COUNTY ROAD N OVER ALKALI CREEK BRIDGE REPLACEMENT HYDRAULIC REPORT CDOT PROJECT NO. 22521

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1.0 INTRODUCTION

The purpose of this project is to replace the bridge carrying County Rd N over Alkali Creek. This report outlines the bridge hydrologic and hydraulic analyses, minimum freeboard requirements, and scour results for the new County Rd N bridge spanning Alkali Creek. The new bridge will be designed to protect against scour and mitigate channel erosion and stream instability.

The project is located in Montezuma County. The bridge is located approximately 1,500 feet southwest from the intersection SR 22 and County Rd N. The project site is approximately 5 miles northwest of Cortez, Colorado. The project includes demolition of existing structures, roadway improvements, local drainage improvements, channel improvements, and the new bridge structure. Figure 1 provides a vicinity map of the project location.

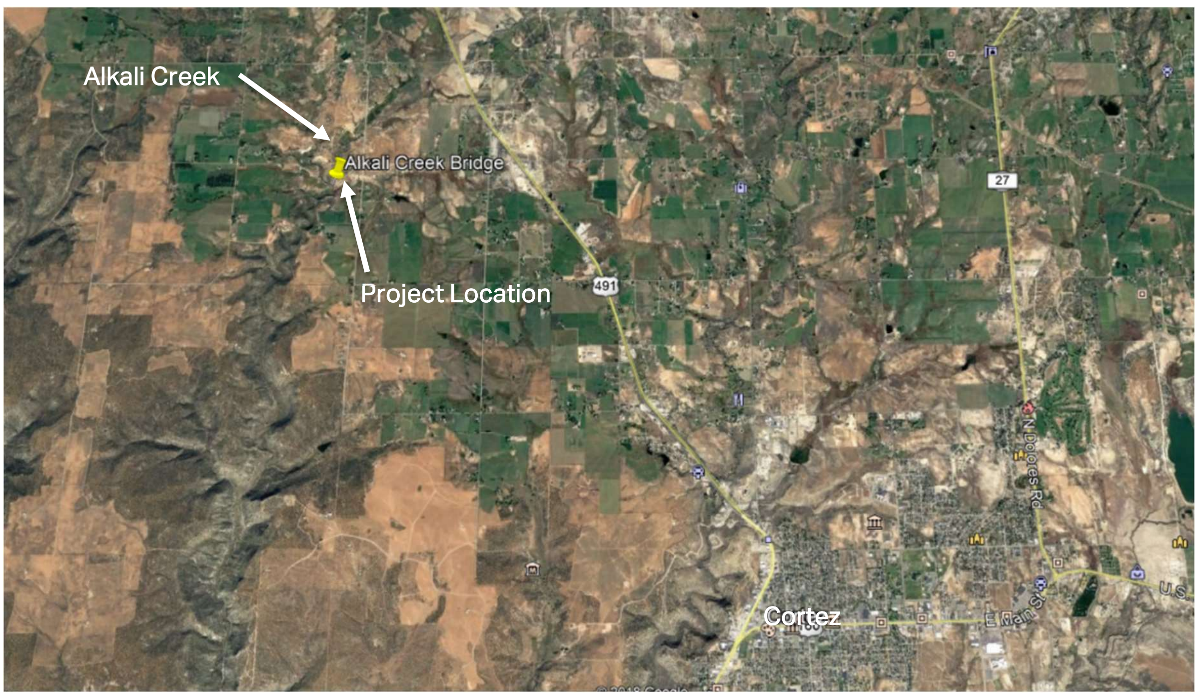


Figure 1. Vicinity Map

The design considers the span length, structure depth, County Rd N roadway profile, and channel grading. The structure needs to safely convey the 25-year flow while meeting freeboard requirements (see Section 4.4) and protect against scour (see Section 5.0). Figure 2 provides a map of the project area. Existing site photos may be found in Appendix A.

Figure 2. Project Area

2.0 BACKGROUND

The existing County Rd N structure, structure number 083000N01.80016, is located in Montezuma County at Alkali Creek, Latitude: 37°23′51.50″N, Longitude: 108°38′58.40″. The structure is a 30′-5″ foot long, single-span steel girder bridge that was built approximately 40 years ago. The bridge has a 27′-10″ clear span with vertical abutments built on piles. The bridge is owned and maintained by Montezuma County.

The structure spans Alkali Creek. Alkali Creek flows from north to south with approximate channel slope of 0.4%. The channel consists of a variable sandy clay soil loess deposit. The bridge is aligned with a skew angle of 90 degrees.

The structure received a sufficiency rating of 48 and was deemed structurally deficient based on an inspection report completed on November 2, 2016. The bridge has heavily corroded metal cribbing at the east abutment and northwest and northeast wingwalls.

3.0 HYDROLOGY

The Alkali Creek watershed is approximately 36.9 square miles. The land use within the basin is largely undeveloped but has several agricultural fields. The watershed drains generally north to south with the highest and lowest elevations of 6,750, and 6,163, respectively. The mean basin slope is approximately 1.5%. Figure 3 illustrates the drainage basin for the County Rd N crossing over Alkali Creek, which was obtained from the USGS software program StreamStats. A portion of the watershed flows into Narraguinnep Reservoir where it is detained. Information regarding the release of water from the reservoir was unable to be obtained. Therefore, to simulate the most conservative hydrologic conditions, the hydraulic effects of the reservoir on its tributary area were ignored.

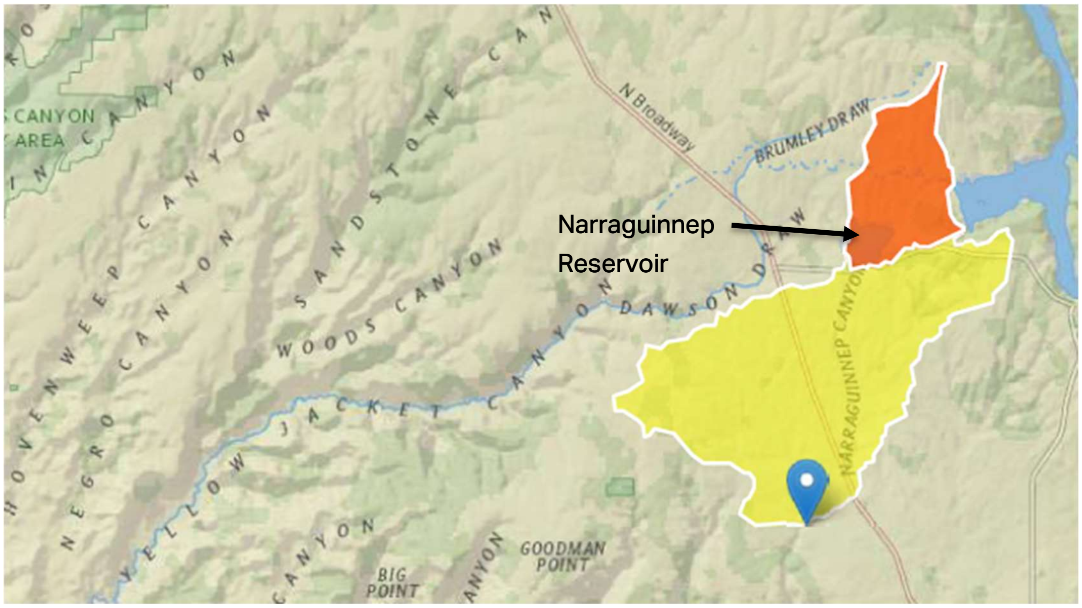


Figure 3. Drainage Basin at County Rd N Crossing Alkali Creek

There are no known published hydrology studies for Alkali Creek and the watershed is currently ungauged. Therefore, the peak discharges for Alkali Creek were estimated by comparing the results from different hydrologic methods. The following sections describe the different hydrologic methods that were analyzed.

3.1 USGS STREAMSTATS

StreamStats is an online Geographic Information System (GIS) developed by the United States Geologic Survey (USGS) and can provide stream flow data and drainage basin characteristics. The peak flows are estimated within the program by utilizing the published USGS Regional Regression equations applicable to the watershed. The only needed input into the online StreamStats program is the latitude and longitude of the outlet location (i.e. the location of the bridge structure over Alkali Creek). A comparison of the StreamStats results are provided in Table 1.

3.2 HEC-HMS MODEL

A program called Watershed Modeling System (Version 11.0) was used to gather basin parameters and composite curve numbers based off of internet databases. This data was imported into HEC-HMS to run hydrology calculations. HEC-HMS (Version 4.2) is a rainfall runoff model developed by the U.S. Army Corps of Engineers (USACE). HEC-HMS simulates the surface runoff response to precipitation of a river basin as an interconnected system of hydrologic and hydraulic components. Two hydrology methods, the frequency storm method and the SCS method, were analyzed in HEC-HMS. The frequency storm method uses statistical data to produce balanced storms with a specific exceedance probability. The SCS hypothetical storm method implements primary precipitation distributions using Natural Resources Conservation Service (NRCS) criteria TR-55. The HEC-HMS model assumed the following:

- Drainage Area: The Alkali Creek watershed is approximately 36.9 miles as delineated in Figure 3.
- Rainfall: All rainfall data was gathered from NOAA 14. The frequency method utilizes rainfall distributions for different storm durations. The SCS method utilizes the 24-hour point precipitation values. A summary of the rainfall data is below in Table 1.

Table 1. Atlas 14 Point Rainfall Depths

Rainfall Duration	25 Year (inches)	100 Year (inches)	500 Year (inches)
5 minutes	0.44	0.67	1.04
15 minutes	0.78	1.20	1.85
1 hour	1.32	1.91	2.73
2 hours	1.59	2.23	3.08
3 hours	1.71	2.35	3.17
6 hours	1.87	2.50	3.30
12 hours	2.00	2.61	3.40
24 hours	2.27	2.86	3.64

- Storm Distribution: For the SCS method, the SCS Type II hypothetical storm distribution was used to model the precipitation within the watershed.
- Loss Method: The SCS Curve Number (CN) method was used to determine losses within the watershed. A composite CN of 80 was utilized for the entire watershed per the existing soils and land conditions within the basin.
- Unit Hydrograph: The SCS unit hydrograph method was utilized.
- Lag Method: The SCS lag time method was utilized.

3.3 SUMMARY OF RESULTS

Table 2 provides a comparison of results from the hydrologic methods that were analyzed.

Table 2. Peak Discharges per Hydrologic Method

Storm Event	USGS StreamStats (cfs)	Frequency Method (cfs)	SCS Method (cfs)
Q_{25}	3,570	2,314	2,210
Q_{100}	6,020	3,969	3,563
Q_{500}	13,200	6,428	5,551

As presented in Table 2, the resulting peak flows vary greatly depending on the hydrologic method analyzed. Based on other hydrologic studies in the area, it was determined that StreamStats was unreasonably high. It was decided that of the two commonly used analyses, frequency method and SCS method, the more conservative of the two would be used, which was the frequency method. The design discharges are presented below in Table 3. The model output files and the StreamStats results may be found in Appendix B.

Table 3. Design Discharges at County Rd N Crossing over Alkali Creek

Storm Event	Peak Flow (cfs)
Q_{25}	2,314
Q_{100}	3,969
Q_{500}	6,428

4.0 HYDRAULIC ANALYSIS

4.1 SOURCE OF MAPPING AND HYDRAULIC MODEL

Project survey and topographic mapping in the vicinity of the County Rd N crossing over Alkali Creek was completed in 2019. The horizontal datum for the survey is modified Colorado State Plane coordinates North American Datum (NAD) 83 (2011). The vertical datum used for survey and the model was NAVD 88 and all elevations in this report reference the NAVD 88 datum.

Using proposed construction plans, existing as-built information, and the survey; the model's bridge geometries, waterway opening approach and exit sections were modeled and necessary adjustments were made to ensure the existing ground and channel conditions at Alkali Creek were modeled correctly. The velocity and depth information obtained from the HEC-RAS model were used in scour analysis, as well as sizing the riprap used for the scour countermeasures.

4.2 FLOODPLAIN INVESTIGATION

The floodplain for Alkali Creek is designated as Zone A with no Base Flood Elevation by the Federal Emergency Management Agency (FEMA). Based on the Flood Insurance Rate Map (FIRM) and the hydraulic model, the 25-year, 100-year, and 500-year flows will be contained within Alkali Creek. The FIRM is included in the Appendix. The project is not located within a regulatory floodway. Also, the bridge replacement will not cause a rise in the 100-year floodplain.

4.3 HYDRAULIC MODEL RESULTS

HEC-RAS version 5.0.7 was used to simulate the 25-year, 50-year, 100-year, and 500-year flows through the proposed structure number 083000N01.80016, carrying County Rd N over Alkali Creek. Manning's roughness coefficients were chosen based on aerial images of the existing topography. Ineffective flow areas were derived upstream and downstream of the bridges by projecting a 1:1 contraction slope and a 2:1 expansion slope out from the upstream and downstream bridge abutments. The contraction and expansion coefficients were modified to 0.3 and 0.5 respectively upstream and downstream of the bridge. Complete HEC-RAS output from the 25-year, 50-year, 100-year, and 500-year events are located in Appendix C.

4.4 FREEBOARD REQUIREMENTS

Chapter 10 of the CDOT Drainage Design Manual states requirements for bridge freeboard. For low to moderate debris streams, the freeboard shall be a minimum of 2 feet where practical. The elevation of the water surface 50 to 100 feet upstream of the bridge should be the elevation to which the freeboard is to be subtracted.

The proposed bridge provides 6.2 feet of freeboard in the 25-year event. The water surface elevation was taken from Cross Section 10466.14, which is located approximately 85 feet upstream of the bridge. Table 4 below summarizes the freeboard calculations.

Table 4. Freeboard at County Rd N Crossing Over Alkali Creek

Low Girder Elevation	25-Yr WSEL	Required Freeboard (ft)	Actual Freeboard (ft)
6179.12	6172.96	2.0	6.2

5.0 BRIDGE SCOUR ANALYSIS

A bridge scour analysis was performed in conformance with Table 10.1 of the CDOT Drainage Design Manual and HEC-18 which lists the scour design flood as the 50-year event and the scour check flood as the 100-year event since the proposed bridged is designed for the 25-year event. The 500-year scour was also analyzed. As recommended by HEC-18, channel horizontal stability, long-term stream degradation, contraction scour, pier scour, and abutment scour were analyzed at the proposed structure 083000N01.80016. The following sections provide the basis for the data used in the scour analysis and the corresponding scour results.

5.1 SITE GEOLOGY

Trautner Geotech, LLC performed a geotechnical analysis of the bridge site on April 25, 2019 and completed a geotechnical report May 17, 2019. The report presents geotechnical observations from field activities and design recommendations. The sieve analysis showed the D50 particle size was less than the #200 sieve and was classified as CL (Sandy Lean Clay). The NRCS web soil survey confirmed the small particle size, classifying the area as Gladel-Pulpit complex with a sandy loam classification. Therefore, a D50 particle size of 0.075mm (the #200 sieve) was used for the scour calculations.

5.2 SCOUR PARAMETERS

Scour is the erosion of streambed or bank material due to flowing water. The 50-year, 100-year and 500-year event was analyzed for scour at the proposed bridge using the procedures described in HEC-18 and were checked using the FHWA Hydraulic Toolbox. The following sections describe channel horizontal stability, stream degradation, and the development of the equation parameters for pier scour, contraction scour, and abutment scour that was analyzed for the proposed bridge on County Rd N crossing Alkali Creek.

5.2.1 CHANNEL HORIZONTAL STABILITY

Alkali Creek has a sharp turn directly upstream of the County Rd N bridge. Historical photos were analyzed and no channel meandering was observed over time. Also, based on photos of the site, no signs of active channel migration were observed. Therefore, scour problems due to channel horizontal instability are not anticipated.

5.2.2 STREAM DEGRADATION

A CDOT inspection report from 2016 notes that a bridge foundation was undermined by scour, however based on the streambed history, the stream has experienced minor aggradation since 2012. The overall slope through the channel reach is 0.40%. There appears to be a grade control structure approximately 800 feet downstream of the project location. Based on the mild stream slope, the aggradation in the inspection reports, and the potential downstream control, stream degradation is not anticipated at the bridge.

5.2.3 CONTRACTION SCOUR

Contraction scour results from the constriction of the flow area at the bridge, which causes an increase in velocity and the removal of material from the bed across the channel. Shear stress on the stream bed at the bridge is further divided into live bed and clear water scour. The parameters used in the contraction scour equations were determined from simulating the 50-year, 100-year and 500-year flood event in the HEC-RAS model. Cross section 10466.14, which is located approximately 85 feet upstream of the bridge, was chosen as the main channel section. This section accurately represents the general channel geometry upstream of the bridge and was compared to the bridge cross section to determine the magnitude of the contraction scour.

Based on average velocity and critical velocity calculations, the live-bed contraction scour equation was used. Due the shape of the channel, the average depths of were calculated to determine instead of using the width top or bottom width of the channel. The flow area and channel flows were determined from the HEC-RAS flow distribution for each cross section. Complete contraction scour calculations are located in Appendix E.

5.2.4 PIER SCOUR

The proposed bridge does not have piers, therefore there will be no pier scour.

5.2.5 ABUTMENT SCOUR

Abutment scour occurs locally and results in the removal of material around the abutments due to the acceleration of flow as the velocity increases through the bridge. The 50-year, 100-year and 500-year events encroach on the sloped abutments. The NCHRP Project 24-20 method was used as is recommended by FHWA to calculate the abutment scour. Complete abutment scour calculations are located in Appendix E.

5.3 SCOUR RESULTS

Table 4 summarizes the results of the scour analysis for the 50-year, 100-year and 500-year event.

Table 4. Summary of Scour Results

Flood Event	Long Term Aggradation / Degradation (ft)	Contraction Scour (ft)	Abutment Scour (ft)	
			Left	Right
50-Year	0.0	0.0	0.0	0.0
100-Year	0.0	0.8	2.4	2.4
500-Year	0.0	1.2	8.6	8.6

5.4 SCOUR PREVENTION

Due to the steep channel banks and the bend prior to the bridge, riprap protection was designed to withstand the predicted velocities on the channel banks for the 25-year event. The proposed riprap has a D50 of 18", a thickness of 3 feet and extends 2 feet above the design water surface. The riprap spans the channel for a majority of the project limits to prevent weak points at the channel bottom which is approximately 10 feet wide. In areas with riprap on only one bank, the riprap is toed in 3 feet. The riprap will be keyed in with a trench at the upstream limits. A geotextile filter will be placed under the abutment riprap to prevent migration of fines underlying the channel bed material. The limits of the riprap bank protection are shown on the Bridge Hydraulic Information Sheet and Riprap Site Plan Sheet attached in Appendix F.

6.0 CONCLUSION

This report presents supporting information and results from the hydrologic, hydraulic, and scour analysis completed for the design of proposed structure number 083000N01.80016 carrying County Rd N over Alkali Creek in Montezuma County. Additional information and calculations are included in the attached appendices.

7.0 REFERENCES

Colorado Department of Transportation (CDOT), 2004. *Drainage Design Manual.* PDF

Urban Drainage and Flood Control District, 2017. *Urban Storm Drainage Criteria Manual: Volume 1 Management, Hydrology, and Hydraulics (USDCM)*, prepared by the Urban Drainage and Flood Control District, Denver, CO. PDF.

U.S. Army Corps of Engineers (USACE), 2010. *HEC-RAS Version 5.0.7.*

U.S. Army Corps of Engineers (USACE), 2010. Hydraulic Modeling System HEC-HMS *Version 4.2.*

U.S Department of Transportation, Federal Highway Administration, 2012. *Hydraulic Engineering Circular No. 18 – Evaluating Scour at Bridges – Fourth Edition.*

U.S Department of Transportation, Federal Highway Administration, 2009. *Hydraulic Engineering Circular No. 23 - Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance – Third Edition.*

U.S. Geological Survey - StreamStats program for Colorado. *http://water.usgs.gov/osw/streamstats/colorado.html*.

APPENDIX A

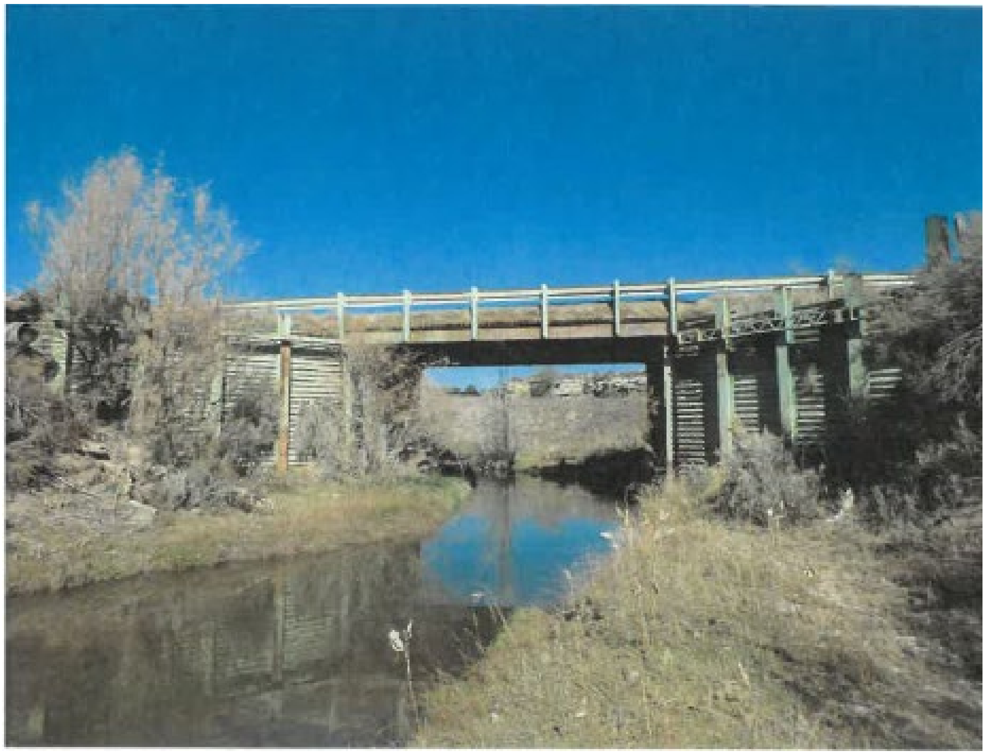
SITE PHOTOS

Looking East at the Upstream Face of Structure

Looking at Alkali Creek Upstream from Structure

Looking at Alkali Creek Downstream from Structure

County Rd N Looking East



Bridge Elevation Looking North

Bridge Corrosion on Wing Wall

APPENDIX B

HYDROLOGIC ANALYSIS

RESPEC

property inundation and associated damage is judged to be severe, a higher design frequency should be considered. The design discharge used in an area that has FEMA mapped floodplain shall be the 100-year discharge.

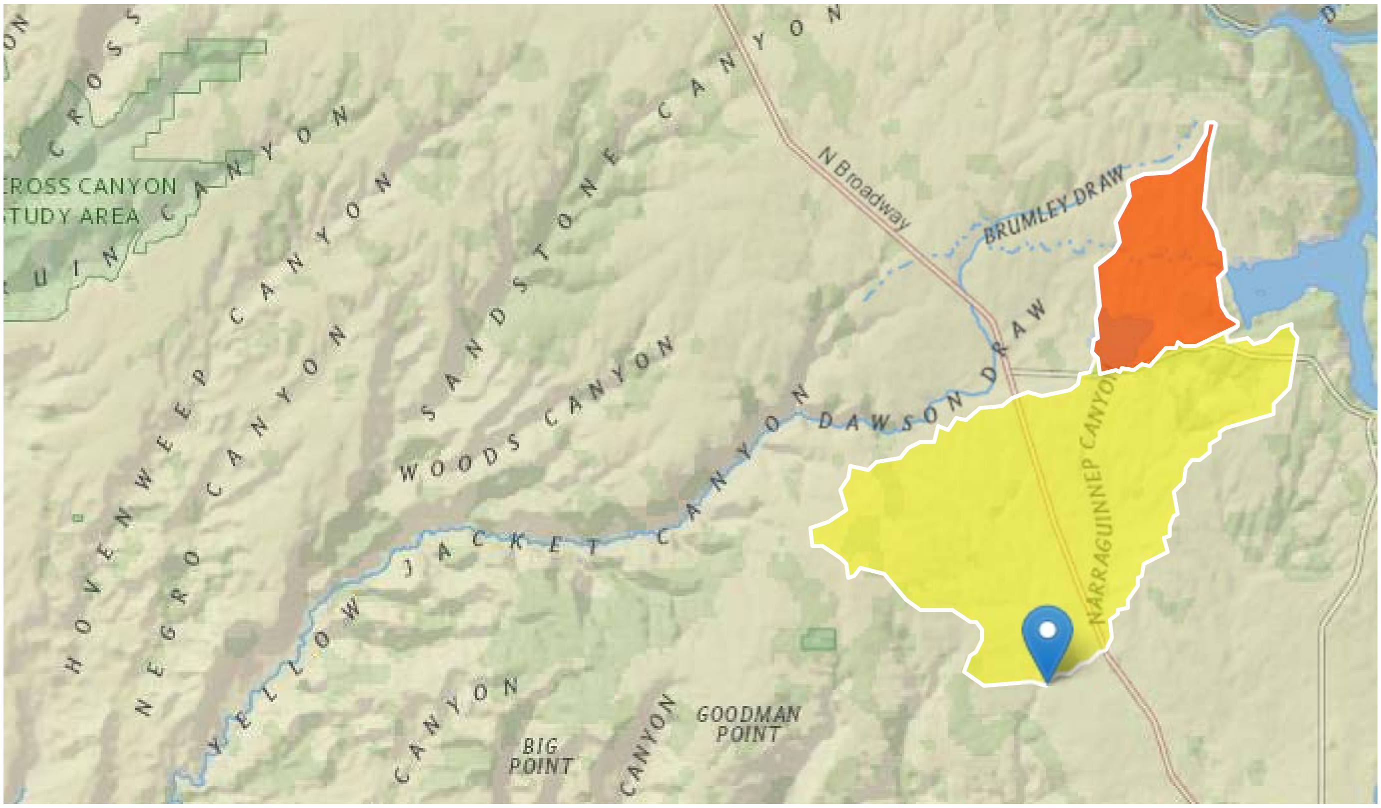
Table 7.2 Table of Design Frequencies

Drainage Type		**Frequency**
A.	Cross Drainage	
	Multilane Roads - including interstate	
	In Urban Areas	100-year*
	In Rural Areas	50-year
	Two-Lane Roads	
	In Urban Areas	100-year
	In Rural Areas	
	Q_{50} : $\geq$ 4000 cfs	50-year
	Q_{50} < 4000 cfs	25-year
	Culvert Outlet Scour Protection	10-year
	Pedestrian Walkways and Bikeways	2 to 5-year
	Bridge Foundation Scour	100 and 500-year
B.	Parallel Drainage	
	Roadway Overtopping and Revetment	Same as for Cross Drainage
	Side Drains	2 to 10-year#
C.	Storm Drains	
	Major System	100-year
	Minor System	2 to 5-year
D.	Detour Culverts	monthly discharges for 2 to 5-year

Notes: *Urban cross culverts (not Interstate); if Q_{100} < 100 cfs, consider designing the culvert using the storm drain Minor System Frequency.
#Side drains shall not cause water to flow onto the highway at a greater probability than applies to cross drainage.

StreamStats Report Montezuma bridge Replacement

Region ID: CO
Workspace ID: CO20181010193239264000
Clicked Point (Latitude, Longitude): 37.39737, -108.64959
Time: 2018-10-10 13:32:58 -0600



Basin Characteristics

Parameter Code	Parameter Description	Value	Unregulated Value	Unit
DRNAREA	Area that drains to a point on a stream	37.2	29.803	square miles
EL7500	Percent of area above 7500 ft	0	0.000	percent

General Disclaimers

Upstream regulation was checked for this watershed.

This watershed is 19.84 percent regulated, computed flows may not apply.

Peak-Flow Statistics Parameters [Southwest Region Peak Flow]

Parameter Code	Parameter Name	Value	Unregulated Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	37.2	29.803	square miles	1	4390
EL7500	Percent above 7500 ft	0	0.000	percent	0	99

Peak-Flow Statistics Flow Report [Southwest Region Peak Flow]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unregulated Value	Unit	SEp
2 Year Peak Flood	473	411	ft^3/s	90
5 Year Peak Flood	1270	1110	ft^3/s	71
10 Year Peak Flood	2080	1820	ft^3/s	67
25 Year Peak Flood	3570	3120	ft^3/s	66
50 Year Peak Flood	4970	4360	ft^3/s	67
100 Year Peak Flood	6860	6020	ft^3/s	69
200 Year Peak Flood	8930	7850	ft^3/s	71
500 Year Peak Flood	13200	11600	ft^3/s	75

Peak-Flow Statistics Citations

Capesius, J.P., and Stephens, V. C.,2009, Regional Regression Equations for Estimation of Natural Streamflow Statistics in Colorado: U. S. Geological Survey Scientific Investigations Report 2009-5136, 32 p. (http://pubs.usgs.gov/sir/2009/5136/http://pubs.usgs.gov/sir/2009/5136/)

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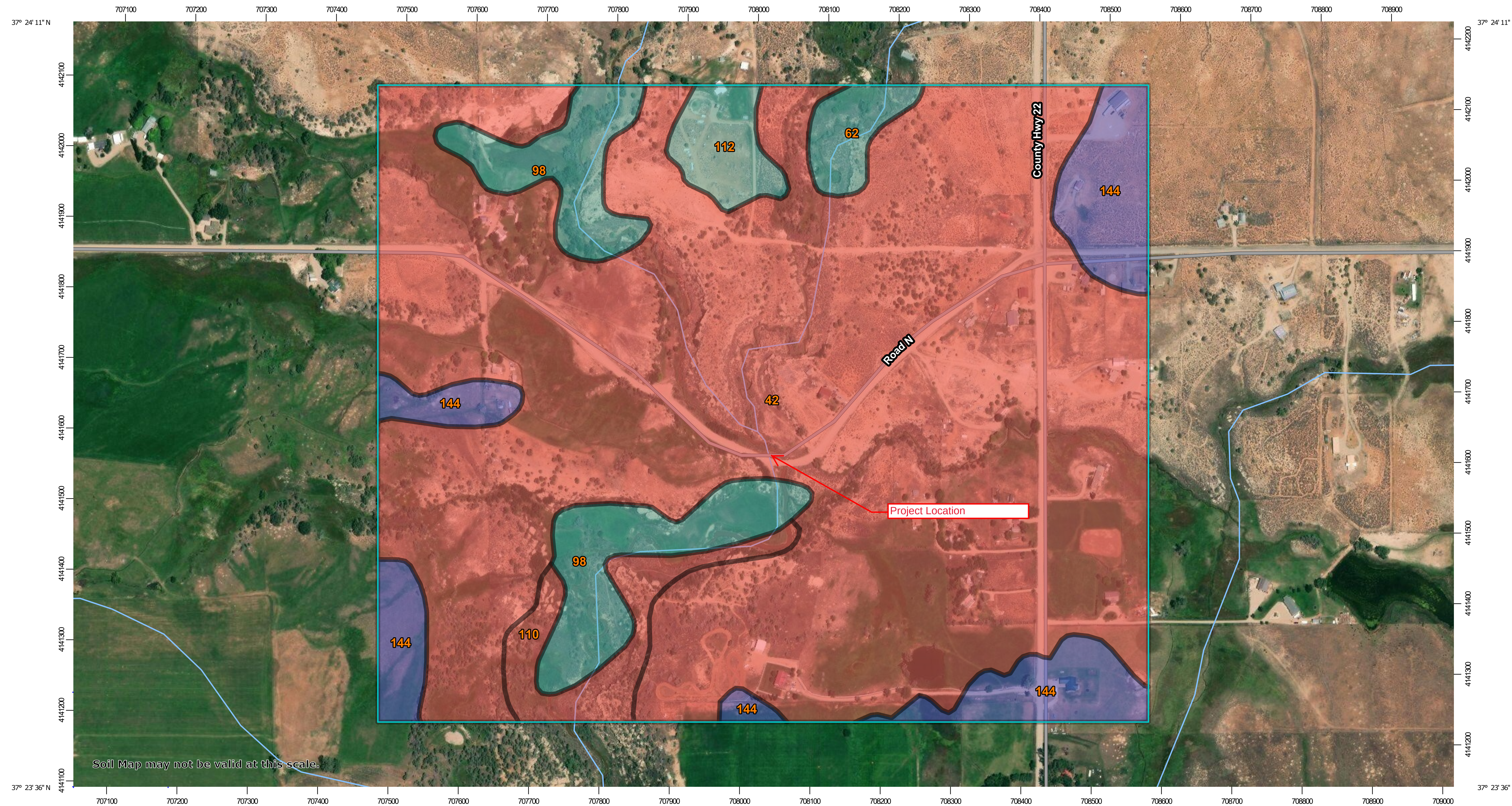
Application Version: 4.2.1



Map Scale: 1:5,290 if printed on B landscape (17" x 11") sheet.

Meters: 0 50 100 200 300

N

Feet: 0 250 500 1000 1500

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 12N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils

Soil Rating Polygons
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Lines
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Points
- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Cortez Area, Colorado, Parts of Dolores and Montezuma Counties
Survey Area Data: Version 13, Jun 5, 2020

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: May 22, 2017—Sep 7, 2017

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
42	Gladel-Pulpit complex, 3 to 9 percent slopes	D	188.8	77.1%
62	Irak loam, 0 to 3 percent slopes	C	3.7	1.5%
98	Ramper loam, 0 to 3 percent slopes	C	18.4	7.5%
110	Romberg, extremely stony-Crosscan, very bouldery-Rock outcrop complex, 25 to 80 percent slopes	D	7.8	3.2%
112	Sharps loam, 3 to 6 percent slopes	C	5.0	2.0%
144	Wetherill loam, 3 to 6 percent slopes	B	21.2	8.7%
Totals for Area of Interest			**245.0**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having 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. These soils have a slow rate of water transmission.

Group D. Soils having 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. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

Cortez Area, Colorado, Parts of Dolores and Montezuma Counties

42—Gladel-Pulpit complex, 3 to 9 percent slopes

Map Unit Setting

National map unit symbol: 2w592
Elevation: 6,200 to 7,400 feet
Mean annual precipitation: 13 to 16 inches
Mean annual air temperature: 45 to 50 degrees F
Frost-free period: 100 to 120 days
Farmland classification: Not prime farmland

Map Unit Composition

Gladel and similar soils: 45 percent
Pulpit and similar soils: 35 percent
Minor components: 20 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Gladel

Setting

Landform: Dip slopes on cuestas
Down-slope shape: Convex
Across-slope shape: Convex
Parent material: Eolian deposits over residuum weathered from sandstone

Typical profile

A - 0 to 3 inches: fine sandy loam
Bw - 3 to 11 inches: sandy loam
Bk - 11 to 18 inches: sandy loam
R - 18 to 59 inches: bedrock

Properties and qualities

Slope: 3 to 9 percent
Depth to restrictive feature: 12 to 20 inches to lithic bedrock
Drainage class: Well drained
Capacity of the most limiting layer to transmit water (Ksat): Low to moderately high (0.01 to 0.57 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum content: 15 percent
Maximum salinity: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Available water capacity: Very low (about 2.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7s

Hydrologic Soil Group: D
Ecological site: R036XY141CO - Shallow Loamy Mesa Top - (Pinyon-Juniper)
Hydric soil rating: No

Description of Pulpit

Setting

Landform: Dip slopes on cuestas
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Eolian deposits over residuum weathered from sandstone

Typical profile

A - 0 to 3 inches: silt loam
Bt - 3 to 10 inches: silt loam
Bk - 10 to 24 inches: silt loam
2R - 24 to 59 inches: bedrock

Properties and qualities

Slope: 3 to 9 percent
Depth to restrictive feature: 20 to 40 inches to lithic bedrock
Drainage class: Well drained
Capacity of the most limiting layer to transmit water (Ksat): Low to moderately high (0.01 to 0.57 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum content: 10 percent
Maximum salinity: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Available water capacity: Low (about 3.9 inches)

Interpretive groups

Land capability classification (irrigated): 4e
Land capability classification (nonirrigated): 4e
Hydrologic Soil Group: C
Ecological site: R036XY142CO - Loamy Mesa Top - (Pinyon-Juniper)
Hydric soil rating: No

Minor Components

Rock outcrop

Percent of map unit: 10 percent
Hydric soil rating: Unranked

Wetherill

Percent of map unit: 5 percent
Landform: Dip slopes on cuestas
Down-slope shape: Linear
Across-slope shape: Linear
Ecological site: R036XY284CO - Loamy Foothills

Hydric soil rating: No

Dolcan

Percent of map unit: 5 percent
Landform: Dip slopes on cuestas
Down-slope shape: Convex
Across-slope shape: Linear
Ecological site: R036XY111CO - Steep Shallow Clay Loam - (Pinyon-Utah Juniper)
Hydric soil rating: No

Data Source Information

Soil Survey Area: Cortez Area, Colorado, Parts of Dolores and Montezuma Counties
Survey Area Data: Version 13, Jun 5, 2020

Project: wmsexport Simulation Run: freq25yr

Start of Run:	20Aug2019, 15:41	Basin Model:	WMS Watershed
End of Run:	21Aug2019, 15:41	Meteorologic Model:	freq25yr
Compute Time:	23Aug2019, 08:24:07	Control Specifications:	WMS Control Info

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
1B	36.9320	2314.2	21Aug2019, 08:11	0.59
3C	36.9320	2314.2	21Aug2019, 08:11	0.59

Project: wmsexport Simulation Run: freq50yr

Start of Run:	20Aug2019, 15:41	Basin Model:	WMS Watershed
End of Run:	21Aug2019, 15:41	Meteorologic Model:	freq50yr
Compute Time:	23Aug2019, 08:26:13	Control Specifications:	WMS Control Info

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
1B	36.9320	3095.1	21Aug2019, 07:56	0.77
3C	36.9320	3095.1	21Aug2019, 07:56	0.77

Project: wmsexport Simulation Run: freq100yr

Start of Run:	20Aug2019, 15:41	Basin Model:	WMS Watershed
End of Run:	21Aug2019, 15:41	Meteorologic Model:	freq100yr
Compute Time:	23Aug2019, 08:25:49	Control Specifications:	WMS Control Info

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
1B	36.9320	3968.6	21Aug2019, 07:56	0.97
3C	36.9320	3968.6	21Aug2019, 07:56	0.97

Project: wmsexport Simulation Run: freq500yr

Start of Run:	20Aug2019, 15:41	Basin Model:	WMS Watershed
End of Run:	21Aug2019, 15:41	Meteorologic Model:	freq500yr
Compute Time:	31Oct2019, 14:02:31	Control Specifications:	WMS Control Info

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
1B	36.9320	6428.3	21Aug2019, 07:56	1.52
3C	36.9320	6428.3	21Aug2019, 07:56	1.52

Project: wmsexport Simulation Run: SCS25yr

Start of Run:	20Aug2019, 15:41	Basin Model:	WMS Watershed
End of Run:	21Aug2019, 16:41	Meteorologic Model:	SCS25yr
Compute Time:	31Oct2019, 14:01:58	Control Specifications:	SCS Control Info

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
1B	36.9320	2210.1	21Aug2019, 07:56	0.66
3C	36.9320	2210.1	21Aug2019, 07:56	0.66

Project: wmsexport Simulation Run: SCS100yr

Start of Run:	20Aug2019, 15:41	Basin Model:	WMS Watershed
End of Run:	21Aug2019, 16:41	Meteorologic Model:	SCS100yr
Compute Time:	31Oct2019, 14:01:37	Control Specifications:	SCS Control Info

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
1B	36.9320	3563.0	21Aug2019, 07:56	1.03
3C	36.9320	3563.0	21Aug2019, 07:56	1.03

Project: wmsexport Simulation Run: SCS500yr

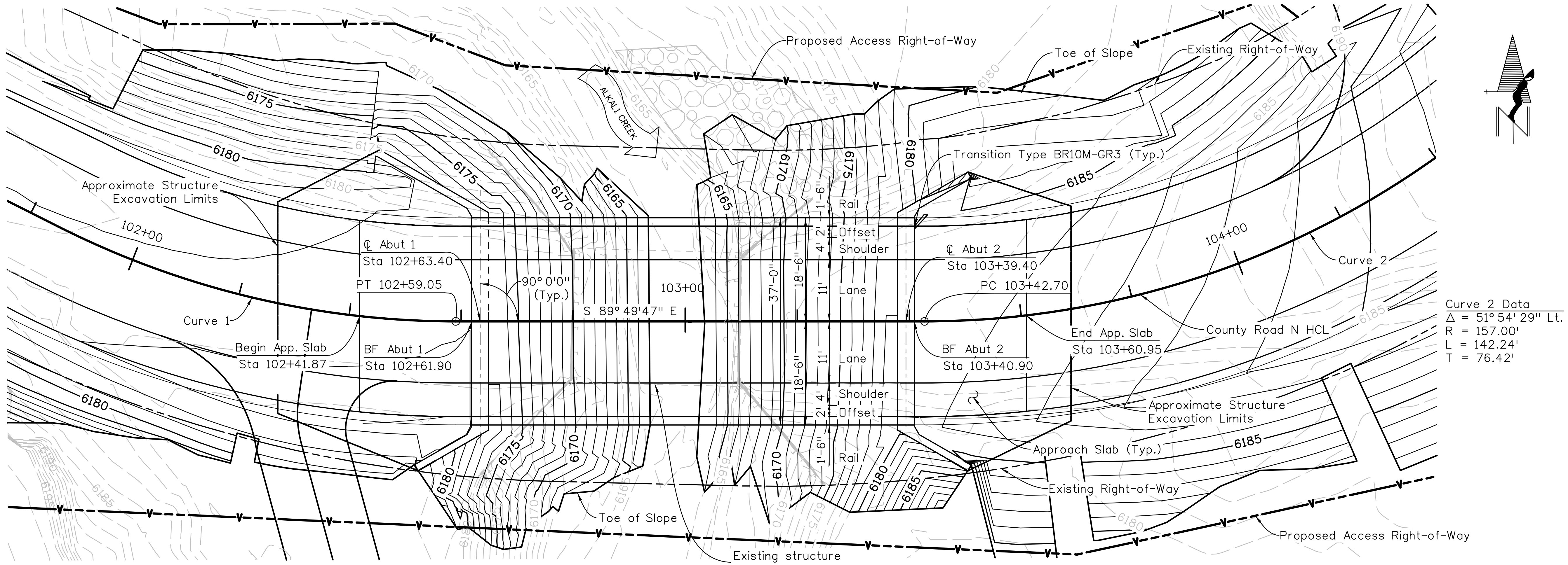
Start of Run:	20Aug2019, 15:41	Basin Model:	WMS Watershed
End of Run:	21Aug2019, 16:41	Meteorologic Model:	SCS500yr
Compute Time:	31Oct2019, 14:01:02	Control Specifications:	SCS Control Info

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
1B	36.9320	5550.6	21Aug2019, 07:41	1.59
3C	36.9320	5550.6	21Aug2019, 07:41	1.59

APPENDIX C

HYDRAULICS RESULTS

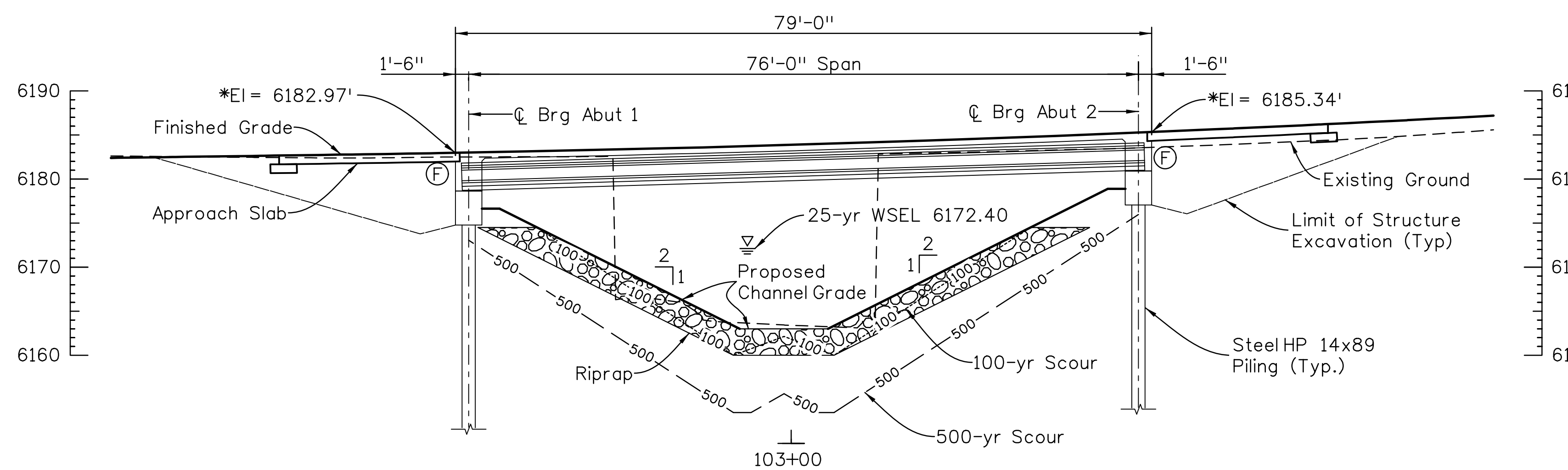
FOR PLANS



Curve 1 Data
Δ = 26° 46' 17" Lt.
R = 157.00'
L = 73.36'
T = 37.36'

Curve 2 Data
Δ = 51° 54' 29" Lt.
R = 157.00'
L = 142.24'
T = 76.42'

PLAN



ELEVATION
(Taken along County Road N HCL)
* Elevations are at finished grade

Know what's below.
Call before you dig.

