Janitell Bridge over Fountain Creek Scour Protection Project

Structure No. EPC 0377-00.50A

COLORADO LICENSED
NOELLE S. BEEGLE
12/27/2022
41284
PROFESSIONAL ENGINEER

Final Drainage Report

El Paso County Transportation

3275 Akers Drive

Colorado Springs, Colorado 80922

EPC PO Number: 8114482

December 2022

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Ms. Werre:

Transmitted herewith is the Final Drainage Report for the scour protection project at the Janitell River Bridge over Fountain Creek in El Paso County, Colorado. This report includes hydraulics analysis using HEC-RAS version 4.1.0 with flowrates from the FEMA Flood Insurance Study Vol. 1-8, Revised: December 7, 2018. The HEC-RAS model is part of this submittal.

The report has been prepared at the request of the El Paso County to document the design of the scour mitigation measures at the Janitell Road Bridge.

We remain available at any time to answer questions or provide specific information relative to this study.

Respectfully submitted,

Noelle S Beegle

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Design Engineer's Statement:

The attached drainage plan and report were prepared under my direction and supervision and are correct to the best of my knowledge and belief. Said drainage report has been prepared according to the criteria established by the County for drainage reports and said report is in conformity with the applicable master plan of the drainage basin. I accept responsibility for any liability caused by any negligent acts, errors, or omissions on my part in preparing this report.

Noelle S Beegle

______________________________ 12/23/2022______________

Noelle S. Beegle, PE #41284 Date

El Paso County:

Filed in accordance with the requirements of the Drainage Criteria Manual, Volumes 1 and 2, El Paso County Engineering Criteria Manual and Land Development Code as amended.

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Jennifer Irvine, P.E. Date

County Engineer / ECM Administrator

Conditions:

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I. PURPOSE

El Paso County has contracted Alfred Benesch & Company (Benesch) to explore three alternatives to mitigate the scour at the bridge abutments at the Janitell Road bridge over Fountain Creek. The alternatives study was completed December 2021 and was approved by the County. The County selected Alternative 1. The purpose of this report is to describe the scour mitigation design to protect the bridge from additional scour and instability.

II. GENERAL LOCATION

The County structure name of the bridge is EPC0377-00.50A. The bridge and project extents are located about 2.5 miles southeast of downtown Colorado Springs, Colorado at the boundary of the Northeast ¼ of the Southeast ¼ of Section 29 and the Northwest ¼ of the Southwest ¼ of Section 28, Township 14 South, Range 66 West of the 6th Prime Meridian. See Figure 1 for a vicinity map.

Figure 1: Vicinity Map

The project site is bounded by private property on all sides. All properties will be impacted by each mitigation alternative. The area is considered part of the Valley Gardens Plat. The upstream side of the bridge and Fountain Creek channel is owned by Jose Luis Garcia, see the map. Downstream of the bridge there are three property owners: the City of Colorado Springs, Me and Thee, LLC, and Recycled Aggregate Products, Inc. The City of Colorado Springs owns property at the center of the bridge where the Pikes Peak Greenway trail is located. An ownership map created by Farnsworth is provided in the Appendix.

III. DESCRIPTION OF PROJECT

A. Limits of Disturbance

The total area of construction impact is 3.57 acres. This includes staging areas and access roads. The total area of surface disturbance is 1.43 acres. This includes riprap placement, excavation, and seeding. Generally, the impact area includes the north bridge abutment and the slopes upstream and downstream of the bridge. An overflow channel will be excavated between Piers 3 and 4. This overflow channel will help redirect and spread larger flow events in the channel.

B. Ground Cover

The ground cover in the project area is sparse. The north abutment is eroded with exposed dirt and rocks. In the channel and along the south banks, the ecological value of the area is low with elm trees and smooth brome as dominant vegetation. There are only a few potential wetlands on the fringes of the creek downstream of the bridge, outside of the impact area, on the north bank. No other wetlands in the area were identified. On the south bank, the bench is 3-5 feet above the channel. The main channel is along the north bank. This limits the available ground water along the south bank. There are some native trees and shrubs in the area including cheatgrass, thistle, wheatgrass, willow, and cottonwood, but no thick stands likely due to lack of groundwater and trampling. There are also some younger cottonwoods east of the bridge and multiple 80-year-old (or older) trees in the area. No cottonwood saplings were noted, which indicates no regenerative growth is occurring.

C. General Topography and Topographical Survey

The general topography is sloping downstream along Fountain Creek from northwest to southeast. The north bank is very steep with 1:1 slopes. The south bank is flatter with terraced banks at 2:1 or less. As part of the initial exploration phase of this alternatives study, Benesch requested that Farnsworth provide topographic survey of the bridge, channel, and overbanks in the vicinity of the bridge. Farnsworth completed the survey of Fountain Creek from downstream of the bridge to upstream where Sand Creek enters Fountain Creek. This survey was used to develop the hydraulic model.

D. Soils and Geotechnical Investigations

The overall soil type was reviewed by looking at the USDA NRCS Web Soil Survey. Soils at the bridge and impact area include Ellicott loamy coarse sand. The hydrologic soils group is A. Type A soils have high infiltration rates and are well drained. See the Appendix for a soils map. See the Appendix for a detailed soils map.

Benesch contracted with CTL Thompson (CTL) to perform geotechnical investigations. The full geotechnical investigation report is a separate document and is included in the Appendix. Benesch provided CTL with six preferred boring locations in the vicinity of the channel and piers. Benesch was considering both a cut off wall and drop structure to provide scour protection. Because of the poor access and high water in the creek, CTL could only complete four borings. See the Appendix for correspondence from CTL and the Geotechnical Report. CTL measured the current ground surface (in the channel) at the western pier at P-5 which was approximately 9 feet below the previous ground elevation.

The CTL report also stated that previous and current observations indicate that shale bedrock is exposed along most of the stream bed in this area of Fountain Creek or is within 2-4 feet of the stream bed. There is an intermittent layer of sand and gravel with scattered cobbles along the bottom of the stream over shale bedrock. Geology maps of the vicinity indicate that the local bedrock is Pierre Shale which is overlain by recent alluvial deposits in the creek bed.

Shale bedrock was encountered in all four borings. The shale has been eroded in the creek channel, near Pier 5 to about 3 to 5 feet below the bedrock surface at the north bank. Benesch compared the as-built plans and ground elevations with the geotechnical measurements to determine that the existing Pier 5 columns have about 6.6 feet penetration into bedrock.

