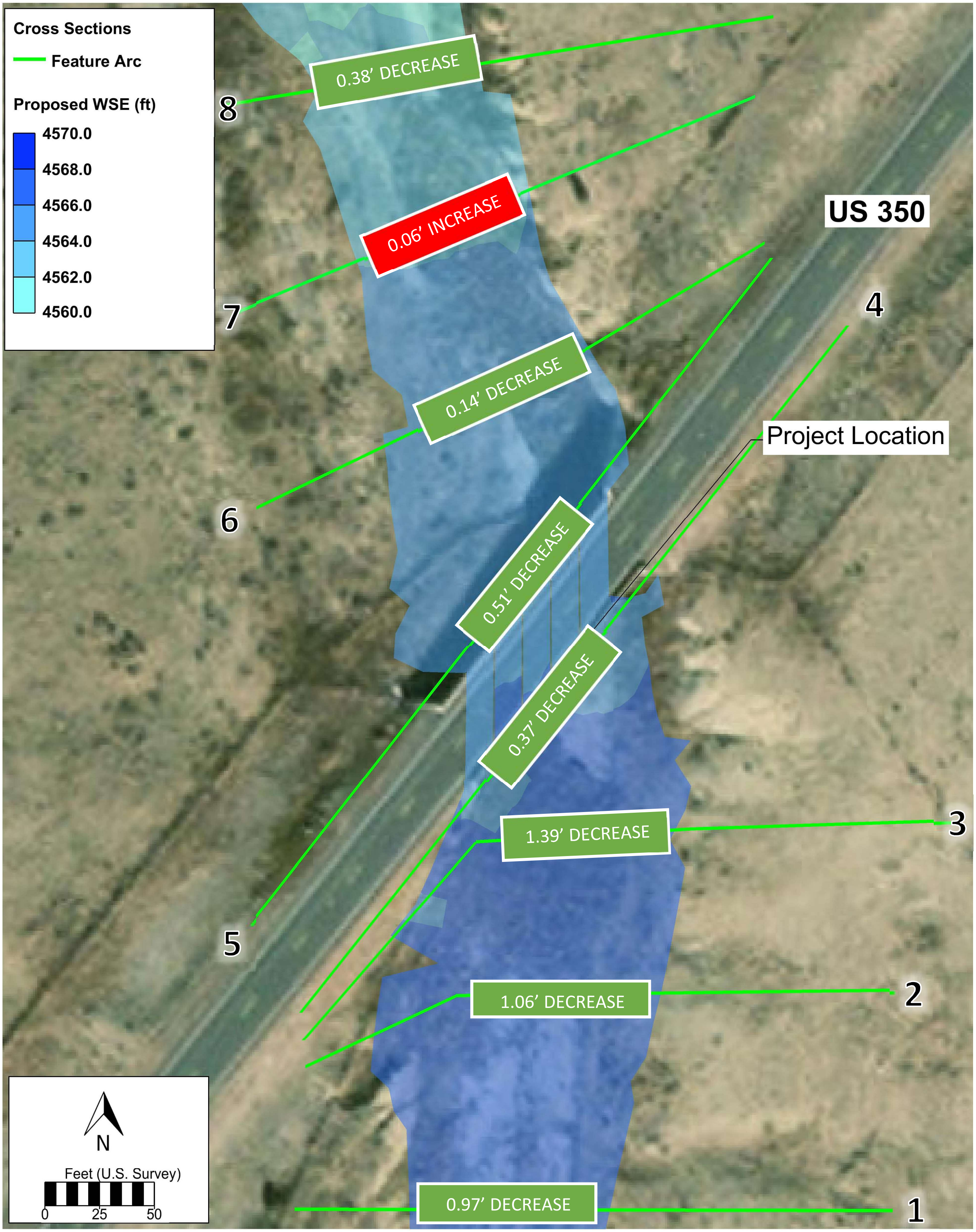


PROPOSED 100-YEAR VELOCITY RESULTS – BRIDGE OPTION

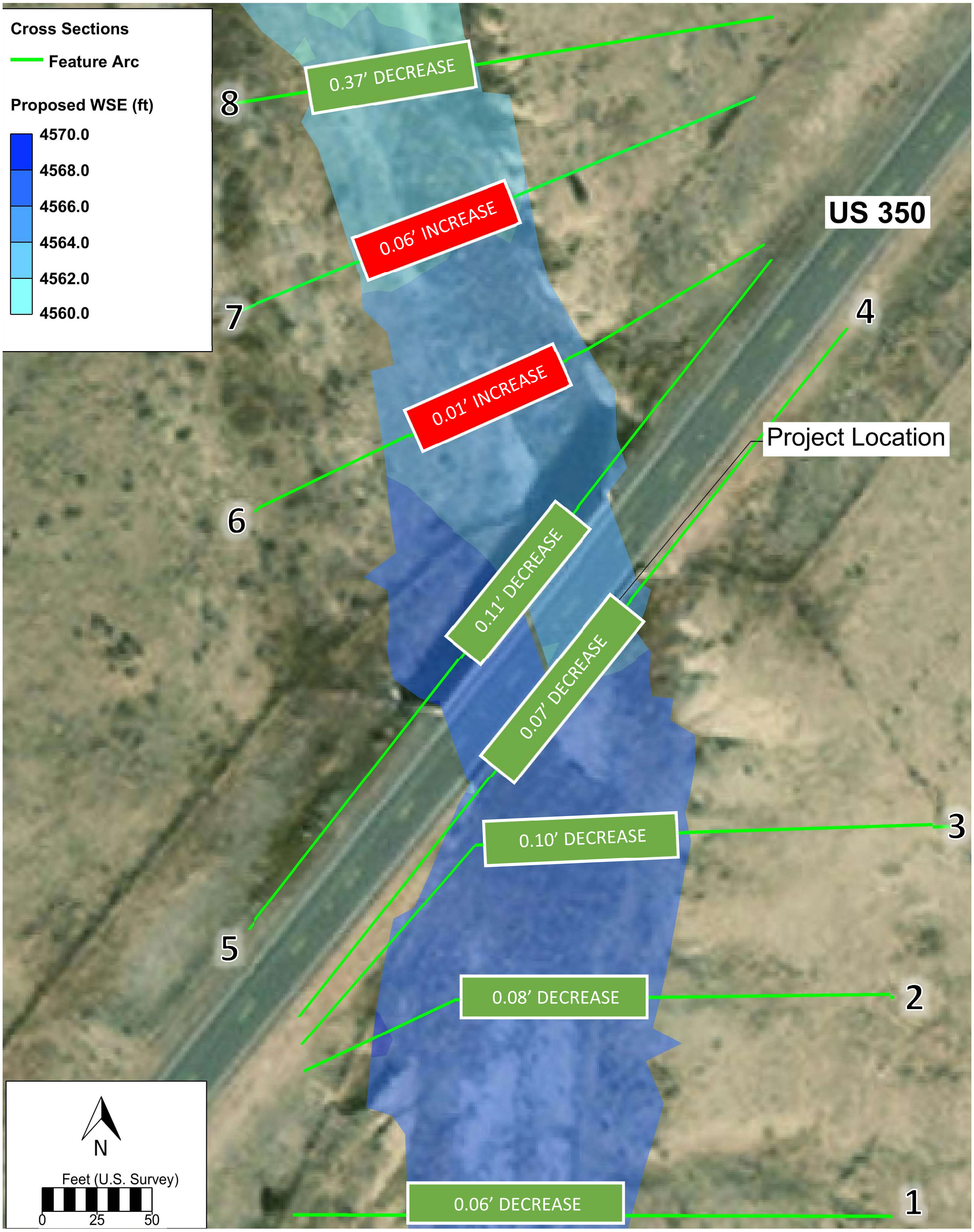
STRUCTURE M-21-C

FIGURE 9

APPENDIX F WATER SURFACE ELEVATION COMPARISON GRAPHICS

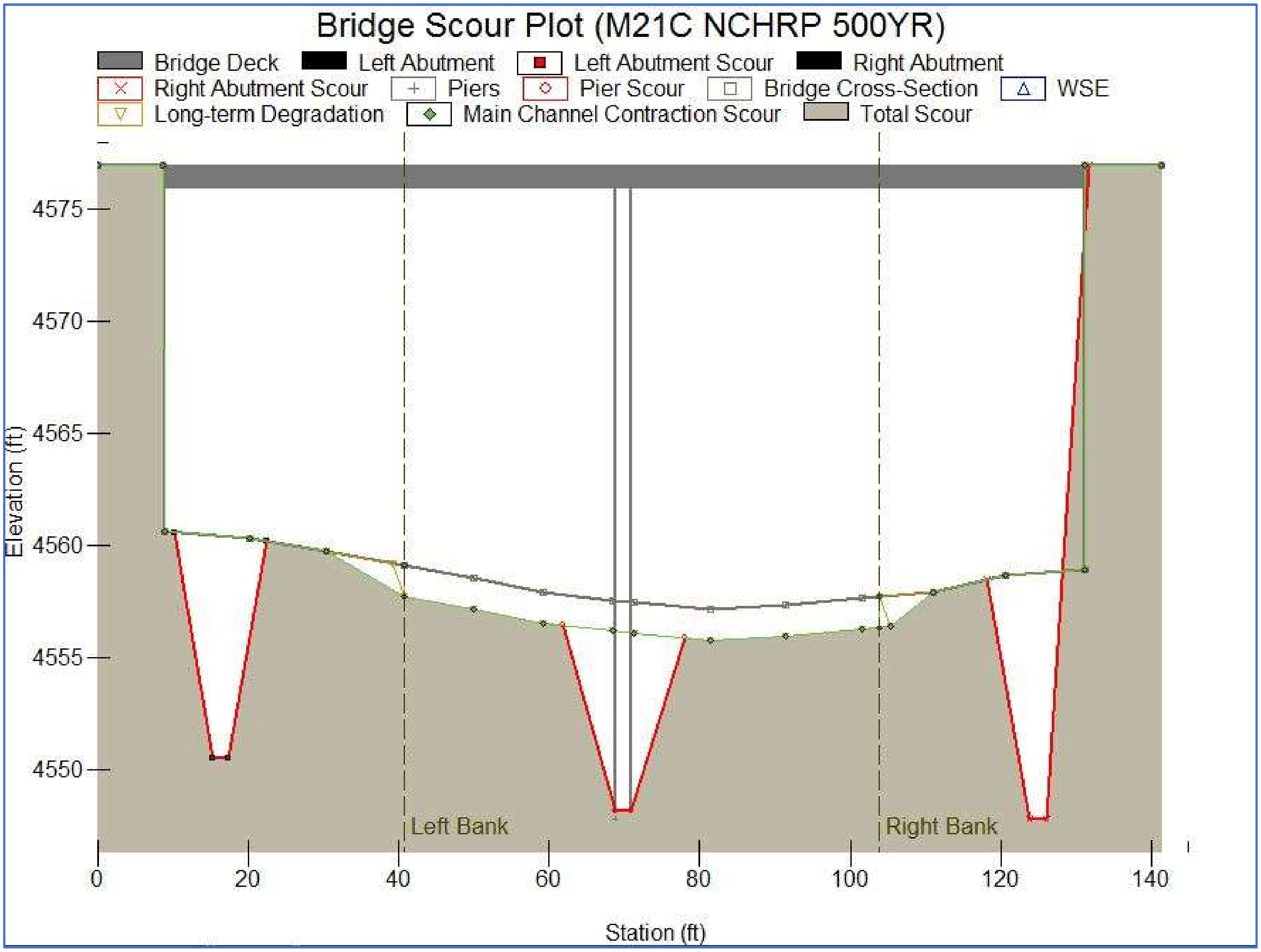


FLOODPLAIN CROSS SECTIONS – RCBC OPTION
STRUCTURE M-21-C
FIGURE 10



FLOODPLAIN CROSS SECTIONS – BRIDGE OPTION
STRUCTURE M-21-C
FIGURE 11

APPENDIX G BRIDGE SCOUR ANALYSIS


Bridge Scour Plot (M21C NCHRP 500YR)
Bridge Deck
Left Abutment
Left Abutment Scour
Right Abutment
Right Abutment Scour
Piers
Pier Scour
Bridge Cross-Section
WSE
Long-term Degradation
Main Channel Contraction Scour
Total Scour
Elevation (ft)
4575
4570
4565
4560
4555
4550
Left Bank
Right Bank
0
20
40
60
80
100
120
140
Station (ft)

Hydraulic Analysis Report

Project Data

Project Title: M-21-C 100YR

Designer: Stanley Consultants

Project Date: Tuesday, December 1, 2020

Project Units: U.S. Customary Units

Bridge Scour Analysis: M21C NCHRP 100YR

Notes

Long Term Degradation

Controlled by Armoring

Long Term Degradation (LTD) 0.80 ft

Minimum Channel Elevation with LTD 4556.34 ft

Contraction Scour

Contraction & Long Term Scour is applied method due to greater scour.