0' 5' 10' 20'

Design			Detail			Quantities		
	INITIAL	DATE		INITIAL	DATE		INITIAL	DATE
Designed By	DMA	2/21	Detailed By	DMA	2/21	Quantities By	DMA	2/21
Checked By	XXX	X/XX	Checked By	MRM	3/21	Checked By	XXX	X/XX

adams 2:47:34 PM I:\0-Projects\ACTIVE PROJECTS\52701_Montezuma County Road N over Alkali Creek sub to Bechtolt\800_Computer Design Files\802_Sheet Files\B3_GEN LAYOUT.dgn

Print Date: 3/19/2021
File Name: B3_GEN LAYOUT.dgn
Horiz. Scale: 1:20 Vert. Scale: Same
Staff Bridge Branch: Unit Leader:
Prime Consultant: BECHTOLT
Sub-Consultant: RockSol Consulting Group, Inc.

Sheet Revisions		
Date:	Comments	Init.

MONTEZUMA County
109 WEST MAIN STREET
CORTEZ, CO 81321
ROOM 260
PHONE: 970-565-3728
FAX: 970-385-3635

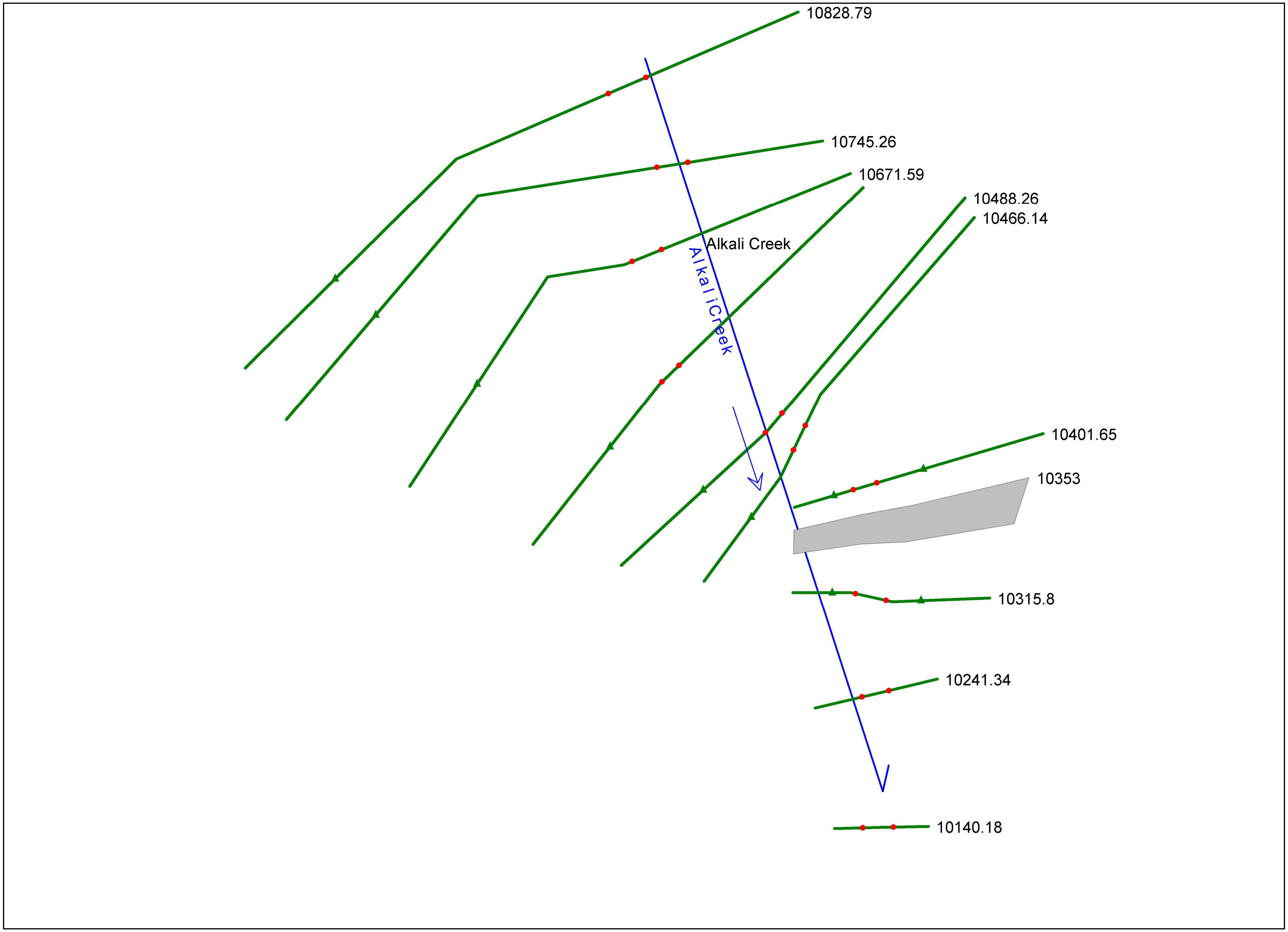
As Constructed
No Revisions:
Revised:
Void:

COUNTY ROAD N OVER ALKALI CREEK
GENERAL LAYOUT
PLAN & ELEVATION

Designer: D. Adams
Detailer: D. Adams
Sheet Subset: BRIDGE
Structure Numbers: 083000N01.79040
Subset Sheets: B3 of

Project No./Code
BRO C320-004
22591
Sheet Number 20

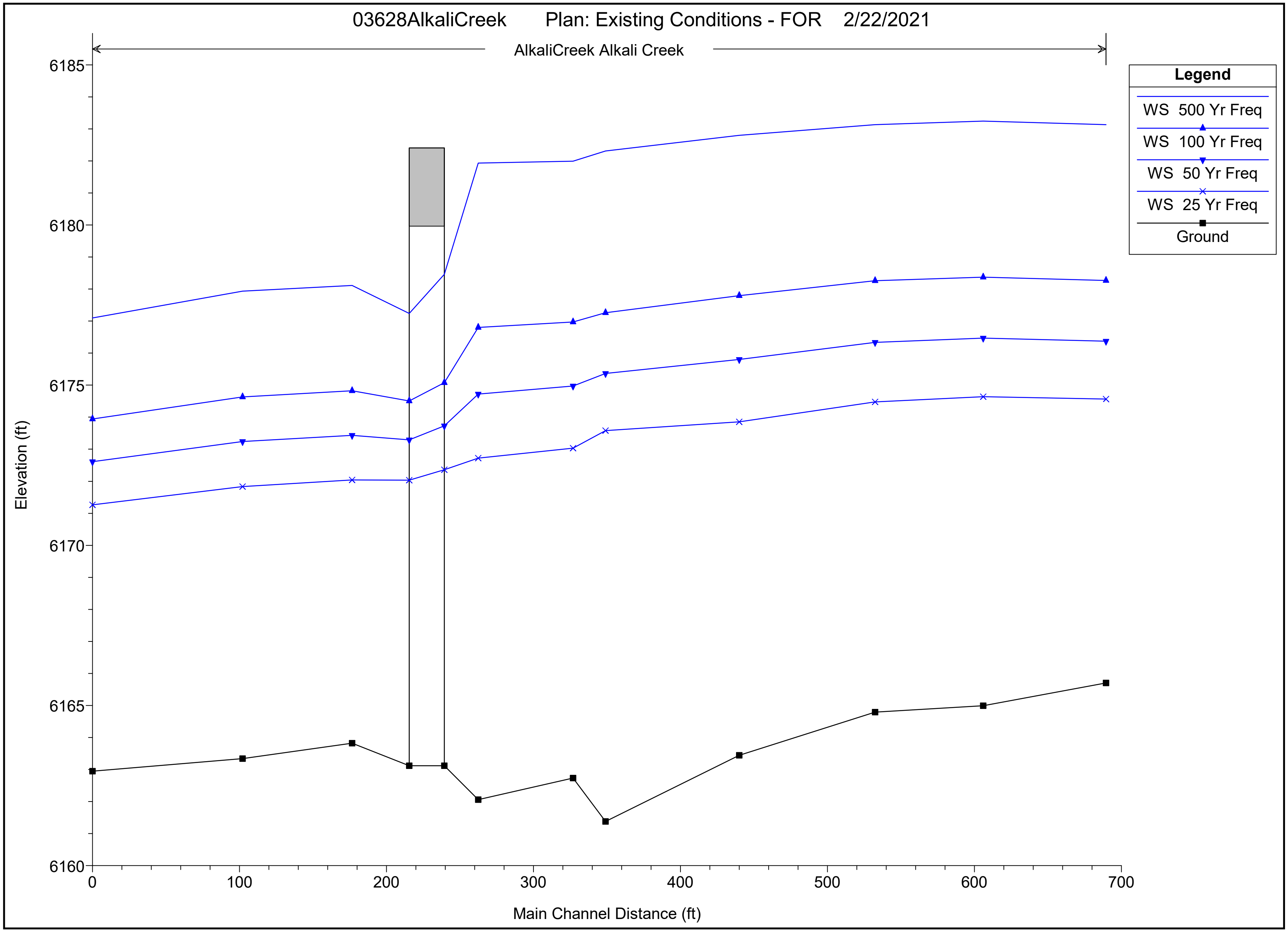
FOR SUBMITTAL - NOT FOR CONSTRUCTION



10828.79
10745.26
10671.59
10488.26
10466.14
Alkali Creek
Alkali Creek
10401.65
10353
10315.8
10241.34
10140.18

HEC-RAS Plan: Ex Cond - FOR River: AlkaliCreek Reach: Alkali Creek

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Alkali Creek	10828.79	500 Yr Freq	6428.30	6165.70	6183.13	6175.79	6183.60	0.000915	6.98	1467.45	220.47	0.32
Alkali Creek	10828.79	100 Yr Freq	3968.60	6165.70	6178.27	6174.04	6178.77	0.001609	7.04	808.16	114.33	0.40
Alkali Creek	10828.79	50 Yr Freq	3095.10	6165.70	6176.38	6173.20	6176.92	0.002242	7.18	601.30	103.37	0.46
Alkali Creek	10828.79	25 Yr Freq	2314.20	6165.70	6174.57	6172.49	6175.16	0.003373	7.36	424.56	92.52	0.54
Alkali Creek	10745.26	500 Yr Freq	6428.30	6164.99	6183.25	6174.09	6183.47	0.000481	5.55	2108.00	269.93	0.24
Alkali Creek	10745.26	100 Yr Freq	3968.60	6164.99	6178.37	6172.52	6178.61	0.000718	5.32	1185.41	152.93	0.28
Alkali Creek	10745.26	50 Yr Freq	3095.10	6164.99	6176.47	6171.70	6176.71	0.000925	5.32	906.15	140.79	0.31
Alkali Creek	10745.26	25 Yr Freq	2314.20	6164.99	6174.64	6170.86	6174.90	0.001249	5.33	660.12	127.85	0.35
Alkali Creek	10671.59	500 Yr Freq	6428.30	6164.79	6183.14	6174.32	6183.43	0.000608	6.33	2100.03	329.94	0.27
Alkali Creek	10671.59	100 Yr Freq	3968.60	6164.79	6178.26	6172.50	6178.54	0.000804	5.77	1111.01	144.69	0.30
Alkali Creek	10671.59	50 Yr Freq	3095.10	6164.79	6176.33	6171.08	6176.63	0.001026	5.79	844.24	132.92	0.33
Alkali Creek	10671.59	25 Yr Freq	2314.20	6164.79	6174.48	6170.59	6174.79	0.001348	5.77	609.47	117.20	0.36
Alkali Creek	10579.28	500 Yr Freq	6428.30	6163.44	6182.80	6175.57	6183.33	0.000962	8.03	1544.54	251.99	0.33
Alkali Creek	10579.28	100 Yr Freq	3968.60	6163.44	6177.79	6173.50	6178.41	0.001470	8.07	770.38	102.81	0.38
Alkali Creek	10579.28	50 Yr Freq	3095.10	6163.44	6175.81	6172.57	6176.47	0.001847	8.15	578.94	89.85	0.42
Alkali Creek	10579.28	25 Yr Freq	2314.20	6163.44	6173.86	6171.54	6174.59	0.002405	8.24	415.56	78.25	0.46
Alkali Creek	10488.26	500 Yr Freq	6428.30	6161.38	6182.32	6174.96	6183.19	0.001301	9.53	1222.91	195.48	0.38
Alkali Creek	10488.26	100 Yr Freq	3968.60	6161.38	6177.26	6172.56	6178.23	0.001863	9.35	628.20	78.40	0.43
Alkali Creek	10488.26	50 Yr Freq	3095.10	6161.38	6175.36	6171.46	6176.27	0.002011	8.85	491.29	65.33	0.44
Alkali Creek	10488.26	25 Yr Freq	2314.20	6161.38	6173.58	6170.29	6174.38	0.002074	8.12	383.83	56.04	0.43
Alkali Creek	10466.14	500 Yr Freq	6428.30	6162.73	6181.99	6176.14	6183.14	0.001556	10.52	1091.78	189.32	0.43
Alkali Creek	10466.14	100 Yr Freq	3968.60	6162.73	6176.97	6173.57	6178.16	0.002207	10.14	562.56	70.21	0.49
Alkali Creek	10466.14	50 Yr Freq	3095.10	6162.73	6174.97	6172.46	6176.19	0.002663	10.00	429.32	63.02	0.52
Alkali Creek	10466.14	25 Yr Freq	2314.20	6162.73	6173.03	6170.88	6174.27	0.003265	9.78	313.61	56.60	0.56
Alkali Creek	10401.65	500 Yr Freq	6428.30	6162.06	6181.93	6176.04	6183.02	0.001476	10.35	906.97	152.03	0.42
Alkali Creek	10401.65	100 Yr Freq	3968.60	6162.06	6176.80	6173.43	6178.01	0.002338	10.49	557.24	75.23	0.51
Alkali Creek	10401.65	50 Yr Freq	3095.10	6162.06	6174.73	6172.39	6176.00	0.002938	10.51	425.33	62.51	0.55
Alkali Creek	10401.65	25 Yr Freq	2314.20	6162.06	6172.72	6171.31	6174.04	0.003719	10.37	309.08	55.00	0.60
Alkali Creek	10353		Bridge									
Alkali Creek	10315.8	500 Yr Freq	6428.30	6163.82	6178.11	6175.05	6180.36	0.003444	13.87	638.47	61.12	0.66
Alkali Creek	10315.8	100 Yr Freq	3968.60	6163.82	6174.82	6172.45	6176.53	0.003592	11.83	446.50	55.64	0.64
Alkali Creek	10315.8	50 Yr Freq	3095.10	6163.82	6173.44	6171.36	6174.92	0.003681	10.91	371.01	53.30	0.63
Alkali Creek	10315.8	25 Yr Freq	2314.20	6163.82	6172.04	6170.30	6173.29	0.003777	9.92	298.90	50.23	0.62
Alkali Creek	10241.34	500 Yr Freq	6428.30	6163.34	6177.94		6180.07	0.003495	13.94	659.82	62.77	0.65
Alkali Creek	10241.34	100 Yr Freq	3968.60	6163.34	6174.64		6176.23	0.003553	11.80	463.41	56.32	0.63
Alkali Creek	10241.34	50 Yr Freq	3095.10	6163.34	6173.24		6174.61	0.003590	10.83	386.81	53.73	0.62
Alkali Creek	10241.34	25 Yr Freq	2314.20	6163.34	6171.84		6172.99	0.003647	9.82	313.03	51.10	0.61
Alkali Creek	10140.18	500 Yr Freq	6428.30	6162.95	6177.10	6174.36	6179.65	0.004000	14.65	602.66	59.07	0.70
Alkali Creek	10140.18	100 Yr Freq	3968.60	6162.95	6173.95	6171.75	6175.82	0.004001	12.31	425.57	53.13	0.67
Alkali Creek	10140.18	50 Yr Freq	3095.10	6162.95	6172.61	6170.63	6174.20	0.004001	11.26	356.36	50.63	0.65
Alkali Creek	10140.18	25 Yr Freq	2314.20	6162.95	6171.26	6169.54	6172.58	0.004008	10.15	289.66	48.10	0.64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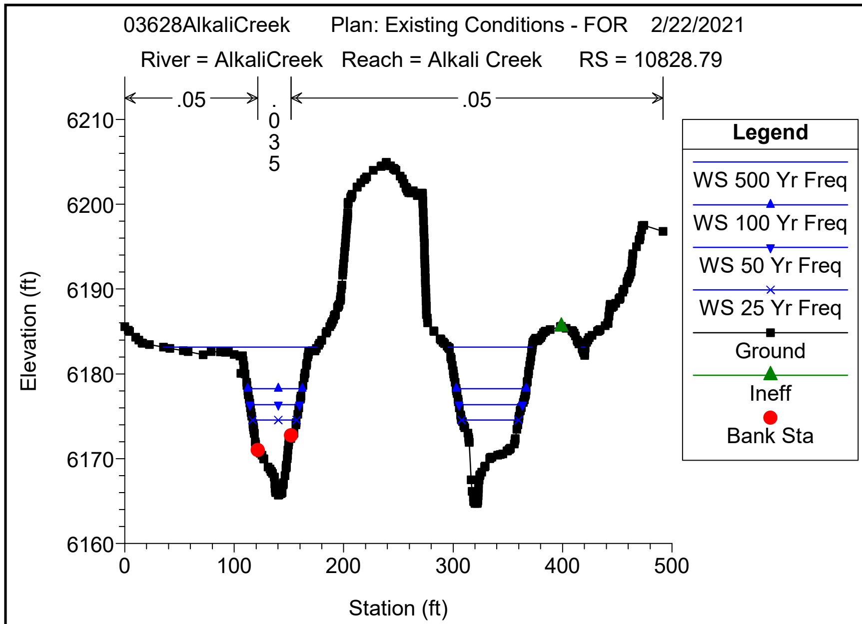
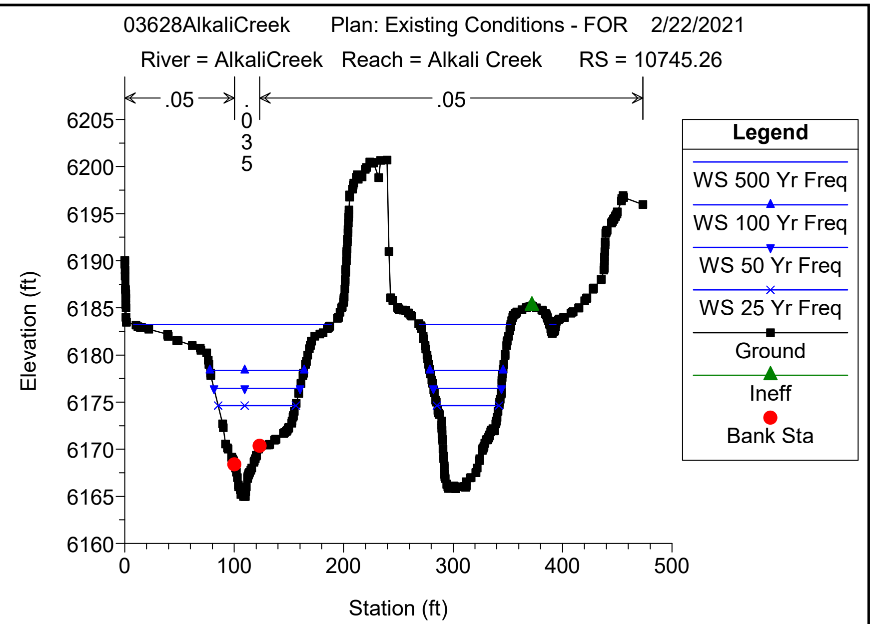
AlkaliCreek Alkali Creek
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Elevation (ft)
6185
6180
6175
6170
6165
6160
0
100
200
300
400
500
600
700
Main Channel Distance (ft)


03628AlkaliCreek Plan: Existing Conditions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10828.79
.05
.035
.05
Elevation (ft)
6160
6170
6180
6190
6200
6210
Station (ft)
0
100
200
300
400
500
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta


03628AlkaliCreek Plan: Existing Conditions - FOR 2/22/2021
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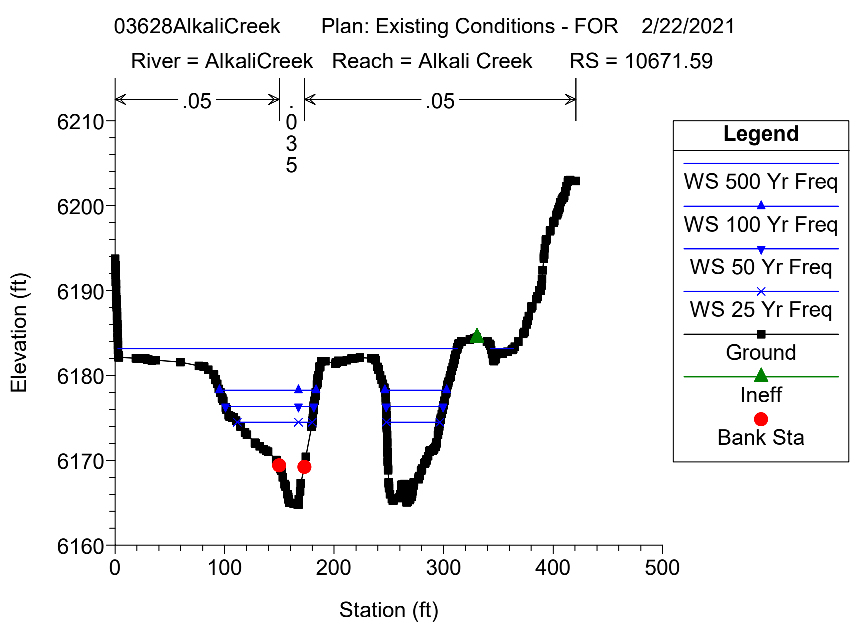
.035
.05
Elevation (ft)
6160
6165
6170
6175
6180
6185
6190
6195
6200
6205
Station (ft)
0
100
200
300
400
500
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta


03628AlkaliCreek Plan: Existing Conditions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10671.59
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.035
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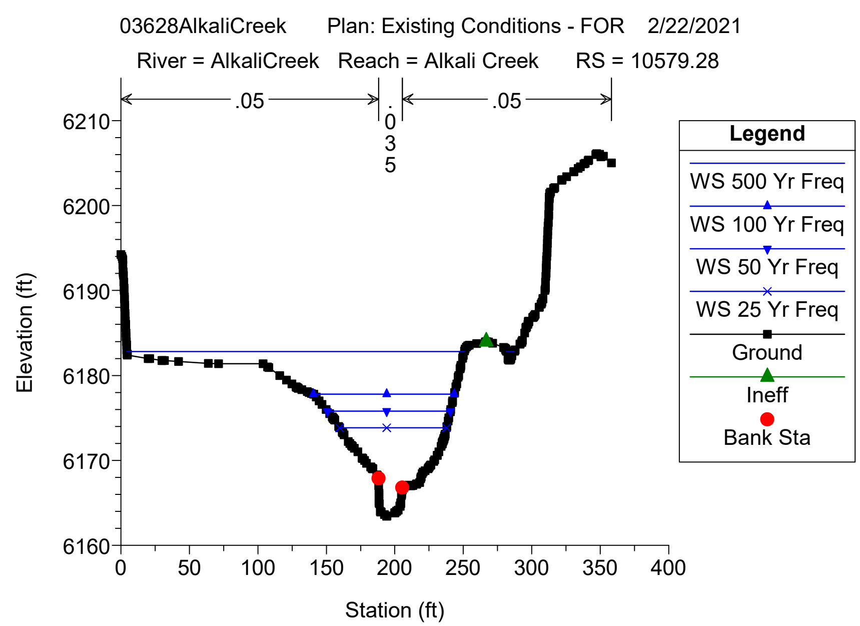
Elevation (ft)
6160
6170
6180
6190
6200
6210
Station (ft)
0
100
200
300
400
500
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta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River = AlkaliCreek Reach = Alkali Creek RS = 10579.28
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.05
Elevation (ft)
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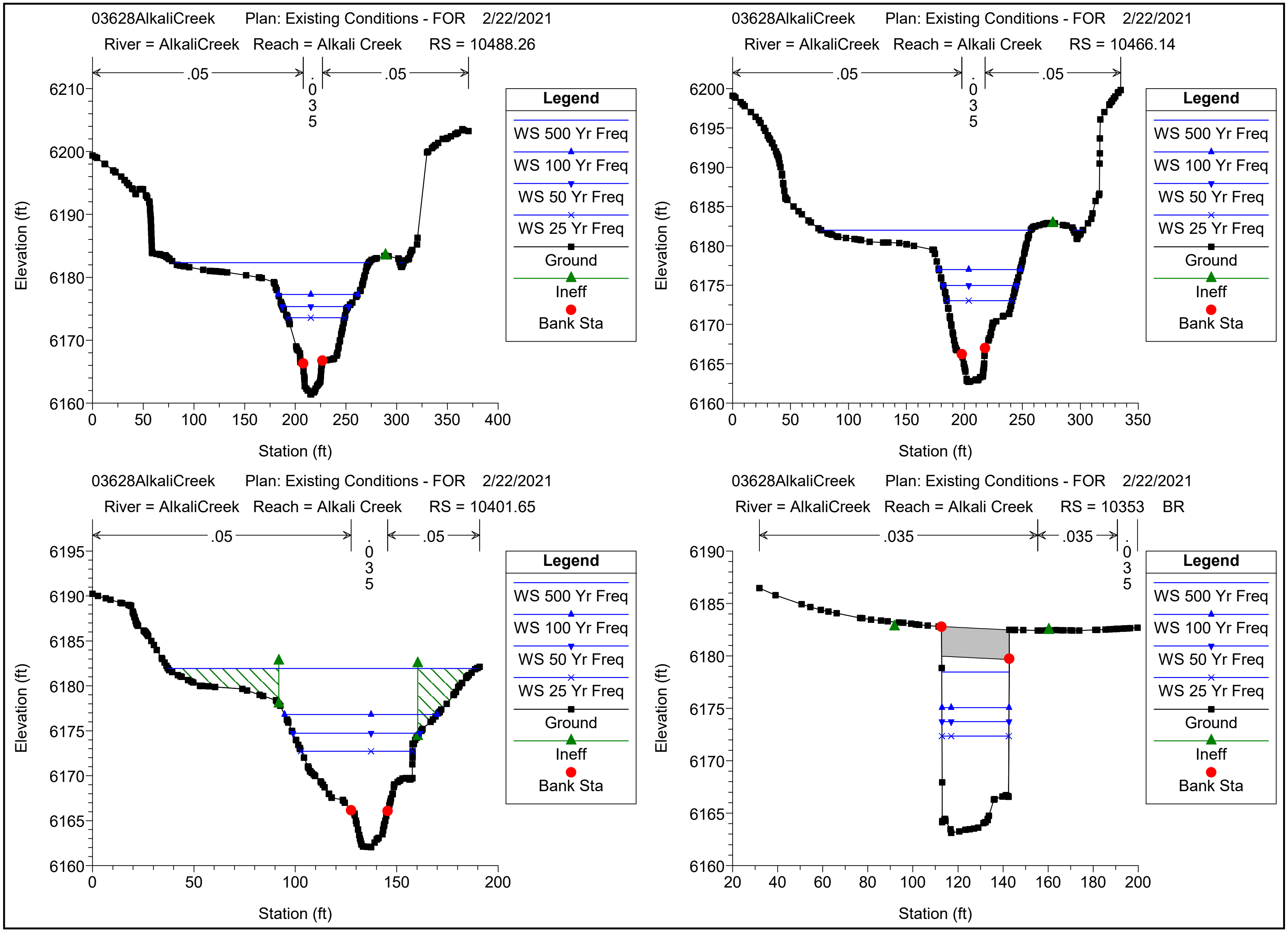
6170
6180
6190
6200
6210
Station (ft)
0
50
100
150
200
250
300
350
400
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta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.05
Elevation (ft)
Station (ft)
Legend
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WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta
03628AlkaliCreek Plan: Existing Conditions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10466.14
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.05
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta
03628AlkaliCreek Plan: Existing Conditions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10401.65
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.035
.05
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta
03628AlkaliCreek Plan: Existing Conditions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10353 BR
.035
.035
.035
Elevation (ft)
Station (ft)
Legend
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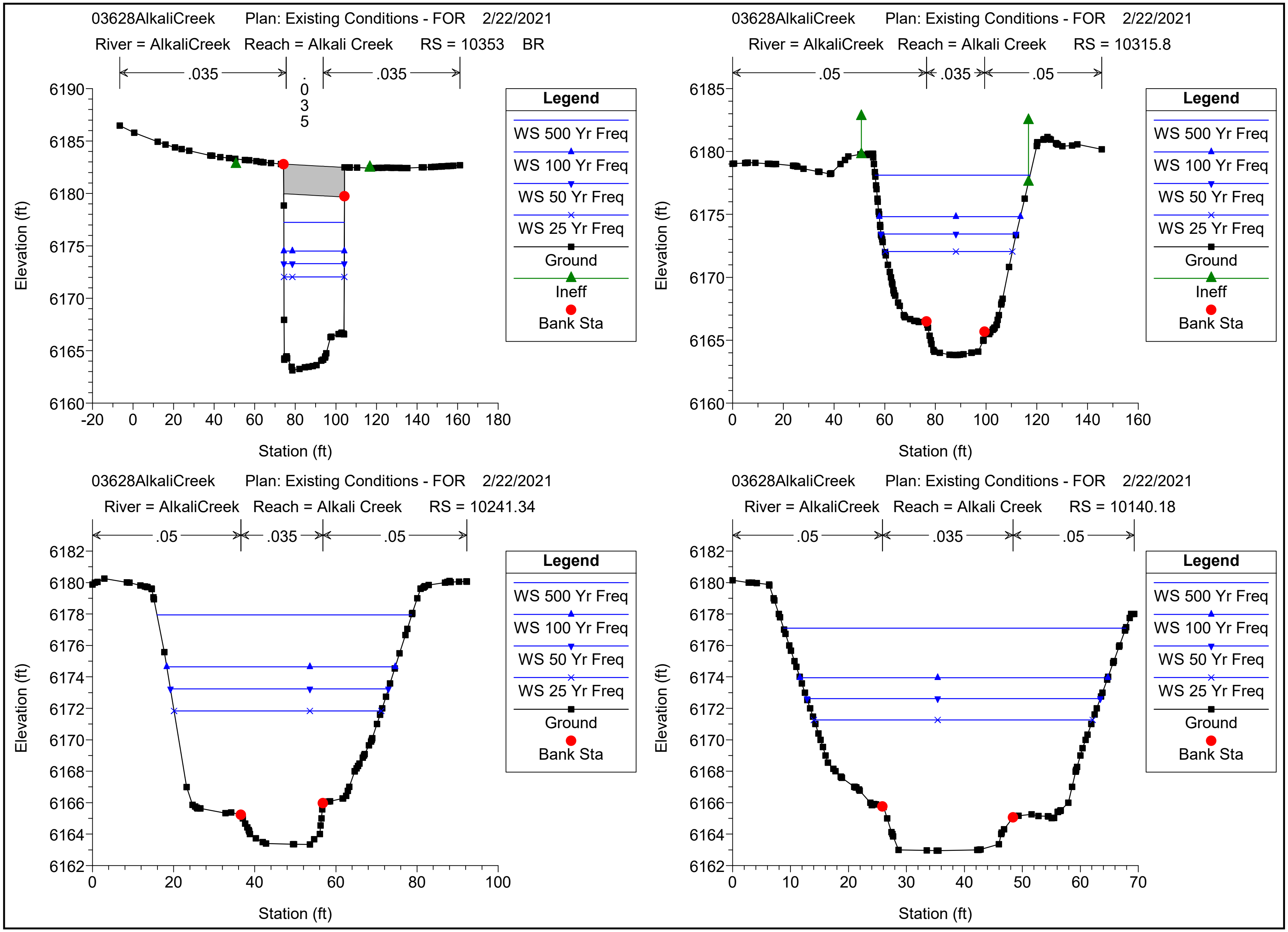
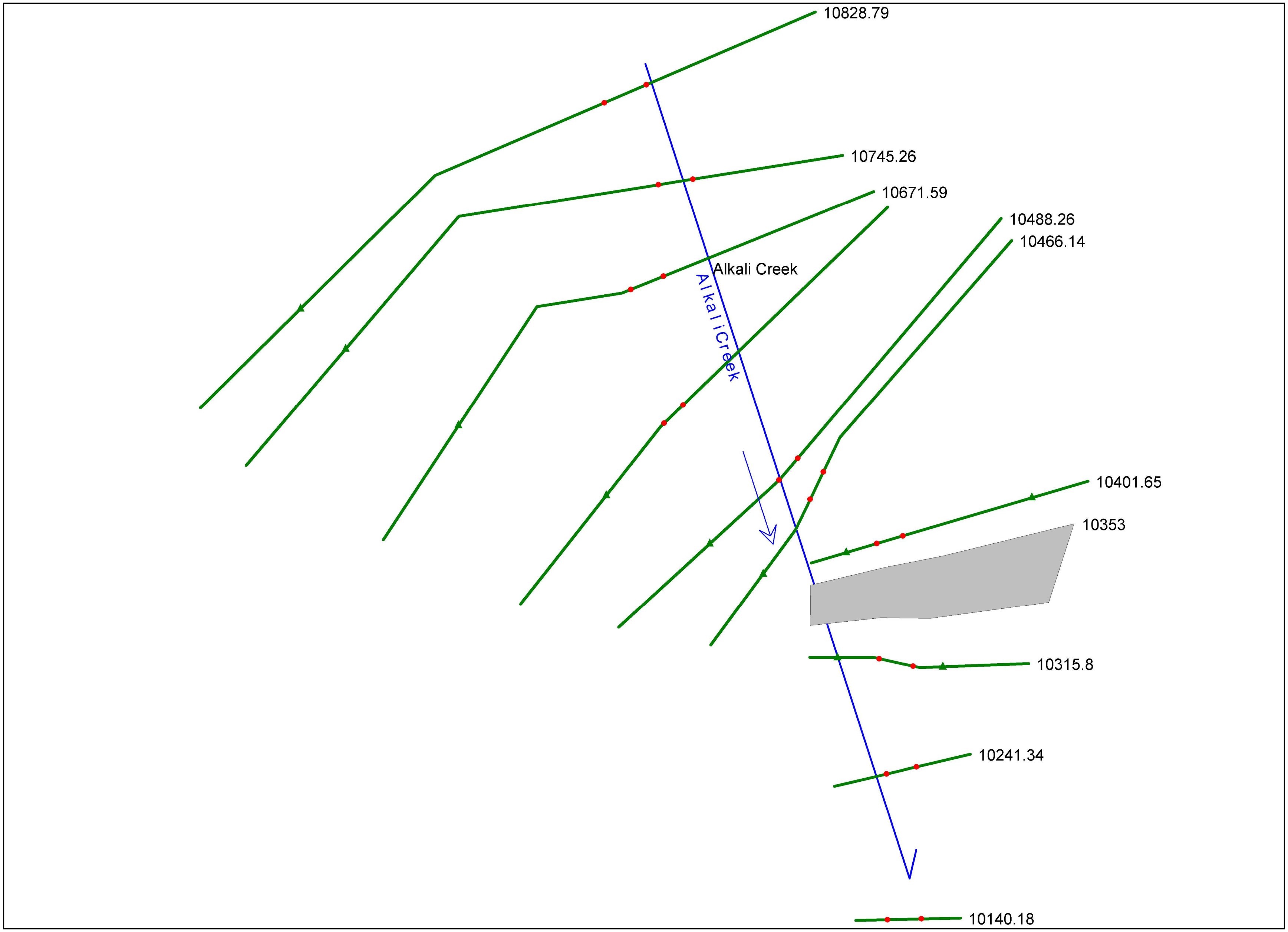
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta

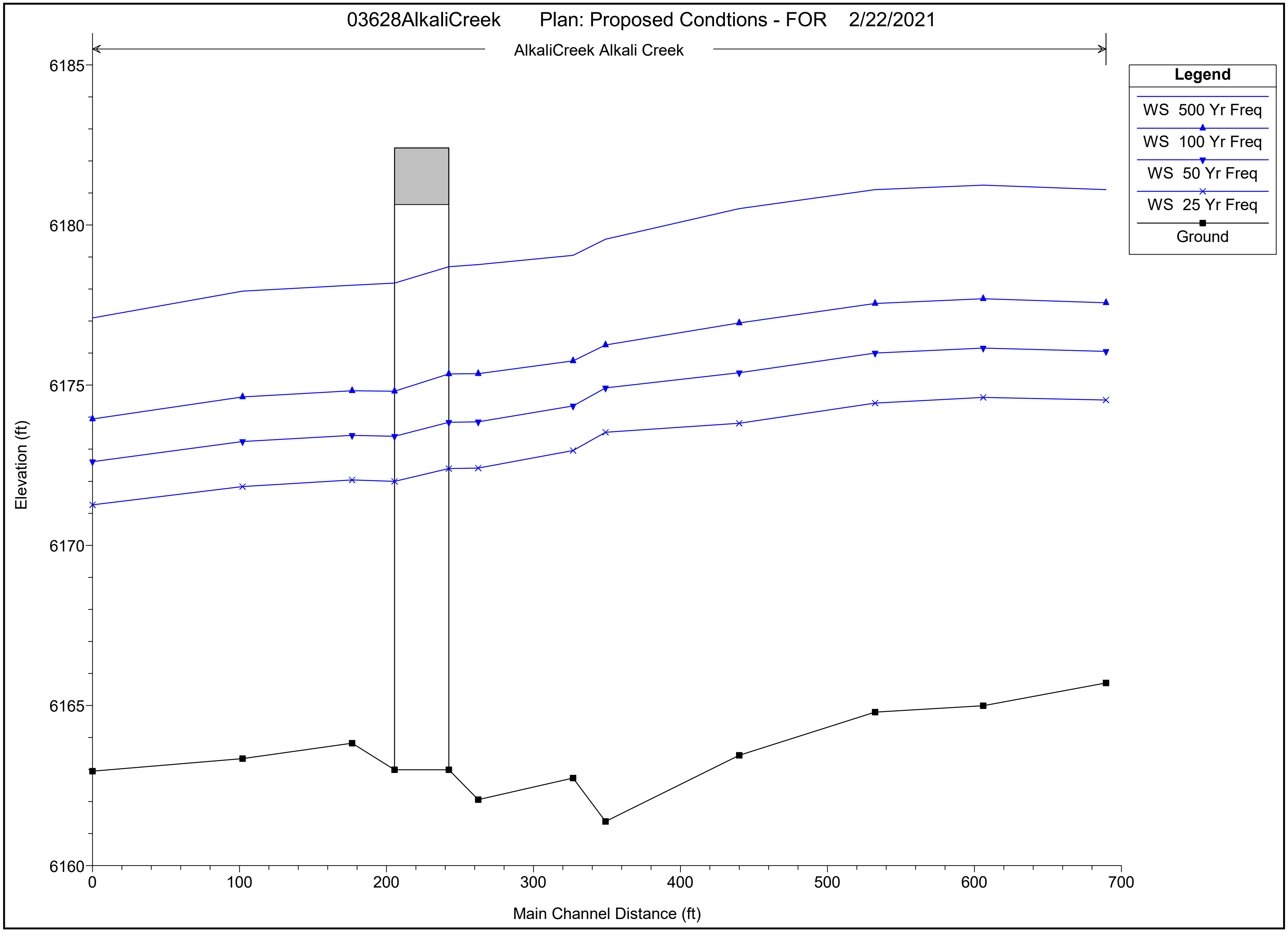


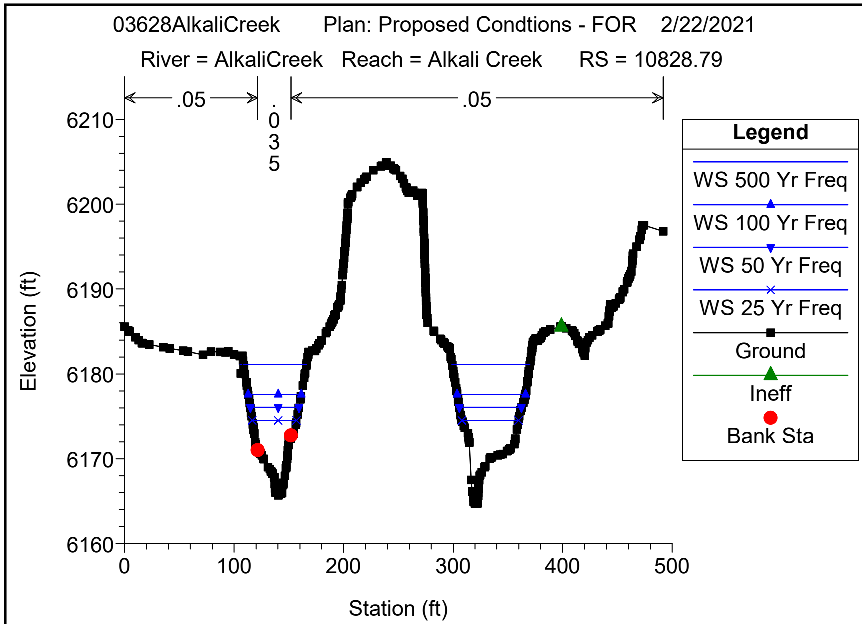
03628AlkaliCreek Plan: Existing Conditions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10353 BR
.035
.035
.035
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta
03628AlkaliCreek Plan: Existing Conditions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10315.8
.05
.035
.05
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta
03628AlkaliCreek Plan: Existing Conditions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10241.34
.05
.035
.05
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Bank Sta
03628AlkaliCreek Plan: Existing Conditions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10140.18
.05
.035
.05
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Bank Sta