To better determine the actual depth of the piers into the bedrock, Olson Engineering was contracted to perform Nondestructive Evaluation (NDE) Investigation for Piers 3, 4, and 5. Each pier has four columns. They performed the evaluation in the field in February-March 2022. The testing evaluated the depth and integrity of the pier's drilled concrete foundations. They use the Sonic Echo/Impulse Response test method. The evaluation report from Olson is in the Appendix. The summary of the findings is that the piers have much greater embedment than first thought. See Table 1. The embedment depth is measured from the ground elevation at the time of the test.

Pier	Column (from east to west)	Embedded Shaft Depth (ft)
3	1	26.0
3	2	19.3
3	3	19.9
3	4	26.0
4	1	17.5
4	2	18.8
4	3	17.6
4	4	16.7
5	1	16.1
5	2	16.4
5	3	14.3
5	4	11.5

Table 1: Results of NDE Investigation: embedment depth of tested Piers 3, 4, and 5.

The results show that the piers have a good amount of embedment into bedrock. The amount of scour observed at the piers occurred over 30 years. Bridges are generally built to provide a 50-year service life. To provide another 20 years of service life, additional scour must be prevented.

E. Structural Evaluation

Benesch performed a structural analysis of Pier 5 to assess the existing structural stability of the pier in its current condition. Based on the findings from the geotechnical investigations and the as-built drawings, structural engineers at Benesch reviewed the pier stability of the bridge. A memo was composed for the County to provide to CDOT stating that the bridge is stable in its current state. The calculations confirmed that the strength and serviceability of the scoured shafts is sufficient. A copy of the memo is in the Appendix. This memo did not preclude additional scour protection.

F. Major Drainageways

Janitell Bridge crosses Fountain Creek which is a major drainageway that runs from north to south through Colorado Springs and El Paso County to its confluence with the Arkansas River in Pueblo. About 750 feet upstream of the Janitell Road bridge is the Spring Creek confluence with Fountain Creek. About 1.7 miles downstream, Sand Creek enters Fountain Creek.

G. Utilities

Farnsworth also performed the subsurface utility engineering (SUE) investigations. The final report was completed in July 2021 and is included in the Appendix. All utility locates were performed to ASCE Quality Level (QL) B standards. No potholing to ASCE QL A was performed. QL A will be performed for the design phase of the project if necessary. The SUE project area encompasses approximately a 300-foot radius from the center of Janitell Road bridge. See the Appendix for the SUE project limits. This area covers the estimated proposed construction improvements. A summary of the utilities' investigation findings is below.

<u>Power:</u> Colorado Springs Utilities (CSU) has multiple overhead electric transmission lines within or near the SUE boundary.

<u>Gas:</u> CSU has two natural gas lines running under the length of the bridge deck and exiting the boundary limits on the north and south sides.

<u>Telecommunications/Fiber Optic:</u>
There are three Century Link copper lines at the southeast side of the bridge. These three lines head south and exit the boundary limits.
One MCI fiber optic line runs in a north-south direction just east of the bridge. Most of the MCI was QL-B except for a portion that cross Fountain Creek.

<u>Storm Sewers and Culverts:</u> Storm outfalls and culverts were located within the boundary limits at six locations. The invert locations were surveyed at the outfalls.

Potable Water, Sanitary Sewers, Irrigation: These utilities were not observed within the boundary limits.

Unknown: Two unknown PVC lines were observed running under the length on the bridge deck. These lines were not locatable due to a lack of locate method and are noted as QL-D.

USGS Gaging Station: USGS 07105530 FOUNTAIN CREEK BLW JANITELL ROAD BLW COLO. SPRINGS, CO The gage box is along the north bank about 50 feet upstream of the bridge. There is a data transmitter attached to the bridge. The gage housing and conduits associated with the gage operations may have to be relocated depending on which alternative is selected.

IV. MAJOR BASIN DESCRIPTIONS

A. Drainage Basin Planning Studies

The City of Colorado Springs has over time completed Drainage Basin Planning Studies (DBPS). A DBPS process is used to define major stormwater improvement needs in the city. Each DBPS identifies needed improvements, environmental impacts, and provides estimated costs. The needs may be in older, existing developed areas, areas that are the City's responsibility, or areas to be developed that developers will be responsible for. Fountain Creek has not been studied as an individual drainage basin but as part of several adjacent, contributing runoff basins. See Figure 2.

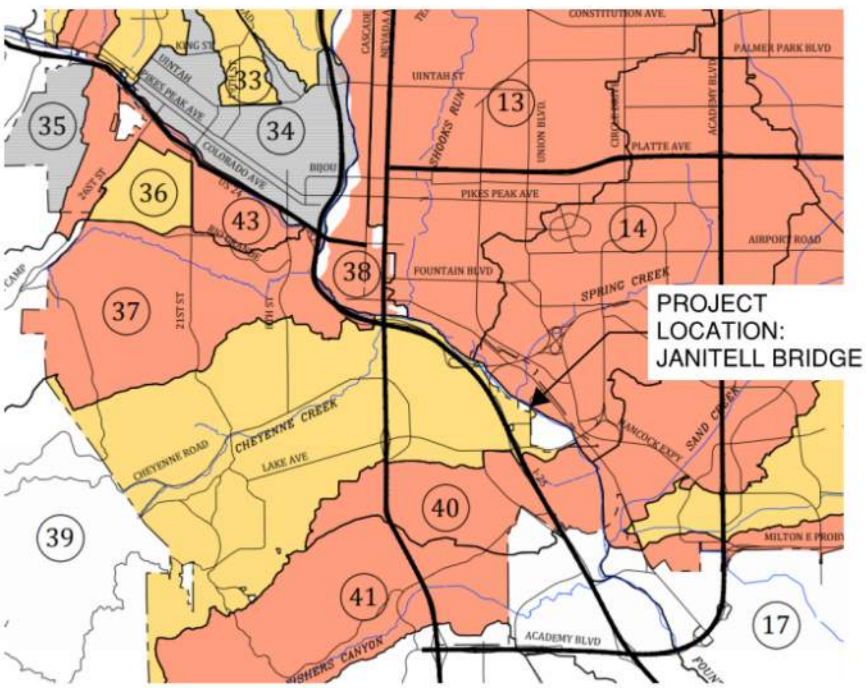

Figure 2: DBPS Inventory Map excerpt.

DBPS 39: Southwest Area – Upper Cheyenne Creek Cheyenne Run and Spring Run, completed in 1984, and 14: Spring Creek, completed in 1993, are adjacent to the Janitell Bridge project. The drainage basins studied in these reports outfall to Fountain Creek. Fountain Creek itself is not the subject of these reports.

