PRELIMINARY HYDRAULICS REPORT
STRUCTURE N-21-F REPLACEMENT

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

A Part of Section 35, Township 26 South, Range 58 West of the 6^{th} P.M.,
County of Otero, Colorado

February 5, 2021

Prepared for:

COLORADO
Department of Transportation

Colorado Department of Transportation
Region 2
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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 N-21-F as a part of the CDOT Region 2 Bridge Bundle Design Build. The project is located within Otero County at Mile Post 48.744 along US 350 between Trinidad and La Junta. Structure N-21-F crosses over the Sheep Canyon Arroyo. Figure 1 below illustrates the project location. The project is in Section 35, Township 26 South, Range 58 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) 0801320275B 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 N-21-F.

Table 1: Summary of Peak Discharge for Bridge N-21-F

River Location	Design Storm	100-year (cfs)	200-year (cfs)	500-year (cfs)
Upstream of Bridge	100-year	4,355	5,289	6,656

3. EXISTING CONDITIONS

3.1 Existing Structure

Existing structure is a four-span concrete deck, steel I beam girder, bridge built in 1937 to span Sheep Canyon Arroyo. The bridge is on a 45-degree skew. The existing bridge consists of four spans of 39’-6”, with a total length of 166’-2”. The width of the existing bridge is 30’-0” curb to curb, 33’-6” out to out of deck. The existing vertical clearance varies from 4’-0” to 12’-6”. The structure has an unidentified utility attached to the south side of the bridge girder. The bridge is located on US 350, southwest of La Junta, at milepost 48.744.

3.2 Watershed Overview

The Sheep Canyon Arroyo is a dry arroyo that flows from the south to the north toward Timpas Creek. The watershed tributary to Sheep Canyon Arroyo is approximately 17.6 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 45 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 at the base of the center pier columns. This is evident by the exposed columns and high soil marks. 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: Bridge #1 Replacement
- Proposed Conditions: Bridge #2 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.

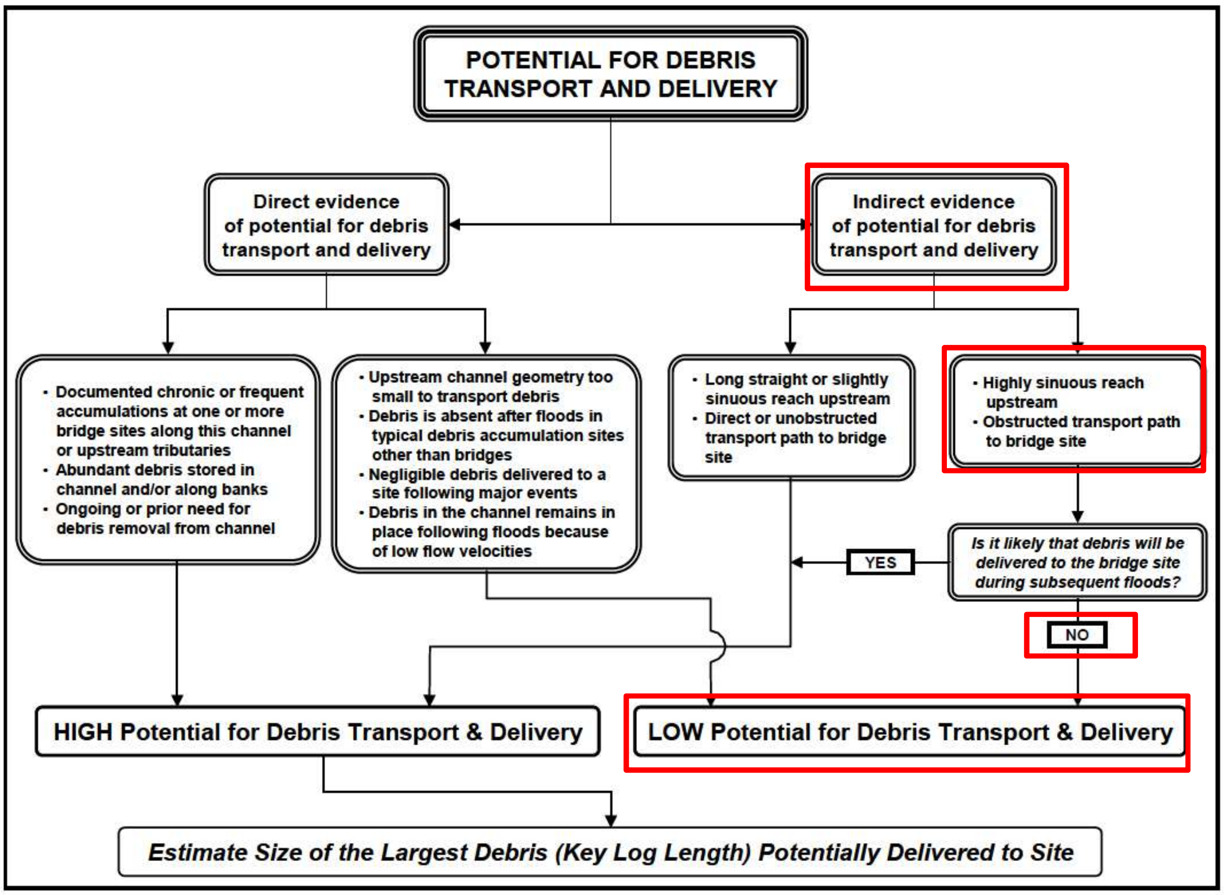


Figure 2: Flow Chart for Potential Debris Production (FHWA, HEC 20)

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 would be required, if a bridge is selected for the proposed conveyance structure. The proposed preliminary design provides 1.89-ft of freeboard which does not meet the 2-ft minimum but due to funding and site constraints, it is not feasible to raise the bridge above the 100-year floodplain.

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. Additionally, a drone was flown by Stanley Consultants, which collected LiDAR data for the railroad bridge that is outside of the CDOT right-of-way. These three 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, railroads 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 7,038 and the mesh extends approximately 1,150 feet upstream of the bridge and 915 feet downstream of the bridge. These extents were chosen due to it encompassing the limits of the survey from CDOT, and to account for the nearby railroad bridge that is downstream of N-21-F.

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
Median	0.045
Open Space	0.040
Riprap	0.050
Railroad	0.025
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	4,355	4601.68

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 4626.0 feet, at the grade center, while the low chord is 4621.65 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 4 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 2.2 feet to 11.0 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. Due to the high discharge and depth of the channel relative to the roadway, two bridge alternatives were modelled. 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 Bridge Alternative #1

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 7,092 mesh elements. The proposed model has a two-span non-symmetrical 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 79.5’ and 89.5’ long from bearing to bearing, with a total length of 174’ centerline to centerline of the abutments. The low chord of the bridge is at 4623.5’ elevation, and the high chord didn’t change from the existing condition. The piers were modelled with a diameter of 2.5’. Roadway embankments were graded at 2:1.

Depths and velocity grids for the proposed bridge show depths from 2.0 to 11.2 feet and velocities from 0.6 to 7.8 ft/s. See Appendix D for 100-year depths and velocities graphics for this option.

Proposed Bridge Alternative #2

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 7,096 mesh elements. The proposed model has a two-span symmetrical 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 58.5’ long from bearing to bearing, with a total length of 122’ centerline to centerline of the abutments. The low chord of the bridge is at 4623.5’ elevation, and the high chord didn’t change from the existing condition. The piers were modelled with a diameter of 2.5’. Roadway embankments were graded at 2:1.

Depths and velocity grids for the proposed bridge show depths from 3.0 to 10.8 feet and velocities from 3.4 to 8.3 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 0801320275B 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 Bridge Alternative #1

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

To perform a comparison between the existing and proposed WSE, eight 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 bridge option, as shown in Appendix F.

For the proposed culvert option, upstream of Bridge N-21-F (Cross Sections 1-4), the WSE decreases between 0.08 and 0.28 feet between existing and proposed. Downstream of Bridge N-21-F (Cross Sections 5,6 & 8), the WSE decreases between 0.00 and 0.14 feet between existing and proposed. Also downstream of Bridge N-21-F (Cross Section 7), the WSE increases a maximum of 0.11 feet between existing and proposed. The WSE comparison at these sections is shown in Table 4.