Applied Contraction Scour Elevation with LTD -1.34 ft

Local Scour at Piers

Pier Name: Pier 1

Computation Method: HEC-18

Pier Scour Depth 7.45 ft

Total Scour at Pier 6.91 ft

Total Scour Elevation at Pier 4548.89 ft

Local Scour at Abutments

Left Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 6.98 ft

Total Scour at Abutment 6.98 ft

Total Scour Elevation at Abutment 4549.36 ft

Right Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 7.23 ft

Total Scour at Abutment 7.23 ft

Total Scour Elevation at Abutment 4549.11 ft

Long Term Details

Long-Term Degradation

Computation Type: Controlled by Armoring

Input Parameters

Shield's Parameter: 0.0470

Depth or Hydraulic Radius: 7.55 ft

Average Channel Velocity: 5.23 ft/s

Unit Weight of Water: 62.40 lb/ft^3

Unit Weight of Sediment: 165.00 lb/ft^3

Bed Material is NOT Coarse Material

Manning's n Value: 0.0350

Armor Thickness Factor: 2

Result Parameters

Boundary Shear Stress: 0.4827 lb/ft^2

Critical Bed Material Size: 0.1001 ft

Percent of Bed Material Coarser than Critical Bed Material Size: 20.00 %

Depth of Degradation until Armor is Expected to Develop: 0.80 ft

Armor Thickness: 0.20 ft

Contraction Scour

Contraction Scour

Computation Type: Clear-Water or Live-Bed Scour

Input Parameters

Average Depth Upstream of Contraction: 5.93 ft

D50: 0.036100 ft

Average Velocity Upstream: 6.32 ft/s

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 4.97 ft/s

Contraction Scour Condition: Live-Bed

Live Bed and/or Clear Water Input Parameters

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section: 0.0129 ft/ft

Flow in Contracted Section: 2709.22 cfs

Flow Upstream that is Transporting Sediment: 4186.61 cfs

Width in Contracted Section: 46.54 ft

Width Upstream that is Transporting Sediment: 111.83 ft

Depth Prior to Scour in Contracted Section: 8.49 ft

Unit Weight of Water: 62.40 lb/ft^3

Unit Weight of Sediment: 165.00 lb/ft^3

Results of Live Bed Method

Shear Velocity: 1.57 ft/s

Fall Velocity: 1.49 ft/s

Average Depth in Contracted Section after Scour: 7.15 ft

Scour Depth for Live Bed: -1.34 ft

Scour may be limited by armoring. Compute all methods to check.

Left Bank Contraction Scour

Contraction Scour

Computation Type: Clear-Water or Live-Bed Scour

Input Parameters

Average Depth Upstream of Contraction: 5.93 ft

D50: 0.036100 ft

Average Velocity Upstream: 6.32 ft/s

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 4.97 ft/s

Contraction Scour Condition: Live-Bed

Live Bed and/or Clear Water Input Parameters

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section: 0.0129 ft/ft

Flow in Contracted Section: 861.75 cfs

Flow Upstream that is Transporting Sediment: 0.00 cfs

Width in Contracted Section: 31.80 ft

Width Upstream that is Transporting Sediment: -5.83 ft

Depth Prior to Scour in Contracted Section: 7.05 ft

Unit Weight of Water: 62.40 lb/ft^3

Unit Weight of Sediment: 165.00 lb/ft^3

Right Bank Contraction Scour

Contraction Scour

Computation Type: Clear-Water or Live-Bed Scour

Input Parameters

Average Depth Upstream of Contraction: 5.93 ft

D50: 0.036100 ft

Average Velocity Upstream: 6.32 ft/s

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 4.97 ft/s

Contraction Scour Condition: Live-Bed

Live Bed and/or Clear Water Input Parameters

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section: 0.0129 ft/ft

Flow in Contracted Section: 541.75 cfs

Flow Upstream that is Transporting Sediment: 1.08 cfs

Width in Contracted Section: 27.16 ft

Width Upstream that is Transporting Sediment: 0.49 ft

Depth Prior to Scour in Contracted Section: 7.36 ft

Unit Weight of Water: 62.40 lb/ft^3

Unit Weight of Sediment: 165.00 lb/ft^3

Results of Live Bed Method

Shear Velocity: 1.57 ft/s

Fall Velocity: 1.49 ft/s

Average Depth in Contracted Section after Scour: 94.14 ft

Scour Depth for Live Bed: 86.79 ft

Scour may be limited by armoring. Compute all methods to check.

Pier Details

Pier Name: Pier 1

Pier Scour

Computation Type: HEC-18

Input Parameters

Pier Shape: Round Nose

Bed Condition: Clear-Water Scour

Depth Upstream of Pier: 6.44 ft

Velocity Upstream of Pier: 12.62 ft/s

Width of Pier: 2.00 ft

Angle of Attack: 22.57 Degrees

Result Parameters

Froude Number Upstream: 0.88

Correction Factor for Pier Nose Shape (K1): 1.00

Correction Factor of Angle of Attack (K2): 1.19

Pier Length to Pier Width (L/a): 1.00

Correction Factor for Bed Condition (K3): 1.10

Scour Depth: 7.45 ft

Left Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment

Angle of Embankment to Flow: 49.73 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 37.44 cfs/ft