B. Flood Plain Statement

Fountain Creek is a FEMA designated Special Flood Hazard Area and has been studied in detail. The flowrates for 20%, 10%, 1%, and 0.2% annual chance peak flood events at the Janitell Road bridge are available from the Federal Emergency Management Administration (FEMA) Flood Insurance Study (FIS) for El Paso County, revised Dec. 7, 2018. The FEMA Flood Insurance Rate Map (FIRM) number 08041CV001A has an effective date of December 7, 2018. See Figure 3 for a clipped image from the effective 2018 FIRM. The full FIRM is in the Appendix. The FIRM shows that the Floodway, 100-year, and 500-year events have been mapped.

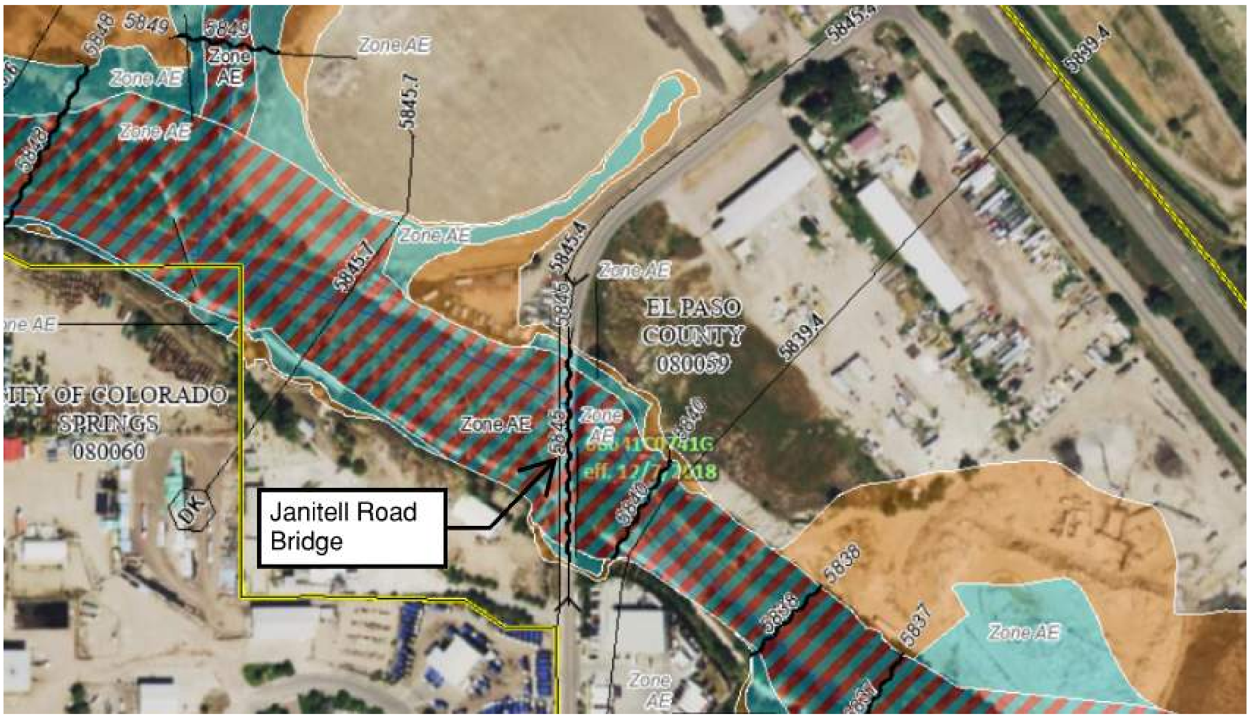

Figure 3: Excerpt from 2018 FIRM.

C. Major Basin Characteristics

Using the StreamStats program from USGS, the drainage area contributing to Fountain Creek at the Janitell bridge was mapped. The drainage area is about 412 sq. miles. The basin starts north at Palmer Lake, west to Woodland Park and Pike's Peak, east to the Black Forest, and south to Cheyenne Mountain. See the Appendix for the StreamStats Report.

V. SUB BASIN CHARACTERISTICS

A. Historic Drainage Patterns

There are two culverts that outfall to Fountain Creek at the Janitell bridge that have been identified in the SUE investigations as part of this project. See the Appendix for the SUE map.

One culvert is on the northside of the creek just upstream of the bridge. The culvert drains roadway runoff from Janitell Road north of the creek. The outfall of this culvert is along the eroded bank of Fountain Creek. The project will not impact the culvert and will be protected in place.

The other culvert is on the southside of the creek adjacent to the upstream abutment of the Janitell bridge. This culvert also conveys roadway and adjacent property runoff from south of the bridge. The outfall of this culvert runs down the upstream abutment and across the Pikes Peak Regional Trail. The project will not impact the culvert and will be protected in place.

B. Offsite Drainage Patterns

There is no offsite drainage from this project. All the work is within the Fountain Creek drainage area. All flows through the project site stay within the creek floodplain. The alternatives study looks at the impact to the floodplain of Fountain Creek.

VI. DRAINAGE DESIGN CRITERIA

A. References

The final design for scour mitigation was evaluated using criteria from the El Paso County Engineering Criteria Manual, the Colorado Springs/El Paso County Drainage Criteria Manual Volumes 1 and 2 and the Mile High Flood District (MHFD) Urban Storm Drainage Criteria Manuals 1-3.

The floodplain impacts were evaluated for no impact or no rise conditions using FEMA Floodplain criteria.

B. Previous Drainage Studies

The bridge and the Fountain Creek floodplain are mapped as part of the El Paso County Flood Insurance Study (FIS), Revised Dec. 7, 2018, by the Federal Emergency Management Administration (FEMA).

VII. FOUR STEP PROCESS

A. Employ Runoff Reduction Practices

This project will not increase impervious area. No drainage or runoff calculations will be performed.

B. Stabilize Drainageways

This project stabilizes the natural channel of Fountain Creek by replacing eroded riprap protection around the bridge. The north abutment and bank will also be stabilized and have less steep slopes. A grading and erosion control permit will be necessary to complete the project. During construction, water diversions and water quality best management practices will be utilized.

C. Provide Water Quality Capture Volume

No Water Quality Capture Volume will be calculated with this project. This project will have minimal water quality impacts. During construction, the runoff from construction activities shall be controlled. Temporary BMPs such as rock check dams and/or erosion control logs will control runoff from side slopes. A temporary access road will be constructed to place the riprap. Permanent BMPs such as seeding, soil retention blankets, and rock check dams will help control runoff. Riprap placement at the abutment and channel bed will limit the amount of scour and hold the channel in place reducing bed removal through the structure.

D. Consider Need for Industrial and Commercial BMPs

This project is not within an industrial or commercial site.