Table 4: WSE Comparison for Bridge #1 Option

Cross Section	Location Relative to Proposed Bridge	Existing WSE (ft)	Proposed WSE (ft)	Proposed vs Existing
1	Upstream	4622.40	4622.32	-0.08
2	Upstream	4621.94	4621.81	-0.13
3	Upstream	4621.62	4621.44	-0.18
4	Upstream	4621.36	4621.08	-0.28
5	Downstream	4620.93	4620.93	0.00
6	Downstream	4620.09	4619.95	-0.14
7	Downstream	4619.36	4619.47	0.11
8	Downstream	4618.39	4618.36	-0.03

Proposed Bridge Alternative #2

Similarly, the model for the proposed bridge will not increase the WSE by more than 1 foot. Although the bridge opening for this option is much shorter than the existing bridge, the area of opening is similar. This is due to the main channel being much deeper than the outer banks, thus conveying most of the flow. The proposed bridge opening only spans the main channel; therefore, no change in WSE is expected.

For the proposed bridge option, upstream of Bridge N-21-F (Cross Sections 1-4), the WSE decreases between 0.00 and 0.12 feet between existing and proposed. Downstream of Bridge N-21-F (Cross Sections 5,6 & 8), the WSE decreases between 0.00 and 0.18 feet between existing and proposed. Also downstream of Bridge N-21-F (Cross Section 7), the WSE increases a maximum of 0.12 feet between existing and proposed.

Appendix F 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 #2 Option

Cross Section	Location Relative to Proposed Bridge	Existing WSE (ft)	Proposed WSE (ft)	Proposed vs Existing
1	Upstream	4622.40	4622.40	0.00
2	Upstream	4621.94	4621.94	0.00
3	Upstream	4621.62	4621.61	-0.01
4	Upstream	4621.36	4621.24	-0.12
5	Downstream	4620.93	4620.75	-0.18
6	Downstream	4620.09	4620.02	-0.07
7	Downstream	4619.36	4619.48	0.12
8	Downstream	4618.39	4618.39	0.00

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 Sheep Canyon 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 Sheep Canyon 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**	**0.6**	**12.6**	**9.2**	**13.2**	**9.8**
500-Year	**0**	**1.0**	**17.6**	**10.8**	**18.6**	**11.8**

*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 is characterized with a slight sinuosity, defined low flow channel and highly erosive soils. The river takes a defined turn immediately upstream of the bridge and the bridge pier will be located in an area of the highest velocities within 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 abutment and wing walls shall be designed with a toe wall extending down to the 100-yr scour depth. The FHWA

Hydraulic Toolbox Version 5.0 (FHWA, 2018) was used to size riprap at the ends of the proposed wing walls and along the roadway embankment. The riprap was sized for the 100-year hydraulic 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 maximum velocity in the channel. 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 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 7. 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

Countermeasure	D50 (in)	Recommended Thickness	Side Slopes (Max)	Toe Down Depth (ft)	Bottom Ref. Elevation (ft)	Top Ref. Elevation (ft)
Riprap	18	36	2:1	14	4597.0	4623.3
Wing Walls	N/A	N/A	N/A	14	4597.0	4623.3

7. CONCLUSIONS

This report presents preliminary analysis and results from the hydrologic and hydraulic study for the Region 2 Bridge Bundle Design Build – Bridge N-21-F. 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 122-foot span length bridge. The recommended freeboard is 2-feet and the proposed WSE 100 feet upstream of the proposed bridge is 4621.61 feet, giving a final recommended bridge low chord of 4623.61 feet. The proposed low chord is 4623.50 feet, which does not meet the 2 feet of freeboard that is required. However, this condition is not worse than the existing condition.

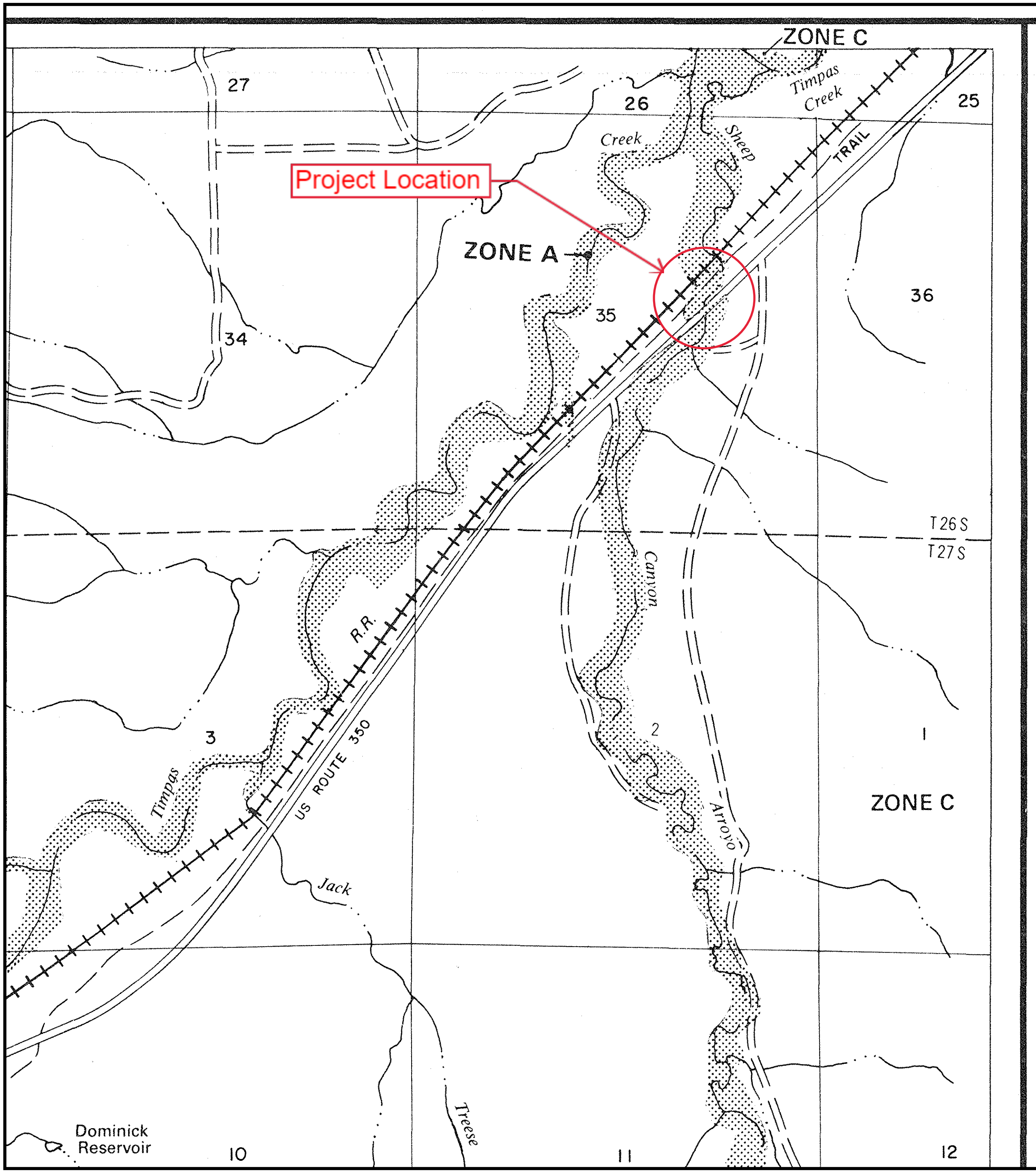
Floodplain analysis demonstrates that the proposed bridge opening will not cause a rise in flood levels during the 100-year design event. This meets guidelines in CFR Sections 60.3 (b). A floodplain development permit is required to be approved through the Otero County floodplain administrator during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

Total design scour for the bridge abutments was determined to be 13.2 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