Unit Discharge in the Constricted Area (q2): 58.21 cfs/ft

D50: 0.036100 ft

Upstream Flow Depth: 5.93 ft

Flow Depth Prior to Scour: 6.52 ft

Result Parameters

q2/q1: 1.55

Average Velocity Upstream: 6.32 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 4.97 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 1.56

Flow Depth including Contraction Scour: 8.65 ft

Maximum Flow Depth including Abutment Scour: 13.51 ft

Scour Hole Depth from NCHRP Method: 6.98 ft

Right Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment

Angle of Embankment to Flow: 130.27 Degrees

Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 37.44 cfs/ft

Unit Discharge in the Constricted Area (q2): 58.21 cfs/ft

D50: 0.036100 ft

Upstream Flow Depth: 5.93 ft

Flow Depth Prior to Scour: 6.28 ft

Result Parameters

q2/q1: 1.55

Average Velocity Upstream: 6.32 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 4.97 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 1.56

Flow Depth including Contraction Scour: 8.65 ft

Maximum Flow Depth including Abutment Scour: 13.51 ft

Scour Hole Depth from NCHRP Method: 7.23 ft

Bridge Scour Analysis: M21C NCHRP 500YR

Notes

Long Term Degradation

Controlled by Armoring

Long Term Degradation (LTD) 1.36 ft

Minimum Channel Elevation with LTD 4555.78 ft

Contraction Scour

Contraction & Long Term Scour is applied method due to greater scour.

Applied Contraction Scour Elevation with LTD -0.52 ft

Local Scour at Piers

Pier Name: Pier 1

Computation Method: HEC-18

Pier Scour Depth 7.59 ft

Total Scour at Pier 8.44 ft

Total Scour Elevation at Pier 4548.18 ft

Local Scour at Abutments

Left Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 9.92 ft

Total Scour at Abutment 9.92 ft

Total Scour Elevation at Abutment 4545.85 ft

Right Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 11.06 ft

Total Scour at Abutment 11.06 ft

Total Scour Elevation at Abutment 4544.72 ft

Long Term Details

Long-Term Degradation

Computation Type: Controlled by Armoring

Input Parameters

Shield's Parameter: 0.0470

Depth or Hydraulic Radius: 8.90 ft

Average Channel Velocity: 7.01 ft/s

Unit Weight of Water: 62.40 lb/ft^3

Unit Weight of Sediment: 165.00 lb/ft^3

Bed Material is NOT Coarse Material

Manning's n Value: 0.0350

Armor Thickness Factor: 2

Result Parameters

Boundary Shear Stress: 0.8220 lb/ft^2

Critical Bed Material Size: 0.1705 ft

Percent of Bed Material Coarser than Critical Bed Material Size: 20.00 %

Depth of Degradation until Armor is Expected to Develop: 1.36 ft

Armor Thickness: 0.34 ft

Contraction Scour

Contraction Scour

Computation Type: Clear-Water or Live-Bed Scour

Input Parameters

Average Depth Upstream of Contraction: 7.89 ft

D50: 0.036100 ft

Average Velocity Upstream: 7.23 ft/s

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 5.21 ft/s

Contraction Scour Condition: Live-Bed

Live Bed and/or Clear Water Input Parameters

Temperature of Water: 60.00 ºF

Slope of Energy Grade Line at Approach Section: 0.0142 ft/ft

Flow in Contracted Section: 4303.24 cfs

Flow Upstream that is Transporting Sediment: 6375.49 cfs

Width in Contracted Section: 49.26 ft

Width Upstream that is Transporting Sediment: 111.83 ft

Depth Prior to Scour in Contracted Section: 10.04 ft

Unit Weight of Water: 62.40 lb/ft^3

Unit Weight of Sediment: 165.00 lb/ft^3

Results of Live Bed Method

Shear Velocity: 1.90 ft/s

Fall Velocity: 1.49 ft/s

Average Depth in Contracted Section after Scour: 9.52 ft

Scour Depth for Live Bed: -0.52 ft

Scour may be limited by armoring. Compute all methods to check.