VIII. HYDROLOGIC CRITERIA

According to the FIS, Fountain Creek was studied by detailed methods. "Flow rates for portions of Fountain Creek, Upper Fountain Creek, and Monument Creek downstream of the U.S. Air Force Academy were adopted from a USACE 1976 hydrology report and USACE Flood Plain Information (FPI) reports. This hydrologic method consists of gage station analysis, whereby stream gaging data have been analyzed to estimate peak flows for the various recurrence intervals." Table 2 shows the peak discharge rates from the 2018 El Paso County FIS from FEMA.

Table 2: Peak discharges for Fountain Creek at Janitell Road Bridge from 2018 FIS.

Peak Event	10-Year (cfs)	50-Year (cfs)	100-Year (cfs)	500-Year (cfs)
Flowrate	11,800	18,800	22,400	32,200

These flowrates were used to define the floodplain and floodway in the FEMA regulatory maps.

IX. GENERAL CONCEPT

The existing structure was built in 1991 and replaced a narrow two lane bridge located about 450 feet upstream of the current bridge. The new bridge deck is approximately 26' above the channel centerline.

See Figure 4 for an excerpt from the as-built drawings from the 1991 bridge design plans. In the 1991 plans excerpt, the main channel of Fountain Creek was at the center of the bridge. The abutments were evenly sloped and protected with riprap. The Pikes Peak Greenway Trail had not been constructed yet. The piers and abutments are numbered in Figure 4 from left to right looking upstream.

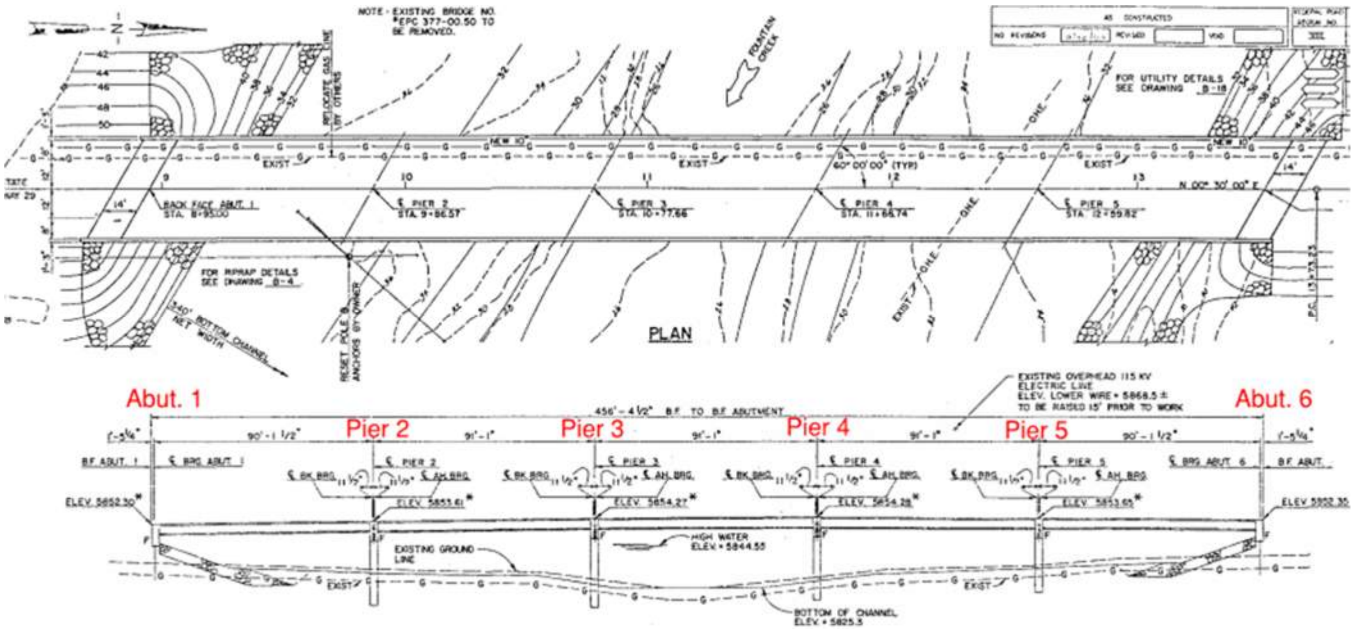

Figure 4: 1991 As-built drawing excerpt.

Since 1991, the channel of Fountain Creek has been migrated north and the channel has been modified downstream of the bridge. See Figure 5 for review of historical images from Google Earth. The channel of Fountain Creek at the Janitell Road bridge has been modified over time from both manmade and natural forces. The creek in this area can see flows greater than 4,000 cfs yearly due to spring runoff. There were also large flooding events in the recent past including events in 1999 and 2013.

Over time, the channel at Pier 5 and the riprap at abutment 6 has been eroded. The channel material at Pier 5 has scoured away and exposed the pier about 9 feet since construction. See the pictures of the channel and abutment erosion at the end of this report.

In March 2020 as part of CDOT's Off-System Bridge Inspection Program, the engineering firm SEH, Inc. inspected the bridge and filed an Essential Bridge Repair Documentation Form. This form described the essential bridge repairs:

> "Place scour countermeasures at the north pier (P5) to protect the columns and inhibit future scour. Place additional countermeasures where the channel is migrating to the north and cutting into the toe of the embankment slope below the north abutment. These counter measures are to be installed in accordance with an engineered design and completed within the next year or as funding allows."

1999

1999: Note that the channel alignment is smooth with a long, gently curved alignment. The downstream bank on the north side does not constrict the channel.

2003

2003: Note that the downstream north bank has been pushed out into the channel. High voltage towers have been installed up and down stream of the bridge (shown with red circles). The bank was filled below the downstream tower changing the stream alignment. It can also be seen that the north abutment has migrated north likely due to flooding events.

2020

2020: North embankment has migrated further north. The channel has undergone a significant alteration in alignment. The channel takes an abrupt turn to the south downstream of the bridge. The Pikes Peak Greenway Trail has been installed along with riprap embankment protection on the south side of the channel downstream of the bridge.

Figure 5: Historical images of the alignment of Fountain Creek at Janitell Road bridge.
(Source: Google Earth)

There are no major offsite drainages to the project area other than a couple culverts: one from the north one from the south. The north outfall will be rebuilt to accommodate the proposed slope stabilization. The south outfall will not be impacted by the project.

X. FEMA DATA

Because the scour protection project is in the regulated floodplain of Fountain Creek, a no-rise condition in the FEMA Effective WSEL at the regulatory cross sections must be met. With a no-rise condition, floodplain map changes requiring a CLOMR/LOMR action is not required.