NATIONAL FLOOD INSURANCE PROGRAM

FIRM

FLOOD INSURANCE RATE MAP

OTERO COUNTY,
COLORADO
(UNINCORPORATED AREAS)

PANEL 275 OF 325

(SEE MAP INDEX FOR PANELS NOT PRINTED)

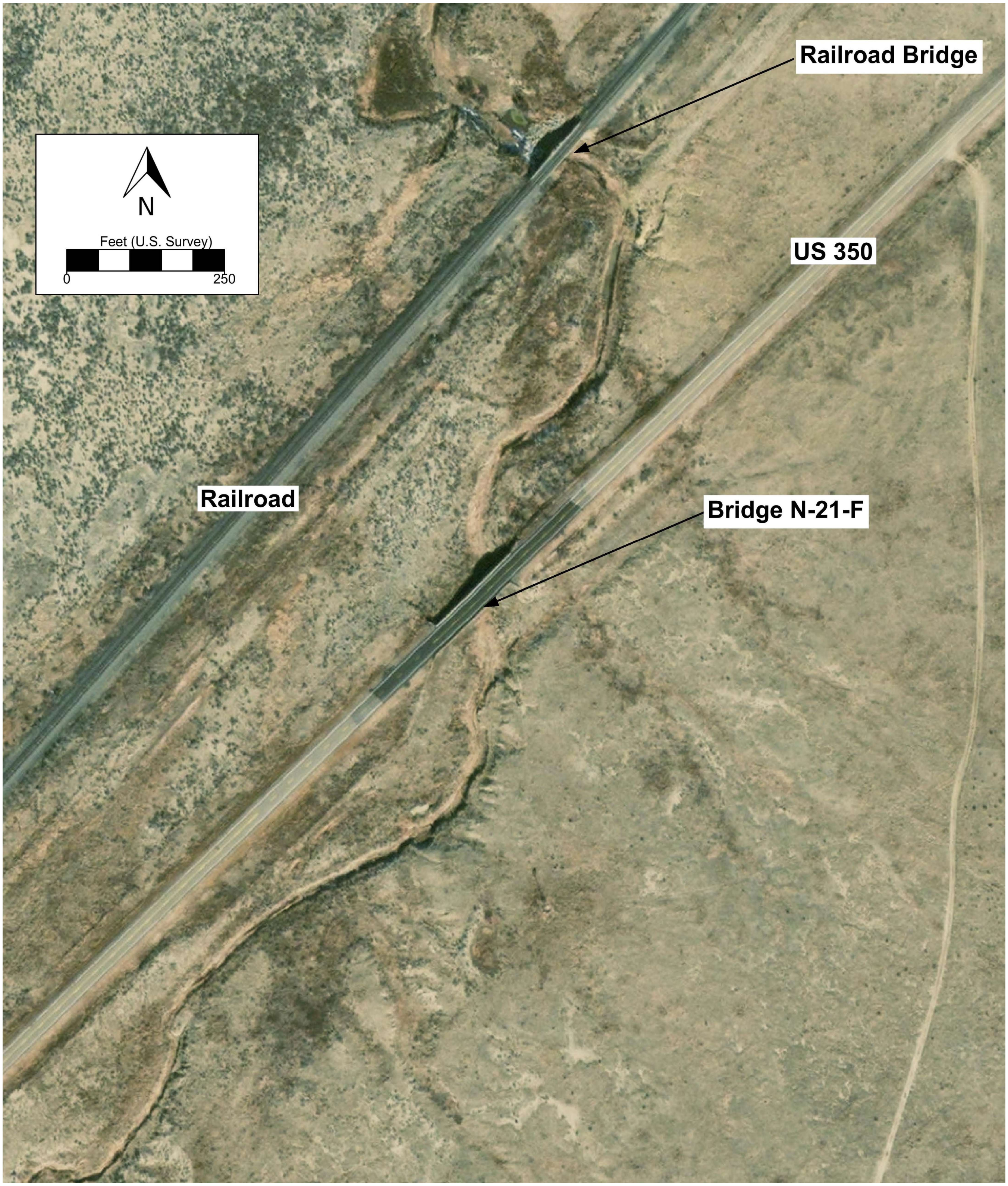
COMMUNITY-PANEL NUMBER
080132 0275 B

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 N-21-F
FIGURE 3

PHOTO 1: BRIDGE STRUCTURE SIGN