Left Bank Contraction Scour

Contraction Scour

Computation Type: Clear-Water or Live-Bed Scour

Input Parameters

Average Depth Upstream of Contraction: 3.74 ft

D50: 0.036100 ft

Average Velocity Upstream: 0.04 ft/s

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 4.60 ft/s

Contraction Scour Condition: Clear-Water

Live Bed and/or Clear Water Input Parameters

Flow in Contracted Section: 1334.83 cfs

Bottom Width in Contracted Section: 31.80 ft

Depth Prior to Scour in Contracted Section: 8.66 ft

Results of Clear Water Method

Diameter of the smallest nontransportable particle in the bed material: 0.045125 ft

Average Depth in Contracted Section after Scour: 7.41 ft

Scour Depth: -1.25 ft

Right Bank Contraction Scour

Contraction Scour

Computation Type: Clear-Water or Live-Bed Scour

Input Parameters

Average Depth Upstream of Contraction: 2.79 ft

D50: 0.036100 ft

Average Velocity Upstream: 0.39 ft/s

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 4.38 ft/s

Contraction Scour Condition: Clear-Water

Live Bed and/or Clear Water Input Parameters

Flow in Contracted Section: 936.58 cfs

Bottom Width in Contracted Section: 27.16 ft

Depth Prior to Scour in Contracted Section: 8.08 ft

Results of Clear Water Method

Diameter of the smallest nontransportable particle in the bed material: 0.045125 ft

Average Depth in Contracted Section after Scour: 6.26 ft

Scour Depth: -1.82 ft

Pier Details

Pier Name: Pier 1

Pier Scour

Computation Type: HEC-18

Input Parameters

Pier Shape: Round Nose

Bed Condition: Clear-Water Scour

Depth Upstream of Pier: 8.54 ft

Velocity Upstream of Pier: 12.81 ft/s

Width of Pier: 2.00 ft

Angle of Attack: 17.73 Degrees

Result Parameters

Froude Number Upstream: 0.77

Correction Factor for Pier Nose Shape (K1): 1.00
Correction Factor of Angle of Attack (K2): 1.16
Pier Length to Pier Width (L/a): 1.00
Correction Factor for Bed Condition (K3): 1.10
Scour Depth: 7.59 ft

Left Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment
Angle of Embankment to Flow: 53.72 Degrees
Centerline Length of Embankment: 0.00 ft
Projected Length of Embankment: 0.00 ft
Width of Flood Plain: 0.00 ft
Unit Discharge, Upstream in Main Channel (q1): 57.01 cfs/ft
Unit Discharge in the Constricted Area (q2): 87.36 cfs/ft
D50: 0.036100 ft
Upstream Flow Depth: 7.89 ft
Flow Depth Prior to Scour: 8.11 ft
Result Parameters
q2/q1: 1.53
Average Velocity Upstream: 7.23 ft/s
Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 5.21 ft/s
Scour Condition: Live Bed
Embankment Length/Floodplain Width Ratio: 0.00
Scour Condition: a (Main Channel)
Amplification Factor: 1.59
Flow Depth including Contraction Scour: 11.38 ft
Maximum Flow Depth including Abutment Scour: 18.03 ft
Scour Hole Depth from NCHRP Method: 9.92 ft

Right Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Vertical-wall abutment
Angle of Embankment to Flow: 126.29 Degrees
Centerline Length of Embankment: 0.00 ft

Projected Length of Embankment: 0.00 ft

Width of Flood Plain: 0.00 ft

Unit Discharge, Upstream in Main Channel (q1): 57.01 cfs/ft

Unit Discharge in the Constricted Area (q2): 87.36 cfs/ft

D50: 0.036100 ft

Upstream Flow Depth: 7.89 ft

Flow Depth Prior to Scour: 6.97 ft

Result Parameters

q2/q1: 1.53

Average Velocity Upstream: 7.23 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 5.21 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 1.59

Flow Depth including Contraction Scour: 11.38 ft

Maximum Flow Depth including Abutment Scour: 18.03 ft

Scour Hole Depth from NCHRP Method: 11.06 ft

Riprap Analysis: Left Abutment

Notes: The Total Bridge Area was adjusted until the characteristic velocity matched the maximum channel velocity. This allows for a more conservative calculation at the abutment. Based on engineering judgement, the D50 is rounded to the next highest class. When results are considered liberal, the maximum channel velocity is used in lieu of the average to achieve more practical results. When results are considered conservative, the average channel velocity is used in lieu of the maximum to achieve more practical results. For this calculation, the average velocity is used.

Input Parameters

Riprap Type: Abutment/Guide Bank

The structure is a guidebank

Set-back Length: 11 ft

The set-back length is the distance from the near edge of the main channel to the toe of abutment

Main Channel Average Flow Depth: 10.5 ft

Flow Depth at Toe of Abutment: 7.5 ft

Calculations will use either total or overbank discharges.