El Paso County reached out to FEMA and requested the latest hydraulic modeling for Fountain Creek. FEMA provided the "Restudy of Fountain Creek" HEC-RAS model, by WHPacific which was completed in March 2011 with FEMA review comments incorporated for the final 2013 version (2013 model). The model was completed in Datum NGVD29 using HEC-RAS version 4.1.0, January 2010.

This 2013 model was run and is considered the effective model. There are four FEMA cross sections lettered from DH to DK that will be impacted by any improvements to the channel and banks in the vicinity of the bridge. See Figure 6 below. The effective model was copied and called corrected. Cross sections DL and DM are upstream of the project but any changes to the water surface elevation due to the project will be noted.

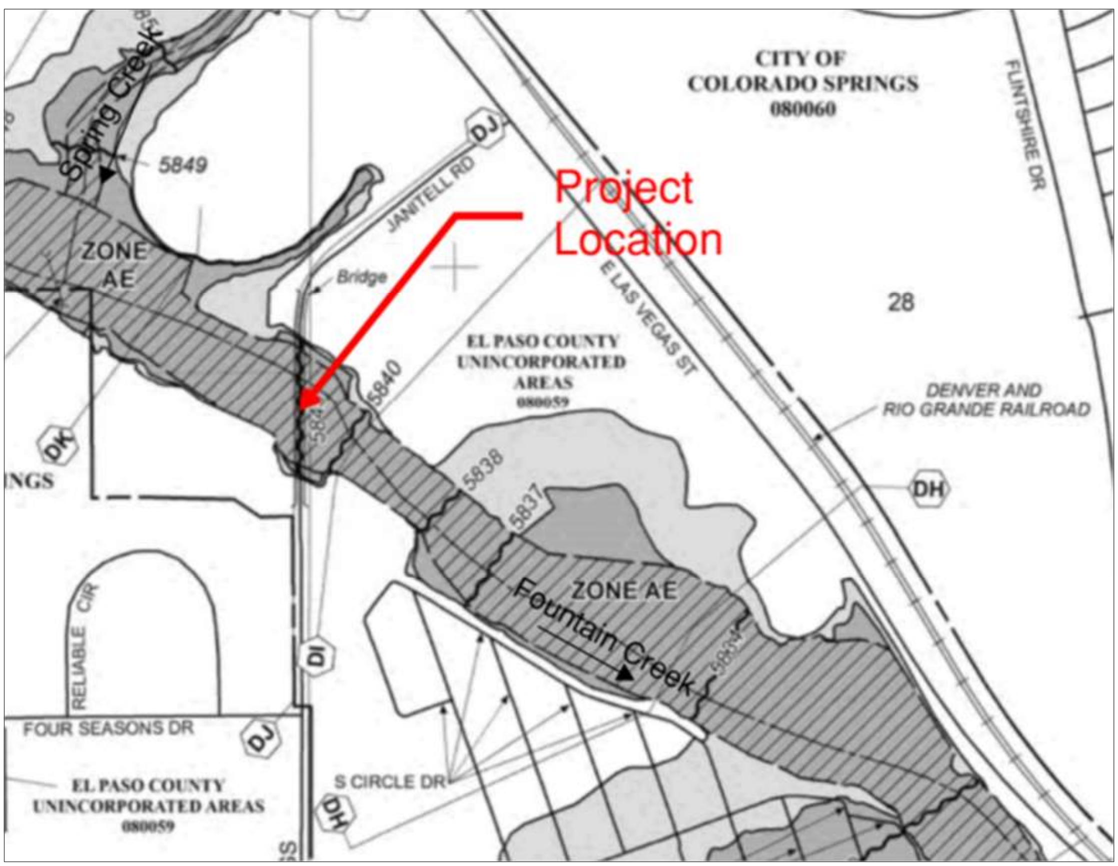

Figure 6: Excerpt from FEMA FIRM 08041C0741G, December 7, 2018.

The 2018 FIS converted older hydraulic model elevations from NGVD29 to NAVD88. These older models were converted from NGVD29 to NAVD88 using an average conversion factor of 3.5 feet (adding 3.5 feet to NGVD29 base flood elevations (BFE)). To confirm that the 2013 effective model was the effective model, the WSEL's of the effective HEC-RAS model river station cross sections that correspond to the lettered FIS/FIRM cross section were compared. They are shown in Table 2. The BFE at NAVD88 of these cross sections from the 2018 FIS are shown as well. The conversion factor (CF) for each cross section was calculated by subtracting the 2013 HEC-RAS BFE from the 2018 FIS BFE. The average CF is 3.5 feet.

Table 2: FEMA cross sections effected by project.

River Station	FIS Cross Section	2013 HEC-RAS Model BFE (NGVD29) (100-Year WSEL) (ft)	2018 FIS BFE (NAVD88) (100-Year WSEL) (ft)	Conversion Factor (ft)
15913	DM	5849.07	5852.6	3.53
15241	DL	5845.13	5848.6	3.47
14376	DK	5842.20	5845.7	3.50
13898	DJ	5841.91	5845.4	3.49
13598	DI	5835.89	5839.4	3.51
12358	DH	5830.76	5834.3	3.54

XI. HEC-RAS MODEL

This existing effective FEMA 2013 HEC-RAS model was modified based on current topography and proposed channel modifications to study projects impact to the BFE.

Detailed topographical survey and utility investigations were performed by Farnsworth in March 2021 at the bridge using Datum NAVD88. The survey included the channel and bank up and downstream of the bridge. Spot elevations at the tops of the pier from the bridge deck were also taken. The bridge deck is well above the 500-year WSEL, so detailed elevation data on the roadway was not required.

The El Paso County GIS department provided LIDAR with 2-foot increment contour data to supplement the topographic survey. To extend the topographic data, LIDAR and survey surfaces were combined to provide a detailed topography in the vicinity of the Janitell Bridge. The FEMA cross sections are wider than the surveyed area and the LIDAR data helped to provide more detail at the effected cross sections. The FEMA FIRM cross sections were imported from the National Flood Hazard Layer GIS program. These lines were imported into AutoCAD on the Colorado State Plane Central Zone NAD '83/(92) (CHARN) coordinate system. The topography of the channel from County LIDAR and project specific ground survey were used as the base for cutting cross sections to import into HEC-RAS. The bridge structure was recreated in the HEC-RAS model using the as-built plans. These plans are in the Appendix.