STRUCTURE N-21-F

APPENDIX A

PHOTO 2: BRIDGE M-21-C EXISTING STRUCTURE

STRUCTURE N-21-F

APPENDIX A

PHOTO 3: LOOKING SOUTH UPSTREAM OF BRIDGE

STRUCTURE N-21-F

APPENDIX A

PHOTO 4: DITCH UNDER THE BRIDGE LOOKING SOUTH

STRUCTURE N-21-F

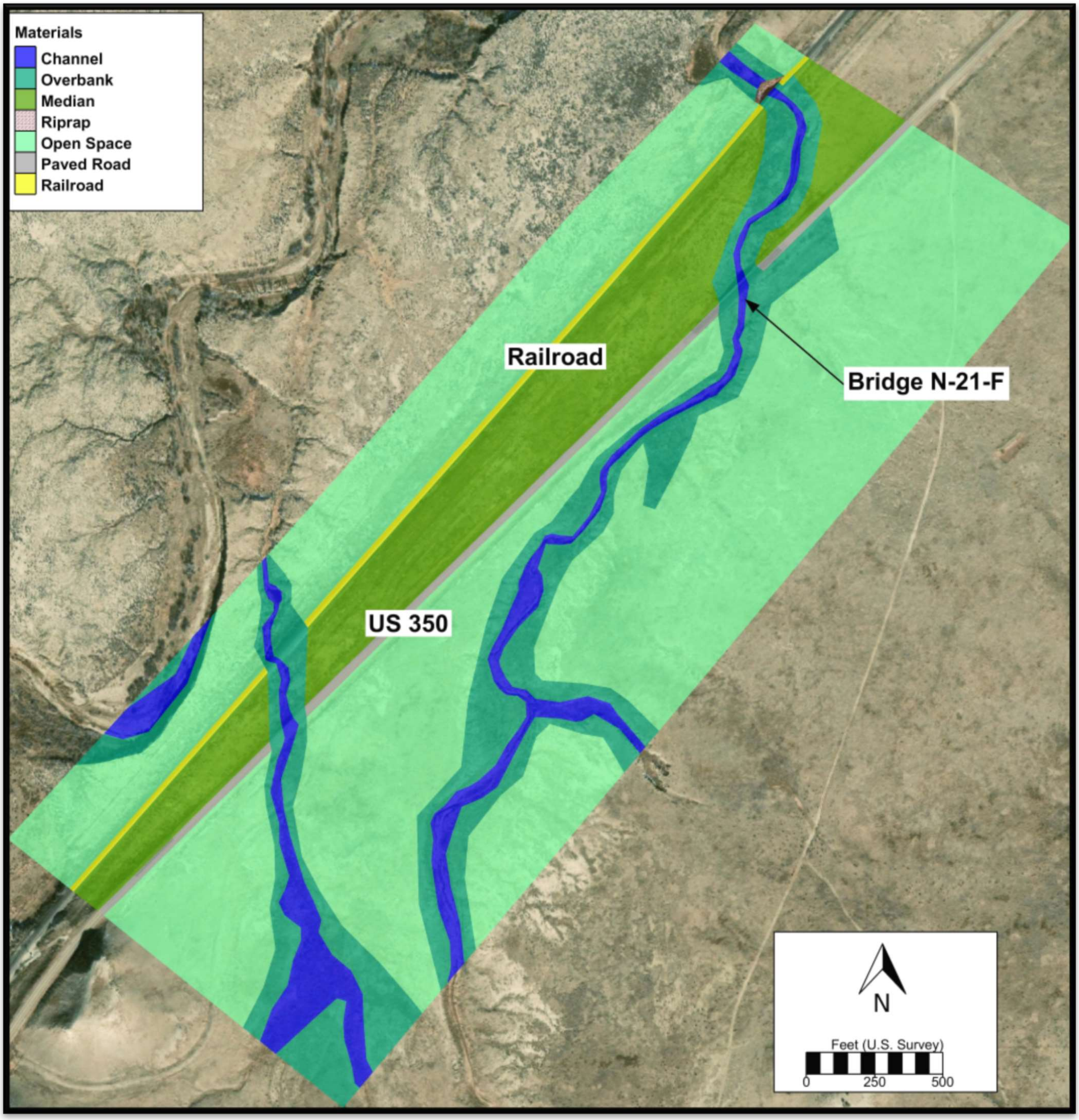
APPENDIX A

PHOTO 5: DOWNSTREAM OF BRIDGE LOOKING NORTH

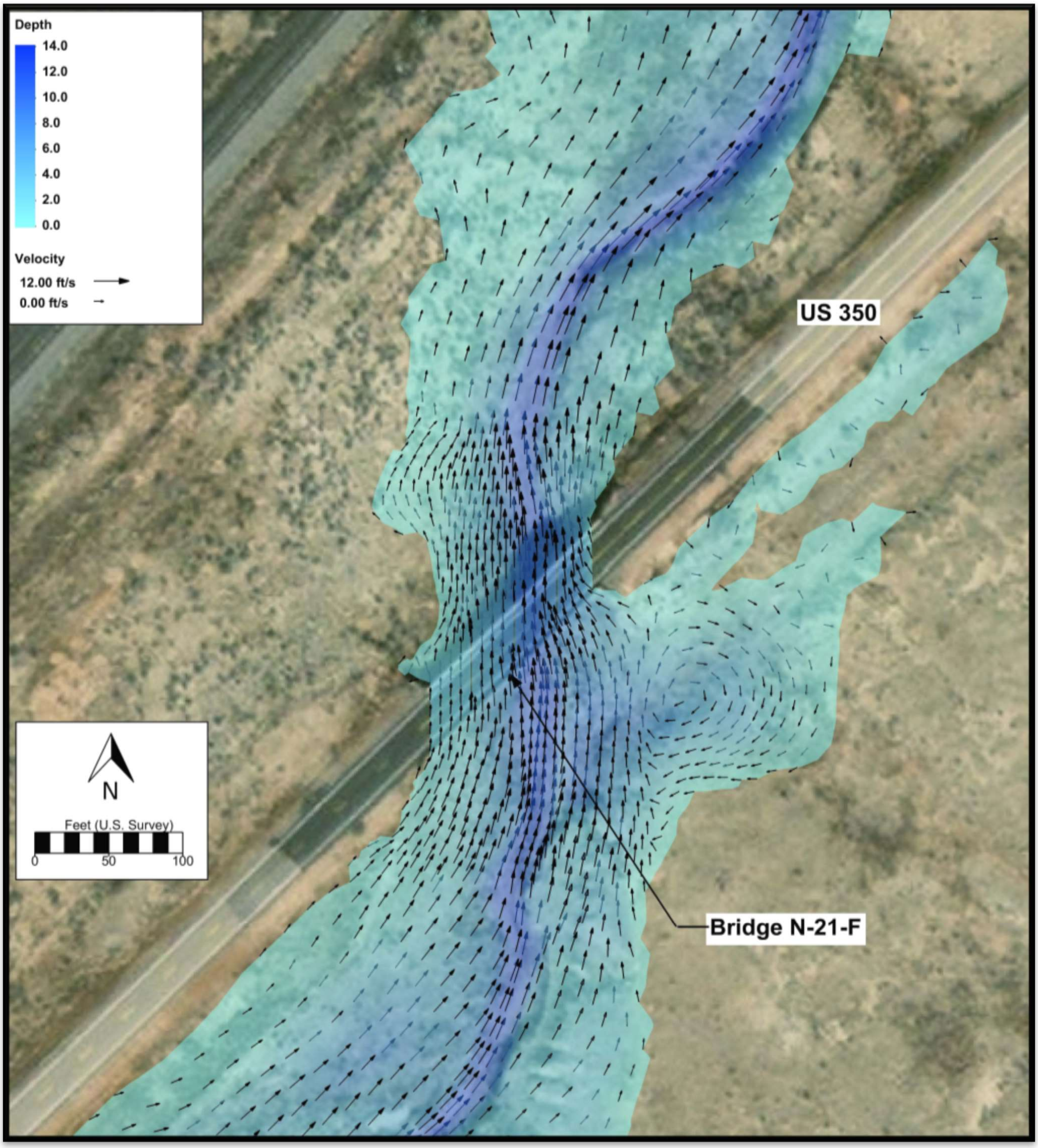