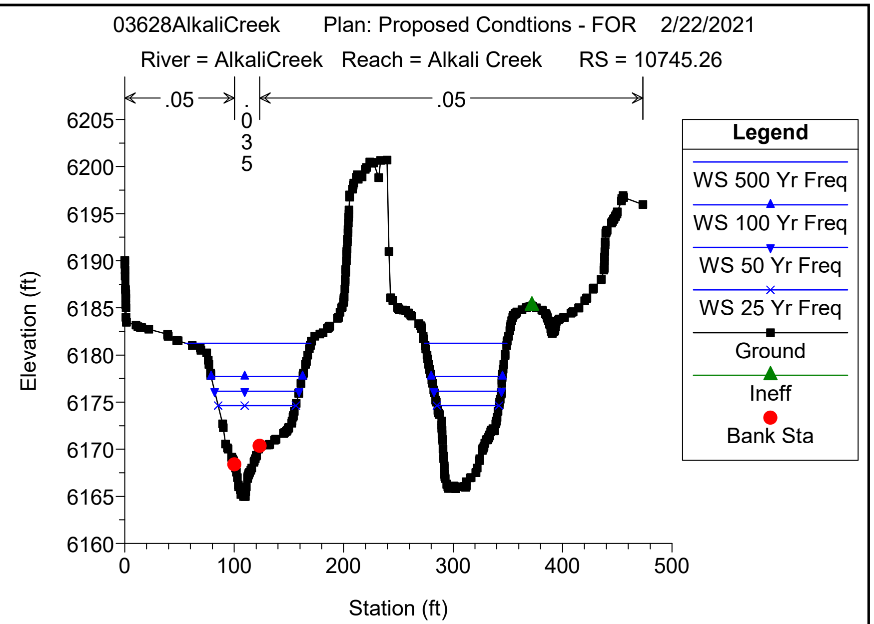

10828.79
10745.26
10671.59
10488.26
10466.14
Alkali Creek
AlkaliCreek
10401.65
10353
10315.8
10241.34
10140.18

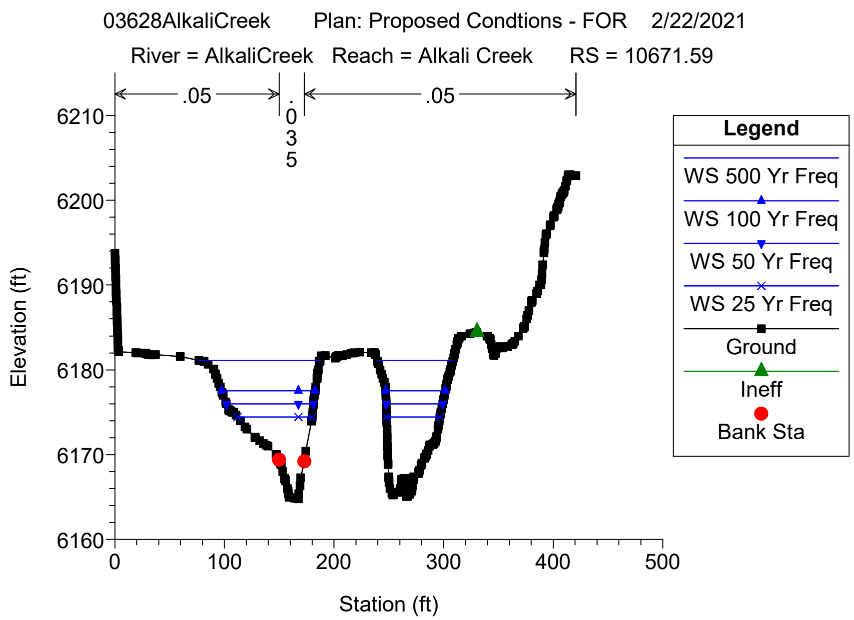
HEC-RAS Plan: Pro Cond - FOR River: AlkaliCreek Reach: Alkali Creek

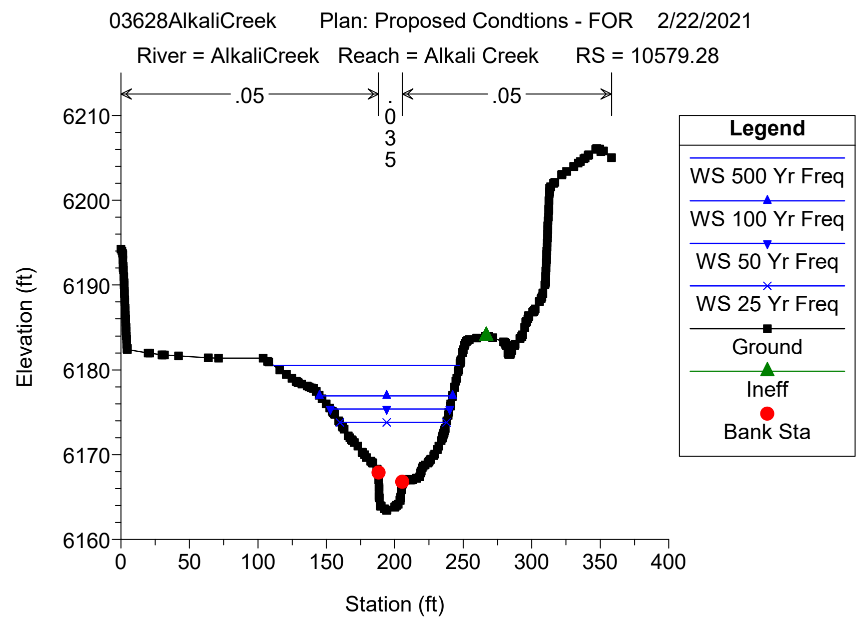
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Alkali Creek	10828.79	500 Yr Freq	6428.30	6165.70	6181.10	6175.79	6181.76	0.001566	8.26	1152.70	129.58	0.41
Alkali Creek	10828.79	100 Yr Freq	3968.60	6165.70	6177.58	6174.04	6178.19	0.002144	7.73	729.99	110.80	0.46
Alkali Creek	10828.79	50 Yr Freq	3095.10	6165.70	6176.06	6173.20	6176.66	0.002613	7.54	568.82	101.03	0.49
Alkali Creek	10828.79	25 Yr Freq	2314.20	6165.70	6174.54	6172.49	6175.13	0.003434	7.41	421.85	92.27	0.54
Alkali Creek	10745.26	500 Yr Freq	6428.30	6164.99	6181.25	6174.09	6181.58	0.000782	6.47	1659.68	190.02	0.31
Alkali Creek	10745.26	100 Yr Freq	3968.60	6164.99	6177.70	6172.52	6177.98	0.000922	5.78	1083.65	148.70	0.32
Alkali Creek	10745.26	50 Yr Freq	3095.10	6164.99	6176.16	6171.70	6176.43	0.001062	5.57	862.66	138.86	0.33
Alkali Creek	10745.26	25 Yr Freq	2314.20	6164.99	6174.62	6170.86	6174.87	0.001269	5.36	656.44	127.63	0.35
Alkali Creek	10671.59	500 Yr Freq	6428.30	6164.79	6181.11	6174.32	6181.50	0.000908	7.09	1559.79	180.07	0.33
Alkali Creek	10671.59	100 Yr Freq	3968.60	6164.79	6177.55	6172.50	6177.90	0.001037	6.29	1010.56	140.01	0.33
Alkali Creek	10671.59	50 Yr Freq	3095.10	6164.79	6176.00	6171.08	6176.34	0.001185	6.08	800.33	131.12	0.35
Alkali Creek	10671.59	25 Yr Freq	2314.20	6164.79	6174.44	6170.59	6174.76	0.001371	5.80	605.64	116.92	0.36
Alkali Creek	10579.28	500 Yr Freq	6428.30	6163.44	6180.52	6175.57	6181.35	0.001685	9.75	1100.26	135.93	0.42
Alkali Creek	10579.28	100 Yr Freq	3968.60	6163.44	6176.94	6173.50	6177.72	0.001978	8.97	685.25	97.18	0.44
Alkali Creek	10579.28	50 Yr Freq	3095.10	6163.44	6175.39	6172.57	6176.15	0.002186	8.65	541.71	87.10	0.45
Alkali Creek	10579.28	25 Yr Freq	2314.20	6163.44	6173.81	6171.54	6174.55	0.002460	8.31	411.93	77.99	0.47
Alkali Creek	10488.26	500 Yr Freq	6428.30	6161.38	6179.56	6174.96	6181.09	0.002571	12.11	823.04	94.42	0.52
Alkali Creek	10488.26	100 Yr Freq	3968.60	6161.38	6176.25	6172.56	6177.48	0.002528	10.38	552.45	72.03	0.50
Alkali Creek	10488.26	50 Yr Freq	3095.10	6161.38	6174.92	6171.46	6175.92	0.002303	9.25	462.83	61.90	0.47
Alkali Creek	10488.26	25 Yr Freq	2314.20	6161.38	6173.53	6170.29	6174.34	0.002112	8.17	381.15	55.88	0.44
Alkali Creek	10466.14	500 Yr Freq	6428.30	6162.73	6179.05	6176.14	6180.99	0.003121	13.27	715.95	77.30	0.59
Alkali Creek	10466.14	100 Yr Freq	3968.60	6162.73	6175.76	6173.57	6177.37	0.003304	11.64	479.98	65.85	0.59
Alkali Creek	10466.14	50 Yr Freq	3095.10	6162.73	6174.35	6172.46	6175.81	0.003367	10.83	391.07	60.77	0.58
Alkali Creek	10466.14	25 Yr Freq	2314.20	6162.73	6172.96	6170.88	6174.23	0.003370	9.89	309.61	56.39	0.57
Alkali Creek	10401.65	500 Yr Freq	6428.30	6162.06	6178.77	6176.30	6180.77	0.003405	13.87	711.91	91.79	0.62
Alkali Creek	10401.65	100 Yr Freq	3968.60	6162.06	6175.36	6173.56	6177.13	0.003841	12.46	463.98	67.68	0.64
Alkali Creek	10401.65	50 Yr Freq	3095.10	6162.06	6173.85	6172.27	6175.54	0.004194	11.90	367.15	60.93	0.65
Alkali Creek	10401.65	25 Yr Freq	2314.20	6162.06	6172.41	6171.28	6173.95	0.004442	11.09	283.97	54.45	0.65
Alkali Creek	10353		Bridge									
Alkali Creek	10315.8	500 Yr Freq	6428.30	6163.82	6178.12	6175.05	6180.36	0.003415	13.81	638.88	61.14	0.65
Alkali Creek	10315.8	100 Yr Freq	3968.60	6163.82	6174.82	6172.45	6176.53	0.003592	11.83	446.50	55.64	0.64
Alkali Creek	10315.8	50 Yr Freq	3095.10	6163.82	6173.44	6171.36	6174.92	0.003681	10.91	371.01	53.30	0.63
Alkali Creek	10315.8	25 Yr Freq	2314.20	6163.82	6172.04	6170.30	6173.29	0.003777	9.92	298.90	50.23	0.62
Alkali Creek	10241.34	500 Yr Freq	6428.30	6163.34	6177.94		6180.07	0.003495	13.94	659.82	62.77	0.65
Alkali Creek	10241.34	100 Yr Freq	3968.60	6163.34	6174.64		6176.23	0.003553	11.80	463.41	56.32	0.63
Alkali Creek	10241.34	50 Yr Freq	3095.10	6163.34	6173.24		6174.61	0.003590	10.83	386.81	53.73	0.62
Alkali Creek	10241.34	25 Yr Freq	2314.20	6163.34	6171.84		6172.99	0.003647	9.82	313.03	51.10	0.61
Alkali Creek	10140.18	500 Yr Freq	6428.30	6162.95	6177.10	6174.36	6179.65	0.004000	14.65	602.66	59.07	0.70
Alkali Creek	10140.18	100 Yr Freq	3968.60	6162.95	6173.95	6171.75	6175.82	0.004001	12.31	425.57	53.13	0.67
Alkali Creek	10140.18	50 Yr Freq	3095.10	6162.95	6172.61	6170.63	6174.20	0.004001	11.26	356.36	50.63	0.65
Alkali Creek	10140.18	25 Yr Freq	2314.20	6162.95	6171.26	6169.54	6172.58	0.004008	10.15	289.66	48.10	0.64


03628AlkaliCreek Plan: Proposed Condtions - FOR 2/22/2021
AlkaliCreek Alkali Creek
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Elevation (ft)
6185
6180
6175
6170
6165
6160
0
100
200
300
400
500
600
700
Main Channel Distance (ft)


03628AlkaliCreek Plan: Proposed Condtions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10828.79
.05
.035
.05
Elevation (ft)
6160
6170
6180
6190
6200
6210
Station (ft)
0
100
200
300
400
500
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta


03628AlkaliCreek Plan: Proposed Condtions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10745.26
.05
.035
.05
Elevation (ft)
6160
6165
6170
6175
6180
6185
6190
6195
6200
6205
Station (ft)
0
100
200
300
400
500
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta


03628AlkaliCreek Plan: Proposed Condtions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10671.59
.05
.035
.05
Elevation (ft)
6160
6170
6180
6190
6200
6210
Station (ft)
0
100
200
300
400
500
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta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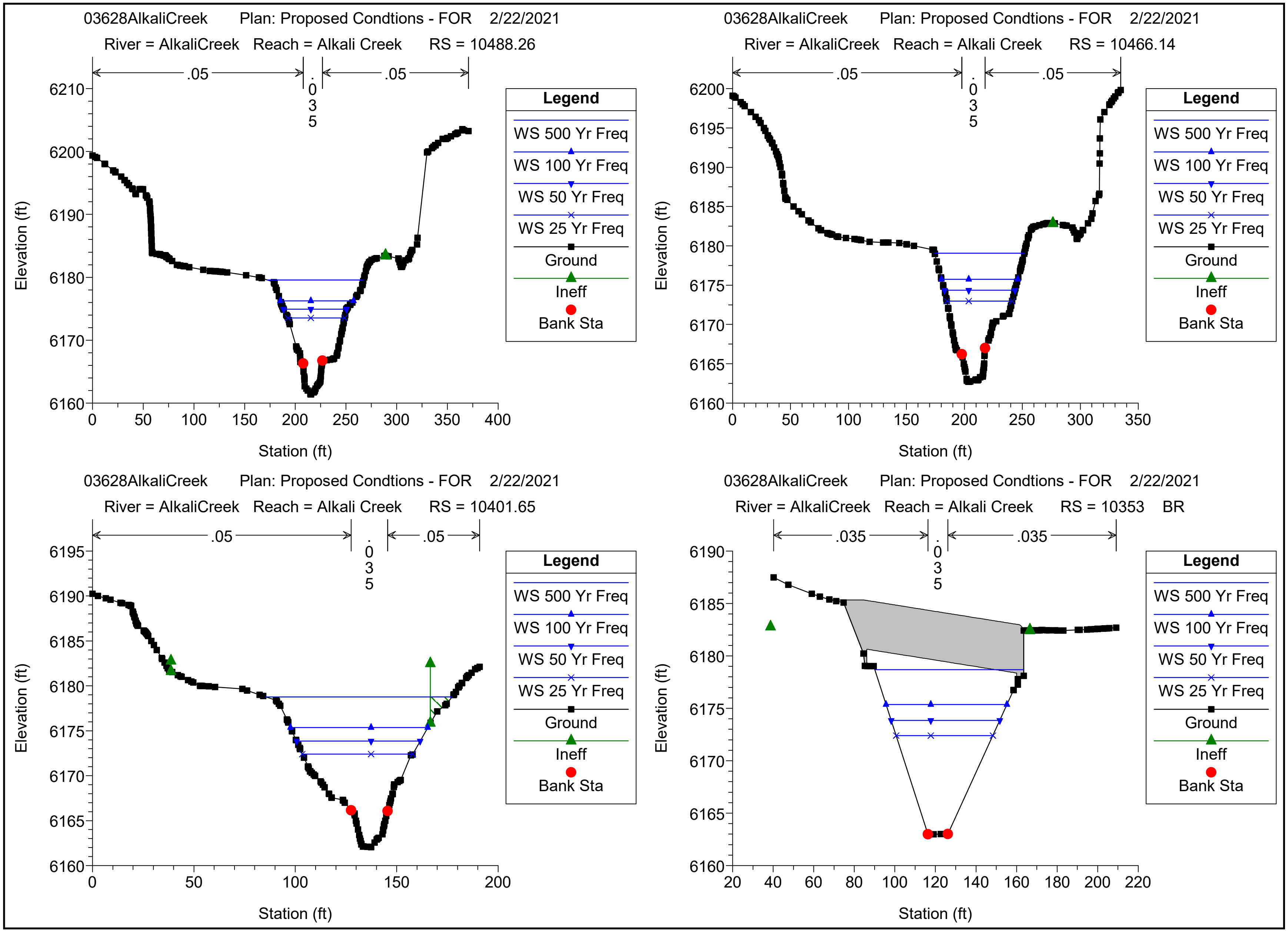
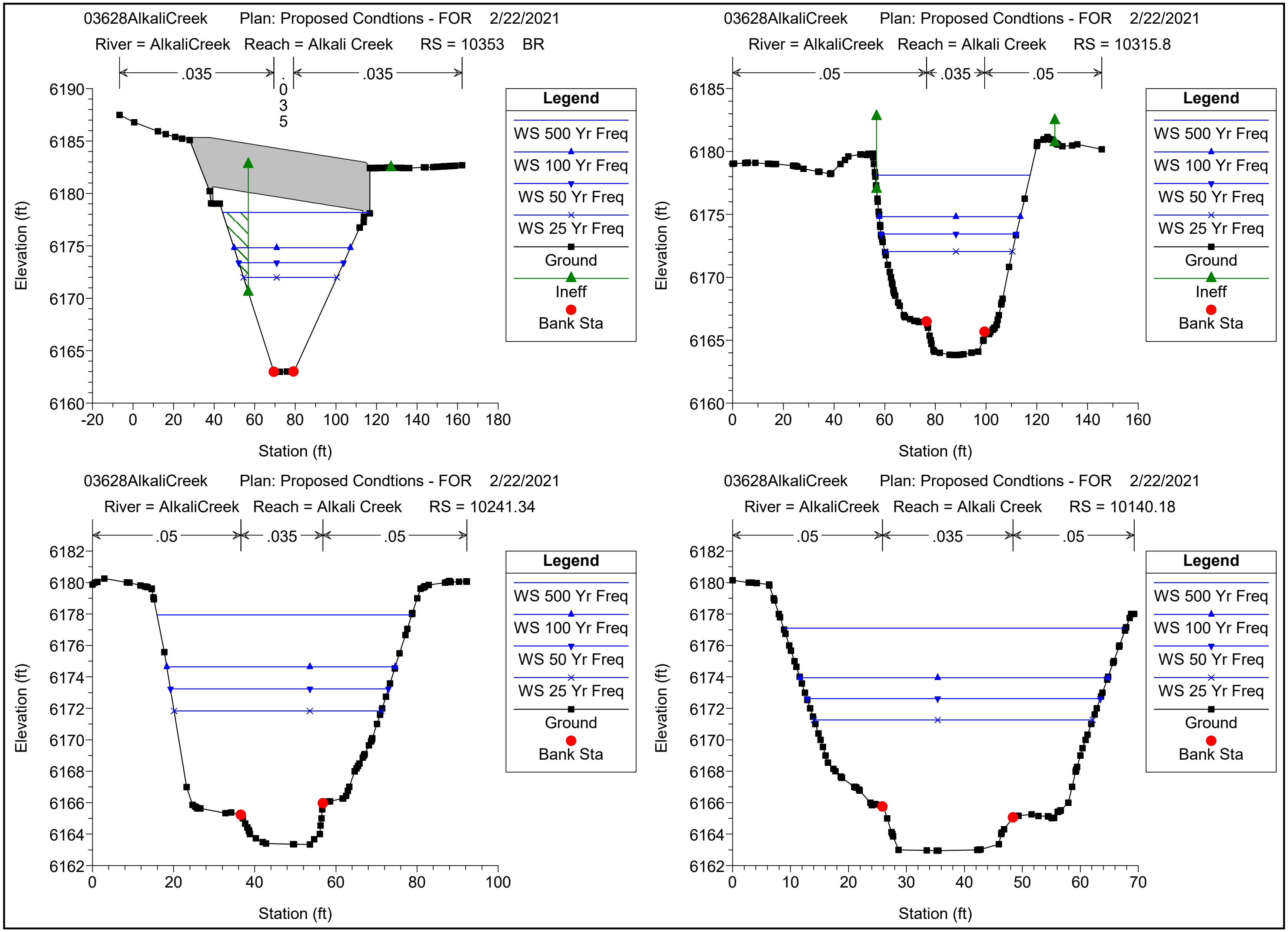
River = AlkaliCreek Reach = Alkali Creek RS = 10579.28
.05
.035
.05
Elevation (ft)
6160
6170
6180
6190
6200
6210
Station (ft)
0
50
100
150
200
250
300
350
400
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta


03628AlkaliCreek Plan: Proposed Condtions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10488.26
.05
.035
.05
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta
03628AlkaliCreek Plan: Proposed Condtions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10466.14
.05
.035
.05
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta
03628AlkaliCreek Plan: Proposed Condtions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10401.65
.05
.035
.05
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta
03628AlkaliCreek Plan: Proposed Condtions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10353 BR
.035
.035
.035
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta



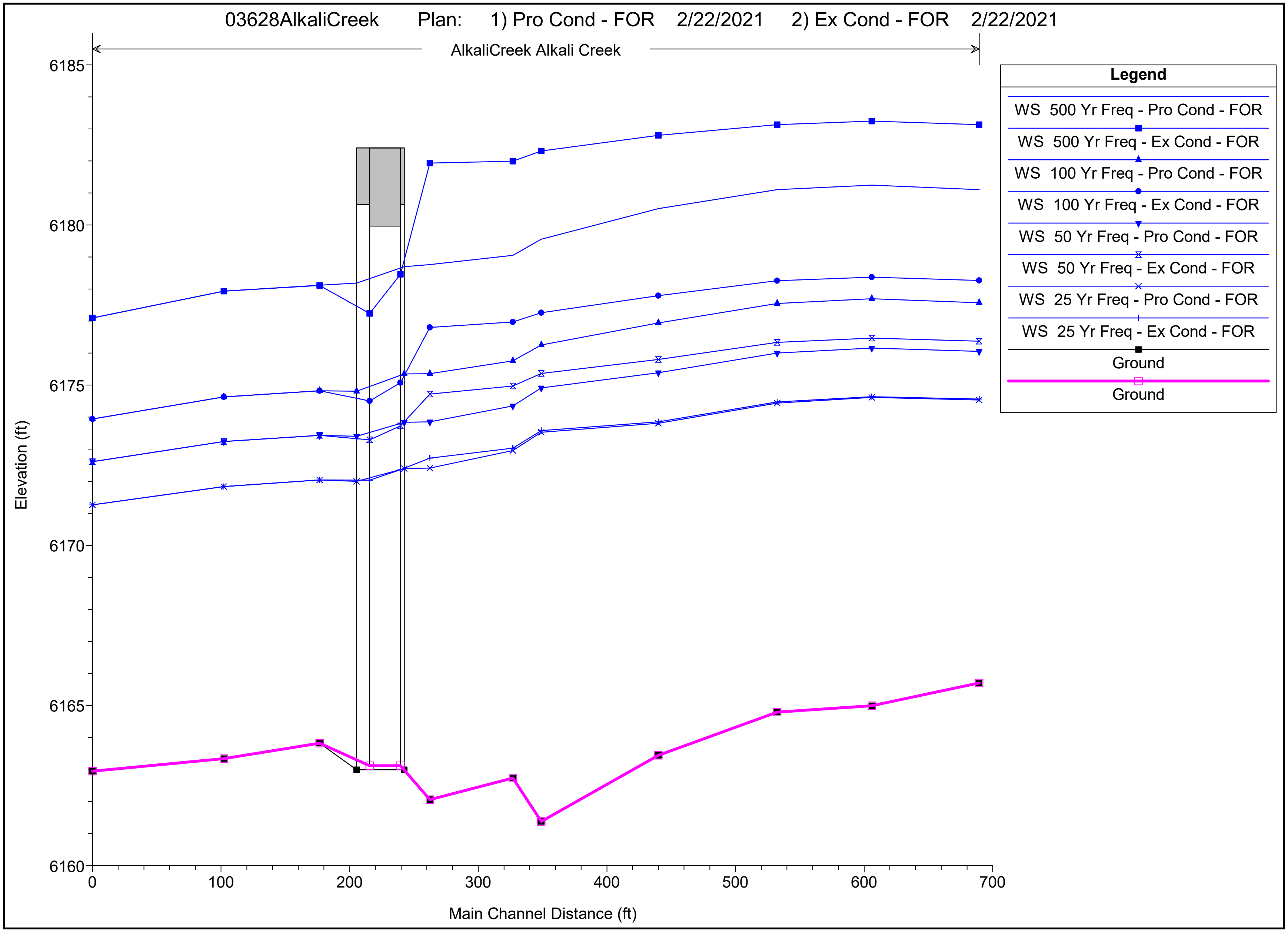
03628AlkaliCreek Plan: Proposed Condtions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10353 BR
.035
.035
.035
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta
03628AlkaliCreek Plan: Proposed Condtions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10315.8
.05
.035
.05
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Ineff
Bank Sta
03628AlkaliCreek Plan: Proposed Condtions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10241.34
.05
.035
.05
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Bank Sta
03628AlkaliCreek Plan: Proposed Condtions - FOR 2/22/2021
River = AlkaliCreek Reach = Alkali Creek RS = 10140.18
.05
.035
.05
Elevation (ft)
Station (ft)
Legend
WS 500 Yr Freq
WS 100 Yr Freq
WS 50 Yr Freq
WS 25 Yr Freq
Ground
Bank Sta

HEC-RAS River: AlkaliCreek Reach: Alkali Creek

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Alkali Creek	10828.79	500 Yr Freq	Pro Cond - FOR	6428.30	6165.70	6181.10	6175.79	6181.76	0.001566	8.26	1152.70	129.58	0.41
Alkali Creek	10828.79	500 Yr Freq	Ex Cond - FOR	6428.30	6165.70	6183.13	6175.79	6183.60	0.000915	6.98	1467.45	220.47	0.32
Alkali Creek	10828.79	100 Yr Freq	Pro Cond - FOR	3968.60	6165.70	6177.58	6174.04	6178.19	0.002144	7.73	729.99	110.80	0.46
Alkali Creek	10828.79	100 Yr Freq	Ex Cond - FOR	3968.60	6165.70	6178.27	6174.04	6178.77	0.001609	7.04	808.16	114.33	0.40
Alkali Creek	10828.79	50 Yr Freq	Pro Cond - FOR	3095.10	6165.70	6176.06	6173.20	6176.66	0.002613	7.54	568.82	101.03	0.49
Alkali Creek	10828.79	50 Yr Freq	Ex Cond - FOR	3095.10	6165.70	6176.38	6173.20	6176.92	0.002242	7.18	601.30	103.37	0.46
Alkali Creek	10828.79	25 Yr Freq	Pro Cond - FOR	2314.20	6165.70	6174.54	6172.49	6175.13	0.003434	7.41	421.85	92.27	0.54
Alkali Creek	10828.79	25 Yr Freq	Ex Cond - FOR	2314.20	6165.70	6174.57	6172.49	6175.16	0.003373	7.36	424.56	92.52	0.54
Alkali Creek	10745.26	500 Yr Freq	Pro Cond - FOR	6428.30	6164.99	6181.25	6174.09	6181.58	0.000782	6.47	1659.68	190.02	0.31
Alkali Creek	10745.26	500 Yr Freq	Ex Cond - FOR	6428.30	6164.99	6183.25	6174.09	6183.47	0.000481	5.55	2108.00	269.93	0.24
Alkali Creek	10745.26	100 Yr Freq	Pro Cond - FOR	3968.60	6164.99	6177.70	6172.52	6177.98	0.000922	5.78	1083.65	148.70	0.32
Alkali Creek	10745.26	100 Yr Freq	Ex Cond - FOR	3968.60	6164.99	6178.37	6172.52	6178.61	0.000718	5.32	1185.41	152.93	0.28
Alkali Creek	10745.26	50 Yr Freq	Pro Cond - FOR	3095.10	6164.99	6176.16	6171.70	6176.43	0.001062	5.57	862.66	138.86	0.33
Alkali Creek	10745.26	50 Yr Freq	Ex Cond - FOR	3095.10	6164.99	6176.47	6171.70	6176.71	0.000925	5.32	906.15	140.79	0.31
Alkali Creek	10745.26	25 Yr Freq	Pro Cond - FOR	2314.20	6164.99	6174.62	6170.86	6174.87	0.001269	5.36	656.44	127.63	0.35
Alkali Creek	10745.26	25 Yr Freq	Ex Cond - FOR	2314.20	6164.99	6174.64	6170.86	6174.90	0.001249	5.33	660.12	127.85	0.35
Alkali Creek	10671.59	500 Yr Freq	Pro Cond - FOR	6428.30	6164.79	6181.11	6174.32	6181.50	0.000908	7.09	1559.79	180.07	0.33
Alkali Creek	10671.59	500 Yr Freq	Ex Cond - FOR	6428.30	6164.79	6183.14	6174.32	6183.43	0.000608	6.33	2100.03	329.94	0.27
Alkali Creek	10671.59	100 Yr Freq	Pro Cond - FOR	3968.60	6164.79	6177.55	6172.50	6177.90	0.001037	6.29	1010.56	140.01	0.33
Alkali Creek	10671.59	100 Yr Freq	Ex Cond - FOR	3968.60	6164.79	6178.26	6172.50	6178.54	0.000804	5.77	1111.01	144.69	0.30
Alkali Creek	10671.59	50 Yr Freq	Pro Cond - FOR	3095.10	6164.79	6176.00	6171.08	6176.34	0.001185	6.08	800.33	131.12	0.35
Alkali Creek	10671.59	50 Yr Freq	Ex Cond - FOR	3095.10	6164.79	6176.33	6171.08	6176.63	0.001026	5.79	844.24	132.92	0.33
Alkali Creek	10671.59	25 Yr Freq	Pro Cond - FOR	2314.20	6164.79	6174.44	6170.59	6174.76	0.001371	5.80	605.64	116.92	0.36
Alkali Creek	10671.59	25 Yr Freq	Ex Cond - FOR	2314.20	6164.79	6174.48	6170.59	6174.79	0.001348	5.77	609.47	117.20	0.36
Alkali Creek	10579.28	500 Yr Freq	Pro Cond - FOR	6428.30	6163.44	6180.52	6175.57	6181.35	0.001685	9.75	1100.26	135.93	0.42
Alkali Creek	10579.28	500 Yr Freq	Ex Cond - FOR	6428.30	6163.44	6182.80	6175.57	6183.33	0.000962	8.03	1544.54	251.99	0.33
Alkali Creek	10579.28	100 Yr Freq	Pro Cond - FOR	3968.60	6163.44	6176.94	6173.50	6177.72	0.001978	8.97	685.25	97.18	0.44
Alkali Creek	10579.28	100 Yr Freq	Ex Cond - FOR	3968.60	6163.44	6177.79	6173.50	6178.41	0.001470	8.07	770.38	102.81	0.38
Alkali Creek	10579.28	50 Yr Freq	Pro Cond - FOR	3095.10	6163.44	6175.39	6172.57	6176.15	0.002186	8.65	541.71	87.10	0.45
Alkali Creek	10579.28	50 Yr Freq	Ex Cond - FOR	3095.10	6163.44	6175.81	6172.57	6176.47	0.001847	8.15	578.94	89.85	0.42
Alkali Creek	10579.28	25 Yr Freq	Pro Cond - FOR	2314.20	6163.44	6173.81	6171.54	6174.55	0.002460	8.31	411.93	77.99	0.47
Alkali Creek	10579.28	25 Yr Freq	Ex Cond - FOR	2314.20	6163.44	6173.86	6171.54	6174.59	0.002405	8.24	415.56	78.25	0.46
Alkali Creek	10488.26	500 Yr Freq	Pro Cond - FOR	6428.30	6161.38	6179.56	6174.96	6181.09	0.002571	12.11	823.04	94.42	0.52
Alkali Creek	10488.26	500 Yr Freq	Ex Cond - FOR	6428.30	6161.38	6182.32	6174.96	6183.19	0.001301	9.53	1222.91	195.48	0.38
Alkali Creek	10488.26	100 Yr Freq	Pro Cond - FOR	3968.60	6161.38	6176.25	6172.56	6177.48	0.002528	10.38	552.45	72.03	0.50
Alkali Creek	10488.26	100 Yr Freq	Ex Cond - FOR	3968.60	6161.38	6177.26	6172.56	6178.23	0.001863	9.35	628.20	78.40	0.43
Alkali Creek	10488.26	50 Yr Freq	Pro Cond - FOR	3095.10	6161.38	6174.92	6171.46	6175.92	0.002303	9.25	462.83	61.90	0.47
Alkali Creek	10488.26	50 Yr Freq	Ex Cond - FOR	3095.10	6161.38	6175.36	6171.46	6176.27	0.002011	8.85	491.29	65.33	0.44
Alkali Creek	10488.26	25 Yr Freq	Pro Cond - FOR	2314.20	6161.38	6173.53	6170.29	6174.34	0.002112	8.17	381.15	55.88	0.44
Alkali Creek	10488.26	25 Yr Freq	Ex Cond - FOR	2314.20	6161.38	6173.58	6170.29	6174.38	0.002074	8.12	383.83	56.04	0.43
Alkali Creek	10466.14	500 Yr Freq	Pro Cond - FOR	6428.30	6162.73	6179.05	6176.14	6180.99	0.003121	13.27	715.95	77.30	0.59
Alkali Creek	10466.14	500 Yr Freq	Ex Cond - FOR	6428.30	6162.73	6181.99	6176.14	6183.14	0.001556	10.52	1091.78	189.32	0.43
Alkali Creek	10466.14	100 Yr Freq	Pro Cond - FOR	3968.60	6162.73	6175.76	6173.57	6177.37	0.003304	11.64	479.98	65.85	0.59
Alkali Creek	10466.14	100 Yr Freq	Ex Cond - FOR	3968.60	6162.73	6176.97	6173.57	6178.16	0.002207	10.14	562.56	70.21	0.49
Alkali Creek	10466.14	50 Yr Freq	Pro Cond - FOR	3095.10	6162.73	6174.35	6172.46	6175.81	0.003367	10.83	391.07	60.77	0.58
Alkali Creek	10466.14	50 Yr Freq	Ex Cond - FOR	3095.10	6162.73	6174.97	6172.46	6176.19	0.002663	10.00	429.32	63.02	0.52
Alkali Creek	10466.14	25 Yr Freq	Pro Cond - FOR	2314.20	6162.73	6172.96	6170.88	6174.23	0.003370	9.89	309.61	56.39	0.57
Alkali Creek	10466.14	25 Yr Freq	Ex Cond - FOR	2314.20	6162.73	6173.03	6170.88	6174.27	0.003265	9.78	313.61	56.60	0.56
Alkali Creek	10401.65	500 Yr Freq	Pro Cond - FOR	6428.30	6162.06	6178.77	6176.30	6180.77	0.003405	13.87	711.91	91.79	0.62
Alkali Creek	10401.65	500 Yr Freq	Ex Cond - FOR	6428.30	6162.06	6181.93	6176.04	6183.02	0.001476	10.35	906.97	152.03	0.42
Alkali Creek	10401.65	100 Yr Freq	Pro Cond - FOR	3968.60	6162.06	6175.36	6173.56	6177.13	0.003841	12.46	463.98	67.68	0.64
Alkali Creek	10401.65	100 Yr Freq	Ex Cond - FOR	3968.60	6162.06	6176.80	6173.43	6178.01	0.002338	10.49	557.24	75.23	0.51
Alkali Creek	10401.65	50 Yr Freq	Pro Cond - FOR	3095.10	6162.06	6173.85	6172.27	6175.54	0.004194	11.90	367.15	60.93	0.65
Alkali Creek	10401.65	50 Yr Freq	Ex Cond - FOR	3095.10	6162.06	6174.73	6172.39	6176.00	0.002938	10.51	425.33	62.51	0.55
Alkali Creek	10401.65	25 Yr Freq	Pro Cond - FOR	2314.20	6162.06	6172.41	6171.28	6173.95	0.004442	11.09	283.97	54.45	0.65
Alkali Creek	10401.65	25 Yr Freq	Ex Cond - FOR	2314.20	6162.06	6172.72	6171.31	6174.04	0.003719	10.37	309.08	55.00	0.60
Alkali Creek	10353			Bridge									
Alkali Creek	10315.8	500 Yr Freq	Pro Cond - FOR	6428.30	6163.82	6178.12	6175.05	6180.36	0.003415	13.81	638.88	61.14	0.65
Alkali Creek	10315.8	500 Yr Freq	Ex Cond - FOR	6428.30	6163.82	6178.11	6175.05	6180.36	0.003444	13.87	638.47	61.12	0.66
Alkali Creek	10315.8	100 Yr Freq	Pro Cond - FOR	3968.60	6163.82	6174.82	6172.45	6176.53	0.003592	11.83	446.50	55.64	0.64
Alkali Creek	10315.8	100 Yr Freq	Ex Cond - FOR	3968.60	6163.82	6174.82	6172.45	6176.53	0.003592	11.83	446.50	55.64	0.64
Alkali Creek	10315.8	50 Yr Freq	Pro Cond - FOR	3095.10	6163.82	6173.44	6171.36	6174.92	0.003681	10.91	371.01	53.30	0.63
Alkali Creek	10315.8	50 Yr Freq	Ex Cond - FOR	3095.10	6163.82	6173.44	6171.36	6174.92	0.003681	10.91	371.01	53.30	0.63
Alkali Creek	10315.8	25 Yr Freq	Pro Cond - FOR	2314.20	6163.82	6172.04	6170.30	6173.29	0.003777	9.92	298.90	50.23	0.62
Alkali Creek	10315.8	25 Yr Freq	Ex Cond - FOR	2314.20	6163.82	6172.04	6170.30	6173.29	0.003777	9.92	298.90	50.23	0.62
Alkali Creek	10241.34	500 Yr Freq	Pro Cond - FOR	6428.30	6163.34	6177.94		6180.07	0.003495	13.94	659.82	62.77	0.65
Alkali Creek	10241.34	500 Yr Freq	Ex Cond - FOR	6428.30	6163.34	6177.94		6180.07	0.003495	13.94	659.82	62.77	0.65
Alkali Creek	10241.34	100 Yr Freq	Pro Cond - FOR	3968.60	6163.34	6174.64		6176.23	0.003553	11.80	463.41	56.32	0.63
Alkali Creek	10241.34	100 Yr Freq	Ex Cond - FOR	3968.60	6163.34	6174.64		6176.23	0.003553	11.80	463.41	56.32	0.63
Alkali Creek	10241.34	50 Yr Freq	Pro Cond - FOR	3095.10	6163.34	6173.24		6174.61	0.003590	10.83	386.81	53.73	0.62
Alkali Creek	10241.34	50 Yr Freq	Ex Cond - FOR	3095.10	6163.34	6173.24		6174.61	0.003590	10.83	386.81	53.73	0.62
Alkali Creek	10241.34	25 Yr Freq	Pro Cond - FOR	2314.20	6163.34	6171.84		6172.99	0.003647	9.82	313.03	51.10	0.61
Alkali Creek	10241.34	25 Yr Freq	Ex Cond - FOR	2314.20	6163.34	6171.84		6172.99	0.003647	9.82	313.03	51.10	0.61
Alkali Creek	10140.18	500 Yr Freq	Pro Cond - FOR	6428.30	6162.95	6177.10	6174.36	6179.65	0.004000	14.65	602.66	59.07	0.70
Alkali Creek	10140.18	500 Yr Freq	Ex Cond - FOR	6428.30	6162.95	6177.10	6174.36	6179.65	0.004000	14.65	602.66	59.07	0.70
Alkali Creek	10140.18	100 Yr Freq	Pro Cond - FOR	3968.60	6162.95	6173.95	6171.75	6175.82	0.004001	12.31	425.57	53.13	0.67
Alkali Creek	10140.18	100 Yr Freq	Ex Cond - FOR	3968.60	6162.95	6173.95	6171.75	6175.82	0.004001	12.31	425.57	53.13	0.67

HEC-RAS   River: AlkaliCreek   Reach: Alkali Creek (Continued)