The original FEMA cross sections and centerline alignment from the 2013 HEC-RAS model was exported to the combined current topography. Cross sections DK through DI were cut with the corrected topography and converted to NAVD29 by subtracting 3.5 feet from the cross section elevation points to import back into the HEC-RAS model to create a new model called Corrected Effective (CorrEff). After reviewing the CorrEff model and determining the limits of the scour mitigation project, additional cross sections were added. See Table 3 for information about the additional cross sections. This new model is called CorrEff/ACS (Additional Cross Sections). Table 3 compares the BFE's from the different plans.

		1	2	3	4	1-3	1-4	3-4	
River Station	FIS Cross Section	Effective 2013 FIS Model BFE (ft)	CorrEff Model BFE (ft)	CorrEff/ ACS Model BFE (ft)	CorrFinal Model BFE (ft)	Eff-CorrEff/ ACS (ft)	Eff-Final (ft)	CorrEff/ ACS-Final (ft)	Cross Section Geometry
15913	DM	5849.07	5849.05	5849.03	5849.03	-0.04	-0.04	0	2013
15241	DL	5845.13	5845.87	5845.6	5845.5	0.47	0.37	-0.1	2013
14715		5844.6	5845.64	5845.29	5845.15	0.69	0.55	-0.14	2013
14512				5844.89	5844.89			0	Addl
14376	DK	5842.2	5840.66	5840.66	5840.66	-1.54	-1.54	0	Corrected
14235				5838.36	5837.59			-0.77	Addl
14065		5841.91	5839.61	5838.25	5837.53	-3.66	-4.38	-0.72	Corrected
13989				5837.85	5837.49			-0.36	Addl
13898	DJ	5841.93	5839.83	5838.56	5837.83	-3.37	-4.1	-0.73	Corrected
13870	Bridge								-
13849		5841.52	5839.06	5837.72	5837.34	-3.8	-4.18	-0.38	Corrected
13782				5837.2	5837.2			0	Addl
13598	DI	5835.89	5835.34	5835.34	5835.34	-0.55	-0.55	0	2013
12887		5833.86	5833.86	5833.86	5833.86	0	0	0	2013
12358	DH	5830.76	5830.76	5830.76	5830.76	0	0	0	2013

Table 3: Base flood elevations for the three models at the FEMA cross sections affected by project.

Column 1-3 takes the difference in BFE between the Effective plan and the Corrected Effective with Additional Cross sections plan. At the upper and lower limits, cross sections 15913 and 12887, the BFE's are within 0.5 feet of each other. FEMA states that for a stream that has a detailed study, an effective tie-in is obtained when the base flood elevations are within 0.5 foot of the effective elevations.

With a working Corrected Effective/ACS plan, the revised channel design cross sections were entered into the Final plan. Column 3-4 shows the change in BFE from the CorrEff/ACS plan and the Final plan. The BFE drops slightly through the project cross sections but ties into the plan outside the project. The project does not cause a rise in the WSEL for the BFE. A No Rise Certification Letter will be submitted to the County for approval.

XII. ENVIRONMENTAL IMPACTS

Pinyon Environmental teamed with Benesch to review the impacts of the project. As stated earlier in this report, there are only a few potential wetlands on the fringes of the creek downstream of the bridge on the north bank.

The project has a footprint of 1.43 acres. It will impact all adjacent property owners. This work falls under a USACE Nationwide Permit 3, Maintenance. This permit authorizes new or additional riprap to protect the existing structure. No Pre-Construction Notification is required. As noted above, a CLOMR/LOMR would not be required for this work.

Riprap will be placed along the north abutment and at the bridge at Piers 3 and 4, outside of the channel. Seeding, soil conditioning, and coir mats will be placed along the new south channel to stabilize the excavated areas. Excavated channel material will be stockpiled to be placed in the new channel.

XIII. MAINTENANCE

Riprap placement is necessary to protect structures in riverine environments. Riprap is very flexible and can shift to fill in eroded areas. Over time, riprap will move or be washed away. The design life of a bridge can be from 50 to 100 years. During that time, large flood events can and will move material in the channel. The channel and bank erosion at the Janitell Road bridge is typical for a structure in a riverine location. Regular bridge inspections provide feedback to the owners of the structure to measure and describe changes. This bridge has seen large flood events since construction, and it is time for maintenance and replacement of riprap protection.

XIV. SCOUR PROTECTION

The Hydraulic Toolbox 4.2 was used to calculate appropriate riprap size using HEC-18 formulas. Riprap protection at Abutment 6 is necessary to prevent further erosion and to reestablish a 2:1 side slope. Review of the original design drawings for the bridge revealed that 24” riprap was placed at the abutments. The calculated riprap size based on the 100-year flow rate was D_{50}=6.50”. However, due to the previous erosion of the original riprap, it was determined that D_{50}=6.50” riprap would not provide sufficient protection. The design specifies D_{50}=24” keyed into bedrock. The extent of the riprap protection was determined using HEC-23 Design Guideline 14.1.

Riprap protection is required around Piers 3, 4 and 5 to prevent additional scour that could impact the structural integrity of the bridge. The calculated riprap size based on the 100-year flow rate was D_{50}=12.5”. However, due to the previous erosion of the original riprap, it was determined that riprap of this size would not provide sufficient protection. Therefore, the riprap protection around the piers is proposed as size D_{50}=24”. Per the HEC-23 Design Guidelines, the depth of riprap should be 3xD_{50} or 6’. The riprap will be keyed into the bedrock having a minimum embedment depth of 1’. Additionally, the riprap placement should extend a length twice the width of the pier. The width of the piers is 3’ so the riprap will extend 6’ around each side of the piers. Upstream and downstream of the pier riprap, the material will be tapered.

The proposed overflow channels will have riprap protection along the slopes through the bridge and the length of the bridge piers. The riprap will be of size D_{50}=12” and will be 2’ thick along the side slopes. The riprap will be keyed into bedrock.

XV. REFERENCES

1. FEMA Flood Insurance Study, El Paso County, Colorado, and Incorporated Areas, Revised: December 7, 2018, Number 08041CV001A, Volumes 1-8
2. Drainage Criteria Manual, El Paso County Public Works, 1994
3. United States Army Corps of Engineers (USACE), Hydraulic Engineering Center-River Analysis System (HEC-RAS), Version 6.0.0, May 2021
4. Fountain Creek Watershed Study, Watershed Management Plan, January 2009, USACE, Albuquerque District
5. CDOT Drainage Criteria Manual, 2004
6. HEC-18, Fifth Edition, April 2012
7. Mile High Flood District (formerly Urban Drainage and Flood Control District) Drainage Criteria Manual, Vol. 1-3

XVI. ATTACHMENTS

HEC-RAS Cross Section Map
Plan and Profile Sheets from Plan Set
No Rise Certification Letter

XVII. APPENDIX

1. As-Built Drawings-Selected Sheets
2. Subsurface Utility Engineering Exhibit
3. Geotechnical Investigation: Geotechnical Report and Correspondence
4. Non-Destructive Testing Report
5. Structural Evaluation Memo
6. FEMA FIS/FIRM References
7. HEC-RAS Model Tables and Cross sections
8. Riprap Design Calculations
9. Environmental
10. USGS Soils Report
11. Property Impacts

XVIII. SITE PHOTOS-EXISTING CONDITIONS

Upstream south abutment, intact riprap.