STRUCTURE N-21-F

APPENDIX A

APPENDIX C EXISTING CONDITIONS MODEL GRAPHICS



MATERIALS COVERAGE
STRUCTURE N-21-F
FIGURE 4

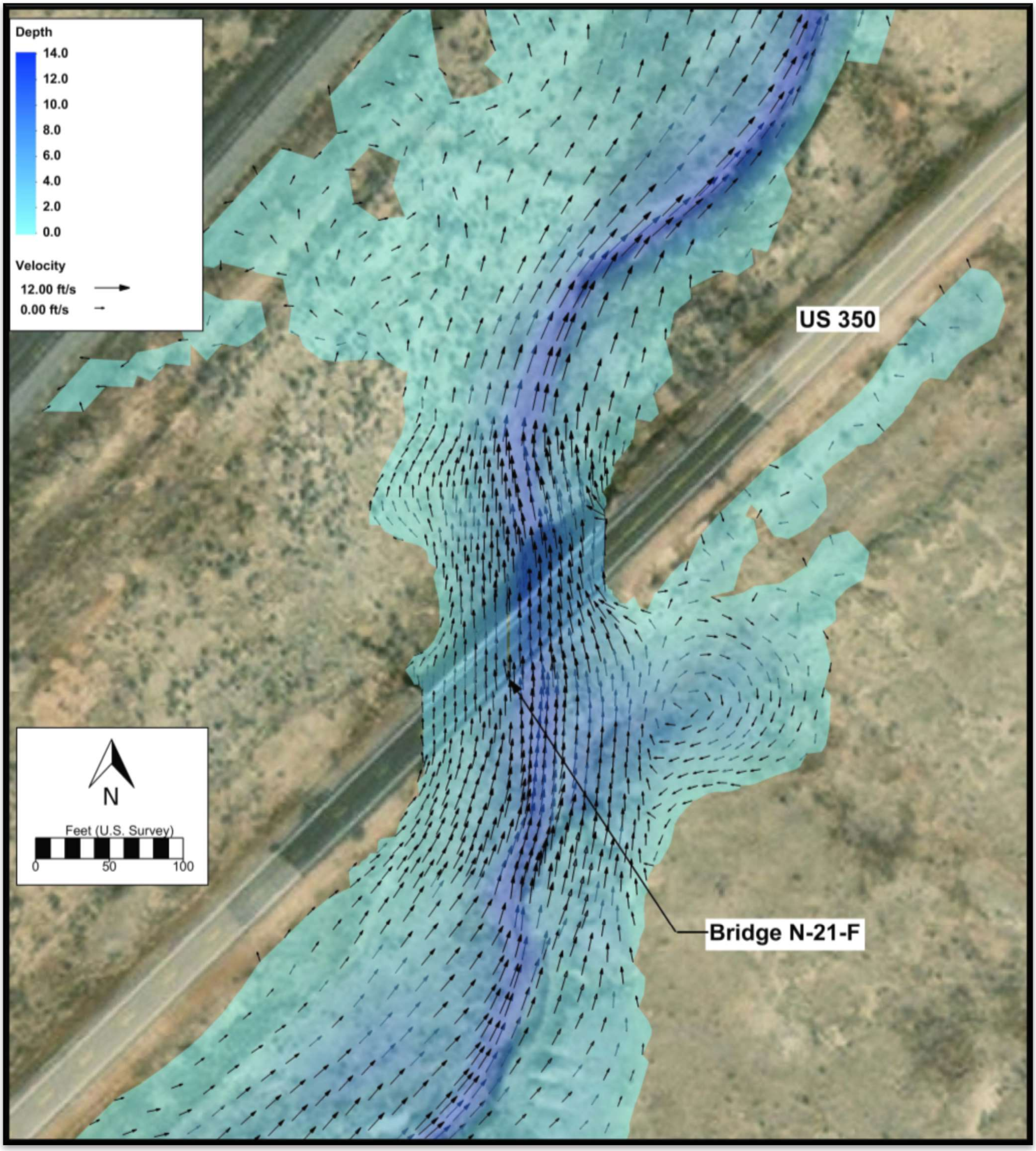


EXISTING CONDITIONS 100-YEAR DEPTH RESULTS

STRUCTURE N-21-F

FIGURE 5

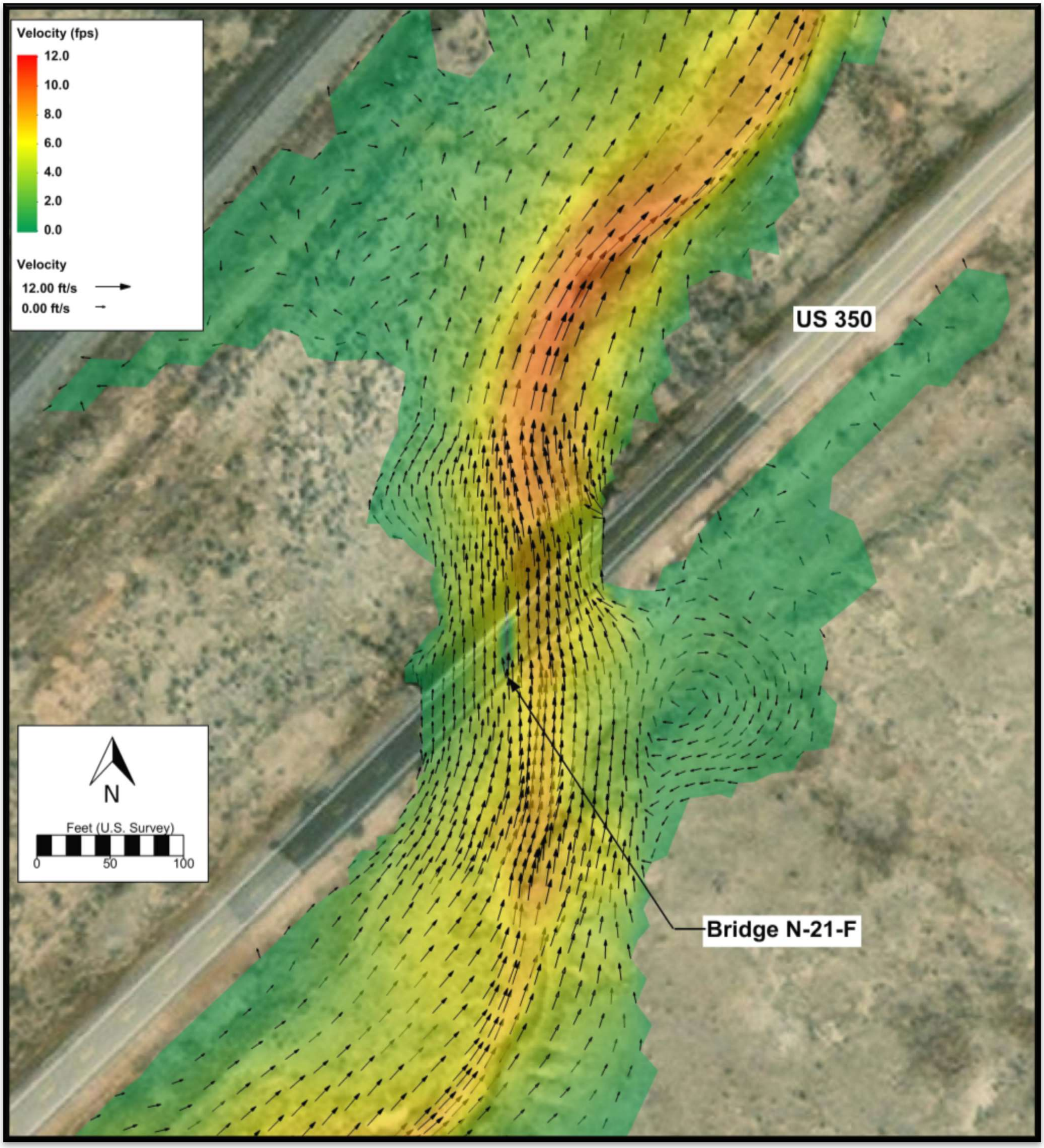
APPENDIX D PROPOSED BRIDGE 1 ALTERNATIVE MODEL GRAPHICS