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Alkali Creek	10140.18	50 Yr Freq	Pro Cond - FOR	3095.10	6162.95	6172.61	6170.63	6174.20	0.004001	11.26	356.36	50.63	0.65
Alkali Creek	10140.18	50 Yr Freq	Ex Cond - FOR	3095.10	6162.95	6172.61	6170.63	6174.20	0.004001	11.26	356.36	50.63	0.65
Alkali Creek	10140.18	25 Yr Freq	Pro Cond - FOR	2314.20	6162.95	6171.26	6169.54	6172.58	0.004008	10.15	289.66	48.10	0.64
Alkali Creek	10140.18	25 Yr Freq	Ex Cond - FOR	2314.20	6162.95	6171.26	6169.54	6172.58	0.004008	10.15	289.66	48.10	0.64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AlkaliCreek Alkali Creek
Legend
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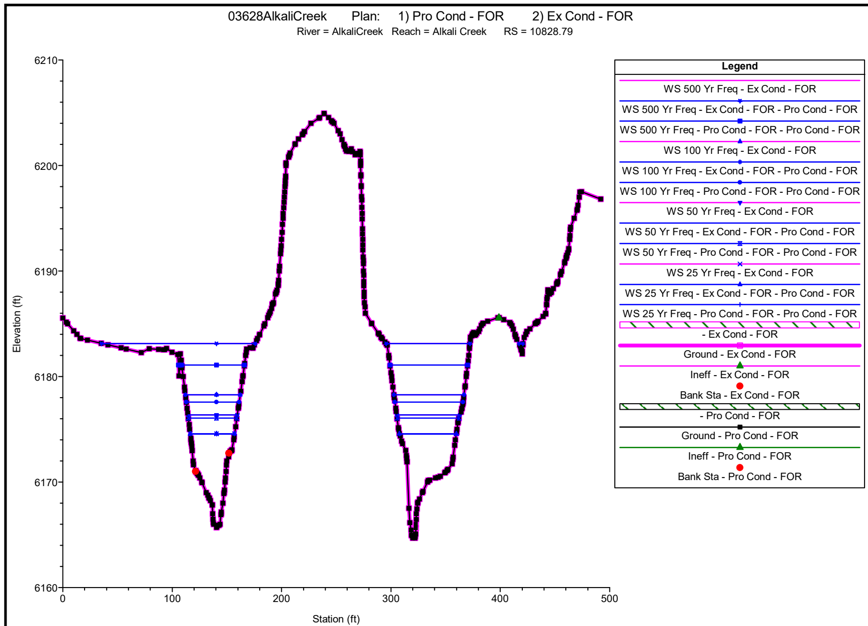
WS 500 Yr Freq - Ex Cond - FOR
WS 100 Yr Freq - Pro Cond - FOR
WS 100 Yr Freq - Ex Cond - FOR
WS 50 Yr Freq - Pro Cond - FOR
WS 50 Yr Freq - Ex Cond - FOR
WS 25 Yr Freq - Pro Cond - FOR
WS 25 Yr Freq - Ex Cond - FOR
Ground
Ground
Elevation (ft)
6185
6180
6175
6170
6165
6160
0
100
200
300
400
500
600
700
Main Channel Distance (ft)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Station (ft)
Legend
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WS 500 Yr Freq - Pro Cond - FOR - Pro Cond - FOR
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WS 100 Yr Freq - Ex Cond - FOR - Pro Cond - FOR
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WS 50 Yr Freq - Ex Cond - FOR - Pro Cond - FOR
WS 50 Yr Freq - Pro Cond - FOR - Pro Cond - FOR
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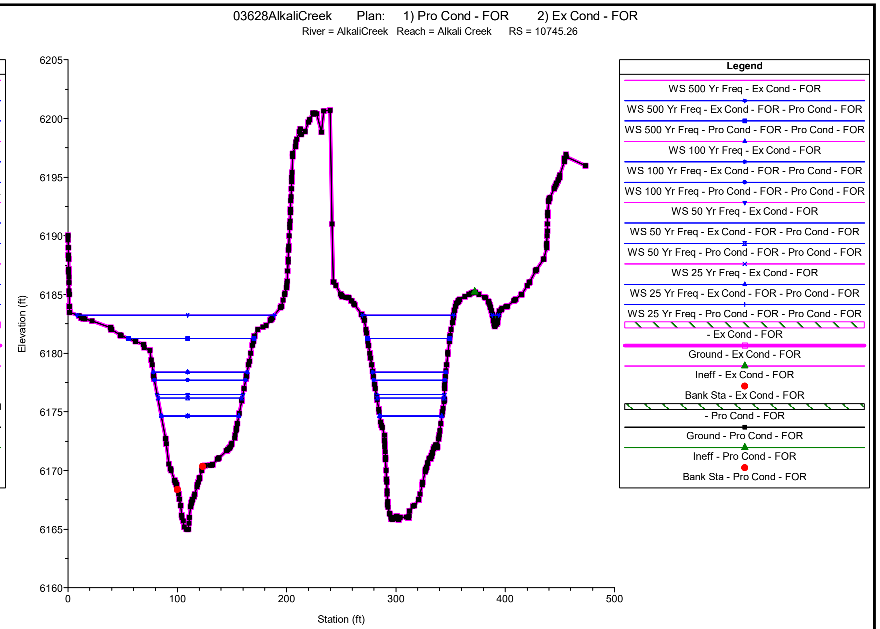
WS 25 Yr Freq - Pro Cond - FOR - Pro Cond - FOR
- Ex Cond - FOR
Ground - Ex Cond - FOR
Ineff - Ex Cond - FOR
Bank Sta - Ex Cond - FOR
- Pro Cond - FOR
Ground - Pro Cond - FOR
Ineff - Pro Cond - FOR
Bank Sta - Pro Cond - FOR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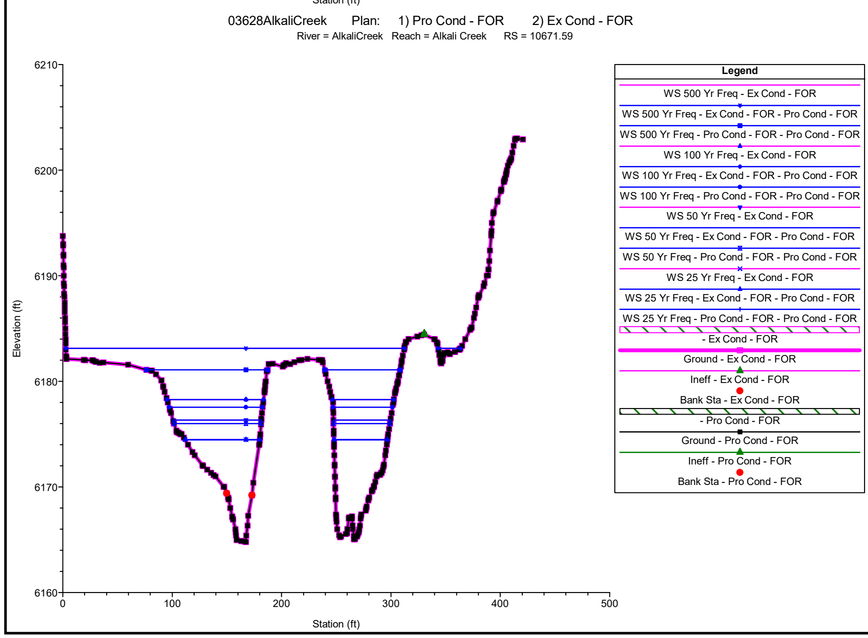
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Elevation (ft)
Station (ft)
Legend
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- Ex Cond - FOR
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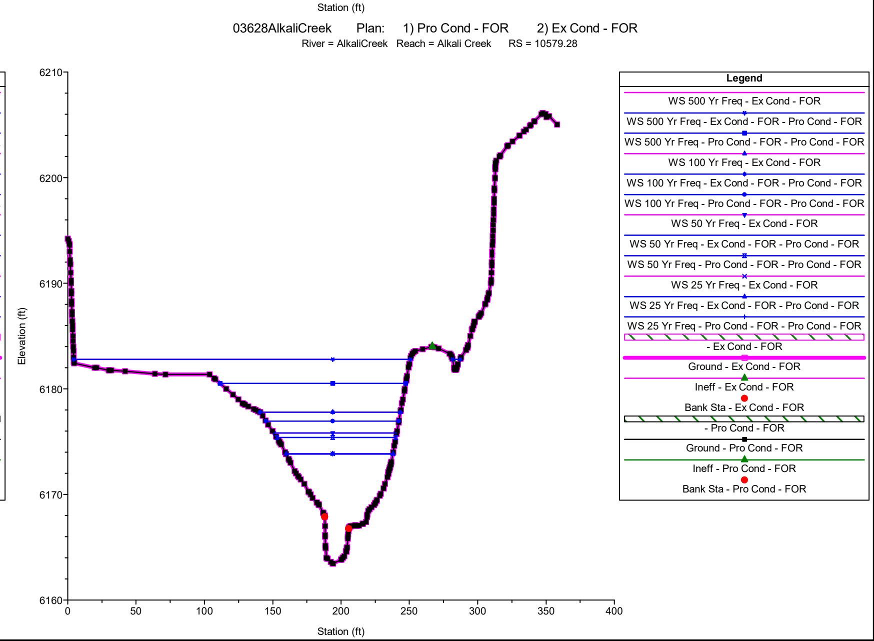
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Elevation (ft)
Station (ft)
Legend
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- Ex Cond - FOR
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Bank Sta - Ex Cond - FOR
- Pro Cond - FOR
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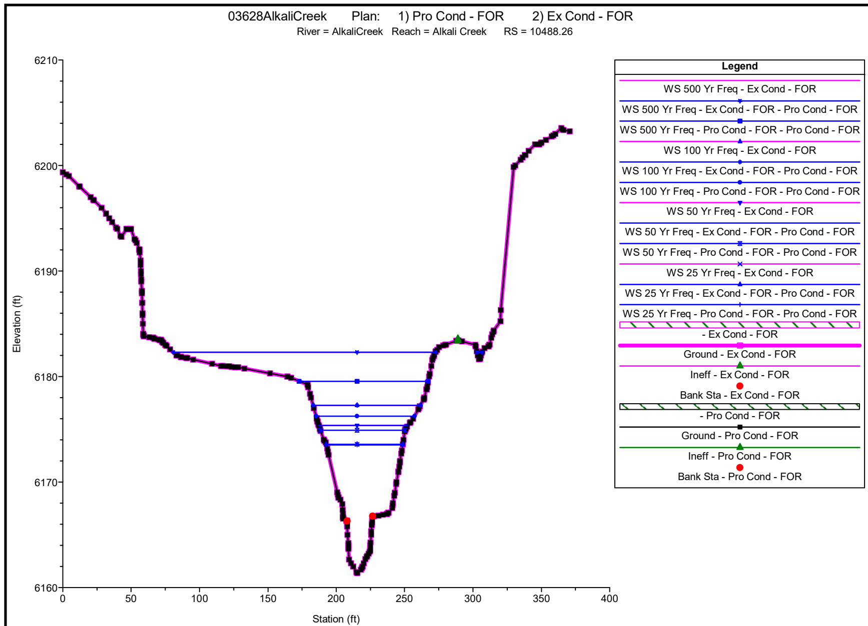
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Elevation (ft)
Station (ft)
Legend
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WS 500 Yr Freq - Pro Cond - FOR - Pro Cond - FOR
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WS 100 Yr Freq - Ex Cond - FOR - Pro Cond - FOR
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- Ex Cond - FOR
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Bank Sta - Ex Cond - FOR
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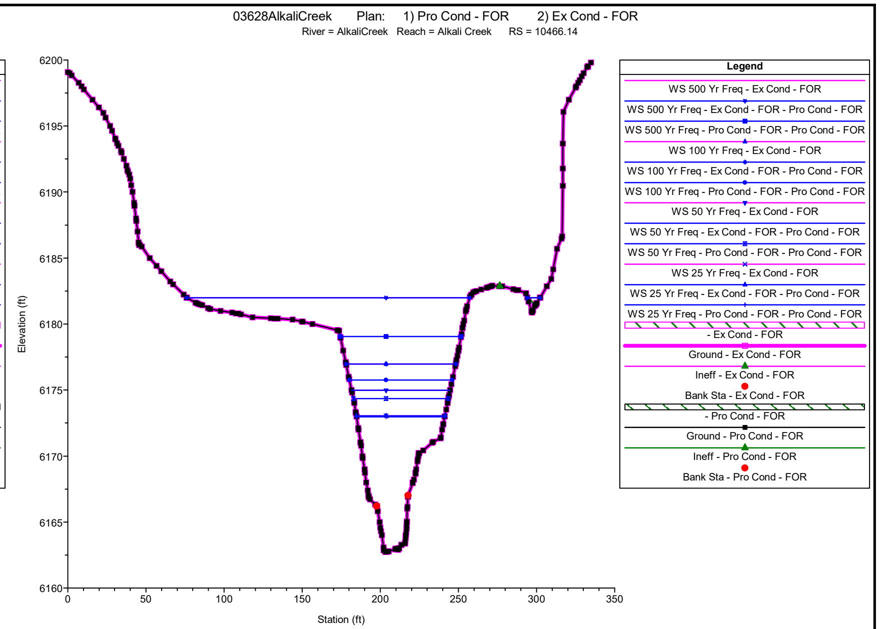
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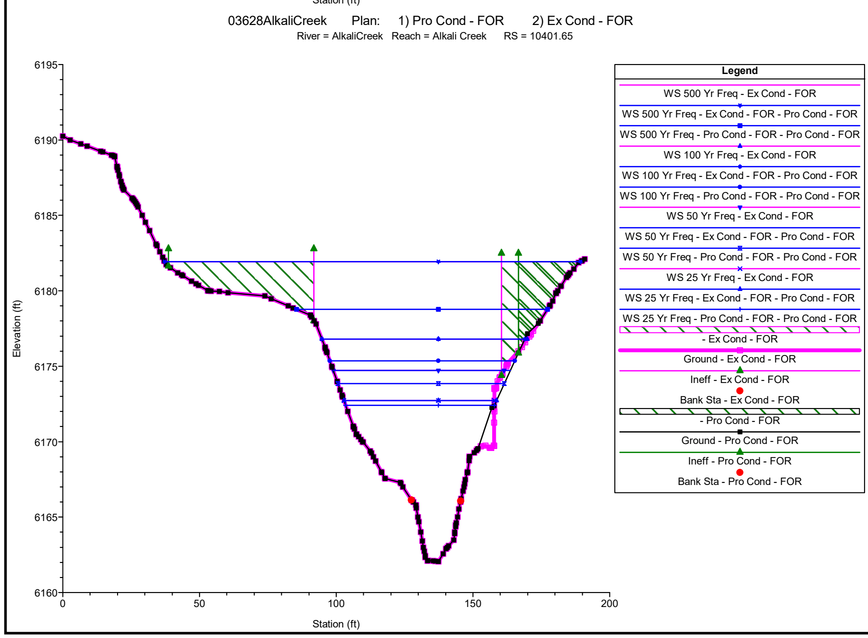
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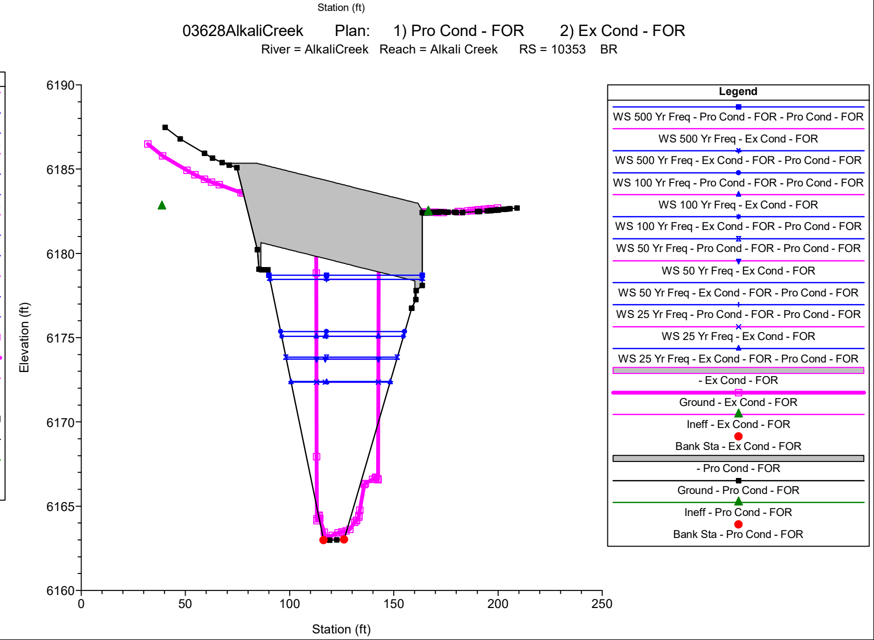
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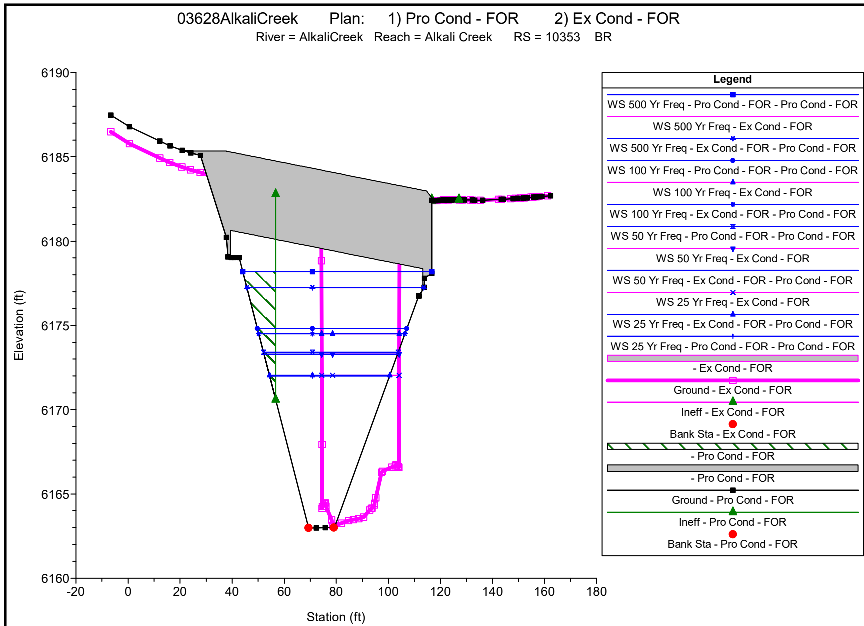
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Legend
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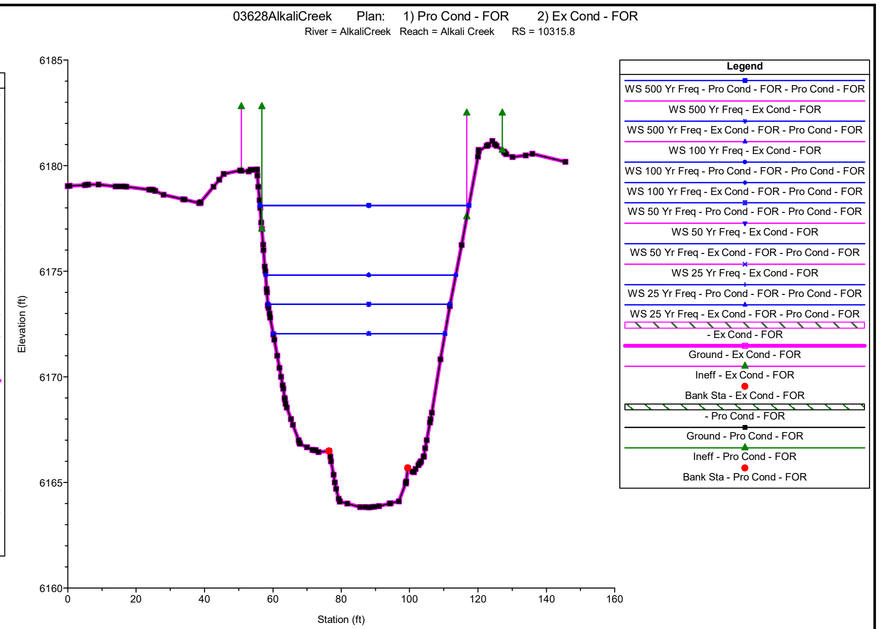
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Station (ft)
Legend
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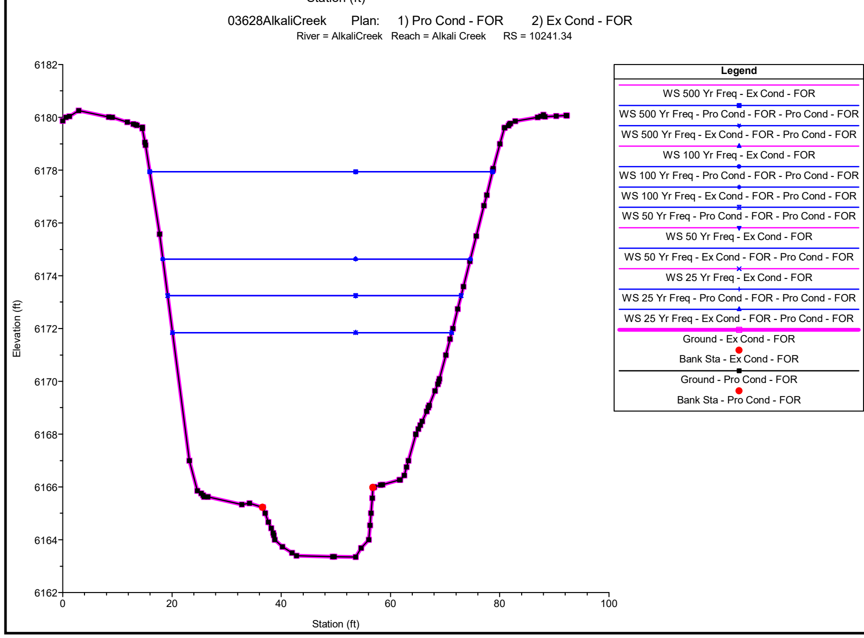
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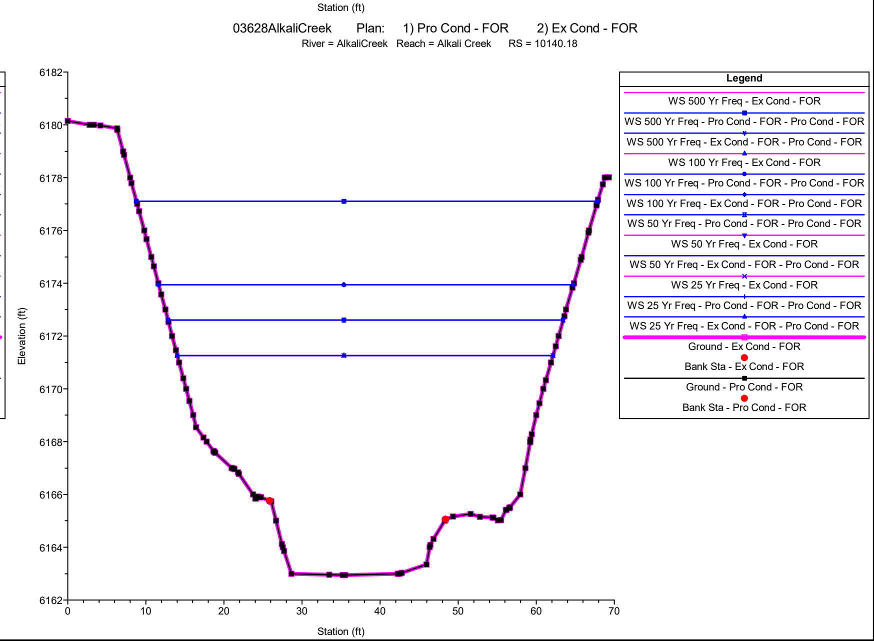
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Station (ft)
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Bank Sta - Ex Cond - FOR
Ground - Pro Cond - FOR
Bank Sta - Pro Cond - FOR

APPENDIX D

CDOT GEOTECHNICAL REPORT

GEOTECHNICAL ENGINEERING STUDY

MONTEZUMA COUNTY, ROAD N/ALKALI CREEK BRIDGE REPLACEMENT PROJECT

(CDOT Project Number: BRO C320-004, Project Identification: 22521)

Montezuma County, Colorado

May 17, 2019

Prepared For:
Mr. Rich Bechtolt, P.E.
Bechtolt Engineering, Inc.
Project Number: 55458GE

1.0 REPORT INTRODUCTION

This report presents our geotechnical engineering recommendations for the proposed Montezuma County, Road N and Alkali Creek Bridge Replacement Project. The Colorado Department of Transportation (CDOT) project number has been designated as BRO C320-004, Project Identification 22521. This report was requested by Mr. Rich Bechtolt, P.E., Bechtolt Engineering, Inc. The field study was performed on April 25, 2019. The laboratory study was completed on May 16, 2019.

The information provided in this report is intended to help develop a design and implementation of construction strategies that are appropriate for the subsurface soil and water conditions at the project site. It is important that we are consulted throughout the design and construction process to verify the implementation of the geotechnical engineering recommendations provided in this report. The recommendations and technical aspects of this report are intended for design and construction personnel who are familiar with construction concepts and techniques, and understand the terminology presented below. We should be contacted if any questions or comments arise as a result of the information presented below.

The following outline provides a synopsis of the various portions of this report;

- Sections 1.0 and 2.0 provide an introduction and an establishment of our scope of service.
- Sections 3.0 and 4.0 of this report present our geotechnical engineering field and laboratory studies
- Sections 5.0 through 7.0 presents our geotechnical engineering design parameters and recommendations which are based on our engineering analysis of the data obtained.
- Section 8.0 provides a brief discussion of construction sequencing and strategies which may influence the geotechnical engineering characteristics of the site.

The discussion and construction recommendations presented in Section 8.0 are intended to help develop site soil conditions that are consistent with the geotechnical engineering recommendations presented previously in the report. The construction considerations section is not intended to address all of the construction planning and needs for the project site, but is intended to provide an overview to aid the owner, design team, and contractor in understanding some construction concepts that may influence some of the geotechnical engineering aspects of the site and proposed development.

The data used to generate our recommendations are presented throughout this report and in the attached figures.

1.1 *Scope of Project*

We understand that the proposed project will consist of designing and constructing a new bridge structure to replace the existing County Road N Bridge that crosses Alkali Creek. We understand that the new bridge will likely be a single span. *At the time of issue of this draft report, it is unclear whether or not the new bridge abutments will be located immediately adjacent to the existing bridge abutments, or will be moved to a different location downstream or upstream of the existing bridge and further away from the banks Alkali Creek. In addition, the elevation of the new bridge deck had not been finalized at the time of issue of this draft report.* New asphalt pavement is proposed for the bridge approach surfaces.

2.0 GEOTECHNICAL ENGINEERING STUDY

The scope of our study which was delineated in our proposal for services, and the order of presentation of the information within this report, is outlined below.

Field Study

- We advanced two test borings at the project within the areas we understand are planned for construction of the proposed bridge support foundation systems. Each test boring was advanced within the County Road N pavement surface, adjacent to each (east and west) existing bridge abutments.
- The test borings were advanced with our approximate 13,000 pound CME-45c track mounted drilling equipment. The field crew consisted of a professional geotechnical engineer and an engineering geologist.
- Select driven sleeve/tube samples, bag samples, and core of the formational materials underlying the project site were obtained from the test borings and returned to our laboratory for testing.

Laboratory Study

- The laboratory testing and analysis of the samples obtained included;
    - Moisture content and dry density of select soil samples obtained from Modified California Barrel samples. In addition, the density of select rock core obtained from the formational materials that underlie the project was determined.
    - Direct shear strength tests performed on select soil samples to help establish a basis for development of lateral earth pressure values for retaining structures.
    - Unconsolidated-Undrained (UU) triaxial strength tests performed on select Modified California Barrel soil samples in order to assess the undrained shear strength versus strain parameters for the site soil materials.

- Unconfined compressive strength tests on select sections of rock core in order to provide engineering design parameters for the formational materials that underlie the project site.
- Swell/consolidation tests to help assess the expansion and consolidation potential of the existing site soil materials.
- Plastic and liquid limit tests to determine the Plasticity Index of the existing site soil materials that overlie the formational materials.
- Sieve analysis tests to determine the gradation of the site soil materials that overlie the formational materials.
- Chemical tests including soluble sulfates, chloride ion, and pH to generally assess the corrosion potential of the site soils on Portland cement concrete and steel components.
- Laboratory resistivity tests to assess the resistivity characteristics of the site soils that overlie the formational materials.
- Modified Proctor (AASHTO T-180/ASTM D1557) tests to determine the laboratory compaction characteristics of the existing roadway subgrade soil materials.
- California Bearing Ratio (CBR) tests to assess the subgrade resilient modulus of the existing roadway subgrade soil materials.

Geotechnical Engineering Recommendations

- This report addresses the geotechnical engineering aspects of the site and provides recommendations including;

Geotechnical Engineering Section(s)

- Subsurface soil and water conditions that may influence the project design and construction conditions.
- Geotechnical engineering foundation design parameters that generally follow AASHTO LRFD bridge design specifications including;

  - ✓ Geotechnical engineering design parameters for drilled caisson and driven pile foundation systems.
  - ✓ LPILE computer modelling parameters for use in modeling laterally loaded deep foundation components.
  - ✓ Lateral Earth Pressure values for design of retaining structures.

Construction Considerations Section

- Fill placement considerations including cursory comments regarding site preparation and grubbing operations.

  - Considerations for excavation cut slopes.
  - Compaction and moisture conditioning recommendations for various types of backfill that may be used for the project.

- This report provides design parameters, but does not provide foundation design or design of structure components. The project structural engineer may be contacted to provide a design based on the information presented in this report.

- Our subsurface exploration, laboratory study and engineering analysis do not address environmental or geologic hazard issues with exception to potential expansive soil conditions.

3.0 FIELD STUDY

3.1 *Project Location*

The proposed bridge replacement project is located at the existing Montezuma County Road N bridge structure that crosses Alkali Creek. The existing bridge will be removed as part of the project. The project site is located within Montezuma County, Colorado, approximately 0.3 miles southwest of the intersection of County Road N and County Road 22. The project site is located west of U.S. Highway 491, approximately 4.6 miles northwest of Cortez, Colorado. The general location of the project site is provided in Figures 3.1and 3.2 presented below. The aerial imagery used for Figures 3.1 and 3.2 were obtained from Google Earth (imagery date: 10/12/2017).

Figure 3.1: Approximate Project Location

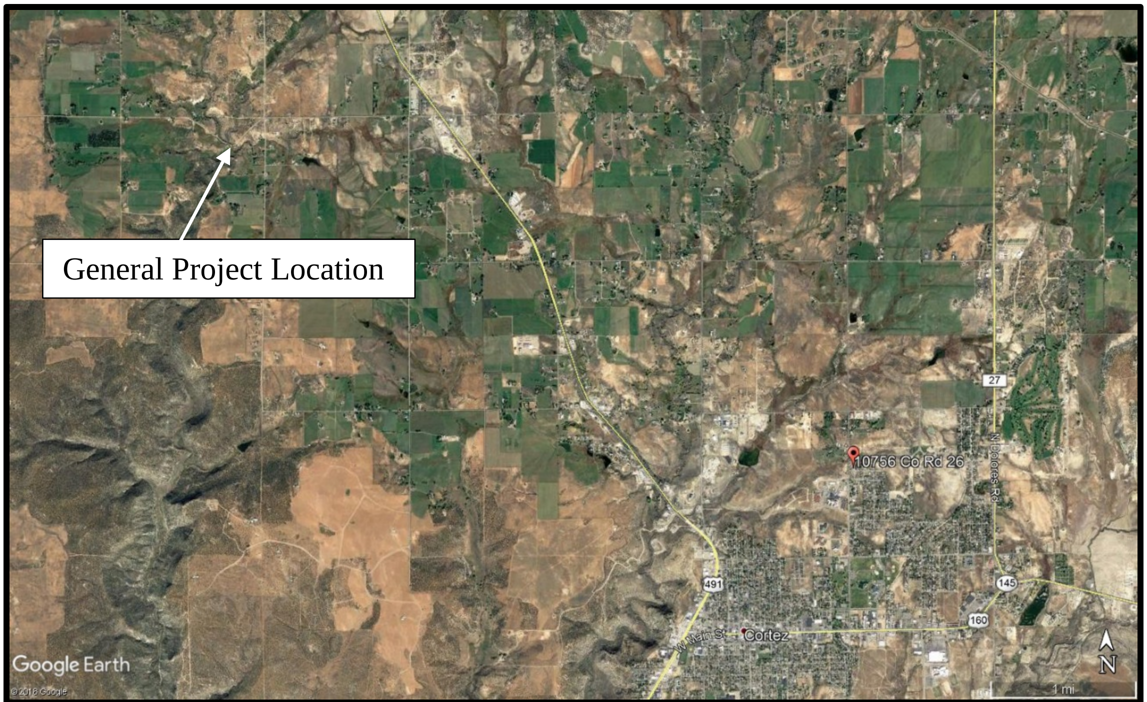


Figure 3.2 Project Location (more detailed view)

3.2 *Site History, Site Description, and Geomorphology*

We understand that the existing bridge was constructed about 40 years ago. The existing bridge length is approximately 28 feet between the east and west abutments. The width of the bridge is approximately 24 feet. The existing bridge deck surface is located approximately 18 to 20 feet above the flowline elevation of Alkali creek. The existing bridge abutment/foundation support elements consist of driven H-piles. Based on our subsurface field study, we anticipate that the existing abutment support piles extend to the formational materials that underlie the project site (subsurface conditions are discussed in more detail in Section 3.3 below). The abutment walls and adjoining wingwalls consist of corrugated metal cribbing that is laterally restrained by driven H-piles. We observed evidence of relatively severe corrosion in the abutment/wingwall cribbing, as well as some evidence of corrosion in the driven H-piles at the stream water elevation that directly support the bridge deck.

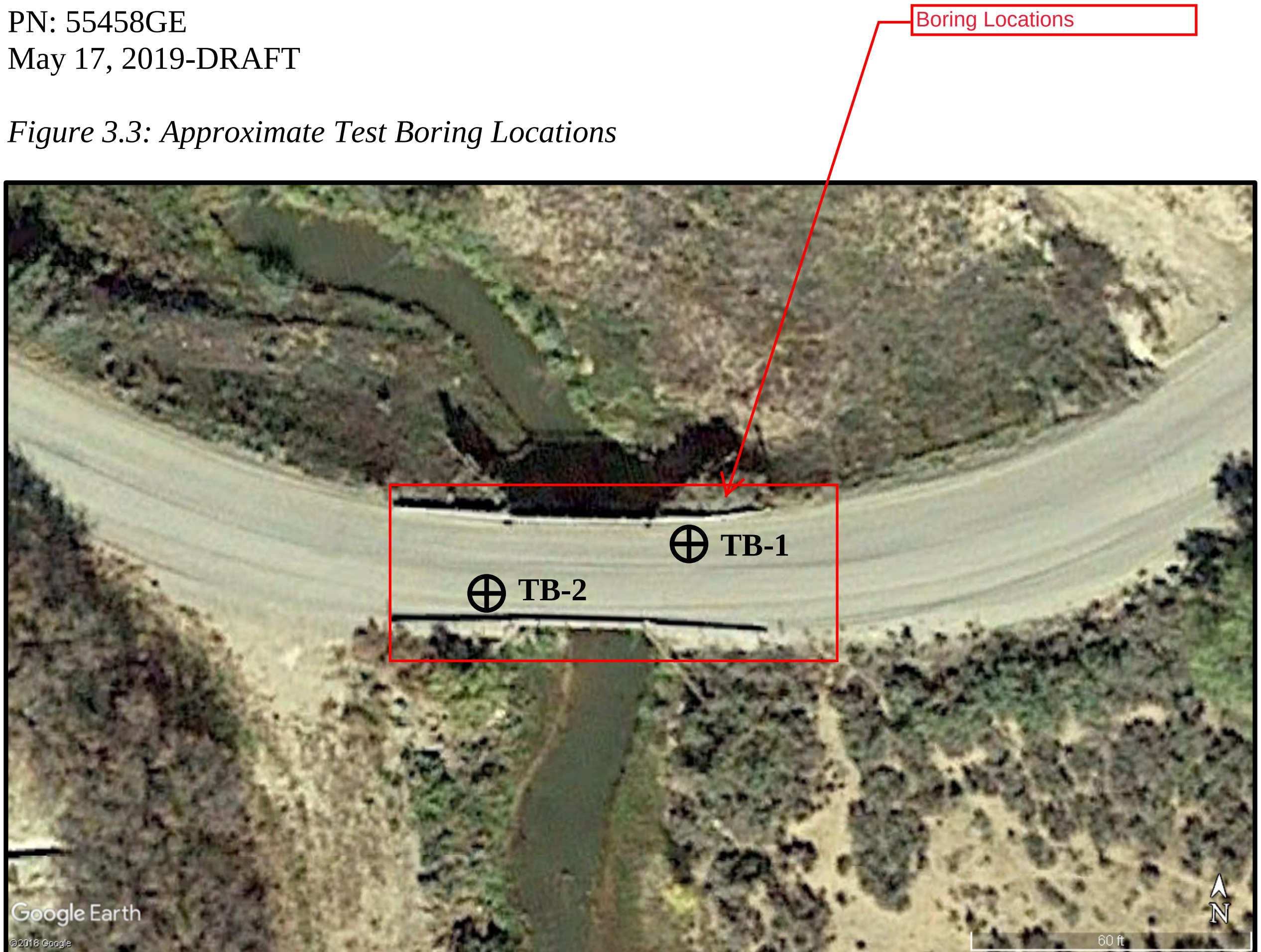
Alkali Creek may be considered as a perennial flowing stream. Water flow within the stream channel is likely primarily influenced by surface water runoff and irrigation water delivery needs downstream of the project site. However, we anticipate that subsurface water flow into the stream channel (via seeps and springs) also occurs. Surface water runoff into the stream channel is influenced by general precipitation/snowmelt conditions, and heavily influenced by extensive agricultural irrigation in areas upstream of the bridge. Fine grained soils consisting of sandy silt/clay soil are predominantly exposed within the bank areas and stream channel in the vicinity of the bridge.

The subsurface soil and rock materials encountered in the vicinity of the project generally consists of a variable sandy clay soil loess deposit that overlies the Dakota Sandstone formation. The clay soil materials encountered in the vicinity of the project site typically exhibit a moderate to high swell potential. The formational materials consist of interbedded layers of tan to white colored sandstone, shale, and claystone materials. Lignite (coal like material) may also be encountered within the formational materials. We observed outcrops of the Dakota Sandstone formation in the vicinity of the bridge. The depth to the formational sandstone materials will be highly variable in relation to the Alkali Creek stream channel.

3.3 *Subsurface Soil and Water Conditions*

We advanced a total of two test borings in the vicinity of the proposed structure. The test borings were advanced adjacent to the east and west existing bridge abutments. We advanced the test borings with 4-inch dimeter continuous flight auger to the upper portions of the formational materials. NW wireline core was obtained within the formational materials to the bottom of the test borings. The approximate locations of our test borings are shown on Figure 3.3 presented below. The imagery used for Figure 3.3 was obtained from Google Earth (imagery date: 10/12/2017). The logs of the soils encountered in our test borings are presented in Appendix A of this report.

Figure 3.3: Approximate Test Boring Locations



The approximate test boring locations shown on the figure above were prepared using notes taken during the field work and are intended to show the approximate test boring locations for reference purposes only. Test Boring TB-1 was advanced about 6 feet east of the existing east bridge abutment within the north lane of County Road N, and Test Boring TB-2 was advanced about 6 feet west of the existing west bridge abutment within the south lane of County Road N. The test borings were patched with asphalt cold-patch material and may be survey located by the project surveyor. A general description of the subsurface conditions encountered in our test borings is provided in the text below. The logs of the subsurface conditions presented in Appendix A of this report should be referenced for a more detailed description of the subsurface conditions.

In general, we encountered about 2 inches of a chip seal roadway surface overlying 2 to 2½ feet aggregate base course material. It was difficult to discern the exact size of the existing aggregate base course materials, however we anticipate that it consists of ¾-inch minus aggregate materials (similar in gradation to CDOT Class 6 material).

Below the existing chip seal surface and aggregate base course section, we generally encountered medium stiff to stiff and moist sandy clay soil material to the surface of the Dakota Sandstone formation which consisted of very hard, tan to white colored sandstone. The formational sandstone materials were encountered at a depth of about 36½ feet below the roadway surface in Test Boring TB-1, and at a depth of about 26 feet below the roadway elevation in Test Boring TB-2.

We anticipate that a substantial depth of the sandy clay soil materials that were encountered in our test borings consists of man placed backfill that is retained by the existing bridge abutment/wingwall cribbing. Standard penetration tests within the sandy clay soils ranged from about N=4 to N=10. We encountered evidence of man placed fill in Test Boring TB-1 to a depth of at least 21 feet below the roadway elevation as evidenced by a section of #3 rebar within a Modified California sample obtained at this depth. A photograph of this sample is provided below.

The depth to the formational sandstone materials will be highly variable. We suspect that the formational materials will be encountered at shallower depths (potentially much shallower depths) at greater distances away from the stream channel alignment and our test boring locations. The upper approximate 1 foot of the formational material was somewhat weathered. Standard penetration tests within about 1½ to 2 feet below the surface of the formational material ranged from about 50 blows for 2 inches to 50 blows for no penetration.

We advanced NW wireline (NQ size) core within the formational materials encountered in our test borings. Core drilling operations were initiated at a depth of 38 feet below the roadway elevation in Test Boring TB-1, and at a depth of 28½ feet below the roadway elevation in Test Boring TB-2.

We obtained nearly 100 percent core recovery in all of the core runs with exception for the first core run in Test Boring TB-2 where we obtained about 85% recovery. Rock Quality Designation (RQD) of the core runs ranged from as low as about 50% to 75% from depths ranging from 28½ to about 36 feet below the roadway elevation in Test Boring TB-2. Otherwise, the RQD of the rock core obtained was about 100%. The photographs presented below indicate the nature of the rock core obtained from our test borings. It should be noted that some of the fractures shown in the photograph presented below are due to mechanical fracturing from the core drilling operation.

<u>Photograph of core, Test Boring TB-1, 38 to 50½ feet below the roadway elevation</u>

Photograph of core, Test Boring TB-2, 28½ to 41 feet below the roadway elevation

Subsurface free water was encountered at depths ranging from about 17 to 18 feet below the roadway elevation at the time of our field study. The subsurface free water elevation will be primarily influenced by the water elevation within the creek.

The logs of the subsurface soil conditions encountered in our test borings are presented in Appendix A. The logs present our interpretation of the subsurface conditions encountered exposed in the test borings at the time of our field work. Subsurface soil and water conditions are often variable across relatively short distances. It is likely that variable subsurface soil and water conditions will be encountered during construction. Laboratory soil classifications of samples obtained may differ from field classifications.

3.4 *Site Seismic Classification*

The seismic site class as defined by the AASHTO LRFD Bridge Design Specifications, Eighth Edition, 2017, is based on some average values of select soil characteristics such as shear wave velocity, standard penetration test result values, undrained shear strength, and plasticity index.

We utilized standard penetration test results and undrained shear strength tests as a basis for the site seismic classification provided below. Based on this information we calculated the Average Standard Penetration Test ($\bar{N}$) using Method B as detailed in Table C3.10.3.1-1 of the AASHTO Bridge Design Specifications.

Based on our calculations we obtained a $\bar{N}$ value equal to about 30 for both Test Borings TB-1 and TB-2. Based on an average $\bar{N}$ value equal to about 20, a seismic site class designation of **Site Class D** should be used for the project seismic design (Table 3.10.3.1-1).

The table below presents the seismic site coefficients for the project site based on a Site Class D designation in conjunction with the mapped zero period acceleration, short period acceleration, and long period acceleration. The spectral response maps and subsequent seismic site coefficients were obtained from the 2017 AASHTO Bridge Design Specifications.

Mapped Spectral Peak Ground Acceleration, PGA (Figure 3.10.2.1-1)	Mapped Spectral Short Period Acceleration S_s (Figure 3.10.2.1-2)	Mapped Spectral 1-second Acceleration S_1 (Figure 3.10.2.1-3)	Zero Period Site Coefficient F_{pga}	Short Term Period Site Coefficient F_a	Long Term Period Site Coefficient F_v
0.05g	0.12g	0.035g	1.6	1.6	2.4

Based on the product of the values obtained for F_v and S_1, the site Seismic Zone obtained from Table 3.10.6-1 is a **Seismic Zone 1**.

3.5 *Estimates of Streambed D_{50} Particle Size*

Often, bridges are located over relatively granular alluvial deposits of gravels, cobbles, and boulder sized particles. In these instances, we typically estimate the streambed D_{50} particle size based on string line measurements of the exposed particle size at set intervals (2 to 3 feet along the string line alignment) to estimate the streambed D_{50} particle size. This is due to the fact that soil samples that accurately represent the granular subsurface soils typically cannot be obtained using auger drilling techniques.

In the case of the subject project bridge, we did not observe notable granular deposits (gravel and cobble sized materials) in the streambed immediately upstream of the bridge. Based on the subsurface soil conditions encountered in our test borings, we recommend performing the project scour analysis (which will be performed by others) based on the grain size distribution of the fine-grained soil materials that were encountered in our test borings below the approximate flowline elevation of Alkali Creek. We feel that the soil samples obtained from our test borings are representative of the grain size distribution of the subsurface soil materials below the flowline elevation of the creek.

Based on the grain size distribution (sieve analyses) for the soil samples encountered below the flowline elevation of the creek in Test Borings TB-1 and TB-2, we recommend assuming a D_{50} particle size equal to the #200 sieve screen (0.075 millimeter). The results of our sieve analyses are discussed in more detail in Section 4.0. The sieve analysis test results for Test Boring TB-1 at depths ranging from about 21 to 22 feet below the existing roadway elevation are presented on Figure 4.2 of Appendix B, while the sieve analysis test results obtained from Test Boring TB-2 at depths ranging from about 19½ to 23½ feet below the roadway elevation are presented on Figure 4.4 of Appendix B.