Looking north from upstream.

Pier 2, looking north downstream side.

Downstream power tower, eroded bank.

Looking downstream, trail rirap abutment.

Looking upstream at Pier 5.

Looking upstream at Pier 4.

Looking upstream ar Pier 5.


Pier 5 and north downstream bank.


Upstream north bank rubble.


Upstream abutment 6 erosion.

Looking upstream, south bank.

Upstream power tower, eroded bank.

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Last Modification Date: 12/6/2022	User: NSB
Full Path: Y:\Denver\152000S\00152064.00\Eng_Docs\Drainage	
Drawing File Name: HEC-RAS Exhibit-2.dwg	
Acad Ver. 2020 Scale: As Noted	Units: ENGLISH

Sheet Revisions

El Paso County
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3275 Akers Drive
Colorado Springs, CO 80922
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benesch
Alfred Benesch & Company, Inc.
7979 E. Tufts Avenue, Suite 800
Denver, CO 80237

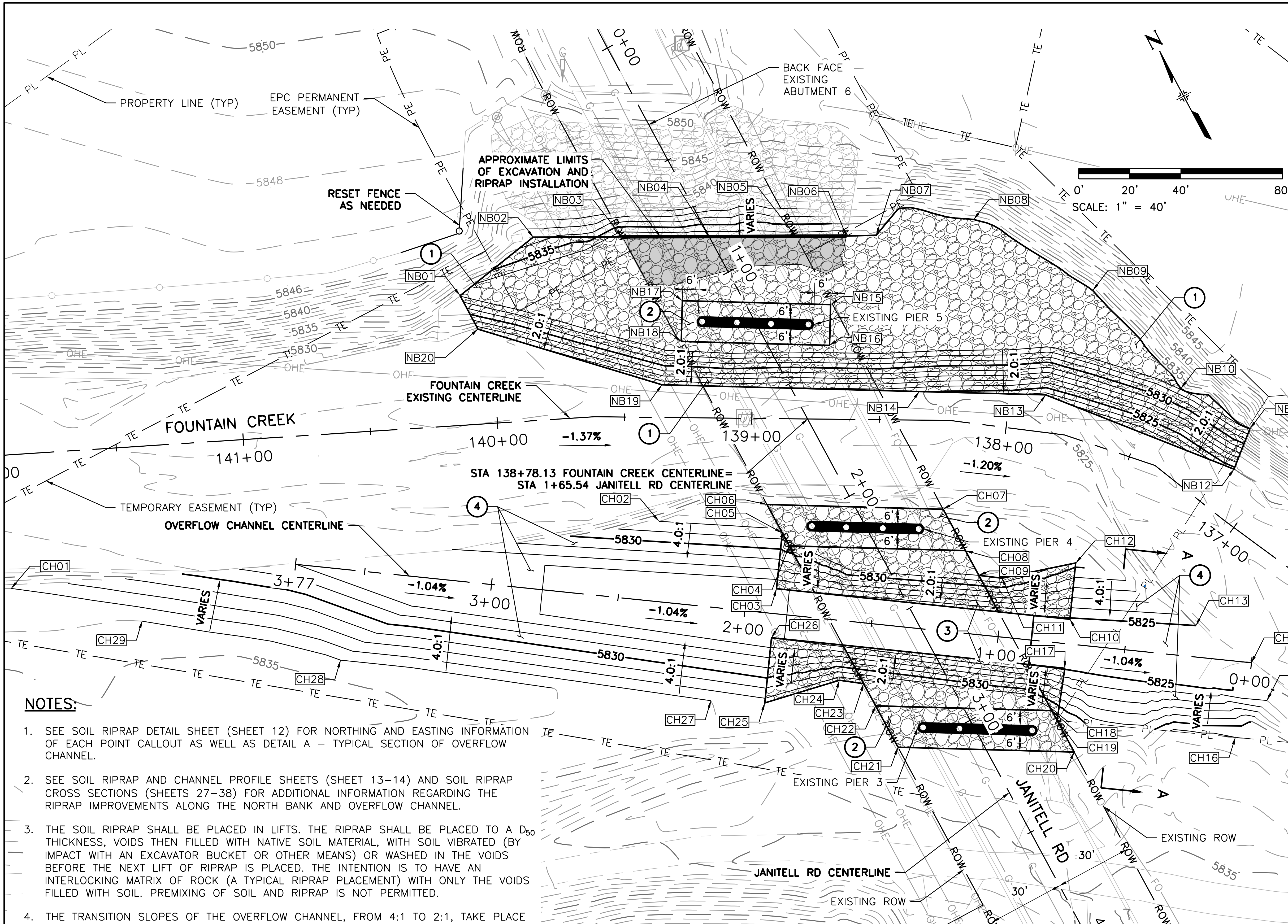
As Constructed
No Revisions:
Revised:
Void:

JANITELL ROAD BRIDGE SCOUR PROTECTION PROJECT HEC-RAS EXHIBIT

Designer: NSB
Detailer:
Sheet Subset:
Structure Numbers
Subset Sheets: of

Project No./Code
EPC PO # 8114482
BENESCH PN 152064
Sheet Number 1 OF 1

Y:\Denver\152000S\00152064.00_EPC_Janitell_Bridge\Eng_Docs\Drainage\HEC-RAS Exhibit-2.dwg - Current tab: Map - Print Time: Tue, 06 Dec 2022 - 11:02am By: nbeegle Last Save: Mon, 05 Dec 2022 - 11:58am



LEGEND:

- EXISTING RIPRAP
- EXISTING RIPRAP/SOIL TO BE REMOVED
- NEW SOIL RIPRAP
- PROPOSED CONTOUR MAJOR
- PROPOSED CONTOUR MINOR
- EXISTING CONTOUR MAJOR
- EXISTING CONTOUR MINOR
- CHXX OVERFLOW CHANNEL POINT
- NBXX NORTH BANK POINT

CONSTRUCTION NOTES:

1. **ABUTMENT PROTECTION**
   REMOVE PORTION OF EXISTING RIPRAP
   INSTALL CLASS VH RIPRAP
   D_{50}=24"
   EMBEDMENT INTO EXISTING BEDROCK=1'