PROPOSED 100-YEAR DEPTH RESULTS – BRIDGE #1 OPTION

STRUCTURE N-21-F

FIGURE 6

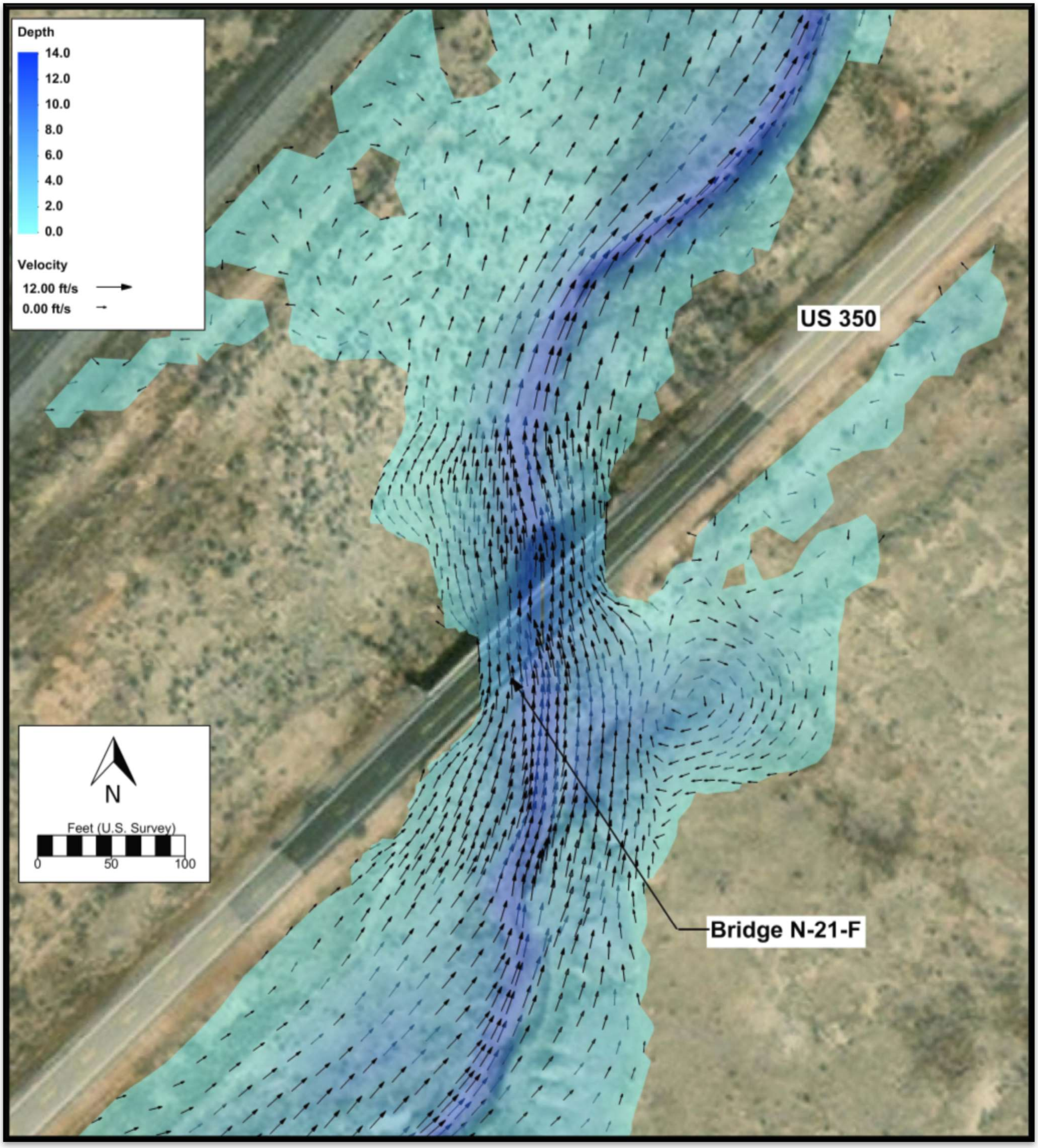


PROPOSED 100-YEAR VELOCITY RESULTS – BRIDGE #1 OPTION

STRUCTURE N-21-F

FIGURE 7

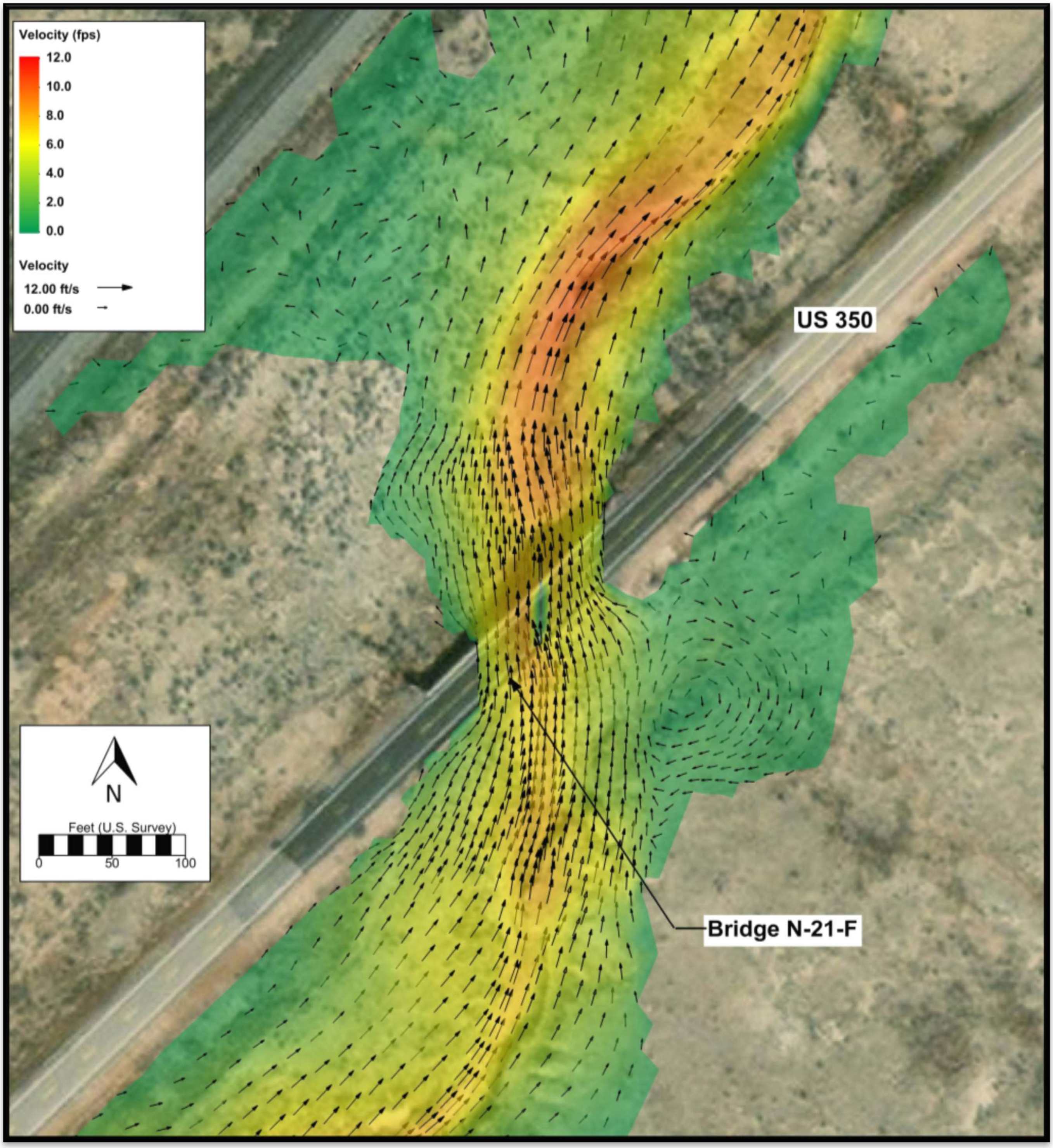
APPENDIX E PROPOSED BRIDGE 2 ALTERNATIVE MODEL GRAPHICS