As discussed in Section 3.3 above, we encountered very hard formational sandstone materials at a depth of approximately 36½ feet below the roadway elevation in Test Boring TB-1, and at a depth of approximately 26 feet below the roadway elevation in Test Boring TB-2. Assuming the flowline elevation of the creek is about 19 feet below the bridge deck elevation, the very hard formational sandstone materials should be located within about 7 to 17 feet below the flowline elevation of the creek. As discussed in Section 4.0 below, the unconfined compressive strength of the upper portions of the formational sandstone materials is in the range of about 2,900 pounds per square inch. We anticipate that significant future scour below the surface elevation of the formational sandstone materials below the streambed channel is unlikely to occur.

4.0 LABORATORY STUDY

We performed the following tests on select samples obtained from our test borings;

Unit Weight; The unit weight and moisture content of select driven Modified California liner samples obtained from the subsurface soil materials, and unit weight of select sections of rock core obtained from the formational sandstone materials were measured. The results of the unit weight measurements for both soil materials and formational sandstone rock core are tabulated below in this section of the report.

Atterberg Limits and Sieve Analysis Tests; the plastic limit, liquid limit and plasticity index in conjunction with the grain size distribution (sieve analysis tests) were performed on select samples obtained from the soil materials encountered in our test borings. The results of the sieve analysis and Atterberg Limits tests are presented on Figures 4.1 through 4.4 of Appendix B.

Based on the results of the sieve analysis and Atterberg Limits tests, the soil materials that were encountered above the formational sandstone materials generally classify as AASHTO type A-6 or USCS type "CL" sandy lean clay material.

Swell-Consolidation Tests; the one-dimensional, swell-consolidation potential of select Modified California soil samples was determined in general accordance with constant volume methodology. The test samples were exposed to varying loads and inundated with water at surcharge pressures ranging from 100 to 500 pounds per square foot. The one-dimensional swell-consolidation response of the soil samples to the loads and the addition of water is represented graphically on Figures 4.5 through 4.7 of Appendix B. A synopsis of some of the pertinent information obtained from the swell-consolidation test results are tabulated in Table 4.1 presented below.

Table 4.1

Sample Designation	Moisture Content (percent)	Dry Density (pcf)	Measured Swell Pressure* (psf)	Swell Potential (%)
TB-1 @ 3 feet	14.5	110.0	none measured	0.0 (100 psf surcharge load)
TB-2 @ 8.5 feet	10.8	114.7	1,270	0.7 (500 psf surcharge load)
TB-2 @ 13.5 feet	12.4	114.7	940	0.2 (500 psf surcharge load

*NOTE: We determine the swell pressure as measured in our laboratory using the constant volume method. The graphically determined swell pressure may be different from that measured in the laboratory.

The site sandy clay soil materials encountered and tested in our test borings exhibit a relatively low swell potential at surcharge pressures of up to 500 pounds per square foot. We feel that potential heave of the site clay soils and the potential influence on the various aspects of the project (such as heave of asphalt pavement or uplift forces on deep foundation components due to expansive soil conditions) is relatively minimal.

Unconsolidated-Undrained (UU) Triaxial Compression Tests; the undrained shear strength (s_u) and general stress-strain relationship of select soil samples extruded from Modified California liners that were obtained at various depths in our test borings was determined in general accordance with ASTM D2850. The results of these tests were used to assess input parameters for LPILE computer modelling, specifically including the undrained shear strength parameters (s_u) and E_{50} parameters that are used for these types of soils in LPILE modelling. The test samples were approximately 2 inches in diameter by 4 inches in length, and were exposed to effective confining pressures that are approximately equal to the estimated effective pressures at the depth the samples were obtained. The test results from UU triaxial compression tests are presented on Figures 4.8 and 4.9 of Appendix B. It should be noted that the presentation of the test results in these figures is not intended to indicate any type of linear relationship between the various test samples, as the test samples consisted of somewhat different soil compositions at various densities, moisture content, and degrees of saturation. Some of the pertinent information obtained from the UU strength tests are tabulated in Table 4.2 presented below.

Table 4.2

Sample Designation and Sample Depth/ (N-value)	**Sample Moisture Content (%)**	**Sample Dry Density (pcf)**	**Initial Void Ratio/Degree of Saturation (S.G.=2.65 assumed)**	**Effective Confining Pressure During Testing (psi)**	**Peak Deviator Stress (psf)**	**Strain at Peak Deviator Stress**	**Undrained Shear Strength (s_u) (psf)**	**Strain at 50% of s_u (E_{50})**
TB-1 @ 8 feet (N=9)	13.1	111.2	0.49/71.0%	6.9	8,570	0.093	4,285	0.005
TB-1 @ 23 feet (N=9)	21.5	105.2	0.57/99.5%	17.4	2,580	>15.0	1,290	0.036
TB-1 @ 28 feet (N=10)	21.4	106.0	0.56/100%	19.5	3,040	>15.0	1,520	0.011
TB-2 @ 18.5 feet (N=9)	16.0	114.7	0.44/95.9%	15.6	5,740	>15.0	2,870	0.020
TB-2 @ 23.5 feet (N=10)	21.0	108.0	0.53/100%	17.8	1,950	13.4	975	0.017

Unconfined Compressive Strength*;* the unconfined compressive strength of select sections of the NWL (NQ diameter) core that was obtained from the formational sandstone materials were tested. The results of the unconfined compressive strength tests are presented in Table 4.3 below.

Table 4.3

Core Boring and Depth (feet below the road surface)	Unit Weight (pcf)	Unconfined Compressive Strength (psi)
TB-1 @ 38 feet	136.4	2,870
TB-1 @ 42 feet	137.1	3,700
TB-2 @ 29 feet	141.8	2,970
TB-2 @ 32 feet	137.0	1,600

Direct Shear Strength tests*;* Direct shear strength tests were performed on select soil samples obtained from the existing suspected man placed bridge abutment backfill materials. The sample that was tested was encountered in Test Boring TB-2 at a depth ranging from about 10 to 14 feet below the roadway elevation. We obtained an angle of internal friction (phi) of about 23 degrees and a cohesion of about 390 pounds per square foot. The results of the direct shear test are presented on Figure 4.10 of Appendix B.

Chemical Tests*;* The water soluble sulfate, chloride ion concentrations, and pH of several samples obtained from our test borings was measured by Green Analytical Laboratories. The results of these tests are tabulated below and provided in Appendix C of this report. We performed soluble sulfate tests in-house for a sample of the formational sandstone material obtained from Test Boring TB-2. The results of these tests are tabulated below in Table 4.4 below.

Table 4.4

Test Boring and Depth (feet)	**Soil Type**	**Sulfate in Water (parts per million)**	**Chloride Ion (parts per million)**	**pH**
TB-1; 13.0-14.5	Sandy clay (existing abutment backfill)	757	180	11.7
TB-2; 9.5-13.5	Sandy clay (existing abutment backfill)	4,140	546	7.9
TB-2; 19.5-23.5	Sandy clay (existing abutment backfill)	4,550	157	7.4
TB-2; 28.5	Formational Sandstone	200	--	--

Based on Section 601.04 of the 2011 CDOT Standard Specifications for Road and Bridge Construction, and the results of the soluble sulfate testing, the severity of sulfate exposure should be considered as ranging from a Class 1 to Class 2 severity of sulfate exposure. Based on the chemical testing that we have performed to date, we recommend that the CDOT requirements for cementitious materials for Class 2 requirements be followed.

A relatively high pH level was obtained for the soils encountered in Test Boring TB-1 at depths ranging from about 13 to 14½ feet below the ground surface elevation. These test results were checked again by Greene Analytical and similar test results were obtained.

Soil Resistivity; we performed resistivity measurements for select soil samples obtained from our test borings. The resistivity testing was performed in the laboratory with a soil box using the Wenner 4-electrode method. The soil samples were remolded to an approximate wet density of about 125 pounds per cubic foot at the existing moisture conditions of the soil materials. The samples tested and obtained from Test Boring TB-1 at a depth of 19½ to 21 feet below the roadway elevation and TB-1 at a depth of 14½ to 18½ feet below the roadway elevation consisted of saturated clay soils (saturated with the existing subsurface free water of Alkali Creek). These tests represent the laboratory soil box resistivity values for fully saturated soils. The results are tabulated in Table 4.5 below.

Table 4.5

Test Boring and Sample Depth	**Remolded Dry Density**	**Moisture Content of Remolded Soil**	**Resistivity**
TB-1; 4’-8’	109.4 pcf	14.3 %	3,300 ohm.cm
TB-1; 19.5’-21’	103.4 pcf	20.9%	3,400 ohm.cm
TB-2; 14.5’-18.5’	103.6 pcf	20.6%	4,200 ohm.cm

Based on Section 10.7.5 of the AASHTO LRFD bridge design specifications, resistivity levels less than 2,000 ohm.cm may be indicative of potential corrosion on steel components.

Moisture content-dry density relationship (modified Proctor) tests; We performed laboratory moisture content-dry density tests to assess the relationship between the soil moisture content and dry density. The Proctor tests were performed in accordance with AASHTO T-180. The tests were performed on a bulk sample of the existing roadway subgrade materials obtained from Test Boring TB-2 at depths ranging from about 2½ to 5 feet below the roadway surface. The results of the modified Proctor tests are presented on Figure 4.11 of Appendix B. We obtained a maximum dry density of about 123 pounds per cubic foot at an optimum moisture content of about 11 percent. We recommend that the moisture-density relationship (modified Proctor) be tested during construction to verify the test results that we obtained (based on limited sampling) are representative of the project-wide subgrade soil materials.

California Bearing Ratio (CBR) Tests; We assessed the pavement section support characteristics of select composite soil samples in general accordance with ASTM D1883. The results of the CBR tests are presented on Figure 4.12 of Appendix B. We obtained a CBR of 3.0 for the existing site subgrade soils that are densified to 90 percent of the maximum dry density as established by the modified Proctor test (see above).

5.0 BRIDGE ABUTMENT FOUNDATION RECOMMENDATIONS

The discussion provided below is based on the assumption that the new bridge abutment foundation system will be constructed within about 10 feet of the existing bridge abutments. The depth to the formational sandstone materials will likely be substantially different (likely located at more shallow depths) in areas away and upslope from our test boring locations. We recommend advancing additional test borings at the actual proposed new bridge abutment locations if the new abutment locations will be constructed a substantial distance away from our test borings locations. We are available to discuss this further with you as the project plans progress.

Based on the subsurface conditions encountered in our test borings, we recommend that the bridge structure be supported by a deep foundation system that extends to the competent formational sandstone materials that underlies the project site. We do not recommend the use of shallow bearing foundation systems such as spread footings due to potential settlement and potential future scour of the fine-grain sandy clay soil materials that are located at and below the streambed elevation. Based on the subsurface conditions encountered in our test borings we feel that there are two primary deep foundation systems that should be considered to support the bridge. These foundation systems are driven piles and drilled shafts. Recommendations for driven piles are provided in Section 5.1 below, and recommendations for drilled shafts are provided in Section 5.2 below. We are available to provide recommendations for alternative deep foundation components such as cased micropiles at your request. Recommended parameters for LPILE computer modeling for laterally loaded deep foundation components are provided in Section 5.3 below. The AASHTO LRFD Bridge Design Specifications, 8th Edition, was used as the primary source for the recommendations provided in this section of the report.

Based on the subsurface conditions encountered, we anticipate that the use of driven piles has advantages over drilled shafts with regard to the ease of installation and general constructability. In addition, due to the relatively high soluble sulfate levels encountered at some depths in our test borings, driven steel piles are advantageous from the perspective of potential sulfate attack on the Portland cement concrete associated with drilled shafts.

The potential disadvantage of driven piles relates to obtaining adequate penetration into the underlying formational sandstone materials to resist lateral loads and potential scour. It may be necessary to pre-drill driven piles to obtain sufficient embedment into the formational sandstone materials to resist lateral loads and potential scour (calculated by others). As discussed in more detail below, we anticipate that driven piles will "set" rapidly within the formational sandstone materials, likely only achieving about 1 to 3 feet of embedment into the upper weathered zone of the formational sandstone materials.

5.1 *Driven Piles*

Driven piles that are end/tip bearing in the competent formational sandstone materials that underlie the project site may be used to support the proposed bridge abutments and potential associated wingwall structures. Based on the subsurface conditions encountered in our test borings, obtaining a tip bearing condition on the very hard formational sandstone materials should be readily obtained for H-section piles. We anticipate that about 1 to 3 feet of penetration into the formational sandstone materials may be obtained for H-section piles. It is also likely feasible to obtain a tip bearing condition for driven pipe piles, however we anticipate that very little penetration into the formational sandstone materials will be obtained for pipe piles.

Based on Section 10.7.3.2.3 of the AASHTO LRFD specifications, the piles may be considered as being point/tip bearing on hard rock. Therefore, the nominal axial compression resistance of the piles is controlled by the structural limit state of the selected pile section. Based on Table 10.5.5.2.3-1 of the AASHTO LRFD specifications, the applicable resistance factor for steel piles shall be based on the structural limit state (Article 6.5.4.2 of the AASHTO LRFD specifications). The project structural engineer should calculate the capacities of the piles based on AASHTO LRFD specifications. The minimum center to center spacing between the individual piles should be 30 inches or 2.5 times the pile diameter, whichever is greater.

We anticipate that relatively immediate refusal will occur once the tip of the pile encounters the formational sandstone materials. We anticipate that damage to the pile could easily and rapidly occur if the potential energy of the hammer is greater than the yield stress of the selected pile section. The piles should be driven with high strength tip protection.

We recommend that the piles be driven with an appropriately sized hammer and/or adjustable stroke/energy hammer to avoid damage to the pile. When the tip elevation seats against the formational sandstone materials, then a set-criteria of 5 blows per 1/2 inch of pile penetration may be used to verify the set of the pile. Again, the energy output of the pile driving equipment must not exceed the structural capacity of the selected pile. We recommend that at least one pile per bridge abutment be monitored with signal matching pile driving analyzer (PDA) equipment, to verify that the needed capacity of the pile is obtained, and that the pile is not damaged at the set criteria discussed above (based on an allowable hammer energy for the selected pile).

Forces due to down drag of the existing bridge abutment backfill materials on the piles may be disregarded based on our understanding of the time frame that the existing abutment backfill materials have been in place (about 40 years). If substantial new fill materials (greater in depth than about 4 feet) are placed over the existing bridge abutment fill materials or native undisturbed soil materials, then down drag forces may need to be considered. We should be contacted to evaluate potential down drag forces on the piles if more than about 4 feet of fill material will be placed over the existing abutment backfill materials or native undisturbed soil materials. In addition, uplift forces acting on the piles due to expansive soil conditions may be disregarded based on the results of our laboratory swell test results.

We anticipate that penetration of the piles into the formational sandstone materials may be necessary to resolve lateral forces that act on the piles. As discussed above, we anticipate that embedment of the piles into the formational sandstone materials will be relatively limited, and the penetration that does occur may cause fracturing/disturbance to the formational materials surrounding the pile. Achieving embedment of the piles in to the formational materials will likely require predrilling the formational materials to the desired depth of pile embedment. The diameter of the predrilled boring must be carefully selected to verify that sufficient contact down the length of the pile installed within the formational materials is achieved in order for the lateral support parameters that we have provided in Section 5.3 below (LPILE parameters) to be applicable. At minimum, we suggest that the predrilled boring diameter be sized slightly under the diagonal distance between the outside edges of the pile flange. Preplacement of fluid grout within the predrilled boring may be considered to assure that full engagement of the surface area of the pile against the adjacent supporting materials is occurring. We are available to provide design parameters for the grout if grouting of the piles is determined to be necessary.

It should be noted that the soil materials that overlie the formational materials generally exhibit a relatively high amount of strain to achieve the full undrained shear strength characteristics of the soils. If the overlying soil materials (soil materials above the surface of the formational materials) will be relied on for lateral support of the pile, and the pile locations will be pre-drilled, then we recommend that fluid grout be placed in the pile borings immediately prior to the installation of the pile. Alternatively, it may be possible to backfill the borings with the drill cuttings prior to installation of the pile. Utilizing battered piles to resolve lateral forces may also be considered for the project.

We performed a number of laboratory tests to assess the corrosion potential of the site soils/formational materials on Portland cement and steel components such as driven piles. These test results are discussed in more detail in Section 4.0 above. Based on our review of Section 10.7.5 of the AASHTO LRFD specifications and our laboratory test results, the site soils should not exhibit a high potential to cause corrosion of steel piles. It should be noted that are chemical tests did not include performing tests on the water within Alkali Creek.

5.2 *Drilled Caissons*

The information provided below provides geotechnical engineering design parameters for drilled caisson elements (Section 5.2.1) and general construction considerations for drilled caissons (Section 5.2.2).

Drilled caissons may be used to support the bridge structure, however we anticipate that substantial constructability issues may arise with drilled caissons when compared to drive piles. The primary considerations for the design and constructability of drilled caissons are;

- The formational materials (which provide the necessary end-bearing support stratum for drilled caissons) were encountered at a depth of about 36½ feet below the roadway elevation at our Test Boring TB-1 location. If drilled caissons were used to support the bridge, assuming that the new bridge abutments will be located near the existing bridge abutments and our test borings, the anticipated total depth of the boring would likely be in the range of at least 40 feet below the existing roadway elevation. This depth presents problems associated with dewatering (discussed below) and visual inspection of the bottom of the caisson boring.
- Subsurface free water was encountered at a depth of about 17 feet below the roadway elevation. The presence of substantial depths of subsurface free water within the borings, which will likely be the case in the vicinity of the existing east bridge abutment, will complicate the successful construction of caissons.
- If the caisson borings cannot be visually inspected for cleanliness, or otherwise verified that the bottom of the boring is clear of loose debris immediately prior to the placement of the caisson concrete, ***then we recommend that only the capacities for side resistance be used to account for axial capacity of the caisson.***
- Based on the results of the soluble sulfate test results, CDOT requirements for cementitious materials for Class 2 requirements/conditions should be followed. Production of concrete that meets Class 2 requirements could be costly in the project region.

If the new bridge abutments are located in areas upslope from the existing bridge abutment locations (east from Test Boring TB-1 and west from Test Boring TB-2), then the formational sandstone materials may be encountered at significantly shallower depths. If this is the case, then the feasibility of using drilled caissons for the project may increase.

5.2.1 Geotechnical Engineering Design Parameters for Drilled Caissons

The tip resistance (end bearing capacity) and side resistance (skin friction capacity) values provided below are based on classifying the formational sandstone materials as competent rock.

- The drilled caisson borings should be advanced a minimum of 2 caisson diameters or 6 feet (whichever is greater) into the competent formational sandstone materials. Additional embedment may be required depending on potential scour depth and lateral resistance requirements. It may be necessary to pilot the caisson borings with smaller diameter drilling equipment to achieve this embedment. We must be contacted to assess the tip bearing elevation if refusal of the drilling equipment occurs prior to this depth of embedment. We must be contacted to log the caisson borings as they are being advanced.
- We recommend that a minimum 30 to 36-inch diameter caisson be considered for the project. This is partially based on the anticipated diameter needed to visually inspect the bottom of the caisson borings. Visual inspection of the caisson borings can occur from the top of the boring using high powered lights and/or sunlight reflecting mirrors, in conjunction with probing equipment. The subsurface free water should be removed from the caisson borings to facilitate inspections of the bottom of the caisson boring.
- The end bearing (tip) and side resistance capacities provided below are based on a minimum distance of at least 3 caisson diameters center to center between adjacent caissons.
- An ultimate or nominal end bearing (tip) capacity (q_p) of 150 kips per square foot may be used provided the end bearing tip elevation of the caisson extends a minimum depth of at least 2.0 caisson diameters or 6 feet (whichever is greater) into the competent formational sandstone material, and the bottom of the caisson boring is verified to be clean of loose debris. As discussed above, we do not recommend accounting for tip resistance if the bottom of the caisson boring cannot be adequately inspected or verified to be clean of loose debris immediately prior to the placement of the caisson concrete.
- A resistance factor of 0.50 should be used for tip resistance based on Table 10.5.5.2.4-1 of the AASHTO LRFD design specifications.
- An ultimate or nominal side friction value (q_s) of 20 kips per square foot may be used for the portion of the caisson that extends into the competent formational material for compression and tensional forces.
- A resistance factor of 0.5 should be used for side resistance based on Table 10.5.5.2.4-1 of the AASHTO LRFD design specifications.
- We do not recommend accounting for side resistance from the soil materials that overlie the formational sandstone materials due to the relatively soft and plastic nature of these materials.
- Post construction settlement of the drilled caissons will be less than 1/2 inch based on the capacities provided above.
- Uplift forces due to expansive soil conditions may be disregarded given the laboratory

swell test results that we obtained.

- Forces due to down drag of the existing bridge abutment backfill materials on the caissons may be disregarded based on our understanding of the time frame that the existing abutment backfill materials have been in place (about 40 years). If substantial new fill materials (greater in depth than about 4 feet) are placed over the existing bridge abutment fill materials or undisturbed native soil deposits, then down drag forces may need to be considered. We should be contacted to evaluate potential down drag forces on the caissons if more than about 4 feet of fill material will be placed over the existing abutment backfill materials or native undisturbed soil materials.

The design parameters presented above are based on a minimum spacing distance of at least 3.0 caisson diameters center to center. In general, we do not recommend placing the caissons closer than 3.0 caisson diameters center to center, primarily due to constructability issues that arise with drilling caissons in close proximity to one another.

5.2.2 General Construction Considerations for Drilled Caissons

Successful installation of drilled caissons on this project site will require a relatively large caisson drilling rig and a very experienced caisson drilling contractor with the appropriate drilling heads to advance the borings though the very hard formational sandstone materials. It may be necessary to pre-drill the formational materials with smaller diameter cutting heads to achieve the needed embedment of the caisson boring into the formational materials.

The caissons should be installed using drilling equipment which is good working order and intended for advancing large diameter borings. Proper performance of the drilled caissons requires appropriate drilling and installation techniques. All drilled caissons must be installed by a contractor who is familiar with caisson construction, including casing and dewatering procedures.

Installation of casing may be necessary during advancement of the drilled caisson borings to prevent caving of the soil materials that overlie the formational sandstone materials, and potentially used to decrease the subsurface water flow rate into the caisson borings. The selected caisson drilling contractor must have sufficient experience with installing and or advancing casing during the drilling process.

We recommend that heavy flow rates of water into the caisson boring be planned for. If possible, water that accumulates in the bottom of the caisson boring(s) should be pumped to within a few inches of the bottom of the boring prior to placement of the caisson concrete. If the water is accessing the caisson boring too rapidly to feasibly pump, then the water may be displaced with a tremie that discharges the concrete at the bottom of the caisson boring until the caisson concrete rises above the water casing joint (if used) or above the subsurface free water elevation, effectively sealing additional water flow from entering the caisson boring. Substantial excess water should then be pumped off the surface of the concrete. The remaining caisson concrete may then be placed to the intended top of caisson elevation. The logistical operations of dealing with heavy water flow into the caisson boring must be thoroughly planned to facilitate proper placement of the caisson concrete.

For cases where subsurface free water exists in the caisson boring, sufficient concrete should be expelled from the top of the caisson boring (using a tremie that is placed at the bottom of the boring) to ensure that the concrete throughout the entire depth of the caisson exhibits the mix design water to cement ratio, and segregation of the concrete aggregates has not occurred. The concrete placement tremie tube must not be raised and lowered during placement of the concrete, as this will potentially increase the water to cement ratio of the concrete and cause segregation of the concrete aggregates. Verifying that high water to cement ratios and/or segregation of the concrete aggregates has not occurred within the actual caisson concrete may involve performing additional concrete field tests such as unit weight measurements and molding extra compressive strength test cylinders from the concrete that is located at the top of the caisson.

CDOT specifications for the caisson concrete should be followed for the project, as well as specifications for the actual placement of the caisson concrete. As discussed in Section 4.0 above, based on the chemical testing that we have performed to date, we recommend that the CDOT requirements for cementitious materials for Class 2 requirements be followed.

The support elevation of the caisson must be thoroughly cleaned prior to placement of the concrete. The caisson support elevation may be cleaned using clean-out tools attached to the drill rig, hand equipment, excavation suction equipment, or a combination of these tools. Under no circumstances should the caisson foundation concrete be placed when loose material exists in the bottom of the borings. We recommend placing the caisson steel reinforcement and concrete as soon as possible after the caisson boring has been completed to prevent soil material from caving into the caisson boring. As discussed in Section 5.2.1 above, if it cannot be substantiated that the bottom of the caisson boring is free of loose materials immediately prior to the placement of the caisson concrete, then the end bearing capacity values provided in Section 5.2.1 should be discounted.

We do not feel that it is necessary to perform load testing of the caissons to substantiate the capacities provided above. However, we should be contacted during the caisson installation to;

- evaluate the drill rig specifications proposed for use in the caisson installation,
- review the concrete mix design proposed for use in the caissons,
- measure the depth of the caisson borings,
- verify the competency of the end bearing support materials,
- verify that the bottom of the caisson borings are clean prior to placement of the caisson concrete, and,
- check the plumbness of the caisson borings.

5.3 *LPILE Computer Modelling Input Parameters*

The LPILE parameters provided below may be utilized for lateral design of the deep foundation components. As discussed in Section 5.1 above, the foundation component must exhibit full lateral contact with the various types of support stratums provided below for the parameters to be applicable. If only partial support of the foundation component is obtained, such as for driven H-piles socketed within the formational materials, then the lateral support parameters may need to be reduced. This should be discussed further between the various members of the design team.

The tables provided below present a summary of soil/formational material parameters for use with LPILE computer analysis program for the different subsurface strata encountered in each of our test borings. The depths of the various layers are based on the depths of materials encountered in our test borings from the existing roadway elevation. The applicability of the depths of the various material types presented below will need to be determined based on the proposed elevations of the deep foundation component relative to the elevation of our test borings.

- The referenced soil layer depths are based on the road surface elevation adjacent to our test boring at the time of the field study. The parameters should be adjusted based on the actual deep foundation component elevation.
- LPILE soil types were obtained from LPILE version 2013 computer software.
- The effective unit weight values are based on laboratory determined densities of select soil samples that we obtained during our field study.
- The LPILE “k” value, or soil modulus value was based on default data provided with the LPILE software. Obtaining project specific “k” values would require full-scale load testing of drilled caissons placed on the project site.
- The values for k_rm are estimated.
- The values for Young’s Modulus are based on estimations.
- The rock quality designation (RQD) values are based on actual RQD measurements performed on rock core that we obtained from the test borings.
- The uniaxial compressive strength values for the formational materials (referenced as weak rock) were estimated from the overall strength characteristics of the formational materials, being partially based on the actual unconfined compressive strength tests that we performed.
- Parameters for the existing aggregate base course materials of the current roadway section are not provided due to the limited depth of these materials.

It should be noted that we anticipate some error messages may be obtained in the LPILE models due to variations between the LPILE values tabulated below and the default values that are provided with the program. We should be contacted to discuss the error messages with the project structural engineer to verify that the computer model is accurate for the actual subsurface conditions encountered and our laboratory test data.

LPILE Parameters for Test Boring TB-1 (East Existing Bridge Abutment)

Top of Layer (ft)	Bottom of Layer (ft)	LPILE Soil Type	Unit Weight (pcf)	Undrained Cohesion (psf)	Static p-y Modulus (k) (pci)	Strain Factor (E_m) or (K_{rm})	Young's Modulus (estimated) (psi)	RQD (%)	Uniaxial Compressive Strength (psi)
0	17	Modified Stiff Clay w/o free water	125.0	3000	750	0.005	--	--	--
17	37	Soft clay w/ free water	125.0	1,000	100	0.02	--	--	--
17	50	Weak Rock	135.0	--	--	0.0005	100,000	90	3,000

LPILE Parameters for Test Boring TB-2 (West Existing Bridge Abutment)

Top of Layer (ft)	Bottom of Layer (ft)	LPILE Soil Type	Unit Weight (pcf)	Undrained Cohesion (psf)	Static p-y Modulus (k) (pci)	Strain Factor (E_m) or (K_{rm})	Young's Modulus (estimated) (psi)	RQD (%)	Uniaxial Compressive Strength (psi)
0	17	Modified Stiff Clay w/o free water	125.0	3000	750	0.005	--	--	--
17	27	Soft clay w/ free water	125.0	1,000	100	0.02	--	--	--
27	40	Weak Rock	135.0	--	--	0.0005	100,000	75	2,000

6.0 LATERAL EARTH PRESSURE VALUES/RETAINING STRUCTURES

This section of the report provides lateral earth pressure values for both the existing sandy clay soil materials that are retained by the existing bridge abutment, and values for imported granular fill materials. The AASHTO LRFD bridge design specifications was the primary source that was used to calculate the various lateral earth pressure values provided below. The lateral earth pressure values provided below are based on the following assumptions;

- The retaining wall structure will exhibit a vertical back face (face of wall directly against the retained soil mass).
- The retaining wall structures will consist of concrete. The values provided below may not be appropriate for steel retaining components such as steel sheet piling
- The top of the retained soil mass consists of level backfill, and will not be directly subjected to surcharge or traffic loads.
- The values provided for imported granular fill materials are based on a minimum angle of internal friction (phi) of 35 degrees.
- The values provided below are appropriate for drained soil conditions, and do not include the influence of hydrostatic pressures.
- Equations 3.11.5.2-1 and 3.11.5.3-1 were used to calculate values for k_o and k_a. Figure 3.11.5.4-1 was used to calculated $k_{p.}$

The values provided below for at-rest, active, and passive earth pressures are based on the calculated lateral earth pressure coefficients (k_o, k_a, k_p) multiplied by the estimated moist unit weight of the soil. A moist unit weight of 125 pounds per cubic foot (pcf) was assumed for the existing sandy clay soil materials, while a moist unit weight of 135 pcf was assumed for imported granular fill materials. The backfill height variable (z) should be analyzed based on the project design heights for the retained soils. In addition, we have not included soil cohesion (c) for passive pressure calculations (c=0). Depending on the design situation, we are available to address cohesional characteristics for passive pressures for cohesive soils at your request.

Lateral Earth Pressure Values

Type of Lateral Earth Pressure	Level Sandy Clay Native Soil Backfill (pounds per cubic foot)	Level Imported Granular Backfill (pounds per cubic foot)
At-rest	76	58
Active	48	33
Passive	400	900

We have provided coefficient of friction values for cast in place concrete placed on the native sandy clay soil materials or on a layer of imported aggregate base course such as CDOT Class 6 material below, with the assumption that spread footings may be used to support retaining structures associated with the project. We must be contacted to evaluate the allowable bearing capacity and potential settlement for footings placed on the site soil materials. This may require additional subsurface data and laboratory testing depending on the location of potential footings.

A preliminary coefficient of friction of 0.30 may be used for concrete footings that are cast directly on the existing site native sandy clay soils, while a preliminary coefficient of friction of 0.50 may be used for concrete footings that are cast direct on imported structural fill material such as CDOT Class 6 material.

The values provided above do not include the influence of hydrostatic pressures developing within the retaining wall backfill materials. The project retaining walls must be designed to allow drainage of subsurface water within the retained soil mass.

Backfill should not be placed and compacted behind the retaining structure unless approved by the project structural engineer. Backfill placed prior to construction of all appropriate supporting structural members, or prior to appropriate curing of the retaining wall concrete (if used) may result in severe damage and/or failure of the retaining structure(s).

6.1 *Considerations for Settlement of New Abutment Backfill Materials*

The existing retained bridge abutment materials appear to be relatively well consolidated (based on the current loads that act on these materials) based on our field observations at the interface of the existing supported bridge deck relative to the existing abutment backfill materials. In addition, we are not aware that settlement of the existing bridge abutment fill materials has been an issue in the recent past.

Some post construction settlement of new backfill material will occur regardless of the backfill material characteristics and regardless of the compaction level of the material. If possible, we recommend that new backfill material consist of imported granular fill material such as a CDOT Class 2 aggregate sub-base course or Class 6 aggregate base course materials. We anticipate that at least 1 to possibly 2 percent post construction settlement could occur within properly densified granular backfill materials. Clay soil backfill materials will likely exhibit a significantly higher post construction settlement potential.

The roadway/bridge design should accommodate the potential for future settlement of the abutment backfill (and supported roadway) relative to the bridge abutments. We anticipate that additional asphalt cement pavement will need to be placed periodically at the interface between the bridge abutments and adjacent roadway for some time after construction of the project if the project will require placement of substantial depths of abutment fill material. We are available to evaluate the potential settlement of potential new fill materials as the project design progresses and fill quantities are known. Lean concrete fill materials or cellular concrete may be considered to help reduce the influence of fill settlement, as well as potentially reducing lateral pressures that act on retaining structures.

7.0 ASPHALT PAVEMENT THICKNESS DESIGN RECOMMENDATIONS

This section of our report provides asphalt pavement thickness design recommendations for the new roadway section associated with the bridge project. Existing traffic count data for the roadway and associated calculated 18kip-equivalent single axle load data (18k ESAL values) has not been provided to us at this time. Therefore, we have provided pavement section design recommendations for various 18k ESAL values. The project civil engineer and/or county may select the appropriate 18k ESAL design section based on the current and projected roadway traffic use. The Colorado Department of Transportation (CDOT) 2014 Pavement Design Manual was utilized as the primary source for the recommendation provided below.

The various factors utilized for our recommendations are itemized below.

- Reliability Factor R=90%
- Overall Standard Deviation, S_o=.44
- Change is serviceability index, Delta PSI=2.0
- Structural Coefficient of new Asphalt Pavement = 0.44
- Structural Coefficient of new CDOT Class 6 aggregate base course materials =0.12
- Structural Coefficient of new CDOT Class 2 aggregate subbase course materials=0.09
- Subgrade resilient modulus for the existing subgrade soils under the existing roadway, M_R= 4,500 psi. This value was obtained by estimating the subgrade resilient modulus based on M_R = 1500(CBR). A CBR value of about 3.0 was obtained from the existing roadway section subgrade materials that are compacted to 90 percent of the maximum dry density as established by AASHTO T-180/ASTM D1557.

The subgrade soil materials should be scarified to a depth of 12 inches, moisture conditioned, and compacted to at least 90 percent of the maximum dry density as defined by AASHTO T180. Proof rolling observations should then be performed over the prepared subgrade surface. We recommend that the moisture content of the subgrade soils be within optimum to 2 percent above optimum moisture content. Any areas of significant yielding should be stabilized as needed prior to placement of the overlying aggregate base course materials. The surface of the subgrade soil should be graded and contoured to be parallel to the finished grade of the asphalt surface.

The asphalt pavement used on this project should be mixed in accordance with a design prepared by a licensed professional engineer, or an asphalt pavement specialist. We should be contacted to review the mix design prior to placement at the project site. We recommend that the asphalt pavement be compacted to between 92 and 96 percent of the maximum theoretical density.

The aggregate materials used within the pavement section should conform to the requirements outlined in the current Specifications for Road and Bridge Construction, Colorado Department of Transportation (CDOT). The aggregate base material should be a three-quarter (3/4) inch minus material that conforms to the CDOT Class 6 aggregate base course specifications and have an R-value of at least 78. The aggregate sub-base course should conform to the CDOT specifications for Class 2 material and should have a minimum R-value 70. Aggregate sub-base and base-course materials should be compacted to at least 95 percent of maximum dry density as defined by AASHTO T-180.

Thorough proof rolling with a fully loaded tandem axle water truck should be performed across the prepared aggregate surface prior to placement of the asphalt cement. Any areas that are observed to yield should be stabilized as necessary. We should be contacted to observe the proof rolling operations and provide recommendations for stabilization if necessary.

We have provided pavement section design thicknesses for 100,000 and 250,000 18k ESAL values below. We are available to provide recommendations for other 18k ESAL values at your request. The structural support characteristics of each section are approximately equal. The project civil engineer, or contractor can evaluate the best combination of materials for economic considerations. We recommend that estimations regarding potential future gas/oil industry type traffic be considered for the roadway. The projected volume of heavily loaded truck traffic will have a major influence on the future condition of the roadway and suitable 18k ESAL value that should be designed for.

Pavement Section Design Thickness
100,000 18k ESAL (Design Critical Lane)

Pavement Section Component	**Alternative Thicknesses of Each Component (inches)**				
Asphalt Concrete	3	3	3	4	4
Class 6	4	6	12	4	9
Class 2	11	8	0	6	0
Reconditioned Subgrade	12	12	12	12	12

Pavement Section Design Thickness
250,000 18k ESAL (Design Critical Lane)

Pavement Section Component	**Alternative Thicknesses of Each Component (inches)**		
Asphalt Concrete	4	4	5
Class 6	6	12	8
Class 2	8	0	0
Reconditioned Subgrade	12	12	12

8.0 CONSTRUCTION CONSIDERATIONS

This section of the report provides comments, considerations and recommendations for aspects of the site construction which may influence, or be influenced by the geotechnical engineering considerations discussed above. The information presented below is not intended to discuss all aspects of the site construction conditions and considerations that may be encountered as the project progresses. If any questions arise as a result of our recommendations presented above, or if unexpected subsurface conditions are encountered during construction we should be contacted immediately.

8.1 *Fill Placement Recommendations*

There are several references throughout this report regarding both natural soil and compacted structural fill recommendations. The recommendations presented below are appropriate for the fill placement considerations discussed throughout the report above.

All areas to receive fill, structural components, or other site improvements should be properly prepared and grubbed at the initiation of the project construction. The grubbing operations should include scarification and removal of organic material and soil. No fill material or concrete should be placed in areas where existing vegetation or poor quality or poorly consolidated fill materials exist.

8.1.1 *Natural Soil Fill*

Any natural soil used for any fill purpose should be free of all deleterious material, such as organic material and construction debris. Natural soil fill includes excavated and replaced material or in-place scarified material.

The natural soils should be moisture conditioned, either by addition of water to dry soils, or by processing to allow drying of wet soils. The proposed fill materials should be moisture conditioned to between about optimum and about 2 percent above optimum soil moisture content.

Moisture conditioning of clay or silt soils may require many hours of processing. Water should be added and thoroughly mixed into fine grained soil such as clay or silt the day prior to establish properly moisture conditioned soils. This technique will allow for development of a more uniform moisture content and will allow for better compaction of the moisture conditioned materials.

The moisture conditioned soil should be placed in lifts that do not exceed the capabilities of the compaction equipment used and compacted to at least 90 percent of maximum dry density as defined by AASHTO T-180. We typically recommend a maximum fill lift thickness of 6 inches for hand operated equipment and 8 to 10 inches for larger equipment. Care should be exercised in placement of utility trench backfill so that the compaction operations do not damage the underlying utilities. Rocks larger that about 3 inches in diameter should be discarded from the fill materials.

8.1.2 *Granular Compacted Fill*

Granular compacted fill is referenced in numerous locations throughout the text of this report. Granular compacted fill should be constructed using an imported commercially produced rock product such as aggregate road base. In general, we recommend that CDOT Class 6 or Class 2 specification products be used for backfill materials. Alternative backfill materials may be appropriate for the project depending on the intended use of the material. We are available to review proposed imported granular fill materials for the project.

All compacted fill below roadway areas or behind retaining wall structures should be moisture conditioned and compacted to at least 90 percent of maximum dry density as defined by AASHTO T-180, Modified Proctor test. Areas where aggregate base course will directly support traffic loads under concrete slabs or asphalt concrete should be compacted to at least 95 percent of maximum dry density as defined by AASHTO T-180.

Clean aggregate fill, if appropriate for the site soil conditions, must not be placed in lifts exceeding 8 inches and each lift should be thoroughly vibrated, preferably with a plate-type vibratory compactor prior to placing overlying lifts of material or structural components. We should be contacted prior to the use of clean aggregate fill materials to evaluate their suitability for use on this project.

8.2 *Excavation Considerations*

Unless a specific classification is performed, the site soils should be considered as an Occupational Safety and Health Administration (OSHA) Type C soil and should be sloped and/or benched according to the current OSHA regulations. Excavations should be sloped and benched to prevent wall collapse. Any soil can release suddenly and cave unexpectedly from excavation walls, particularly if the soils are very moist, or if fractures within the soil are present. Daily observations of the excavations should be conducted by OSHA competent site personnel to assess safety considerations.

If possible, excavations should be constructed to allow for water flow from the excavation the event of precipitation during construction. If this is not possible it may be necessary to remove water from snowmelt or precipitation from the foundation excavations to help reduce the influence of this water on the soil support conditions and the site construction characteristics.

8.2.1 *Excavation Cut Slopes*

We anticipate that both permanent and temporary excavation or embankment fill slopes will be included with the project. Temporary cut slopes should not exceed 5 feet in height and should not be steeper than about 1:1, horizontal to vertical. Permanent excavation or embankment fill slopes of greater than 5 feet or steeper than 2½:1, h:v must be analyzed on a site specific basis.

9.0 CONSTRUCTION MONITORING AND TESTING

Construction monitoring including engineering observations and materials testing during construction is a critical aspect of the geotechnical engineering contribution to any project. Unexpected subsurface conditions are often encountered during construction. The site foundation excavation should be observed by the geotechnical engineer or a representative during the early stages of the site construction to verify that the actual subsurface soil and water conditions are consisting with the subsurface materials encountered in our test borings. If the subsurface conditions encountered during construction are different than those that were the basis of the geotechnical engineering report then modifications to the design may be implemented prior to placement of fill materials or foundation concrete.

Compaction testing of fill material should be performed throughout the project construction so that the engineer and contractor may monitor the quality of the fill placement techniques being used at the site. We recommend that compaction testing be performed for any fill material that is placed as part of the site development. Compaction tests should be performed on each lift of material placed in areas proposed for support of structural components. In addition to compaction testing we recommend that the grain size distribution, clay content and swell potential be evaluated for any imported materials that are planned for use on the site. Concrete tests should be performed on foundation concrete and flatwork. We are available to develop a testing program for soil, aggregate materials, concrete and asphaltic concrete for this project.

10.0 CONCLUSIONS AND CONSIDERATIONS

The information presented in this report is based on our understanding of the proposed construction that was provided to us and on the data obtained from our field and laboratory studies. We recommend that we be contacted during the design and construction phase of this project to aid in the implementation of our recommendations. Please contact us immediately if you have any questions, or if any of the information presented above is not appropriate for the proposed site construction.

The recommendations presented above are intended to be used only for this project site and the proposed construction which was provided to us. The recommendations presented above are not suitable for adjacent project sites, or for proposed construction that is different than that outlined for this study.

Our recommendations are based on limited field and laboratory sampling and testing. Unexpected subsurface conditions encountered during construction may alter our recommendations. We should be contacted during construction to observe the exposed subsurface soil conditions to provide comments and verification of our recommendations. We are available to review and tailor our recommendations as the project progresses and additional information which may influence our recommendations becomes available.

Please contact us if you have any questions, or if we may be of additional service.