Total Discharge: 4359 cfs

Overbank Discharge: 396 cfs

Total Bridge Area: 513 ft^2

Setback Area: 192.5 ft^2

Maximum Channel Velocity: 8.5 ft/s

Specific Gravity of Riprap: 2.65

Result Parameters

Set-back ratio: 1.04762

Characteristic Velocity: 8.49708 ft/s

Froude Number at the Abutment Toe: 0.546999

Abutment Coefficient: 1.02

Computed D50: 16.6468 in

Design D50 = 18 in

Thickness = 36 in

Design D50 > Computed D50

18 in > 16.6468 in

Riprap Class

Riprap shape should be angular

Riprap Class Name: CLASS V

Riprap Class Order: 5

The following values are an **'average'** of the size fraction range for the selected riprap class.

d100: 36 in

d85: 25.5 in

d50: 18.5 in

d15: 13 in

Layout Recommendations

Minimum Riprap Thickness: 36 in

Minimum Horizontal Extent of the Toe Apron from the Abutment Toe: 15 ft

Minimum Extent of "Wrap Around" beyond the Abutment Radius, along the Approach Embankment: 25 ft

See HEC 23, Figure 14.7

No channel used in calculations

Riprap Analysis: Right Abutment

Notes: The Total Bridge Area was adjusted until the characteristic velocity matched the maximum channel velocity. This allows for a more conservative calculation at the abutment. Based on engineering judgement, the D50 is rounded to the next highest class. When results are considered liberal, the maximum channel velocity is used in lieu of the average to achieve more practical results. When results are considered conservative, the average channel velocity is used in lieu of the maximum to achieve more practical results. For this calculation, the average velocity is used.

Input Parameters

Riprap Type: Abutment/Guide Bank

The structure is a guidebank

Set-back Length: 35 ft

The set-back length is the distance from the near edge of the main channel to the toe of abutment

Main Channel Average Flow Depth: 10.15 ft

Flow Depth at Toe of Abutment: 6.6 ft

Calculations will use either total or overbank discharges.

Total Discharge: 4359 cfs

Overbank Discharge: 1194 cfs

Total Bridge Area: 513 ft^2

Setback Area: 595 ft^2

Maximum Channel Velocity: 8.5 ft/s

Specific Gravity of Riprap: 2.65

Result Parameters

Set-back ratio: 3.44828

Characteristic Velocity: 8.49708 ft/s

Froude Number at the Abutment Toe: 0.583103

Abutment Coefficient: 1.02

Computed D50: 16.6468 in

Design D50 = 18 in

Thickness = 36 in

Design D50 > Computed D50

18 in > 16.6468 in

Riprap Class

Riprap shape should be angular

Riprap Class Name: CLASS V

Riprap Class Order: 5

The following values are an **'average'** of the size fraction range for the selected riprap class.

d100: 36 in

d85: 25.5 in

d50: 18.5 in

d15: 13 in

Layout Recommendations

Minimum Riprap Thickness: 36 in

Minimum Horizontal Extent of the Toe Apron from the Abutment Toe: 13.2 ft

Minimum Extent of "Wrap Around" beyond the Abutment Radius, along the Approach Embankment: 25 ft