PROPOSED 100-YEAR DEPTH RESULTS – BRIDGE #2 OPTION

STRUCTURE N-21-F

FIGURE 8

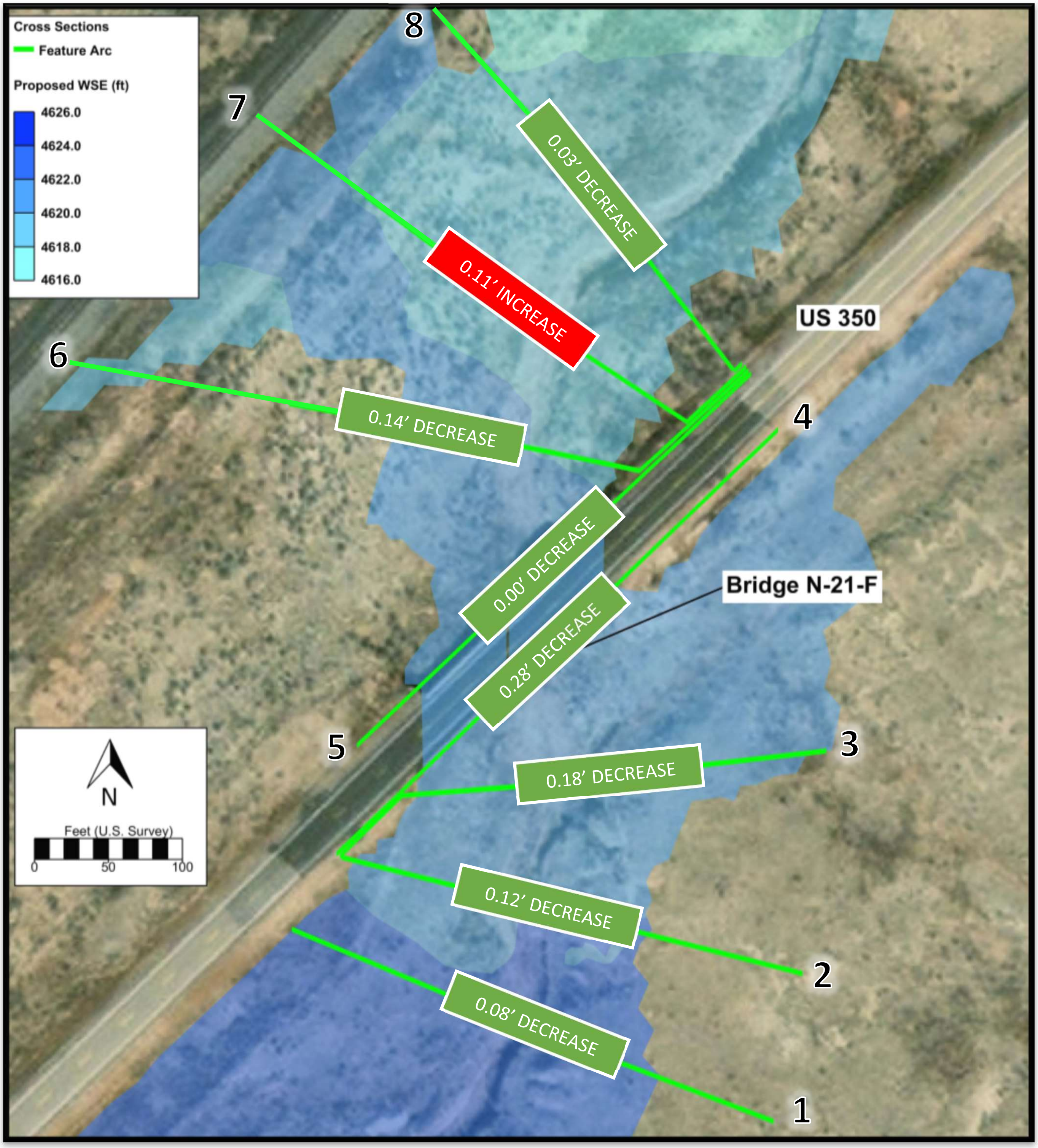


PROPOSED 100-YEAR VELOCITY RESULTS – BRIDGE #2 OPTION

STRUCTURE N-21-F

FIGURE 9

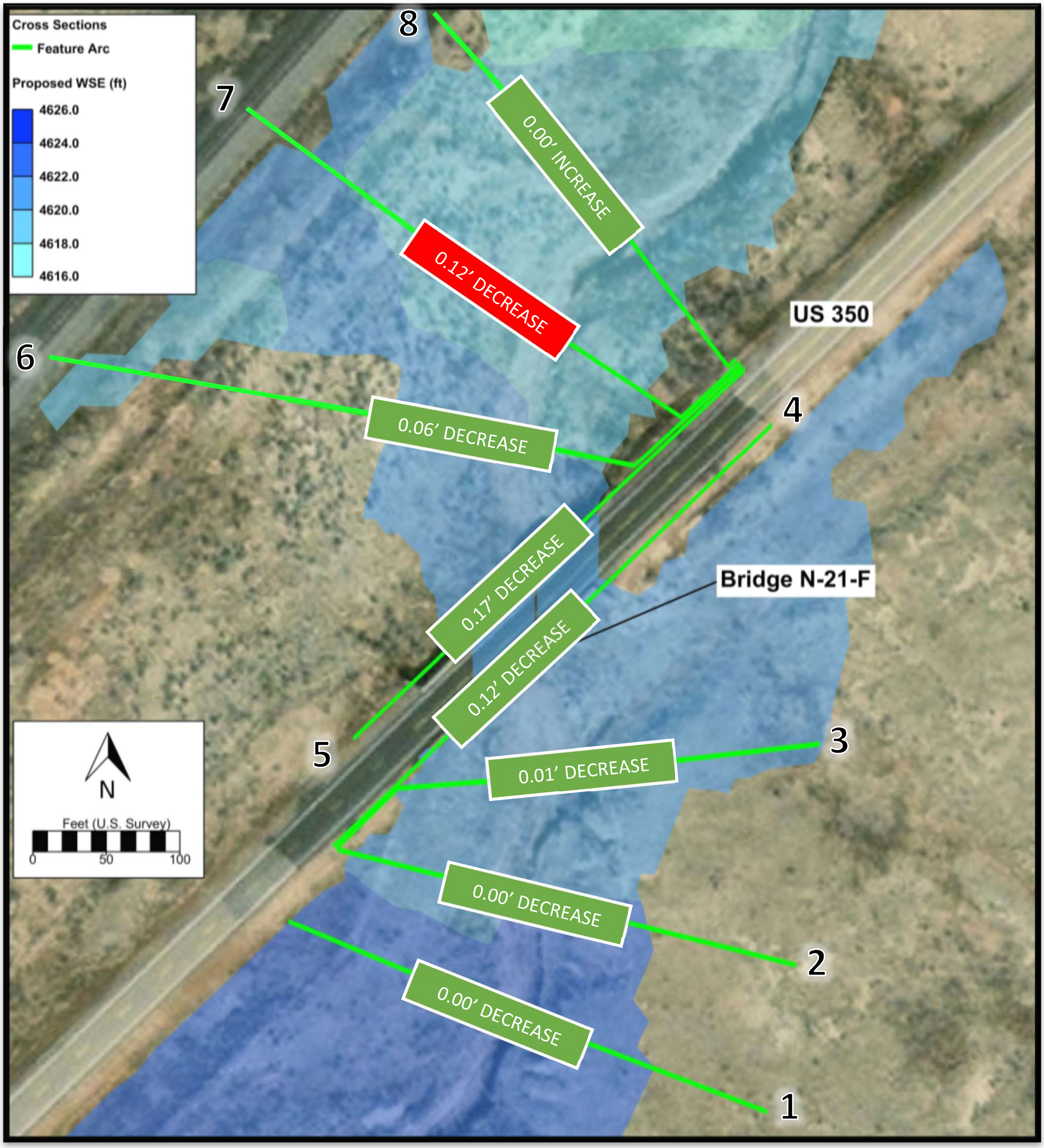
APPENDIX F WATER SURFACE ELEVATION COMPARISON GRAPHICS