Respectfully submitted,
TRAUTNER GEOTECH

Jonathan P. Butler, P.E.
Staff Geotechnical Engineer

APPENDIX A

Logs of Test Borings

TRAUTNER GEOTECH LLC

Field Engineer	: J. Butler
Hole Diameter	: 4 inch/NWL
Drilling Method	: 4" Solid Auger/NWL wireline
Sampling Method	: Mod. California/Core
Date Drilled	: 04/25/2019
Total Depth	: 50.5 feet
Location	: 6' E. of E. Bridge Abutment : C.L. of Westbound Lane
Elevation	: Approx. 19' above creek F.L.
Water Table	: Approx. 17 feet

LOG OF TEST BORING TB-1

Montezuma County Road N
Bridge Replacement Project
Montezuma County, Colorado
Bechtolt Engineering Inc., Mr. Rich Bechtolt

PN: 55458GE

Sample Type: Bag Sample; Core Run; Standard Split Spoon; Mod. California Sampler

Water Level: Water Level During Drilling; Water Level After Drilling

Depth in feet	DESCRIPTION	USCS	Blow count/Run	Water Level	REMARKS,RECOVERY,R.Q.D.
0–2	2 inch Chip Seal Surface over 22 inches of Aggregate Base Course				
2–10	CLAY, sandy, medium stiff to stiff, very moist to wet, dark brown	CL	2/6 3/6 (3–4 ft); 4/6 5/6 (8–9 ft)		
10–14.5	CLAY, sandy, gravels, few cobbles, stiff, moist, brown	CL	6/6 5/6 5/6 (13–14.5 ft)		
14.5–17.5	CLAY, GRAVEL, sandy, few cobbles, stiff/medium dense, moist, brown	CL/GC		▽ (about 17 ft)	Water Level After Drilling
17.5–20.5	CLAY, SAND, few gravels, stiff/medium dense, wet, tan	CL/SC	3/6 5/6 7/6 (18–19.5 ft)		
20.5–25.5	SAND, clayey, medium dense, wet, tan	SC	5/6 9/6 (21 ft); 4/6 5/6 (23–24 ft)		
25.5–37	CLAY, slightly sandy, stiff, wet, dark brown	CL	5/6 5/6 (28–29 ft)		
37–38	Dakota Sandstone Formation at 36.5 feet				
38–50	Begin Core at 38', Dakota Sandstone Formation, Medium to Coarse Grained Sandstone, very hard, low fracturing, white		First Run		38 feet to 40.7 inches Recovery=100% R.Q.D.=100%
			Second Run		40.7 feet to 45.7 inches Recovery=100% R.Q.D.=98%
			Third Run		45.7 feet to 50.5 feet Recovery=100% R.Q.D.=83%
50–51	Shale, highly fractured, black				
52	Bottom of Test Core at 50.5 feet				

Soil profile near channel elevation.

TRAUTNER GEOTECH LLC

LOG OF TEST BORING TB-2

Field Engineer	: J. Butler
Hole Diameter	: 4 inch/NWL
Drilling Method	: 4" Solid Auger/NWL wireline
Sampling Method	: Mod. California/Core
Date Drilled	: 04/25/2019
Total Depth	: 41 feet
Location	: 6' W. of W. Bridge Abutment
	: C.L.of Eastbound Lane
Elevation	: Approx. 19' above creek F.L.
Water Table	: Approx. 17.5 feet

Montezuma County Road N
Bridge Replacement Project
Montezuma County, Colorado
Bechtolt Engineering Inc., Mr. Rich Bechtolt

PN: 55458GE

Sample Type
- Bag Sample
- Core Run
- Standard Split Spoon
- Mod. California Sampler

Water Level
- ▼ Water Level During Drilling
- ▽ Water Level After Drilling

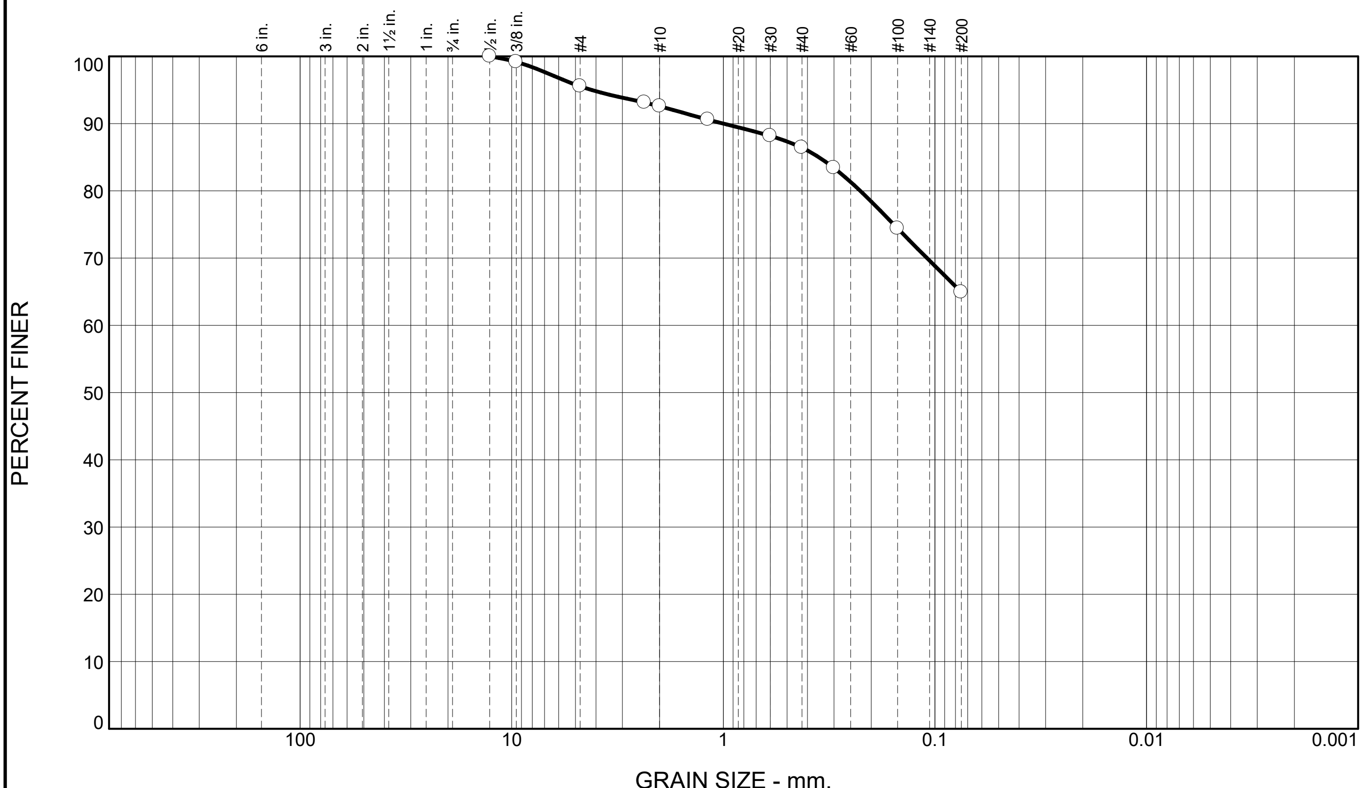
Depth in feet	DESCRIPTION	USCS	GRAPHIC	Samples	Blow count/Run	Water Level	REMARKS,RECOVERY,R.Q.D.
0–2.5	2 inch Chip Seal Surface over 28 inches of Aggregate Base Course			Bag Sample			
2.5–15	CLAY, sandy, medium stiff to stiff, moist, tan	CL		Bag Sample			
3.5–5				Standard Split Spoon	3/6 2/6 2/6		
8.5–9.5				Mod. California Sampler	4/6 5/6		
13.5–14.5				Mod. California Sampler	4/6 6/6		
15–26	CLAY, sandy, few gravels, stiff, very moist to wet, dark brown, some gypsum crystals	CL					
17.5						▽	Water Level After Drilling
18.5–19.5				Mod. California Sampler	3/6 6/6		
23.5–24.5				Mod. California Sampler	4/6 6/6		
26–28.5	Dakota Sandstone Formation at 26 feet, Sandstone, very hard, fractured, wet, white						
28.5				Standard Split Spoon	50/2		
28.5–30	Begin Core at 28.5 feet, Dakota Sandstone Formation, Medium to Coarse Grained Sandstone, very hard			Core Run	First Run		28.5 feet to 31 feet Recovery=85% R.Q.D.=50%
30–31	Moderately to Highly Fractured, tan			Core Run			
31–36	Moderately Fractured			Core Run	Second Run		31 feet to 36 feet Recovery=100% R.Q.D.=75%
36–41	Medium to Coarse Grained Sandstone, low fracturing, white			Core Run	Third Run		36 feet to 41 feet Recovery=100% R.Q.D.=96%
41	Bottom of Test Core at 41 feet						

Soil profile near channel
elevation

APPENDIX B

Laboratory Test Result

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	4	3	7	21	65	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.50	100		
.375	99		
#4	96		
#8	93		
#10	93		
#16	91		
#30	88		
#40	86		
#50	83		
#100	74		
#200	65		

* (no specification provided)

Material Description

CL Sandy Lean Clay

Atterberg Limits

PL= 14 LL= 29 PI= 15

Coefficients

D_{90}= 1.0017 D_{85}= 0.3539 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(7)

Remarks

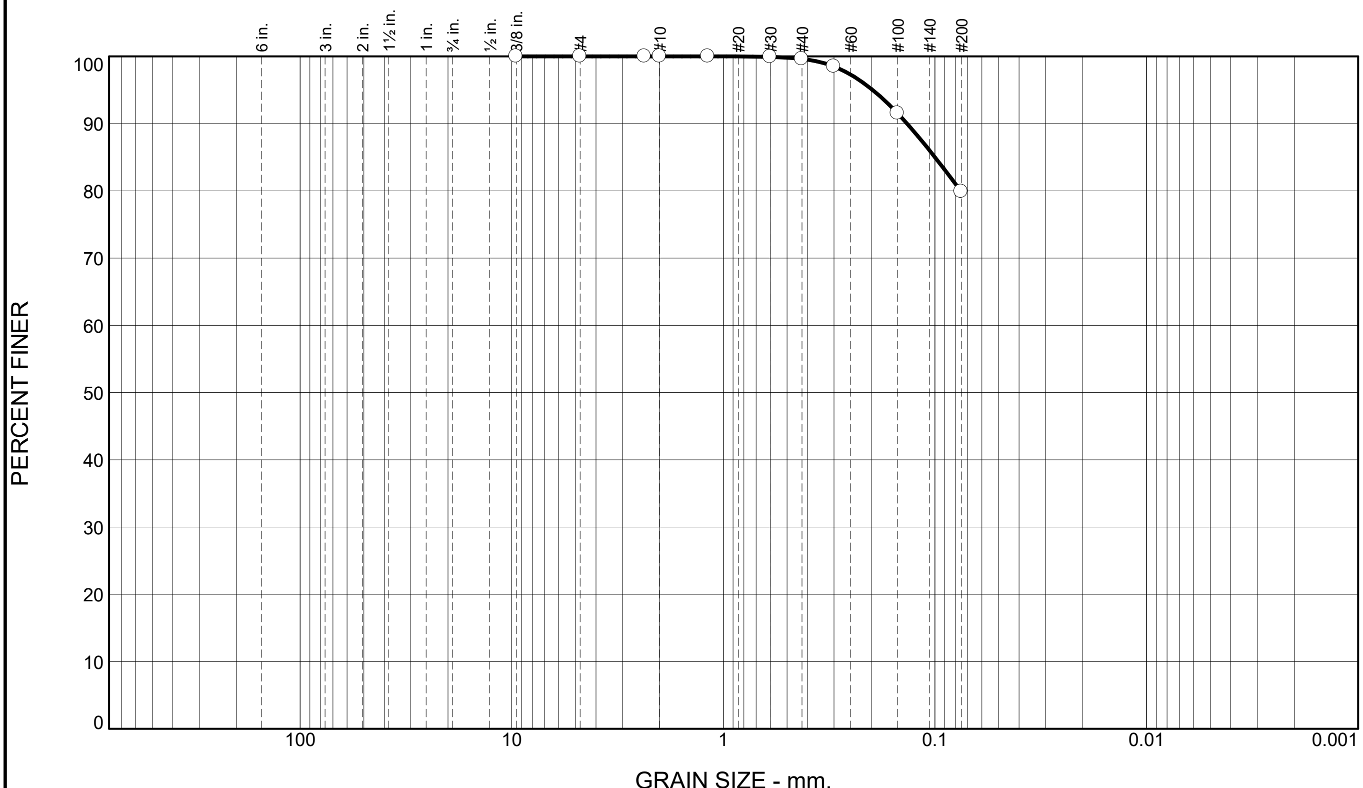
Location: TB-1
Sample Number: C10223-d **Depth:** 2'-3' **Date:** 4/26/19

TRAUTNER GEOTECH LLC

Client: Bechtolt Engineering Inc., Mr. Rich Bechtolt
Project: County Road N Bridge Replacement Project
Project No: 55458GE **Figure** 4.1

Tested By: R. Barrett **Checked By:** J. Butler

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	0	20	80	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375	100		
#4	100		
#8	100		
#10	100		
#16	100		
#30	100		
#40	100		
#50	98		
#100	92		
#200	80		

* (no specification provided)

Soil Description

CL Lean Clay with Sand

Atterberg Limits

PL= 16 LL= 31 PI= 15

Coefficients

D_{90}= 0.1354 D_{85}= 0.1000 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(10)

Remarks

Location: TB-1
Sample Number: C10223-L+M **Depth:** 22' **Date:** 4/26/19

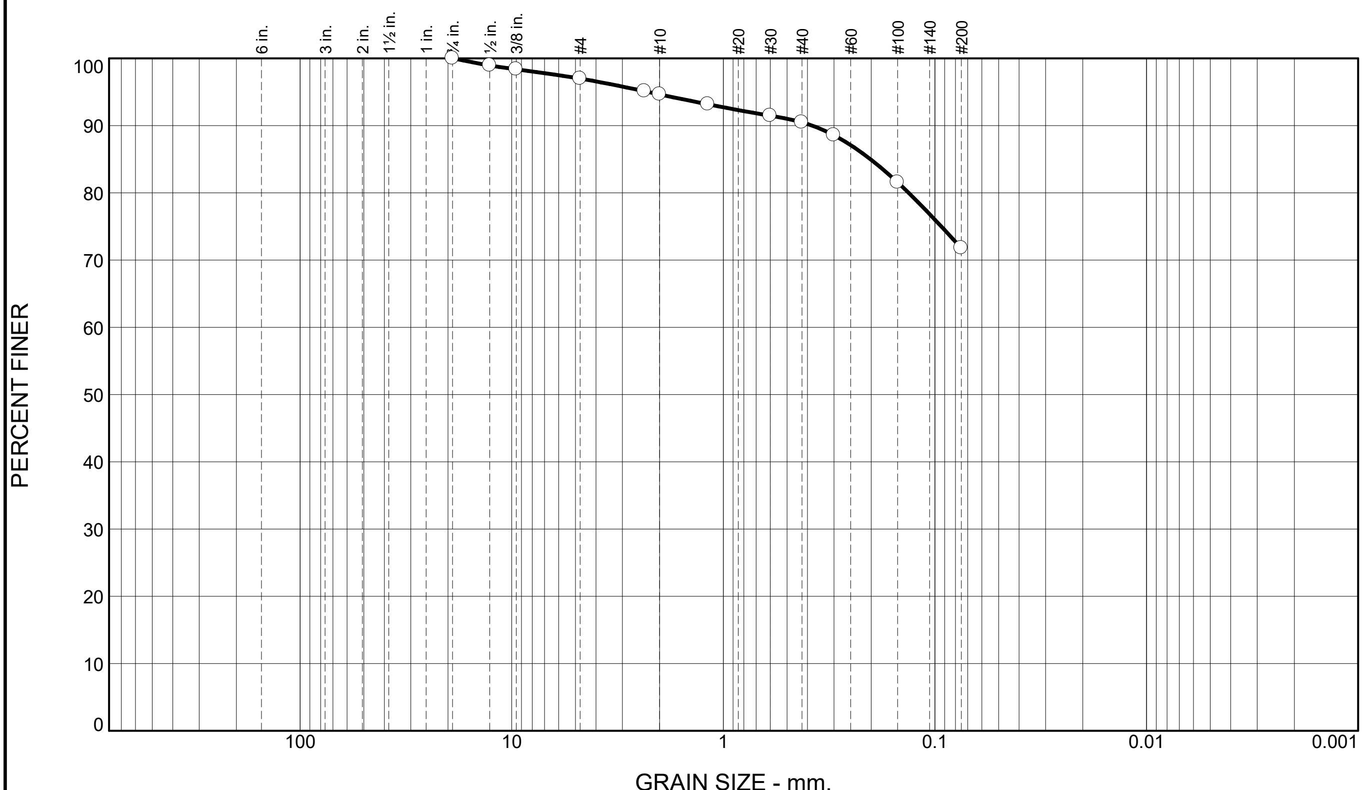
TRAUTNER GEOTECH LLC

Client: Bechtolt Engineering Inc., Mr. Rich Bechtolt
Project: County Road N Bridge Replacement Project

Project No: 55458GE **Figure** 4.2

Tested By: R. Barrett **Checked By:** J. Butler

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	3	2	5	18	72	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75	100		
.50	99		
.375	98		
#4	97		
#8	95		
#10	95		
#16	93		
#30	91		
#40	90		
#50	89		
#100	82		
#200	72		

* (no specification provided)

Material Description

CL Lean Clay with Sand

Atterberg Limits

PL= 16 LL= 32 PI= 16

Coefficients

D_{90}= 0.3809 D_{85}= 0.2017 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(9)

Remarks

Location: TB-2
Sample Number: C10223-W **Depth:** 9.5'-13.5' **Date:** 4/26/19

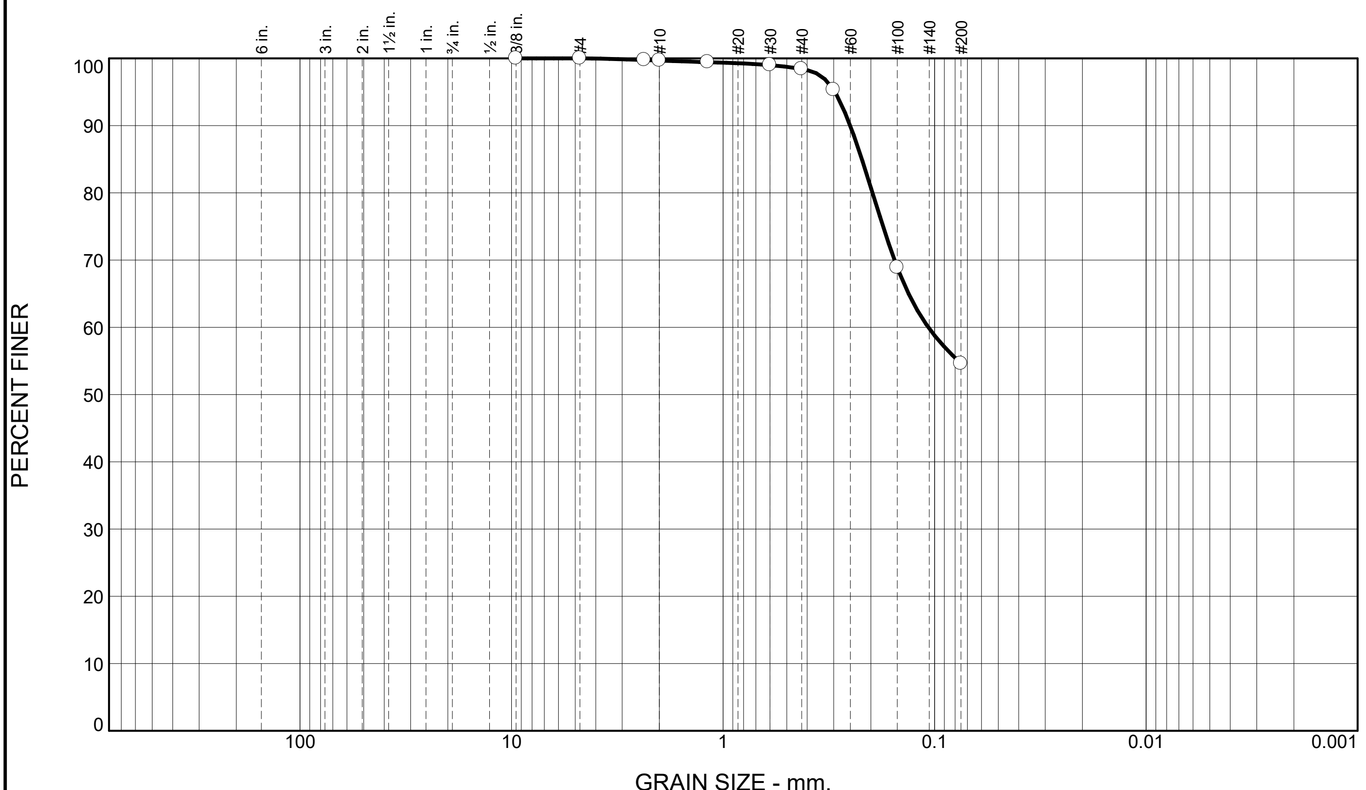
TRAUTNER GEOTECH LLC

Client: Bechtolt Engineering Inc., Mr. Rich Bechtolt
Project: County Road N Bridge Replacement Project

Project No: 55458GE **Figure** 4.3

Tested By: R. Barrett **Checked By:** J. Butler

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	2	43	55	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375	100		
#4	100		
#8	100		
#10	100		
#16	99		
#30	99		
#40	98		
#50	95		
#100	69		
#200	55		

* (no specification provided)

Material Description

CL Sandy Lean Clay

Atterberg Limits

PL= 14 LL= 24 PI= 10

Coefficients

D_{90}= 0.2505 D_{85}= 0.2207 D_{60}= 0.1070
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-4(2)

Remarks

Location: TB-2
Sample Number: C10223-CC **Depth:** 19.5'-23.5' **Date:** 4/26/19

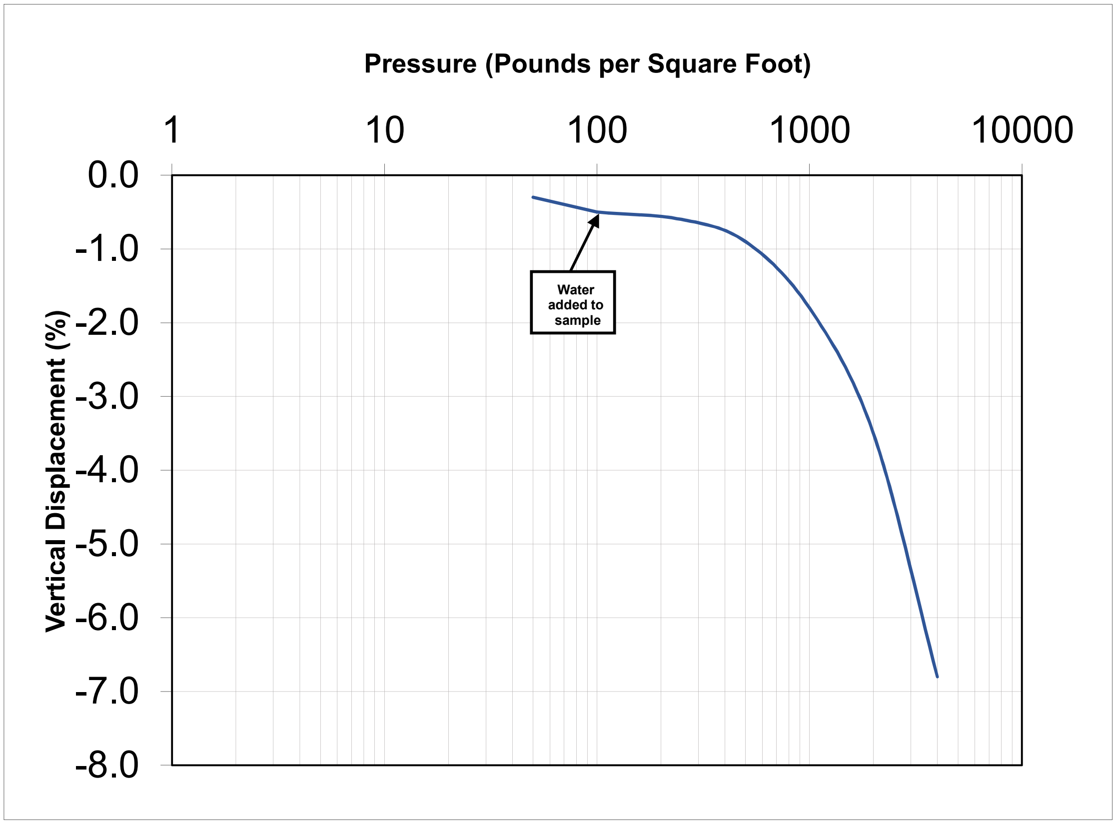
TRAUTNER GEOTECH LLC

Client: Bechtolt Engineering Inc., Mr. Rich Bechtolt
Project: County Road N Bridge Replacement Project

Project No: 55458GE **Figure** 4.4

Tested By: R. Barrett **Checked By:** J. Butler

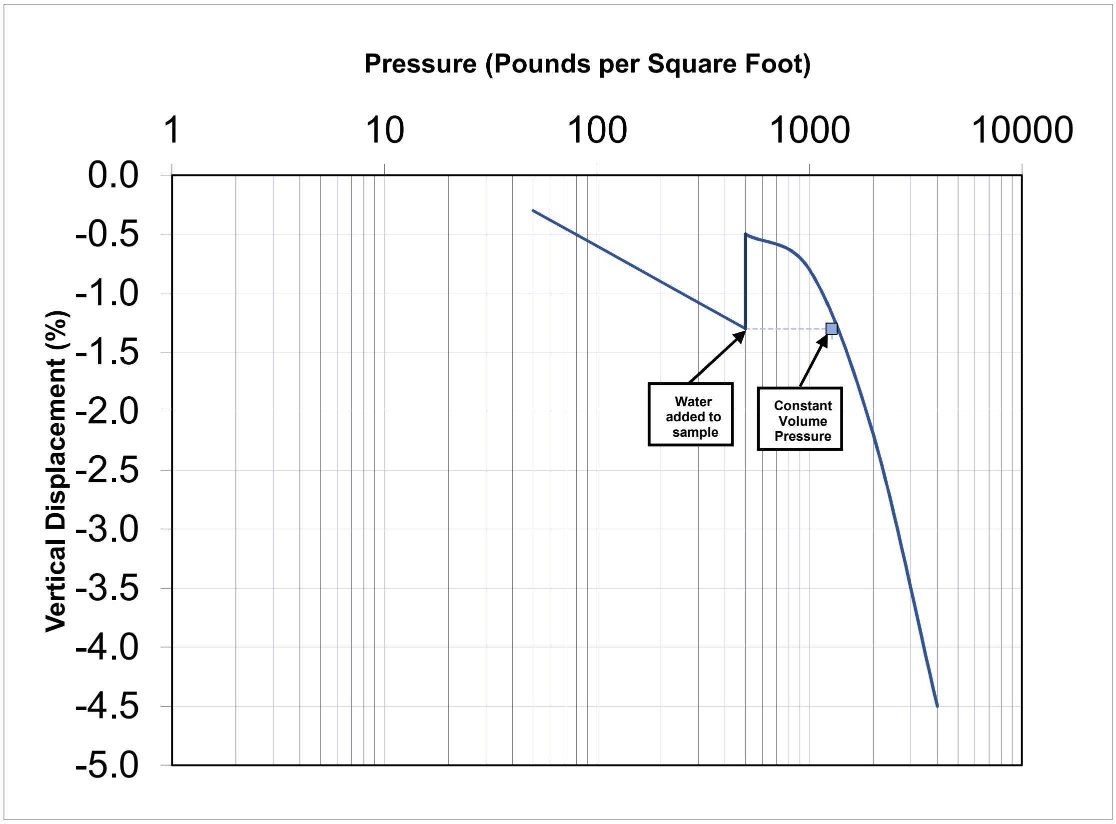
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-1@3'	
Visual Soil Description:	CL Sandy Lean Clay	
Swell Potential (%)	Consolidated	
Constant Volume Swell Pressure (lb/ft^2):	N/A	
	Initial	**Final**
Moisture Content (%):	14.5	17.2
Dry Density (lb/ft^3):	110.0	115.6
Height (in.):	1.000	0.932
Diameter (in.):	1.94	1.94

Project Number:	55458GE
Sample ID:	C10223-E
Figure:	4.5

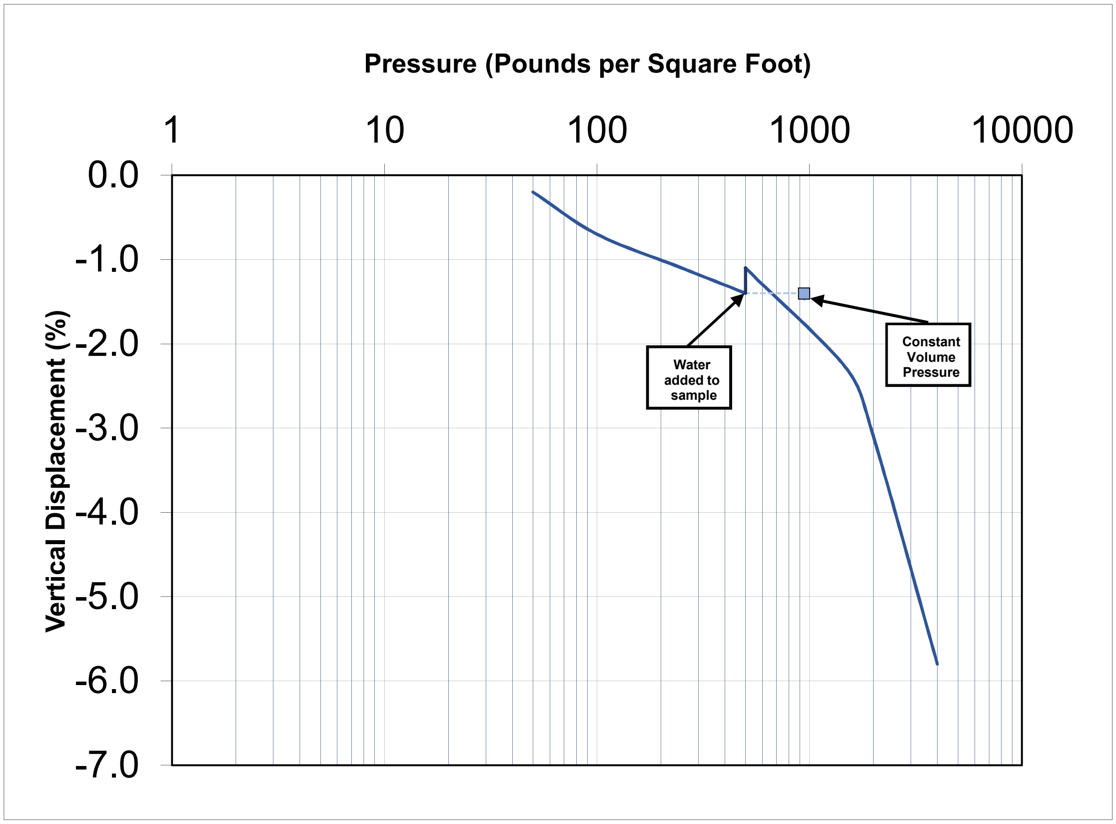
SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-2@8.5'	
Visual Soil Description:	CL Lean Clay with Sand	
Swell Potential (%)	0.8%	
Constant Volume Swell Pressure (lb/ft²):	1,270	
	Initial	**Final**
Moisture Content (%):	10.5	18.5
Dry Density (lb/ft³):	115.0	116.0
Height (in.):	1.000	0.955
Diameter (in.):	1.94	1.94

Project Number:	55458GE
Sample ID:	C10223-V
Figure:	4.6

SWELL - CONSOLIDATION TEST



SUMMARY OF TEST RESULTS		
Sample Source:	TB-2@13.5'	
Visual Soil Description:	CL Lean Clay with Sand	
Swell Potential (%)	0.3%	
Constant Volume Swell Pressure (lb/ft^2):	940	
	Initial	**Final**
Moisture Content (%):	10.5	18.5
Dry Density (lb/ft^3):	115.0	117.6
Height (in.):	1.000	0.942
Diameter (in.):	1.94	1.94

Project Number:	55458GE
Sample ID:	C10223-X
Figure:	4.7

FIGURE: 4.8

UU Test Results for TB-1 Samples

Unconsolidated Undrained Triaxial Test (ASTM D2850)

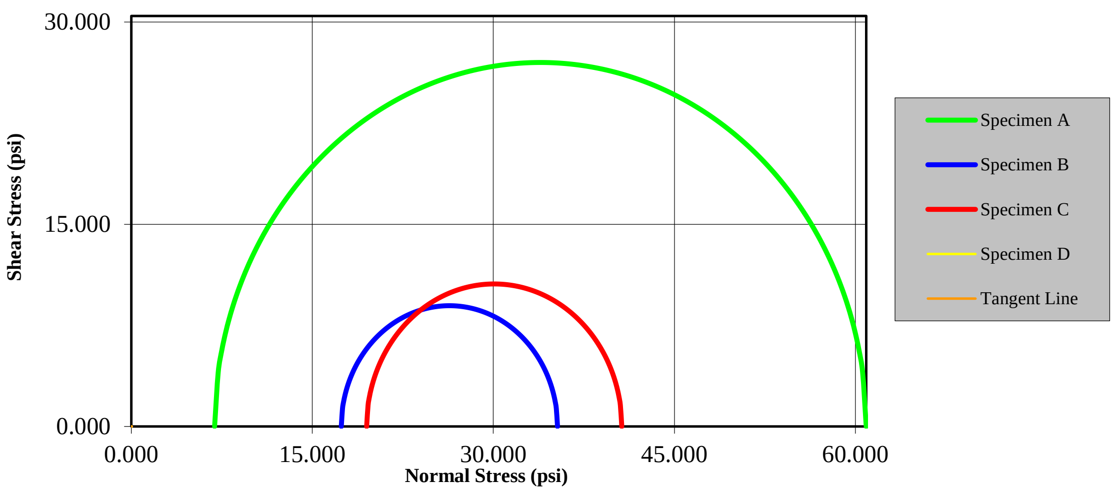
Date :

Checked By:

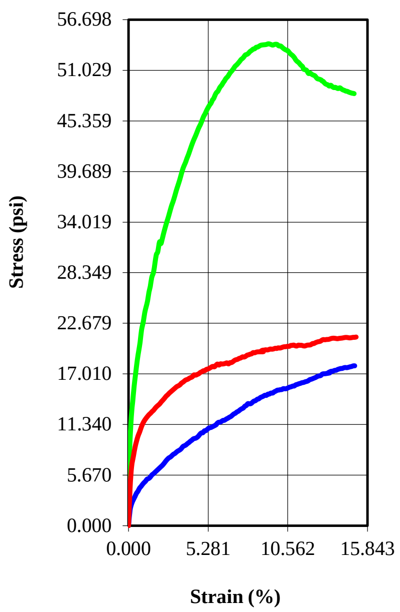
Date:

Tested By:

Mohr Circles



Stress-Strain Curve



	Specimen			
Before Test	**A**	**B**	**C**	**D**
Water Content (%)	13.08	21.49	21.37	0.00
Dry Density (pcf)	111.19	105.21	105.97	0.00
Saturation (%)	71.04	99.48	100.0	0.00
Void Ratio	0.49	0.57	0.56	0.00
Diameter (in)	1.935	1.940	1.940	0.000
Height (in)	4.004	3.995	3.970	0.000
Liquid Limit				
Plastic Limit				
Specific Gravity	2.650	2.650	2.650	
After Test	**A**	**B**	**C**	**D**
Water Content (%)	n/a	n/a	n/a	0.00
Test Data	**A**	**B**	**C**	**D**
Strain Rate (in/min)	0.040	0.040	0.040	0.00
Peak Deviator Stress (psi)	53.998	17.916	21.142	0.000
Axial Strain @ Failure (%)	9.261	14.994	15.088	0.000
	Cell Pressure			
Cell (psi)	6.9	17.4	19.5	0.0
Back (psi)	n/a	n/a	n/a	n/a
	Principle Stresses at Failure			
σ1 (psi)	60.9	35.3	40.6	0.0
σ3 (psi)	6.9	17.4	19.5	0.0

Mohr-Coulomb Strength Parameters		**Sample Description**
C (psi)	Section 4.0 of Report	Sandy clay (CL) AASHTO A-6
Friction Angle Ø	n/a	

Project Information			
Project Name:	CR N-Alkali Creek Bridge		
Project Number:	55458GE	CDOT Number:	BRO C320-004
Location:	Sample A @ 8', B @23', C @28'	Boring Number:	TB-1
Client:	Bechtolt Engineering	Sample Number:	C10223G, O, P
Remarks:	Test Results Discussed in Section 4.0 of Report		

Figure 4.9

UU Test Results for TB-2 Samples

Unconsolidated Undrained Triaxial Test (ASTM D2850)

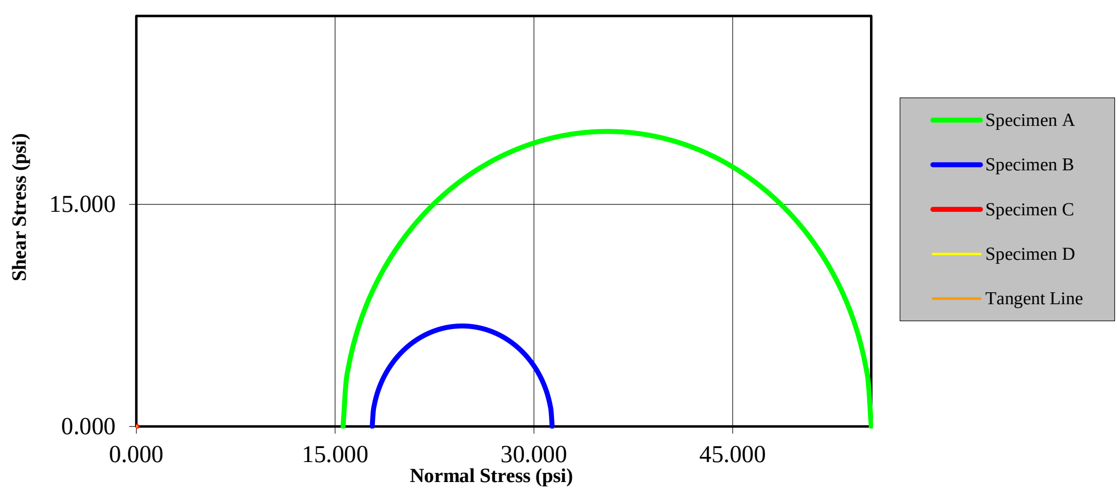
Date :

Checked By:

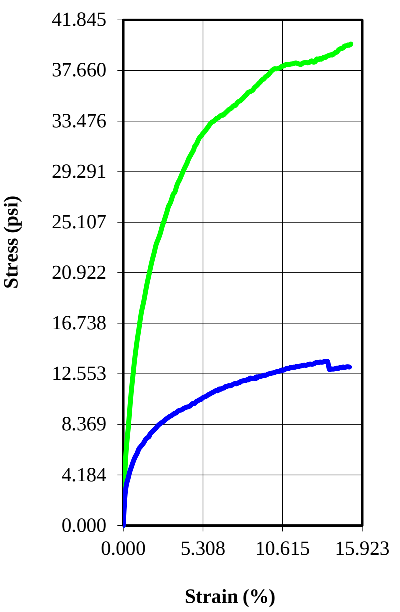
Date:

Tested By:

Mohr Circles



Stress-Strain Curve



	Specimen			
Before Test	**A**	**B**	**C**	**D**
Water Content (%)	16.00	21.00	0.00	0.00
Dry Density (pcf)	114.71	108.02	0.00	0.00
Saturation (%)	95.88	100.0	0.00	0.00
Void Ratio	0.44	0.53	0.00	0.00
Diameter (in)	1.934	1.935	0.000	0.000
Height (in)	3.950	3.975	0.000	0.000
Liquid Limit				
Plastic Limit				
Specific Gravity	2.650	2.650		
After Test	**A**	**B**	**C**	**D**
Water Content (%)	n/a	n/a	0.00	0.00
Test Data	**A**	**B**	**C**	**D**
Strain Rate (in/min)	0.040	0.040	0.00	0.00
Peak Deviator Stress (psi)	39.852	13.574	0.000	0.000
Axial Strain @ Failure (%)	15.165	13.409	0.000	0.000
	Cell Pressure			
Cell (psi)	15.6	17.8	0.0	0.0
Back (psi)	n/a	n/a	n/a	n/a
	Principle Stresses at Failure			
σ1 (psi)	55.5	31.4	0.0	0.0
σ3 (psi)	15.6	17.8	0.0	0.0

Mohr-Coulomb Strength Parameters		**Sample Description**
C (psi)	Section 4.0 of Report	Sandy Clay (CL) AASHTO A-6
Friction Angle Ø	n/a	

Project Information			
Project Name:	CR N-Alkali Creek Bridge		
Project Number:	55548GE	CDOT Job Number:	BRO C320-004
Location:	Sample A @18.5', B @23.5'	Boring Number:	TB-2
Client:	Bechtolt Engineering	Sample Number:	C10223BB, EE
Remarks:	Test Results Discussed in Section 4.0 of Report		

Direct Shear Test Results

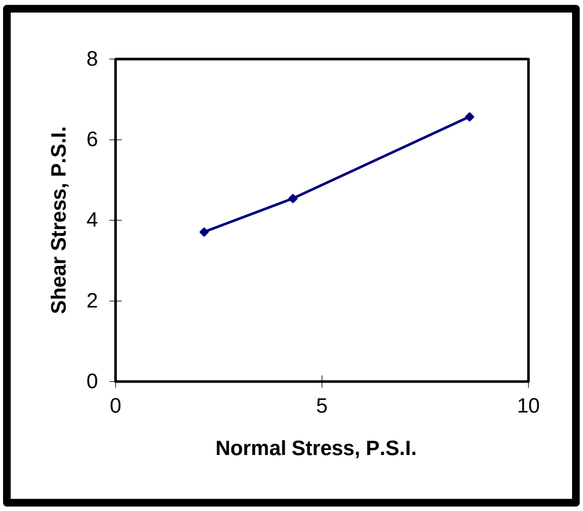
ASTM D3080-90

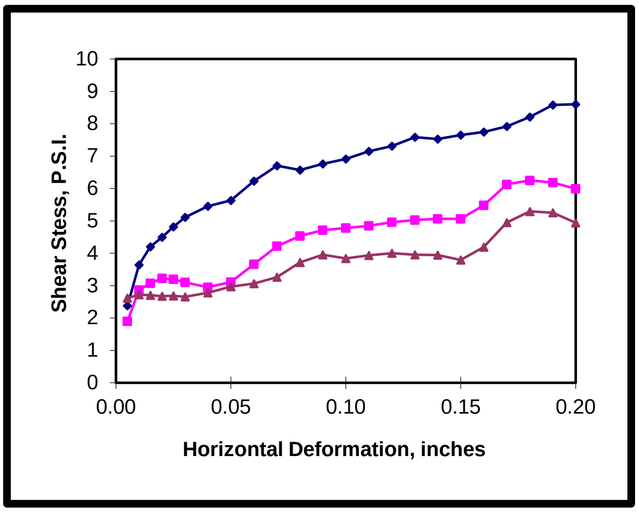
Project: CR N-Alakali Creek Bridge
Project Number: 55458GE
Laboratory Number: C10223W
Date: 4/16/2019
Project Technician: R.B.
Figure 4.10

Visual Soil Description: Sandy Clay (CL)
Type of Specimen: Remolded
Diameter 1.946 in.
Thickness 2.0 in
Sample Source: TB-2; 9.5'-13.5'