See HEC 23, Figure 14.7

No channel used in calculations

APPENDIX H GEOTECHNICAL INFORMATION

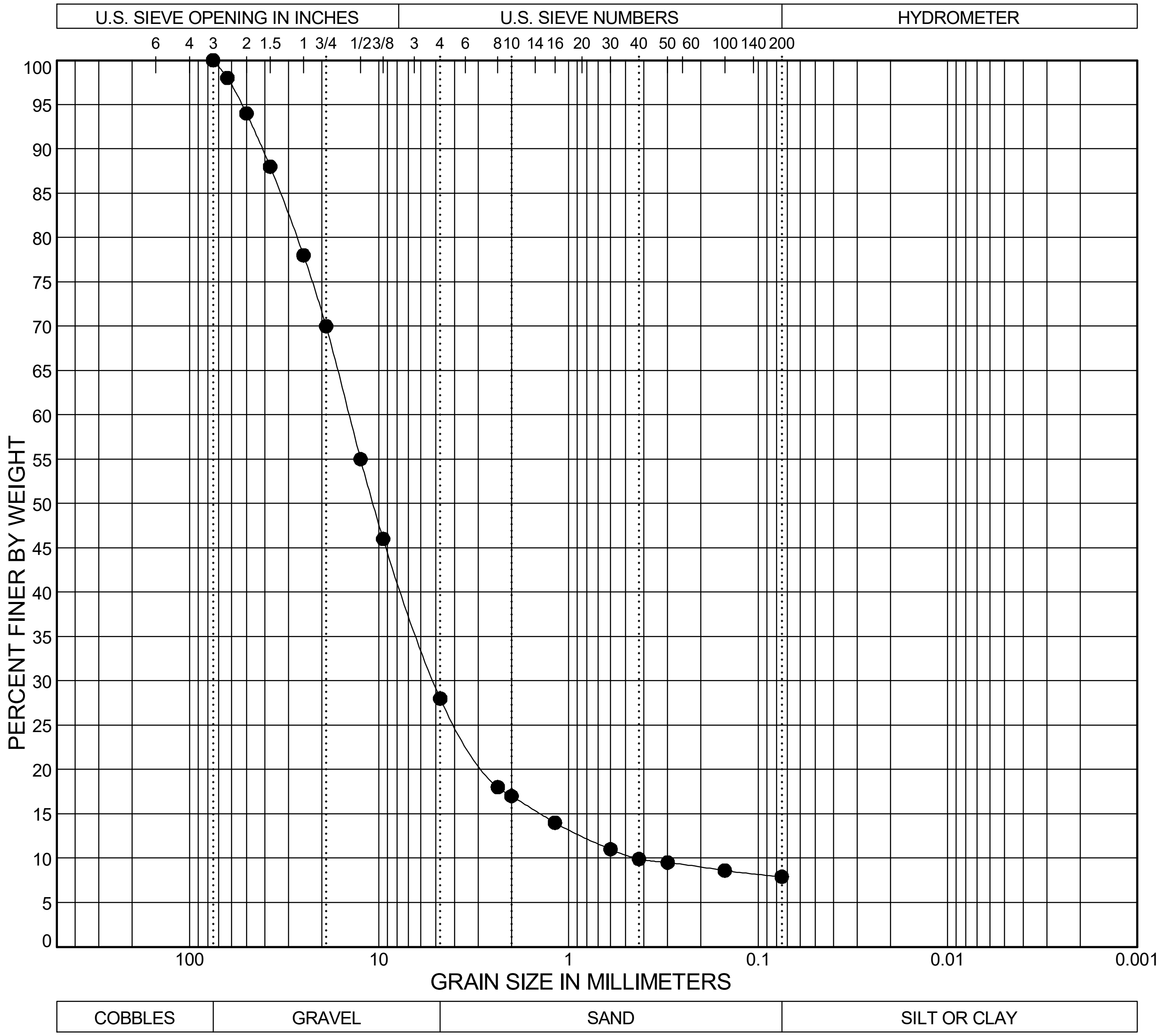
Yeh and Associates, Inc.
Geotechnical • Geological • Construction Services

Colorado Springs Lab

Summary of Laboratory Test Results

Project No: 220-063 Project Name: CDOT Region 2 Bridge Bundle - Scour Test Results Date: 11-06-2020

Sample Location			Natural Moisture Content (%)	Natural Dry Density (pcf)	Gradation			Atterberg			pH	Water Soluble Sulfate (%)	Water Soluble Chloride (%)	Resistivity (ohm-cm)	Swell (+) / Collapse (-) (% at Load in psf)	Unconf. Comp. Strength (psi)	R-Value	Classification	
Boring No.	Depth (ft)	Sample Type			Gravel > #4 (%)	Sand (%)	Fines < #200 (%)	LL	PL	PI								AASHTO	USCS
M-21-B Scour	0	BULK	6.1		4.0	14.9	81.1												
M-21-C Scour	0	BULK	3.5		72.0	20.1	7.9												
M-21-I Scour	0	BULK	4.5		0.0	5.3	94.7												
M-21-J Scour	0	BULK	7.3		1.0	3.5	95.5												
M-22-U Scour	0	BULK	5.9		31.0	24.3	44.7												
M-22-Y Scour	0	BULK	11.9		1.0	11.9	87.1												
N-21-C Scour	0	BULK	1.8		61.0	21.0	18.0												
N-21-F Scour	0	BULK	11.8		2.0	16.4	81.6												
O-19-D Scour	0	BULK	2.7		6.0	56.7	37.3												
P-19-G Scour	0	BULK	1.1		21.0	53.4	25.6												



BOREHOLE	DEPTH (ft)	AASHTO Classification	USCS Classification	LL	PL	PI	%Gravel	%Sand	%Fines: %Silt	%Fines: %Clay
● **M-21-C Scour**	**0.0**						**72.0**	**20.1**	**7.9**	

Yeh and Associates, Inc.
Geotechnical • Geological • Construction Services

SIEVE ANALYSIS

FIGURE **S- 2**

Project No.	220-063	Date:	11-06-2020
Report By:	D. Gruenwald	Yeh Lab:	Colorado Springs
Checked By:	J. McCall		

CDOT Region 2 Bridge Bundle
Scour Test Results