FLOODPLAIN CROSS SECTIONS – BRIDGE #1 OPTION

STRUCTURE N-21-F

FIGURE 10

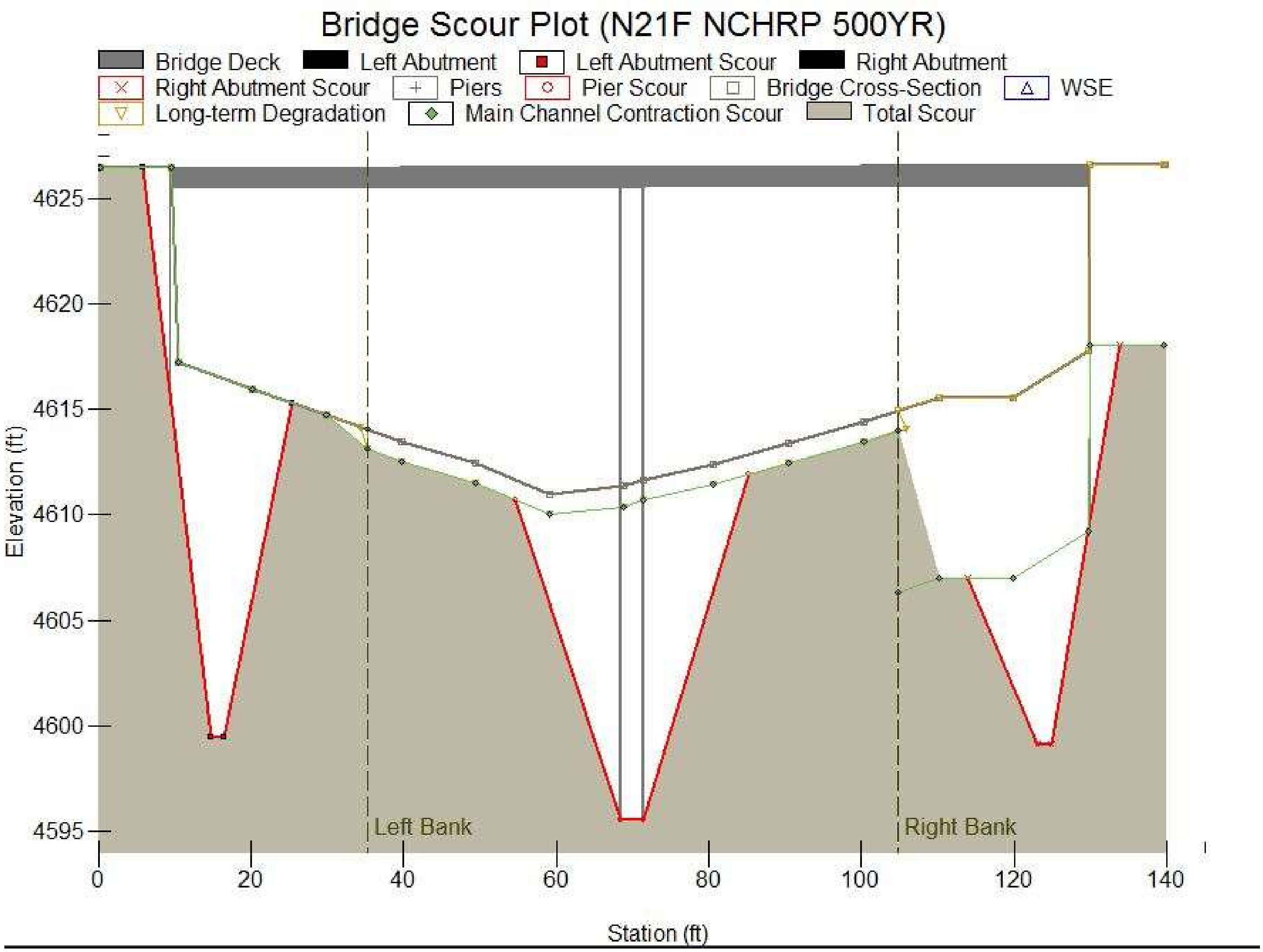


FLOODPLAIN CROSS SECTIONS – BRIDGE #2 OPTION

STRUCTURE N-21-F

FIGURE 11

APPENDIX G BRIDGE SCOUR ANALYSIS


Bridge Scour Plot (N21F 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
4625
4620
4615
4610
4605
4600
4595
Elevation (ft)
Left Bank
Right Bank
0
20
40
60
80
100
120
140
Station (ft)

Hydraulic Analysis Report

Project Data

Project Title: N-12-F 100YR

Designer: Stanley Consultants

Project Date: Wednesday, December 9, 2020

Project Units: U.S. Customary Units

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 maximum velocity is used.

Input Parameters

Riprap Type: Abutment/Guide Bank

The structure is a guidebank

Set-back Length: 10 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: 8.23 ft

Flow Depth at Toe of Abutment: 3.784 ft

Calculations will use either total or overbank discharges.

Total Discharge: 4355 cfs

Overbank Discharge: 262 cfs

Total Bridge Area: 523 ft^2

Setback Area: 62.54 ft^2

Maximum Channel Velocity: 8.33 ft/s

Specific Gravity of Riprap: 2.65

Result Parameters

Set-back ratio: 1.21507

Characteristic Velocity: 8.32696 ft/s

Froude Number at the Abutment Toe: 0.754672

Abutment Coefficient: 1.02

Computed D50: 15.9869 in

Design D50 = 18 in

Thickness = 36 in

Design D50 > Computed D50

18 in > 15.9869 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: 432 in

Minimum Horizontal Extent of the Toe Apron from the Abutment Toe: 7.568 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 maximum velocity is used.

Input Parameters

Riprap Type: Abutment/Guide Bank

The structure is a guidebank

Set-back Length: 20 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: 8.23 ft

Flow Depth at Toe of Abutment: 3.076 ft

Calculations will use either total or overbank discharges.

Total Discharge: 4355 cfs

Overbank Discharge: 262 cfs

Total Bridge Area: 523 ft^2

Setback Area: 111.5 ft^2

Maximum Channel Velocity: 8.33 ft/s

Specific Gravity of Riprap: 2.65

Result Parameters

Set-back ratio: 2.43013

Characteristic Velocity: 8.32696 ft/s

Froude Number at the Abutment Toe: 0.83703

Abutment Coefficient: 0.69

Computed D50: 14.6859 in

Design D50 = 18 in

Thickness = 36 in

Design D50 > Computed D50

18 in > 14.6859 in

Riprap Class

Riprap shape should be angular

Riprap Class Name: CLASS IV

Riprap Class Order: 4

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

d100: 30 in

d85: 21 in

d50: 15.5 in

d15: 10.5 in

Layout Recommendations

Minimum Riprap Thickness: 360 in

Minimum Horizontal Extent of the Toe Apron from the Abutment Toe: 6.152 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

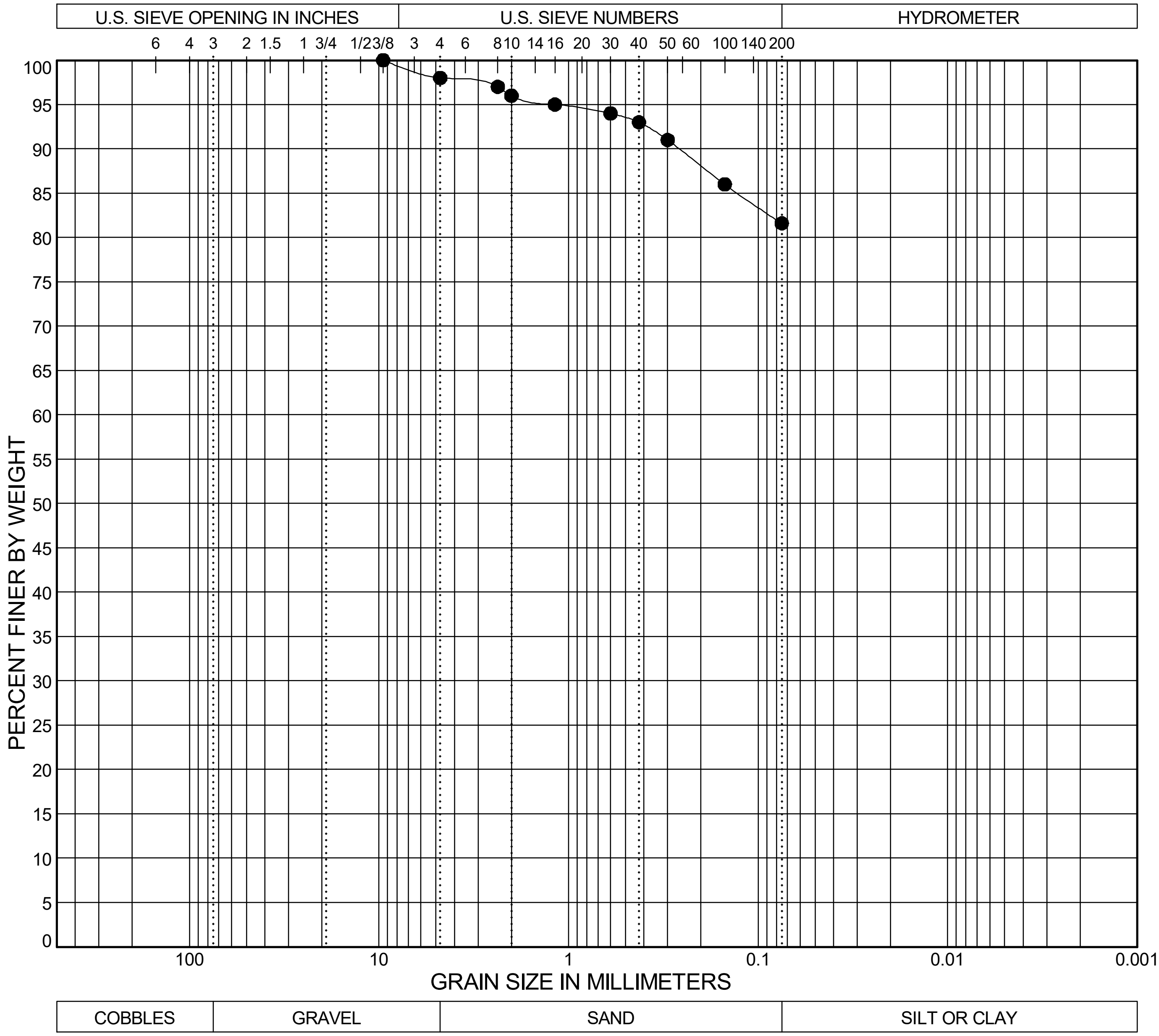
Yeh and Associates, Inc.
Geotechnical • Geological • Construction Services

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	%Clay
● N-21-F Scour	0.0						2.0	16.4	81.6	

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

SIEVE ANALYSIS

FIGURE

S- 8

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