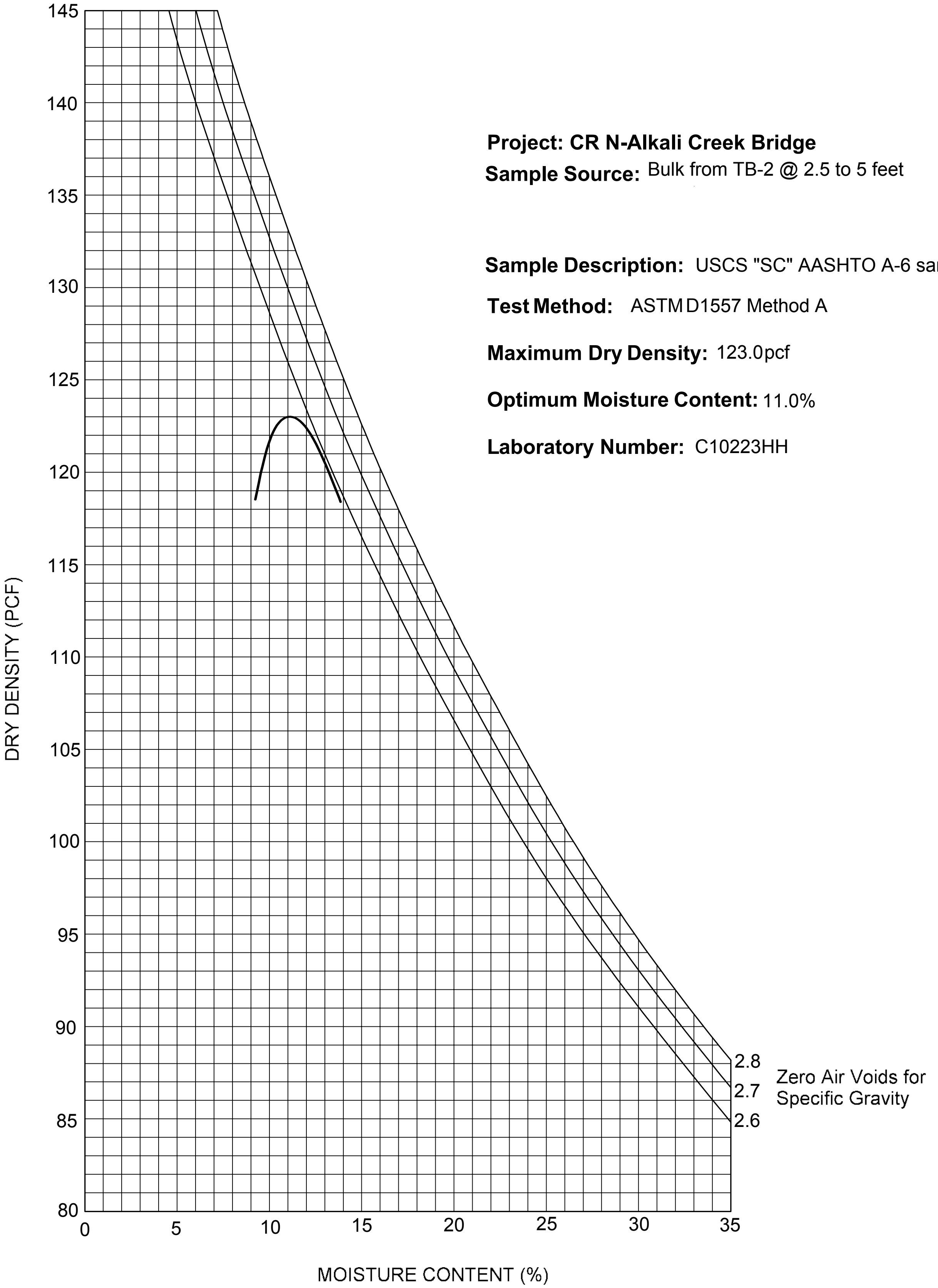
Summary of Sample Data:	
Initial Moisture Content (%)	15.6
Intial Dry Density (P.C.F)	116.6
Final Moisture Content (%)	15.8
Final Dry Density (P.C.F)	116.8

Residual Direct Shear Test Results:			
Normal Stress (P.S.I)	2.14	4.29	8.57
Max. Shear Stress (P.S.I)	3.71	4.54	6.57





ESTIMATED STRENGTH PARAMETERS	
Angle of Internal Friction, phi	23
Cohesion, P.S.F.	390



Project: CR N-Alkali Creek Bridge

Sample Source: Bulk from TB-2 @ 2.5 to 5 feet

Sample Description: USCS "SC" AASHTO A-6 sandy clay

Test Method: ASTM D1557 Method A

Maximum Dry Density: 123.0pcf

Optimum Moisture Content: 11.0%

Laboratory Number: C10223HH

TRAUTNER GEOTECH LLC

Project No. : 55458GE	
Date: 4/26/19	
Figure: 4.11	

California Bearing Ratio Test Results
ASTM D1883

PROJECT NAME: CR N/Alkali Creek Bridge | PROJ NO: 55458GE | Date: 5/14/19
TECHNICIAN: JB

C10223HH

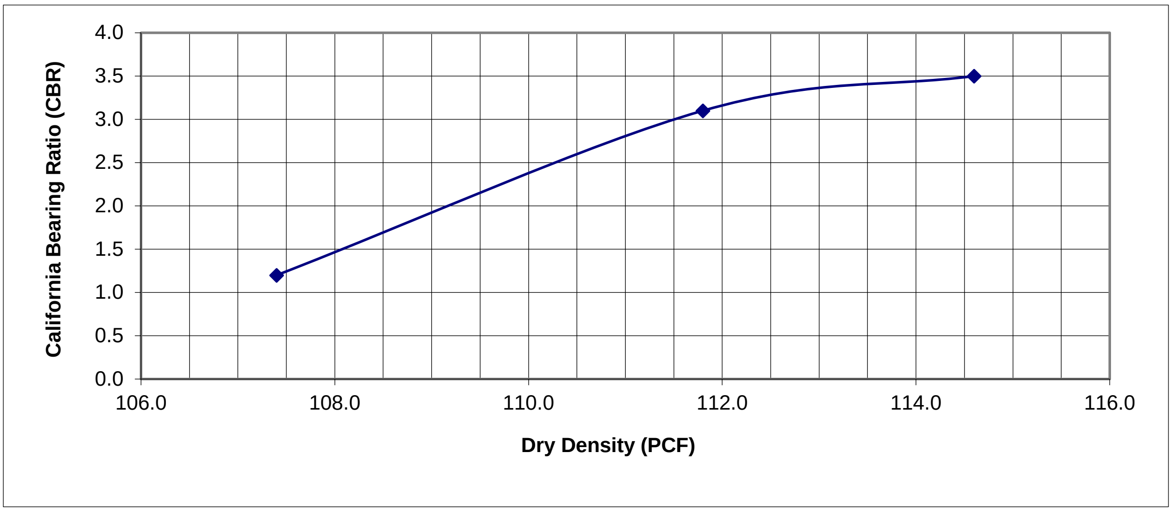
Figure 4.12

Proctor Method: ASTM D1557-A
Max Dry Density: 123.0 pcf
Optimum Moisture Content: 11.0%

Condition: soaked
Surcharge: 15 Lbs

Sample Source: TB-2; 2.5-5 feet

Pre-Soak			After 72 hour Soak			
Dry Density (PCF)	Moisture Content (%)	Relative Compaction (%)	Dry Density (PCF)	Moisture Content of Top One (1) Inch (%)	Swell (%)	CBR (0.100" penetration)
107.4	10.7	87	104.4	23.4	4.1	1.2
111.8	9.9	91	108.5	21.7	4.2	3.1
114.6	10.4	93	111.0	20.7	3.9	3.5



APPENDIX C

Chemical Test Result

dzufelt@greenanalytical.com p: 970.247.4220 f: 970.247.4227 75 Suttle Street Durango, CO 81303

www.GreenAnalytical.com

Trautner Geotech	Project: SO4, Cl, and pH	
649 Tech Ctr. Dr	Project Name / Number: [none]	**Reported:**
Durango CO, 81301	Project Manager: Ross Barrett	05/10/19 07:34

ANALYTICAL REPORT FOR SAMPLES

Sample ID	Laboratory ID	Matrix	Date Sampled	Date Received	Notes
TB-2 @ 19.5'-23.5' C10223 CC	1905002-01	Solid	04/30/19 08:08	04/30/19 08:13	
TB-1 @ 13' C10223 - H	1905002-02	Solid	04/30/19 08:08	04/30/19 08:13	
TB-2 @ 9.5'-13.5' C10223 - W	1905002-03	Solid	04/30/19 08:08	04/30/19 08:13	

Green Analytical Laboratories

Debbie Zufelt

Debbie Zufelt, Reports Manager

dzufelt@greenanalytical.com p: 970.247.4220 f: 970.247.4227 75 Suttle Street Durango, CO 81303
www.GreenAnalytical.com

Trautner Geotech	Project: SO4, Cl, and pH	
649 Tech Ctr. Dr	Project Name / Number: [none]	**Reported:**
Durango CO, 81301	Project Manager: Ross Barrett	05/10/19 07:34

TB-2 @ 19.5'-23.5' C10223 CC

1905002-01 (Solid)

Analyte	**Result**	RL	MDL	Units	Dilution	Analyzed	Method	Notes	Analyst
General Chemistry									
% Dry Solids	**81.2**			%	1	05/06/19	EPA160.3/1684		VJL
Soluble (DI Water Extraction)									
Chloride	**157**	12.3	1.26	mg/kg dry	10	05/07/19	EPA300.0		AES
pH	**7.39**			pH Units	1	05/01/19	9040C		VJL
Sulfate	**4550**	123	26.2	mg/kg dry	100	05/07/19	EPA300.0		AES

Green Analytical Laboratories

Debbie Zufelt

Debbie Zufelt, Reports Manager

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Trautner Geotech 649 Tech Ctr. Dr Durango CO, 81301	Project: SO4, Cl, and pH Project Name / Number: [none] Project Manager: Ross Barrett	**Reported:** 05/10/19 07:34

TB-1 @ 13' C10223 - H

1905002-02 (Solid)

Analyte	Result	RL	MDL	Units	Dilution	Analyzed	Method	Notes	Analyst
General Chemistry									
% Dry Solids	**90.4**			%	1	05/06/19	EPA160.3/1684		VJL
Soluble (DI Water Extraction)									
Chloride	**180**	11.1	1.13	mg/kg dry	10	05/07/19	EPA300.0		AES
pH	**11.7**			pH Units	1	05/02/19	9040C		VJL
Sulfate	**757**	22.1	4.71	mg/kg dry	20	05/07/19	EPA300.0		AES

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Trautner Geotech	Project: SO4, Cl, and pH	
649 Tech Ctr. Dr	Project Name / Number: [none]	**Reported:**
Durango CO, 81301	Project Manager: Ross Barrett	05/10/19 07:34

TB-2 @ 9.5'-13.5' C10223 - W

1905002-03 (Solid)

Analyte	**Result**	RL	MDL	Units	Dilution	Analyzed	Method	Notes	Analyst
General Chemistry									
% Dry Solids	**84.7**			%	1	05/06/19	EPA160.3/1684		VJL
Soluble (DI Water Extraction)									
Chloride	**546**	23.6	2.42	mg/kg dry	20	05/07/19	EPA300.0		AES
pH	**7.86**			pH Units	1	05/01/19	9040C		VJL
Sulfate	**4140**	236	50.3	mg/kg dry	200	05/07/19	EPA300.0		AES

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Trautner Geotech	Project:	SO4, Cl, and pH	
649 Tech Ctr. Dr	Project Name / Number:	[none]	**Reported:**
Durango CO, 81301	Project Manager:	Ross Barrett	05/10/19 07:34

General Chemistry - Quality Control

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch B905049 - General Prep - Wet Chem										
Duplicate (B905049-DUP1)	**Source: 1905002-01**		Prepared & Analyzed: 05/06/19							
% Dry Solids	81.5		%		81.2			0.373	20	

Soluble (DI Water Extraction) - Quality Control

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch B905005 - General Prep - Wet Chem										
Duplicate (B905005-DUP1)	**Source: 1905002-01**		Prepared & Analyzed: 05/01/19							
pH	7.36		pH Units		7.39			0.407	20	
Reference (B905005-SRM1)			Prepared & Analyzed: 05/01/19							
pH	6.97		pH Units	7.00		99.6	98.5-101.4			
Batch B905042 - General Prep - Wet Chem										
Blank (B905042-BLK1)			Prepared: 05/06/19 Analyzed: 05/07/19							
Chloride	ND	10.0	mg/kg wet							
Sulfate	ND	10.0	mg/kg wet							
LCS (B905042-BS1)			Prepared: 05/06/19 Analyzed: 05/07/19							
Chloride	248	10.0	mg/kg wet	250		99.2	85-115			
Sulfate	246	10.0	mg/kg wet	250		98.4	85-115			
LCS Dup (B905042-BSD1)			Prepared: 05/06/19 Analyzed: 05/07/19							
Chloride	249	10.0	mg/kg wet	250		99.7	85-115	0.535	20	
Sulfate	246	10.0	mg/kg wet	250		98.3	85-115	0.114	20	

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Trautner Geotech	Project:	SO4, Cl, and pH	
649 Tech Ctr. Dr	Project Name / Number:	[none]	**Reported:**
Durango CO, 81301	Project Manager:	Ross Barrett	05/10/19 07:34

Notes and Definitions

DET	Analyte DETECTED
ND	Analyte NOT DETECTED at or above the reporting limit
NR	Not Reported
dry	Sample results reported on a dry weight basis
	*Results reported on as received basis unless designated as dry.
RPD	Relative Percent Difference
LCS	Laboratory Control Sample (Blank Spike)
RL	Report Limit
MDL	Method Detection Limit

APPENDIX E

FLOODPLAIN

NOTES TO USERS

...s for use in administering the National Flood Insurance Program. It ...ecessarily identify all areas subject to flooding, particularly from local ...ources of small size. The **community map repository** should be ...or possible updated or additional flood hazard information.

...more detailed information in areas where **Base Flood Elevations** .../or **floodways** have been determined, users are encouraged to consult ...Profiles and Floodway Data and/or Summary of Stillwater Elevations ...ained within the Flood Insurance Study (FIS) report that accompanies ... Users should be aware that BFEs shown on the FIRM represent ...hole-foot elevations. These BFEs are intended for flood insurance ...oses only and should not be used as the sole source of flood elevation ... Accordingly, flood elevation data presented in the FIS report should ... in conjunction with the FIRM for purposes of construction and/or ...management.

...ase Flood Elevations shown on this map apply only landward of 0.0' ...rican Vertical Datum of 1988 (NAVD88). Users of this FIRM should be ...coastal flood elevations are also provided in the Summary of Stillwater ...table in the Flood Insurance Study report for this jurisdiction. Elevations ... the Summary of Stillwater Elevations table should be used for ...n and/or floodplain management purposes when they are higher than ...ons shown on this FIRM.

... of the **floodways** were computed at cross sections and interpolated ...oss sections. The floodways were based on hydraulic considerations ... to requirements of the National Flood Insurance Program. Floodway ... other pertinent floodway data are provided in the Flood Insurance ...rt for this jurisdiction.

...as not in Special Flood Hazard Areas may be protected by **flood** ...**uctures**. Refer to section 2.4 "Flood Protection Measures" of the Flood ...Study report for information on flood control structures for this ...

...**ction** used in the preparation of this map was Universal Transverse ...TM) zone 12. The **horizontal datum** was NAD 83, GRS80 spheroid. ... in datum, spheroid, projection or UTM zones used in the production of ...adjacent jurisdictions may result in slight positional differences in map ...cross jurisdiction boundaries. These differences do not affect the ... of this FIRM.

...ations on this map are referenced to the North American Vertical Datum ...AVD 88). These flood elevations must be compared to structure and ...evations referenced to the same **vertical datum**. For information ...conversion between the National Geodetic Vertical Datum of 1929 and ...American Vertical Datum of 1988, visit the National Geodetic Survey ... http://www.ngs.noaa.gov/ or contact the National Geodetic Survey at ...g address:

...nation Services
...GS12
...eodetic Survey
...9202
...West Highway
...g, MD 20910-3282

...current elevation, description, and/or location information for **bench** ...wn on this map, please contact the Information Services Branch of the ...Geodetic Survey at (301) 713-3242 or visit its website at ...ngs.noaa.gov/.

... information shown on this FIRM was provided in digital format by ...a County. These data are current as of 2006.

...eflects more detailed and up-to-date **stream channel configurations** ...**plain delineations** than those shown on the previous FIRM for this ... The floodplains and floodways that were transferred from the previous ... have been adjusted to conform to these new stream channel ...ons. As a result, the Flood Profiles and Floodway Data tables *in the* ...*rance Study Report (which contains authoritative hydraulic data)* may ...am channel distances that differ from what is shown on this map.

... **limits** shown on this map are based on the best data available at the ...blication. Because changes due to annexations or de-annexations may ...red after this map was published, map users should contact appropriate ... officials to verify current corporate limit locations.

...er to the separately printed **Map Index** for an overview map of the ...wing the layout of map panels; community map repository addresses; ...ng of Communities table containing National Flood Insurance Program ...each community as well as a listing of the panels on which each ... is located.

...**EMA Map Service Center** at 1-800-358-9616 for information on ...roducts associated with this FIRM. Available products may include ...ssued Letters of Map Change, *a Flood Insurance Study Report*, and/or ...ons of this map. The FEMA Map Service Center may also be reached ...-800-358-9620 and its website at http://www.msc.fema.gov/.

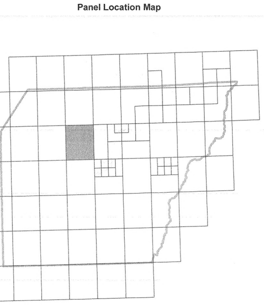
... **questions about this map** or questions concerning the National Flood ...Program in general, please call 1-877-FEMA Map (1-877-336-2627) or ...MA website at http://www.fema.gov/.

Montezuma County Vertical Datum Offset Table

Flooding Source	Vertical Datum Offset (ft)
Chicken Creek	4.0
Lost Canyon Creek	4.0
Mancos River	4.0

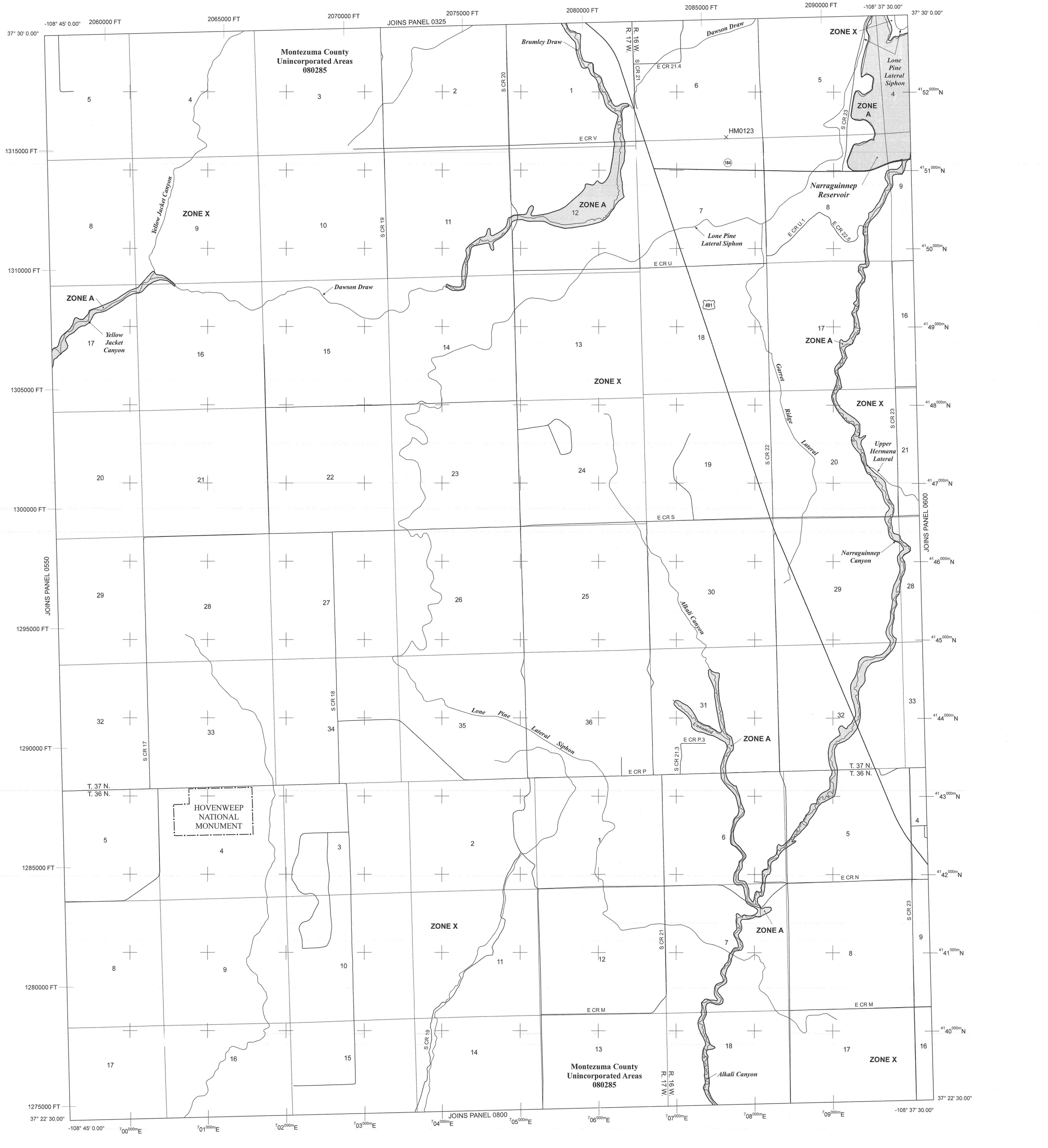
...vert Chicken Creek elevations to NAVD 88, 4.0 feet were added to the NGVD 29 elevations.

Panel Location Map



...al Flood Insurance Rate Map (DFIRM) was produced through a ...ng Technical Partner (CTP) agreement between the State of Colorado ...nservation Board (CWCB) and the Federal Emergency Management ...EMA).

Additional Flood Hazard information and resources are available from local communities and the Colorado Water Conservation Board.



LEGEND

SPECIAL FLOOD HAZARD AREAS (SFHAS) SUBJECT ... INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the f... that has a 1% chance of being equaled or exceeded in any given year. The Special F... Hazard Area is the area subject to flooding by the 1% annual chance flood. Area... Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base F... Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base F... Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); ave... depths determined. For areas of alluvial fan flooding, velocities ... determined.
ZONE AR	Special Flood Hazard Area Formerly protected from the 1% annual ch... flood by a flood control system that was subsequently decertified. ... AR indicates that the former flood control system is being restore... provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal f... protection system under construction; no Base Flood Eleva... determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base F... Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base F... Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that mus... kept free of encroachment so that the 1% annual chance flood can be carried wit... substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood ... average depths of less than 1 foot or with drainage areas less tha... square mile; and areas protected by levees from 1% annual chance flo...

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard ...

	Floodplain boundary
	Floodway boundary
	Zone D Boundary
	CBRS and OPA boundary
	Boundary dividing Special Flood Hazard Areas of different ... Flood Elevations, flood depths or flood velocities.
513	Base Flood Elevation line and value; elevation in feet*
(EL 987)	Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988 (NAVD 88)

A — A	Cross section line
23 - - - 23	Transect line
97° 07' 30.00" 32° 22' 30.00"	Geographic coordinates referenced to the North American Datum of 1983 (NAD 83)
$^{42}75^{000m}$N	1000-meter Universal Transverse Mercator grid ticks, zone 12
6000000 FT	5000-foot ticks: Colorado State Plane coordinate system, south zone (FIPSZONE 0503), Lambert Conformal Conic projection
DX5510 ×	Bench mark (see explanation in Notes to Users section of this FIRM panel)
• M1.5	River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index

EFFECTIVE DATE OF COUNTYWIDE
FLOOD INSURANCE RATE MAP
SEPTEMBER 26, 2008

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

For community map revision history prior to countywide mapping, refer to the Commu... Map History Table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insura... agent or call the National Flood Insurance Program at 1-800-638-6620.

MAP SCALE 1" = 2000'

1000 0 2000 4000 FEET

600 0 600 1200 METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0575C

FIRM

FLOOD INSURANCE RATE ...

MONTEZUMA COU...
COLORADO
AND INCORPORATED A...

PANEL 0575 OF 1450

(SEE MAP INDEX FOR FIRM PANEL...

CONTAINS:

COMMUNITY	NUMBER	PANEL
MONTEZUMA COUNTY	080285	0575

Notice to User: The **Map Number** shown below sh... used when placing map orders: the **Community N...** shown above should be used on insurance applicat... subject community.

MAP NU...
08083C...

EFFECTIVE ...
SEPTEMBER 26...

Federal Emergency Management ...

National Flood Hazard Layer FIRMette

FEMA

37°24'6.76"N

108°39'20.75"W

108°38'43.30"W

37°23'38.18"N

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

1:6,000

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- B 20.2 / 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- 8 Coastal Transect
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 10/30/2019 at 6:49:21 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

APPENDIX F

BRIDGE SCOUR ANALYSIS

PROJECT: **County Road N Over Alkali Creek Bridge Replacement**
DATE: **19-Jul-21**
EVENT: **50-year**

CONTRACTION SCOUR

Live Bed Transport of bed material in the upstream reach into the bridge cross section (HIGH VELOCITIES AND SMALL BED MATERIAL WILL CREATE LIVE BED SCOUR)

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

VARIABLE		Description	Source
A1=	264.2	Flow area of the stream main channel (ft2)	Main channel flow area XS 10466.14 (RAS)
A2=	310.65	Flow area of the contracted section (ft2)	Bridge opening area (RAS)
y1=	10.49	Average depth in the stream main channel, (ft)	XS 10466.14 Active Flow Distribution
yo=	9.99	Average depth in the contracted section, (ft)	BR US Active Flow Distribution
Q1=	2575.77	Flow in the upstream channel transporting sediment, (cfs)	Main Channel Flow XS 10466.14
Q2=	2972.01	Flow in the contracted channel, (cfs)	Flow in channel at bridge (RAS)
W1=	28.42	Bottom/top width of the upstream main channel, (ft)	Main Channel TW XS 10466.14
W2=	37.38	Bottom/top width of the main channel in the contracted section, (ft)	Bridge US section bottom deck width
S1=	0.002416	Slope of the energy grade line of the main channel (ft/ft)	Average E.G. slope upstream of bridge
omega (w)=	0.016404	Find D50 then see figure (right) - Fall velocity (ft/s)	Estimate from Boring (#200 sieve)
V*=	0.90	Shear Velocity (ft/s)	
V*/omega(w)=	55.07		
k1=	0.69		See Table 1
y2=	9.82	Average Depth in the contracted section (scoured) (ft)	
Contraction Scour=	**0.00**	**Average Contraction Scour Depth (ft)**	

Vavg > Vc, Use Live Bed Scour Equation

Clear-Water

VARIABLES

yo=	9.99	Average depth in the contracted section,
Q=	2972.01	Discharge through the bridge or on the setback overbank area at the bridge associated with the width W
W=	37.38	Bottom or top width of the contracted section
D50=	0.000246063	Median diameter (ft) - Sieve Analysis
Dm	0.0003	Diameter of the smallest non-transferable particle in the bed material (1.25 D50) in the contracted section (ft)
Ku=	0.0077	Constant (English Units)
y2=	53.16	Avg equilibrium depth in the contracted section after contraction scour (ft)
Contraction Scour=	**43.17**	**Average Contraction Scour Depth (ft)**

PROJECT: County Road N Over Alkali Creek Bridge Replacement
DATE: 19-Jul-21
EVENT: 50-year

ABUTMENT SCOUR
HEC-18 NCHRP 24-20 Abutment Scour Approach

$y_{max} = \alpha_A y_c$ or $y_{max} = \alpha_B y_c$

$y_s = y_{max} - y_0$

Live Bed Conditions:

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

Clear Water Conditions:

$$y_c = \left(\frac{q_{2f}}{K_u D_{50}^{1/3}}\right)^{6/7}$$

Variable		Description	Source
q_1=	90.6323012	Upstream unit discharge, ft²/s (m²/s)	HEC Output Q/W
q_{2c}=	79.50802568	Unit discharge in the constricted opening accounting for non-uniform flow distribution, ft²/s (m²/s)	HEC Output Q/W
q_{2c}/q_1=	0.877259262	Ratio of upstream unit discharge to unit discharge in the constricted opening	
Bed Conditions=		If Live bed conditions leave blank, if clear-water conditions use "C"	
α_A or α_B=	1.20	Amplification factor for live-bed (α_A) or clear-water (α_B) conditions. From Figs. 8.9 through 8.12	Figures 8.9 through 8.12 (HEC-18)
y_1=	10.49	Upstream flow depth, ft (m)	HEC Output
K_u=	11.17	11.17 English units, 6.19 SI units	HEC-18
D_{50}=	0.000246063	Particle size with 50 percent finer, ft (m)	Grain size distribution
y_c=	9.38	Flow depth including clear-water contraction scour, ft (m)	HEC Output
y_{max}=	11.25	Maximum flow depth resulting from abutment scour, ft (m)	HEC Output
y_0(left)=	10.84	Flow depth at left abutment prior to scour, ft (m)	HEC Output
y_0(right)=	10.84	Flow depth at right abutment prior to scour, ft (m)	HEC Output
y_s (left)=	0.41	Left abutment scour depth, ft (m)	
y_s (right)=	0.41	Right abutment scour depth, ft (m)	

PROJECT: **County Road N Over Alkali Creek Bridge Replacement**
DATE: **19-Jul-21**
EVENT: **100-year**

CONTRACTION SCOUR

Live Bed Transport of bed material in the upstream reach into the bridge cross section (HIGH VELOCITIES AND SMALL BED MATERIAL WILL CREATE LIVE BED SCOUR)

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

VARIABLE		Description	Source
A1=	398.38	Flow area of the stream main channel (ft2)	Main channel flow area XS 10466.14 (RAS)
A2=	391.19	Flow area of the contracted section (ft2)	Bridge opening area (RAS)
y1=	11.62	Average depth in the stream main channel, (ft)	XS 10466.14 Active Flow Distribution
yo=	11.18	Average depth in the contracted section, (ft)	BR US Active Flow Distribution
Q1=	3652.71	Flow in the upstream channel transporting sediment, (cfs)	Main Channel Flow XS 10466.14
Q2=	3823.93	Flow in the contracted channel, (cfs)	Flow in channel at bridge (RAS)
W1=	41.61	Bottom/top width of the upstream main channel, (ft)	Main Channel TW XS 10466.14
W2=	42.45	Bottom/top width of the main channel in the contracted section, (ft)	Bridge US section bottom deck width
S1=	0.002251	Slope of the energy grade line of the main channel (ft/ft)	Average E.G. slope upstream of bridge
omega (w)=	0.016404	Find D50 then see figure (right) - Fall velocity (ft/s)	Estimate from Boring (#200 sieve)
V*=	0.92	Shear Velocity (ft/s)	
V*/omega(w)=	55.93		
k1=	0.69		See Table 1
y2=	11.91	Average Depth in the contracted section (scoured) (ft)	
Contraction Scour=	**0.74**	**Average Contraction Scour Depth (ft)**	

Vavg > Vc, Use Live Bed Scour Equation

Clear-Water

VARIABLES

yo=	11.18	Average depth in the contracted section,
Q=	3823.93	Discharge through the bridge or on the setback overbank area at the bridge associated with the width W
W=	42.45	Bottom or top width of the contracted section
D50=	0.000246063	Median diameter (ft) - Sieve Analysis
Dm	0.0003	Diameter of the smallest non-transferable particle in the bed material (1.25 D50) in the contracted section (ft)
Ku=	0.0077	Constant (English Units)
y2=	59.16	Avg equilibrium depth in the contracted section after contraction scour (ft)
Contraction Scour=	**47.99**	**Average Contraction Scour Depth (ft)**

PROJECT: County Road N Over Alkali Creek Bridge Replacement
DATE: 19-Jul-21
EVENT: 100-year

ABUTMENT SCOUR
HEC-18 NCHRP 24-20 Abutment Scour Approach

$$y_{max} = \alpha_A y_c \text{ or } y_{max} = \alpha_B y_c$$

$$y_s = y_{max} - y_0$$

Live Bed Conditions:

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

Clear Water Conditions:

$$y_c = \left(\frac{q_{2f}}{K_u D_{50}^{1/3}}\right)^{6/7}$$

Variable		Description	Source
q_1=	87.784	Upstream unit discharge, ft²/s (m²/s)	HEC Output Q/W
q_{2c}=	90.081	Unit discharge in the constricted opening accounting for non-uniform flow distribution, ft²/s (m²/s)	HEC Output Q/W
q_{2c}/q_1=	1.026	Ratio of upstream unit discharge to unit discharge in the constricted opening	
Bed Conditions=		If Live bed conditions leave blank, if clear-water conditions use "C"	
α_A or α_B=	1.39	Amplification factor for live-bed (α_A) or clear-water (α_B) conditions. From Figs. 8.9 through 8.12	Figures 8.9 through 8.12 (HEC-18)
y_1=	11.62	Upstream flow depth, ft (m)	HEC Output
K_u=	11.17	11.17 English units, 6.19 SI units	HEC-18
D_{50}=	0.000246063	Particle size with 50 percent finer, ft (m)	Grain size distribution
y_c=	11.88	Flow depth including clear-water contraction scour, ft (m)	HEC Output
y_{max}=	16.51	Maximum flow depth resulting from abutment scour, ft (m)	HEC Output
y_0(left)=	12.35	Flow depth at left abutment prior to scour, ft (m)	HEC Output
y_0(right)=	12.35	Flow depth at right abutment prior to scour, ft (m)	HEC Output
y_s (left)=	4.16	Left abutment scour depth, ft (m)	
y_s (right)=	4.16	Right abutment scour depth, ft (m)	

PROJECT: **County Road N Over Alkali Creek Bridge Replacement**
DATE: **19-Jul-21**
EVENT: **500-year**

CONTRACTION SCOUR

Live Bed Transport of bed material in the upstream reach into the bridge cross section (HIGH VELOCITIES AND SMALL BED MATERIAL WILL CREATE LIVE BED SCOUR)

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

VARIABLE		Description	Source
A1=	663.72	Flow area of the stream main channel (ft2)	Main channel flow area XS 10466.14 (RAS)
A2=	599.78	Flow area of the contracted section (ft2)	Bridge opening area (RAS)
y1=	14.15	Average depth in the stream main channel, (ft)	XS 10466.14 Active Flow Distribution
yo=	13.78	Average depth in the contracted section, (ft)	BR US Active Flow Distribution
Q1=	6290.67	Flow in the upstream channel transporting sediment, (cfs)	Main Channel Flow XS 10466.14
Q2=	6298.04	Flow in the contracted channel, (cfs)	Flow in channel at bridge (RAS)
W1=	58.31	Bottom/top width of the upstream main channel, (ft)	Main Channel TW XS 10466.14
W2=	53.30	Bottom/top width of the main channel in the contracted section, (ft)	Bridge US section bottom deck width
S1=	0.002005	Slope of the energy grade line of the main channel (ft/ft)	Average E.G. slope upstream of bridge
omega (w)=	0.016404	Find D50 then see figure (right) - Fall velocity (ft/s)	Estimate from Boring (#200 sieve)
V*=	0.96	Shear Velocity (ft/s)	
V*/omega(w)=	58.26		
k1=	0.69		See Table 1
y2=	15.07	Average Depth in the contracted section (scoured) (ft)	
Contraction Scour=	**1.28**	**Average Contraction Scour Depth (ft)**	

Vavg > Vc, Use Live Bed Scour Equation

Clear-Water

VARIABLES

yo=	13.78	Average depth in the contracted section,
Q=	6298.04	Discharge through the bridge or on the setback overbank area at the bridge associated with the width W
W=	53.3	Bottom or top width of the contracted section
D50=	0.000246063	Median diameter (ft) - Sieve Analysis
Dm	0.0003	Diameter of the smallest non-transferable particle in the bed material (1.25 D50) in the contracted section (ft)
Ku=	0.0077	Constant (English Units)
y2=	74.65	Avg equilibrium depth in the contracted section after contraction scour (ft)
Contraction Scour=	**60.87**	**Average Contraction Scour Depth (ft)**

PROJECT: **County Road N Over Alkali Creek Bridge Replacement**
DATE: **19-Jul-21**
EVENT: **500-year**

ABUTMENT SCOUR
HEC-18 NCHRP 24-20 Abutment Scour Approach

$y_{max} = \alpha_A y_c$ or $y_{max} = \alpha_B y_c$

$y_s = y_{max} - y_0$

Live Bed Conditions:

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

Clear Water Conditions:

$$y_c = \left(\frac{q_{2f}}{K_u D_{50}^{1/3}}\right)^{6/7}$$

Variable		Description	Source
q_1=	107.88	Upstream unit discharge, ft^2/s (m^2/s)	HEC Output Q/W
q_{2c}=	118.16	Unit discharge in the constricted opening accounting for non-uniform flow distribution, ft^2/s (m^2/s)	HEC Output Q/W
q_{2c}/q_1=	1.10	Ratio of upstream unit discharge to unit discharge in the constricted opening	
Bed Conditions=		If Live bed conditions leave blank, if clear-water conditions use "C"	
α_A or α_B=	1.60	Amplification factor for live-bed (α_A) or clear-water (α_B) conditions. From Figs. 8.9 through 8.12	Figures 8.9 through 8.12 (HEC-18)
y_1=	14.15	Upstream flow depth, ft (m)	HEC Output
K_u=	11.17	11.17 English units, 6.19 SI units	HEC-18
D_{50}=	0.000246063	Particle size with 50 percent finer, ft (m)	Grain size distribution
y_c=	15.29	Flow depth including clear-water contraction scour, ft (m)	HEC Output
y_{max}=	24.47132573	Maximum flow depth resulting from abutment scour, ft (m)	HEC Output
y_0(left)=	15.69	Flow depth at left abutment prior to scour, ft (m)	HEC Output
y_0(right)=	15.69	Flow depth at right abutment prior to scour, ft (m)	HEC Output
y_s (left)=	8.78	Left abutment scour depth, ft (m)	
y_s (right)=	8.78	Right abutment scour depth, ft (m)	

Hydraulic Analysis Report

FHWA Hydraulic Toolbox v5.1 Report showing scour results for the 50-, 100-, and 500-yr events.

Project Data

Project Title:

Designer:

Project Date: Friday, February 19, 2021

Project Units: U.S. Customary Units

Notes:

Bridge Scour Analysis:Bridge Scour Analysis 50yr

Notes:

Scenario: Bridge Scour Analysis - FOR - 50yr

Long Term Degradation

User-Specified Scour Depth

Long term degradation of 0.0' assumed. See report for assumptions.

Contraction Scour Summary

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

Local Scour at Piers Summary

Scour at left and right abutment have the same parameters. Left abutment scour depths are applied to the right abutment.

Local Scour at Abutments Summary

Left Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 0.41 ft

Total Scour at Abutment 0.41 ft

Long Term Details

Long-Term Degradation

Computation Type:

Input Parameters

D50: 0.075000 mm

Main Channel Contraction Scour

Computation Type: Clear-Water or Live-Bed Scour

Input Parameters

Average Depth Upstream of Contraction: 10.49 ft

D50: 0.075000 mm

Average Velocity Upstream: 9.75 ft/s

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 1.04 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: 0.0024 ft/ft

Flow in Contracted Section: 2972.00 cfs

Flow Upstream that is Transporting Sediment: 2575.77 cfs

Width in Contracted Section: 37.38 ft

Width Upstream that is Transporting Sediment: 28.42 ft

Depth Prior to Scour in Contracted Section: 9.99 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: 0.90 ft/s

Fall Velocity: 0.02 ft/s

Average Depth in Contracted Section after Scour: 9.82 ft

Scour Depth for Live Bed: -0.17 ft

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

Pier Details

Left Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Spill-through abutment

Angle of Embankment to Flow: 10.00 Degrees

Centerline Length of Embankment: 100.00 ft

Projected Length of Embankment: 17.36 ft

Width of Flood Plain: 100.00 ft

Unit Discharge, Upstream in Main Channel (q1): 90.63 cfs

Unit Discharge in the Constricted Area (q2): 79.51 cfs/ft

D50: 0.200000 mm

Upstream Flow Depth: 10.49 ft

Flow Depth Prior to Scour: 10.84 ft

Result Parameters

q2/q1: 0.88

Average Velocity Upstream: 8.64 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 1.44 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 1.20

Flow Depth including Contraction Scour: 9.38 ft

Maximum Flow Depth including Abutment Scour: 11.25 ft

Scour Hole Depth from NCHRP Method: 0.41 ft

Bridge Scour Analysis:Bridge Scour Analysis 100yr

Notes:

Long term degradation of 0.0' assumed. See report for assumptions.

Scenario: Bridge Scour Analysis - FOR - 100yr

Long Term Degradation

User-Specified Scour Depth

Contraction Scour Summary

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

Applied Contraction Scour Depth 0.74 ft

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

Pressure Scour Depth 0.74 ft

Live Bed Contraction Scour Depth 0.74 ft

Scour at left and right abutment have the same parameters. Left abutment scour depths are applied to the right abutment.

Local Scour at Piers Summary

Local Scour at Abutments Summary

Left Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 4.12 ft

Total Scour at Abutment 4.12 ft

Long Term Details

Long-Term Degradation

Computation Type:

Input Parameters

Shield's Parameter: 0.0000

Manning's n Value: 0.0000

Main Channel Contraction Scour

Computation Type: Clear-Water or Live-Bed Scour

Input Parameters

Average Depth Upstream of Contraction: 11.62 ft

D50: 0.074981 mm

Average Velocity Upstream: 9.63 ft/s

Results of Scour Condition

Critical velocity above which bed material of size D and smaller will be transported: 1.05 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: 0.0023 ft/ft

Flow in Contracted Section: 3823.93 cfs

Flow Upstream that is Transporting Sediment: 3652.71 cfs

Width in Contracted Section: 42.45 ft

Width Upstream that is Transporting Sediment: 41.61 ft

Depth Prior to Scour in Contracted Section: 11.18 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: 0.93 ft/s

Fall Velocity: 0.02 ft/s

Average Depth in Contracted Section after Scour: 11.92 ft

Scour Depth for Live Bed: 0.74 ft

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

Pier Details

Left Abutment Details

Abutment Scour

Computation Type: NCHRP

Input Parameters

NCHRP Method

Abutment Type: Spill-through abutment

Angle of Embankment to Flow: 10.00 Degrees

Centerline Length of Embankment: 100.00 ft

Projected Length of Embankment: 17.36 ft

Width of Flood Plain: 100.00 ft

Unit Discharge, Upstream in Main Channel (q1): 87.78 cfs

Unit Discharge in the Constricted Area (q2): 90.08 cfs/ft

D50: 0.074981 mm

Upstream Flow Depth: 11.62 ft

Flow Depth Prior to Scour: 12.35 ft

Result Parameters

q2/q1: 1.03

Average Velocity Upstream: 7.55 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 1.05 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 1.39

Flow Depth including Contraction Scour: 11.88 ft

Maximum Flow Depth including Abutment Scour: 16.47 ft

Scour Hole Depth from NCHRP Method: 4.12 ft

Bridge Scour Analysis:Bridge Scour Analysis 500yr

Notes:

Scenario: Bridge Scour Analysis - FOR - 500yr

Long Term Degradation

Long term degradation of 0.0' assumed. See report for assumptions.

User-Specified Scour Depth

Contraction Scour Summary

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

Applied Contraction Scour Depth 1.29 ft

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

Pressure Scour Depth 1.29 ft

Live Bed Contraction Scour Depth 1.29 ft

Local Scour at Piers Summary

Local Scour at Abutments Summary

Scour at left and right abutment have the same parameters. Left abutment scour depths are applied to the right abutment.

Left Abutment

Abutment Scour Method: NCHRP Method

Abutment Scour Depth 8.79 ft

Total Scour at Abutment 8.79 ft

Long Term Details

Long-Term Degradation

Computation Type:

Input Parameters

Shield's Parameter: 0.0000

Manning's n Value: 0.0000

Main Channel Contraction Scour

Computation Type: Clear-Water or Live-Bed Scour

Input Parameters

Average Depth Upstream of Contraction: 14.15 ft

D50: 0.074981 mm

Average Velocity Upstream: 9.48 ft/s

Results of Scour Condition
Critical velocity above which bed material of size D and smaller will be transported: 1.09 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: 0.0020 ft/ft

Flow in Contracted Section: 6298.04 cfs

Flow Upstream that is Transporting Sediment: 6290.67 cfs

Width in Contracted Section: 53.30 ft

Width Upstream that is Transporting Sediment: 58.31 ft

Depth Prior to Scour in Contracted Section: 13.78 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: 0.95 ft/s

Fall Velocity: 0.02 ft/s

Average Depth in Contracted Section after Scour: 15.07 ft

Scour Depth for Live Bed: 1.29 ft

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

Pier Details

Left Abutment Details

Abutment Scour
Computation Type: NCHRP

Input Parameters

NCHRP Method
Abutment Type: Spill-through abutment

Angle of Embankment to Flow: 10.00 Degrees

Centerline Length of Embankment: 100.00 ft

Projected Length of Embankment: 17.36 ft

Width of Flood Plain: 100.00 ft

Unit Discharge, Upstream in Main Channel (q1): 107.88 cfs

Unit Discharge in the Constricted Area (q2): 118.16 cfs/ft

D50: 0.074981 mm

Upstream Flow Depth: 14.15 ft

Flow Depth Prior to Scour: 15.69 ft

Result Parameters

q2/q1: 1.10

Average Velocity Upstream: 7.62 ft/s

Critical Velocity above which Bed Materal of Size D and Smaller will be Transported: 1.09 ft/s

Scour Condition: Live Bed

Embankment Length/Floodplain Width Ratio: 0.00

Scour Condition: a (Main Channel)

Amplification Factor: 1.60

Flow Depth including Contraction Scour: 15.30 ft

Maximum Flow Depth including Abutment Scour: 24.48 ft

Scour Hole Depth from NCHRP Method: 8.79 ft

Hydraulic Analysis Report

FHWA Hydraulic Toolbox Report showing scour results for the 25-yr event (Design Flood)

Project Data

Project Title:

Designer:

Project Date: Tuesday, February 16, 2021

Project Units: U.S. Customary Units

Notes:

Analysis includes Straight Channel, Outside Bend, and Inside Bend. The recommended riprap size is d50=18: based on the outside and inside bend calculations.

Riprap Analysis: Riprap Analysis - Straight

Notes:

Input Parameters

Riprap Type: Revetment

The channel is a natural channel

Local Depth of Flow: 10.1085 ft

Riprap Shape is Angular

Stability Coefficient: 0.3

This value is updated by the selected Riprap Shape

Blanket Thickness Coefficient: 1

Channel Cross-sectional Average Velocity: 7.5758 ft/s

Which is the Average Velocity with Spurs

Centerline Radius of Curvature of Channel Bend: 1e+009 ft

Width of Water Surface at Upstream End of Channel Bend: 50.4338 ft

Bank Angle: 2 :1 H:V

.966 < Bank Angle < 4.011

The location of the revetment protection is on a straight channel

Specific Gravity of Riprap: 2.65

Safety Factor: 1.1

Result Parameters

Side slope Correction Factor: 0.871418

Velocity Distribution Coefficient: 1

Design Velocity: 7.5758 ft/s

Design velocity never less than average channel velocity

Computed D30: 2.90788 in

Computed D50: 3.48945 in

Riprap Class

Riprap Class Name: CLASS I

Riprap Class Order: 1

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

d100: 12 in

d85: 9 in

d50: 6.5 in

d15: 4.5 in

Layout Recommendations

Minimum Riprap Thickness: 144 in

Report for channel

Channel Analysis: Channel Analysis

Notes:

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width: 10.0000 ft

Longitudinal Slope: 0.0043 ft/ft

Manning's n: 0.0400

Lining Type: Rock Riprap - 300 mm (12-inch)

Flow: 2314.0000 cfs

Result Parameters

Depth: 10.1085 ft

Area of Flow: 305.4463 ft^2

Wetted Perimeter: 55.2064 ft

Hydraulic Radius: 5.5328 ft

Average Velocity: 7.5758 ft/s

Top Width: 50.4338 ft

Froude Number: 0.5425

Critical Depth: 7.5163 ft

Critical Velocity: 12.2985 ft/s

Critical Slope: 0.0156 ft/ft

Critical Top Width: 40.07 ft

Calculated Max Shear Stress: 2.6808 lb/ft^2

Calculated Avg Shear Stress: 1.4673 lb/ft^2

Riprap Analysis: Riprap Analysis - 25yr - Outside Bend

Notes:

Input Parameters

Riprap Type: Revetment

The channel is a natural channel

Local Depth of Flow: 10.1085 ft

Riprap Shape is Angular

Stability Coefficient: 0.3

This value is updated by the selected Riprap Shape

Blanket Thickness Coefficient: 1

Channel Cross-sectional Average Velocity: 7.5758 ft/s

Which is the Average Velocity with Spurs

Centerline Radius of Curvature of Channel Bend: 100 ft

Width of Water Surface at Upstream End of Channel Bend: 50.4338 ft

Bank Angle: 2 :1 H:V

.966 < Bank Angle < 4.011

The location of the revetment protection is on the outside of a bend

Specific Gravity of Riprap: 2.5

Safety Factor: 1.1

Result Parameters

Side slope Correction Factor: 0.871418

Velocity Distribution Coefficient: 1.22354

Design Velocity: 12.0108 ft/s

Design velocity never less than average channel velocity

Computed D30: 12.6851 in

Computed D50: 15.2221 in

Riprap Class

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

Report for channel

Channel Analysis: Channel Analysis

Notes:

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width: 10.0000 ft

Longitudinal Slope: 0.0043 ft/ft

Manning's n: 0.0400

Lining Type: Rock Riprap - 300 mm (12-inch)

Flow: 2314.0000 cfs

Result Parameters

Depth: 10.1085 ft

Area of Flow: 305.4463 ft^2

Wetted Perimeter: 55.2064 ft

Hydraulic Radius: 5.5328 ft

Average Velocity: 7.5758 ft/s

Top Width: 50.4338 ft

Froude Number: 0.5425

Critical Depth: 7.5163 ft

Critical Velocity: 12.2985 ft/s

Critical Slope: 0.0156 ft/ft

Critical Top Width: 40.07 ft

Calculated Max Shear Stress: 2.6808 lb/ft^2

Calculated Avg Shear Stress: 1.4673 lb/ft^2

Riprap Analysis: Riprap Analysis - 25yr - Inside Bend

Notes:

Input Parameters

Riprap Type: Revetment

The channel is a natural channel

Local Depth of Flow: 10.1085 ft

Riprap Shape is Angular

Stability Coefficient: 0.3

This value is updated by the selected Riprap Shape

Blanket Thickness Coefficient: 1

Channel Cross-sectional Average Velocity: 7.5758 ft/s

Which is the Average Velocity with Spurs

Centerline Radius of Curvature of Channel Bend: 100 ft

Width of Water Surface at Upstream End of Channel Bend: 50.4338 ft

Bank Angle: 2 :1 H:V

.966 < Bank Angle < 4.011

The location of the revetment protection is on the inside of a bend

Specific Gravity of Riprap: 2.5

Safety Factor: 1.1

Result Parameters

Side slope Correction Factor: 0.871418

Velocity Distribution Coefficient: 1

Design Velocity: 12.0108 ft/s

Design velocity never less than average channel velocity

Computed D30: 10.3675 in

Computed D50: 12.441 in

Riprap Class

Riprap Class Name: CLASS III

Riprap Class Order: 3

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

d100: 24 in

d85: 17 in

d50: 12.5 in

d15: 9 in

Layout Recommendations

Minimum Riprap Thickness: 288 in

Report for channel

Channel Analysis: Channel Analysis

Notes:

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width: 10.0000 ft

Longitudinal Slope: 0.0043 ft/ft

Manning's n: 0.0400

Lining Type: Rock Riprap - 300 mm (12-inch)

Flow: 2314.0000 cfs

Result Parameters

Depth: 10.1085 ft

Area of Flow: 305.4463 ft^2

Wetted Perimeter: 55.2064 ft

Hydraulic Radius: 5.5328 ft

Average Velocity: 7.5758 ft/s

Top Width: 50.4338 ft

Froude Number: 0.5425

Critical Depth: 7.5163 ft

Critical Velocity: 12.2985 ft/s

Critical Slope: 0.0156 ft/ft

Critical Top Width: 40.07 ft

Calculated Max Shear Stress: 2.6808 lb/ft^2

Calculated Avg Shear Stress: 1.4673 lb/ft^2

The EM-1601 equation can be used with uniform or gradually varying flow. Coefficients are included to account for the desired safety factor for design, specific gravity of the riprap stone, bank slope, and bendway character. The EM-1601 equation is:

$$d_{30} = y(S_f C_S C_V C_T)\left[\frac{(V_{des})}{\sqrt{K_1(S_g - 1)gy}}\right]^{2.5} \quad (4.1)$$

where:

- d_{30} = Particle size for which 30% is finer by weight, ft (m)
- y = Local depth of flow, ft (m)
- S_f = Safety factor (must be > 1.0)
- C_S = Stability coefficient (for blanket thickness = d_{100} or $1.5d_{50}$, whichever is greater, and uniformity ratio d_{85}/d_{15} = 1.7 to 5.2)
  - = 0.30 for angular rock
  - = 0.375 for rounded rock
- C_V = Velocity distribution coefficient
  - = 1.0 for straight channels or the inside of bends
  - = 1.283 - 0.2log(R_c/W) for the outside of bends (1.0 for R_c/W > 26)
  - = 1.25 downstream from concrete channels
  - = 1.25 at the end of dikes
- C_T = Blanket thickness coefficient given as a function of the uniformity ratio d_{85}/d_{15}. C_T = 1.0 is recommended because it is based on very limited data.
- V_{des} = Characteristic velocity for design, defined as the depth-averaged velocity at a point 20% upslope from the toe of the revetment, ft/s (m/s)

  For natural channels, $V_{des} = V_{avg}(1.74 - 0.52\log(R_c/W))$

  $V_{des} = V_{avg}$ for $R_c/W > 26$

  For trapezoidal channels, $V_{des} = V_{avg}(1.71 - 0.78\log(R_c/W))$

  $V_{des} = V_{avg}$ for $R_c/W > 8$

- V_{avg} = Channel cross-sectional average velocity, ft/s (m/s)
- K_1 = Side slope correction factor

$$K_1 = \sqrt{1 - \left(\frac{\sin(\theta - 14^\circ)}{\sin(32^\circ)}\right)^{1.6}}$$

  where: θ is the bank angle in degrees

- R_c = Centerline radius of curvature of channel bend, ft (m)
- W = Width of water surface at upstream end of channel bend, ft (m)
- S_g = Specific gravity of riprap (usually taken as 2.65)
- g = Acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2)

4.2.7 Edge Treatment and Termination Details

Riprap revetment should be toed down below the toe of the bank slope to a depth at least as great as the depth of anticipated long-term bed degradation plus toe scour (see Volume 1, Section 4.3.5). Installations in the vicinity of bridges must also consider the potential for contraction scour.

Recommended freeboard allowance calls for the riprap to be placed on the bank to an elevation at least 2.0 feet greater than the design high water level. Upstream and downstream terminations should utilize a key trench that is dimensioned in relation to the d_{50} size of the riprap. Where the design water level is near or above the top of bank, the riprap should be carried to the top of the bank. Figures 4.2, 4.3, and 4.4 are schematic diagrams that summarize these recommendations. If toe down cannot be placed below the anticipated contraction scour and degradation depth (Figure 4.2), a mounded toe approach (Figure 4.3) is suggested.

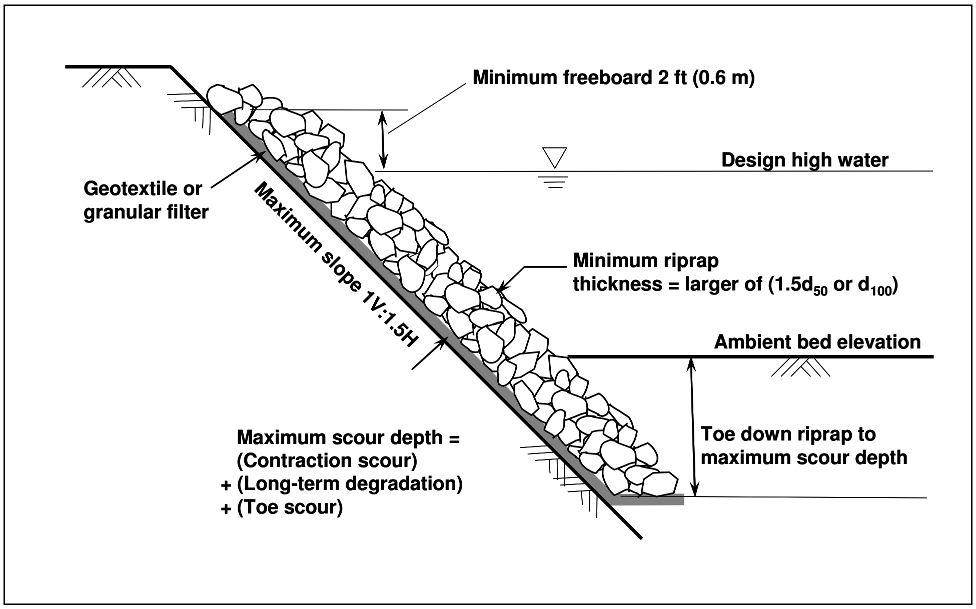


Figure 4.2. Riprap revetment with buried toe.

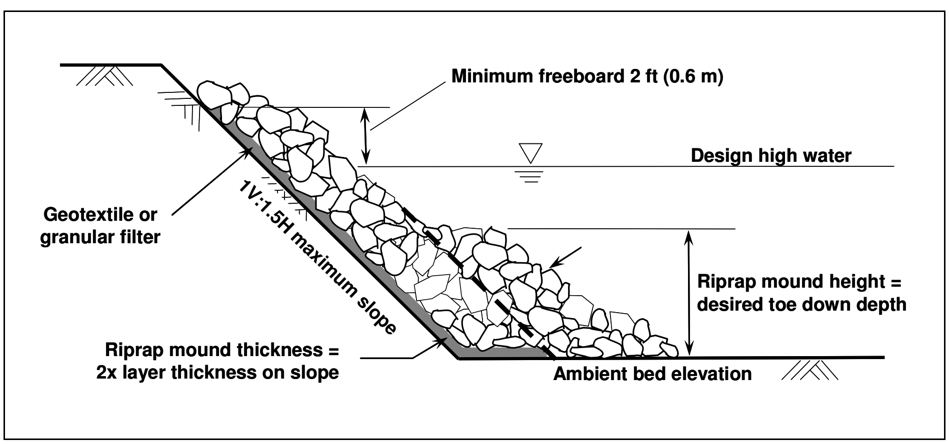


Figure 4.3. Riprap revetment with mounded toe.

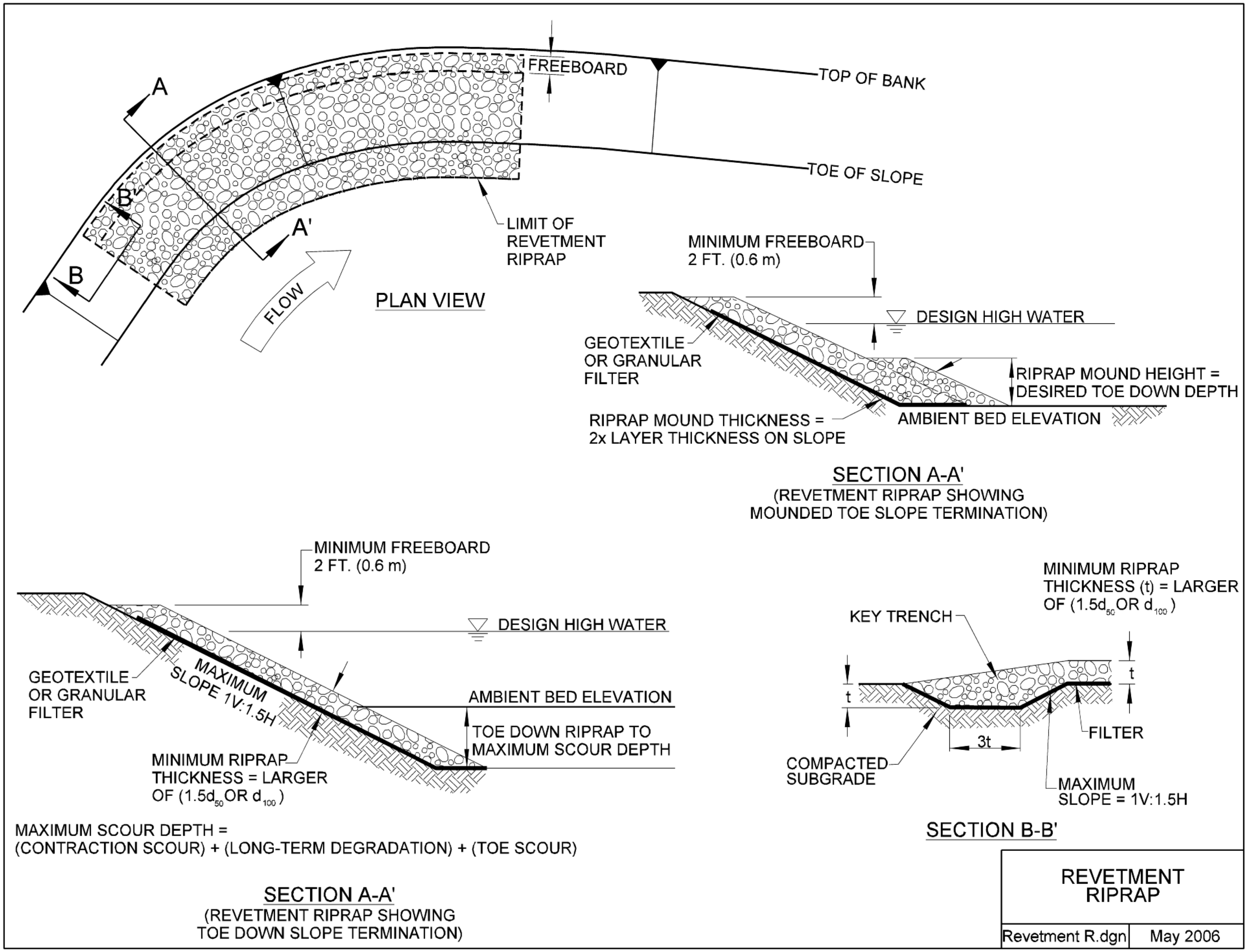
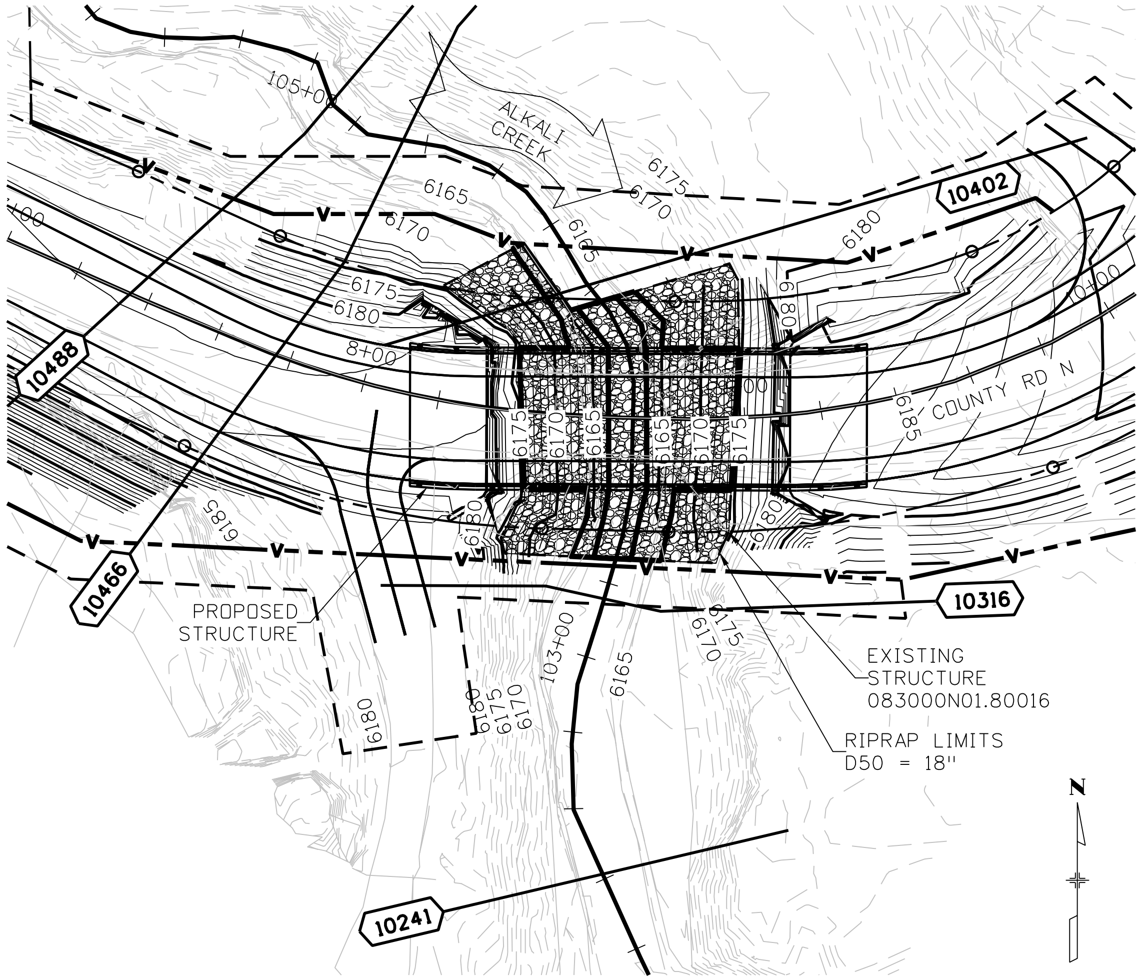


Figure 4.4. Riprap revetment details.

APPENDIX G

HYDRAULIC DESIGN PLAN SHEETS

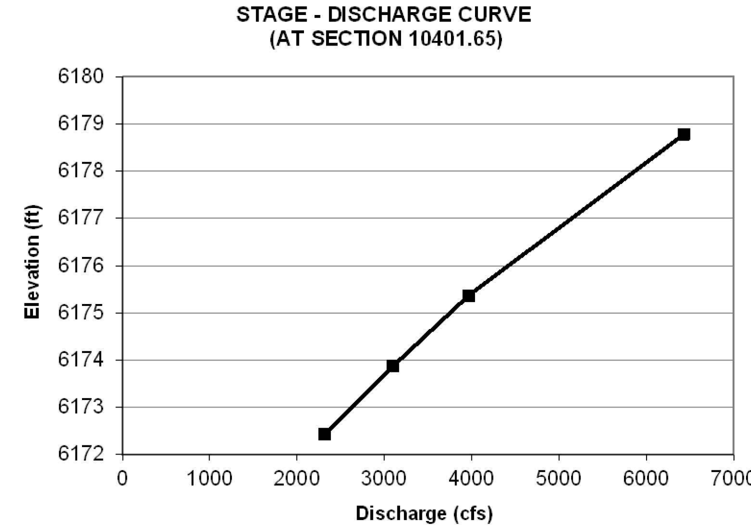
RESPEC

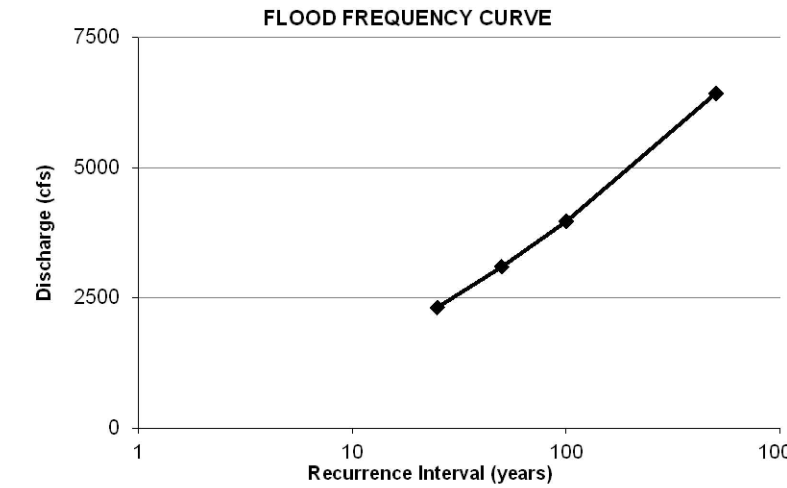


PLAN VIEW - COUNTY RD N OVER ALKALI CREEK

NTS

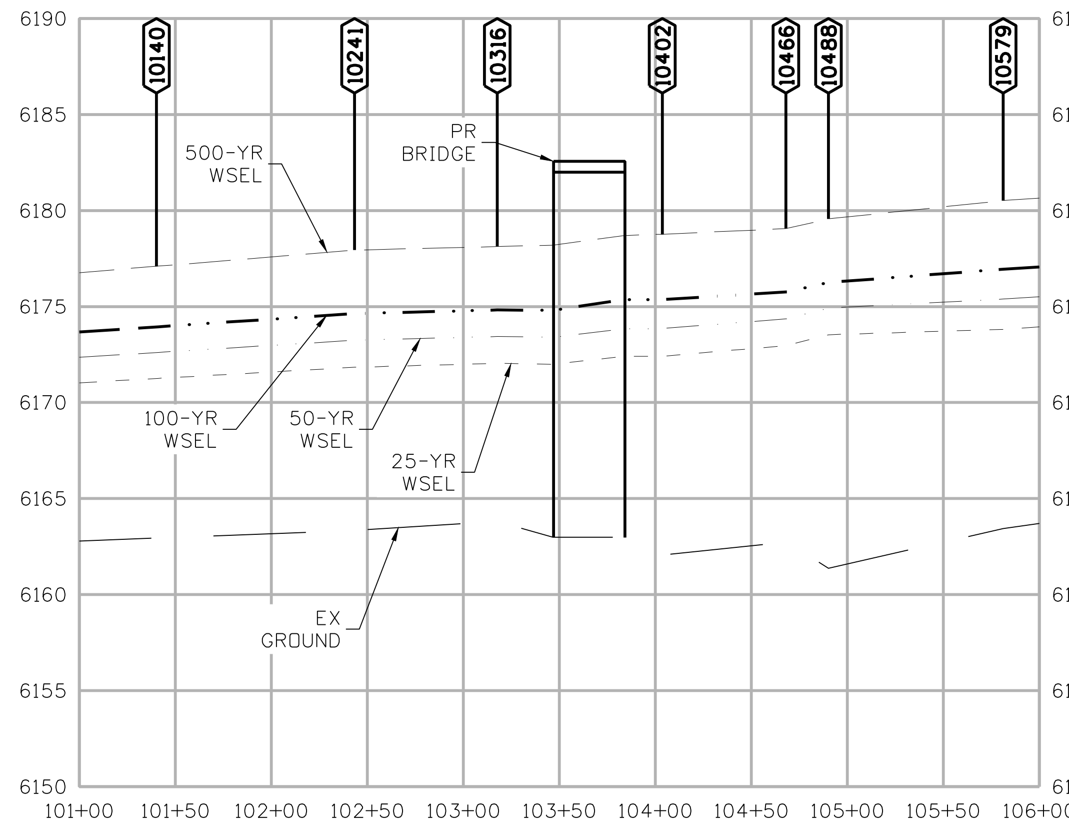
0' 25' 50' 100'

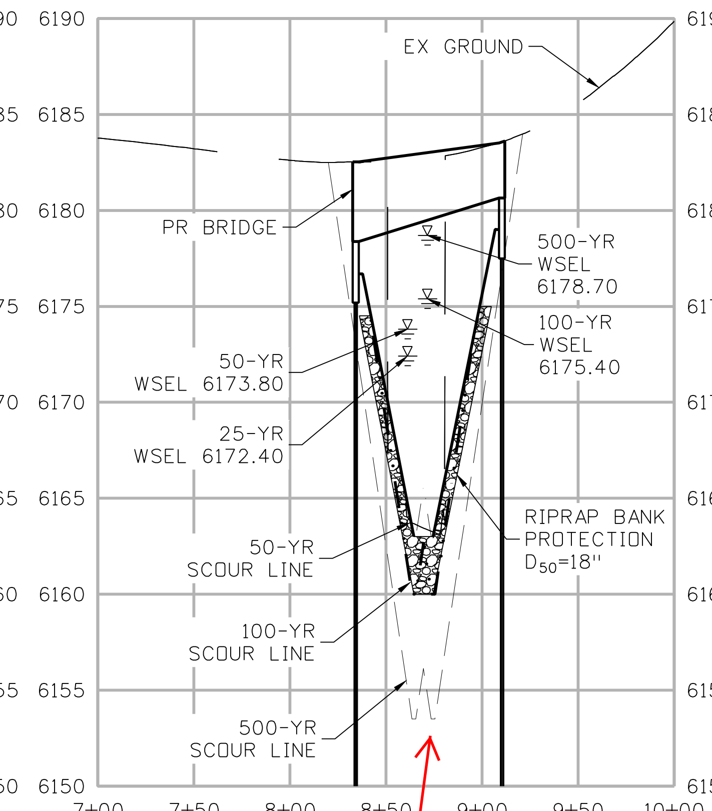




CHANNEL DESCRIPTION

Bottom Material – Cohesive ☐ Non-Cohesive ☒
Bottom Material – Size – Clay ☒ Silt ☐ Sand ☒ Gravel ☐
Cobbles ☐ Other ☐
Stream Form – Straight ☐ Meandering ☒ Braided ☐
Mannings "n" For Design – Channel = 0.035
Overbanks = 0.05
Debris – Brush ☒ Trees/Logs ☒ Ice ☐ Other ☐
Drainage Area = 36.9 sq. mi





Plot to be updated to match values in report based on results of updated Hydraulic Toolbox version.

COMPARISON OF HYDRAULICS*

HEC-RAS Model	Velocity (avg) (ft/s)	Freeboard Required (ft)	Freeboard Actual (U/S) (ft)	WSEL (ft)
Existing	10.3	2.00	4.79	6174.97
Proposed	9.3	2.00	4.76	6174.36

*25-Year Event, velocity from downstream internal bridge section. WSEL obtained from profile 85 ft upstream of bridge at XS 10466.14.

SCOUR ANALYSIS

Event	Discharge (cfs)	WSEL**	Long Term Aggradation / Degradation (ft)	Contraction Scour (ft)	Abutment Scour Left (ft)	Abutment Scour Right (ft)
50-yr	3095.1	6173.8	0.0	0.0	0.0	0.0
100-yr	3968.6	6175.4	0.0	0.8	2.4	2.4
500-yr	6428.3	6178.7	0.0	1.2	8.6	8.6

**WSEL obtained from the Upstream Bridge XS.

Table to be updated to match values in report based on results of updated Hydraulic Toolbox version.

PRELIMINARY

Print Date: March 29, 2021	
File Name: 22801HYDR_BHI.dgn	
Horiz. Scale: As Noted	Vert. Scale: As Noted
Unit Information	Unit Leader Initials

RESPEC
720 SOUTH COLORADO BOULEVARD
SUITE 410 S
DENVER, CO 80246
PHONE (303) 757-3655

Sheet R
Date: Com

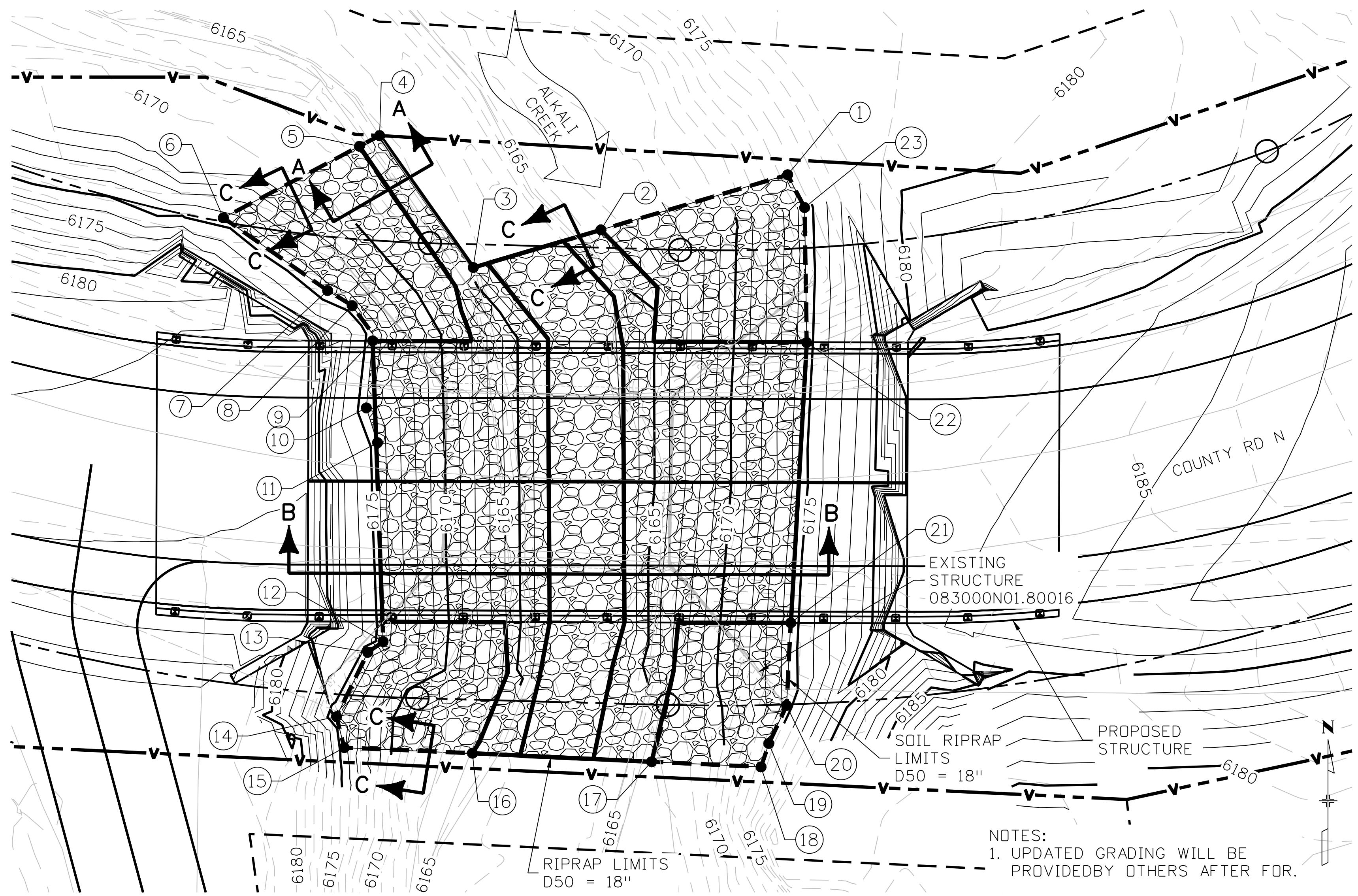
MONTEZUMA County
109 WEST MAIN STREET
CORTEX, CO 81321
ROOM 260
PHONE: 970-565-3728
FAX: 970-385-3635

As Constructed
No Revisions:
Revised:
Void:

ALKALI CREEK

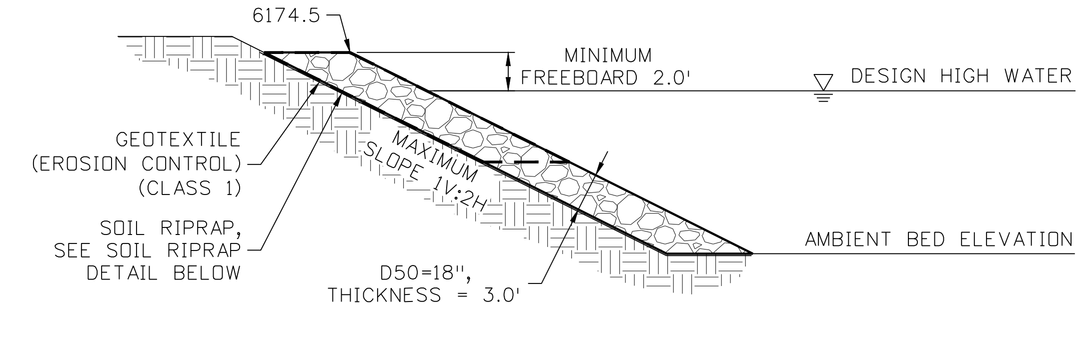
BRIDGE HYDRAULIC INFORMATION

Designer: LDR	Structure Numbers	083000N01.80016	22521
Detailer: ACT			
Sheet Subset: HYDR	Subset Sheets:		Sheet Number



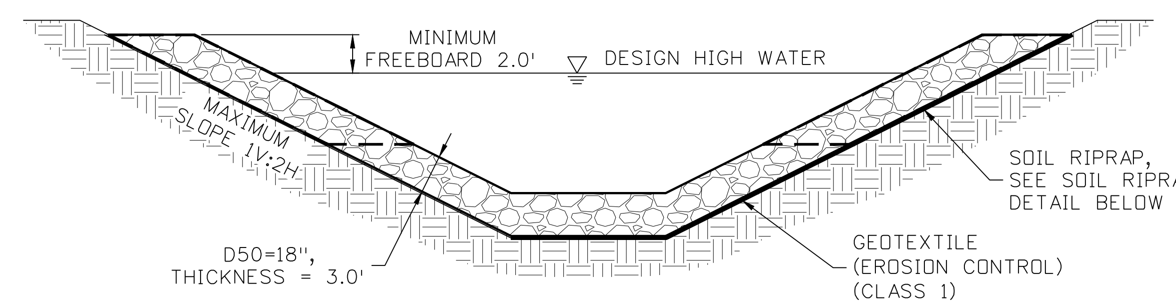
RIPRAP LAYOUT

SCALE: 1" = 20'



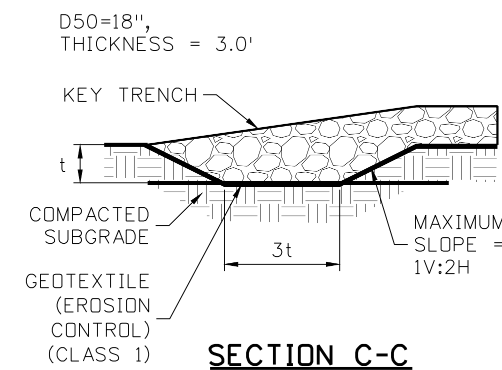
SECTION A-A

NTS



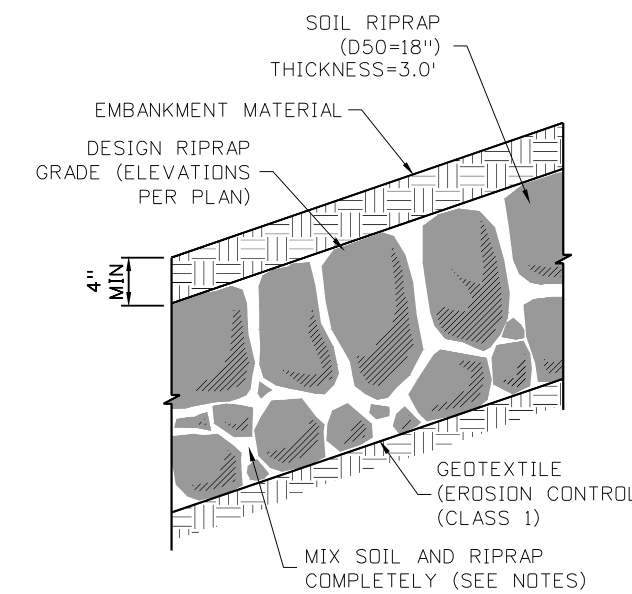
SECTION B-B

NTS



SECTION C-C

NTS



SOIL RIPRAP

NTS

NOTES:
1. VOIDS IN RIPRAP SHALL BE FILLED WITH EMBANKMENT MATERIAL. MIX UNIFORMLY 65% RIPRAP BY VOLUME WITH 35% OF APPROVED SOIL BY VOLUME PRIOR TO PLACEMENT.
2. PLACE STONE-SOIL MIX TO RESULT IN SECURELY INTERLOCKED ROCK AT THE DESIGN THICKNESS AND GRADE. COMPACT AND LEVEL TO ELIMINATE ALL VOIDS AND ROCKS PROJECTING ABOVE DESIGN RIPRAP TOP OF GRADE.
3. PROVIDING AND PLACING SOIL FOR SOIL RIPRAP AND COVER SHALL BE INCLUDED IN COST OF SOIL RIPRAP.

POINT TABLE

POINT #	NORTHING	EASTING	ELEVATION
1	1281619.22	2085270.09	6174.50
2	1281611.96	2085245.48	6166.41
3	1281607.00	2085228.65	6164.16
4	1281624.37	2085216.31	6164.50
5	1281622.97	2085213.64	6166.91
6	1281613.54	2085195.70	6174.50
7	1281603.96	2085209.45	6174.50
8	1281601.96	2085212.70	6174.50
9	1281597.32	2085215.38	6174.50

POINT TABLE

POINT #	NORTHING	EASTING	ELEVATION
10	1281588.49	2085214.55	6174.50
11	1281583.91	2085216.03	6174.50
12	1281557.69	2085216.76	6174.50
13	1281556.28	2085214.75	6174.50
14	1281547.80	2085210.62	6174.50
15	1281543.72	2085211.64	6174.50
16	1281542.97	2085228.54	6165.73
17	1281541.86	2085252.16	6166.97
18	1281541.17	2085266.58	6174.50

POINT TABLE

POINT #	NORTHING	EASTING	ELEVATION
19	1281544.26	2085267.61	6174.50
20	1281549.27	2085269.95	6174.50
21	1281560.16	2085270.50	6174.50
22	1281597.11	2085272.61	6174.50
23	1281614.89	2085272.33	6174.50

PRELIMINARY

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SUITE 410 S
DENVER, CO 80246
PHONE (303) 757-3655

Sheet Revisions

Date:	Comments	Init.

MONTEZUMA County
109 WEST MAIN STREET
CORTEX, CO 81321
ROOM 260
PHONE: 970-565-3728
FAX: 970-385-3635

As Constructed
No Revisions:
Revised:
Void:

ALKALI CREEK
RIPRAP LAYOUT PLAN

Designer: LDR
Detailer: ACT
Sheet Subset: HYDR
Structure Numbers: 083000N01.80016
Subset Sheets:

Project No./Code
22521
Sheet Number