2. **PIER PROTECTION**
   INSTALL TYPE VH RIPRAP
   D_{50}=24"
   EMBEDMENT INTO EXISTING BEDROCK=1'

3. **OVERFLOW CHANNEL**
   INSTALL CLASS M RIPRAP
   (SLOPES UNDER BRIDGE)
   D_{50}=12"
   EMBEDMENT INTO EXISTING BEDROCK=1'

4. **OVERFLOW CHANNEL**
   INSTALL SEEDING, TOPSOIL, AND COIR MAT (ALONG OVERFLOW CHANNEL WHERE THERE IS NO RIPRAP)
   THICKNESS=6"

NOTES:

1. SEE SOIL RIPRAP DETAIL SHEET (SHEET 12) FOR NORTHING AND EASTING INFORMATION OF EACH POINT CALLOUT AS WELL AS DETAIL A – TYPICAL SECTION OF OVERFLOW CHANNEL.
2. SEE SOIL RIPRAP AND CHANNEL PROFILE SHEETS (SHEET 13–14) AND SOIL RIPRAP CROSS SECTIONS (SHEETS 27–38) FOR ADDITIONAL INFORMATION REGARDING THE RIPRAP IMPROVEMENTS ALONG THE NORTH BANK AND OVERFLOW CHANNEL.
3. THE SOIL RIPRAP SHALL BE PLACED IN LIFTS. THE RIPRAP SHALL BE PLACED TO A D_{50} THICKNESS, VOIDS THEN FILLED WITH NATIVE SOIL MATERIAL, WITH SOIL VIBRATED (BY IMPACT WITH AN EXCAVATOR BUCKET OR OTHER MEANS) OR WASHED IN THE VOIDS BEFORE THE NEXT LIFT OF RIPRAP IS PLACED. THE INTENTION IS TO HAVE AN INTERLOCKING MATRIX OF ROCK (A TYPICAL RIPRAP PLACEMENT) WITH ONLY THE VOIDS FILLED WITH SOIL. PREMIXING OF SOIL AND RIPRAP IS NOT PERMITTED.
4. THE TRANSITION SLOPES OF THE OVERFLOW CHANNEL, FROM 4:1 TO 2:1, TAKE PLACE FOR APPROXIMATELY 20' BEYOND THE EXTENTS OF THE RIPRAP.

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Acad Ver. 2020	Scale: 1"=40' Units: English

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303-771-6868 Job No. 152064.01

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Revised:
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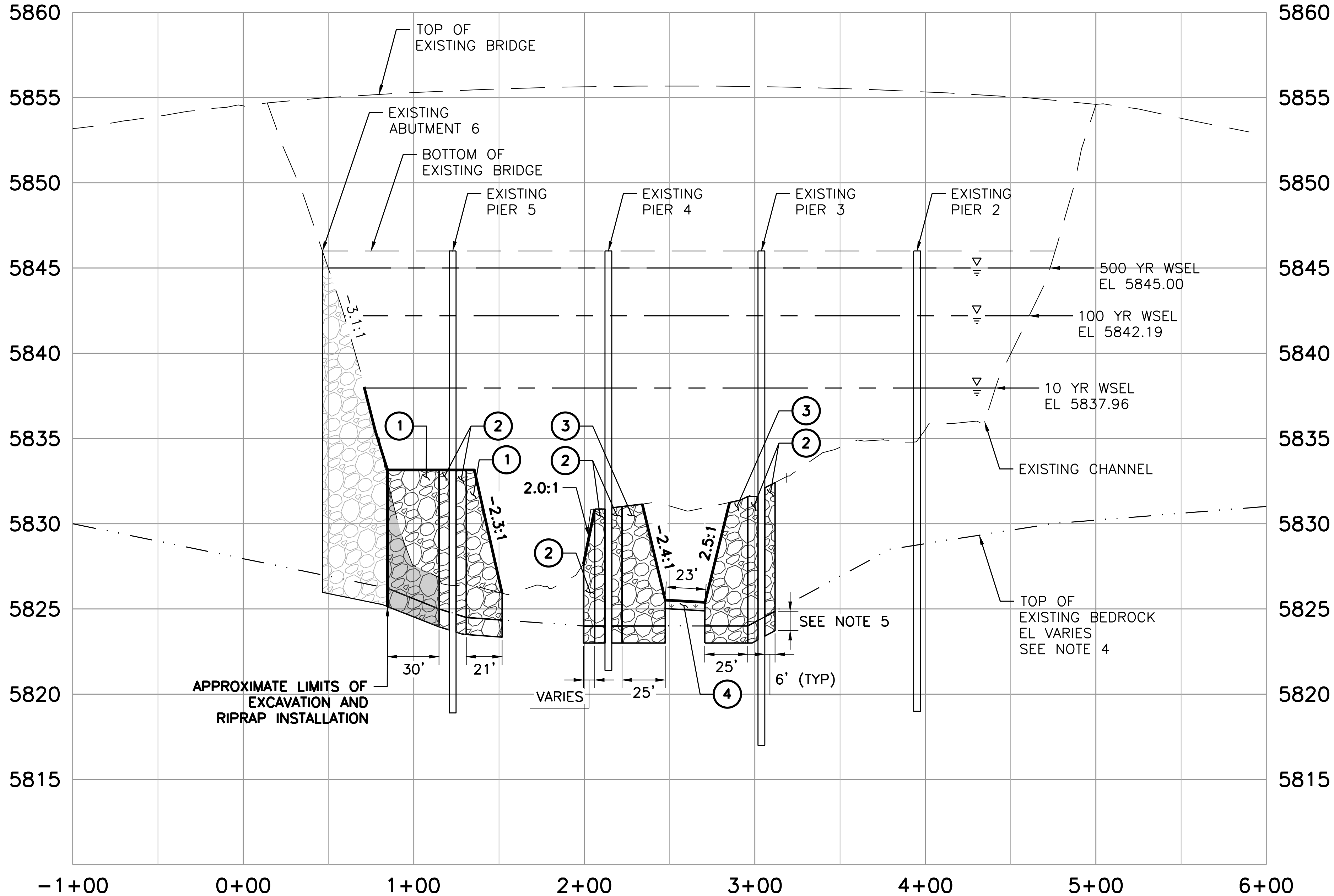
JANITELL RD BRIDGE SCOUR PROTECTION

SOIL RIPRAP PLAN

Designer:	MES	Structure Numbers	EPC 0377-00.50A
Detailer:	LMW		
Sheet Subset:	PLAN	Subset Sheets:	1 of 2

Project No./Code



PROFILE VIEW – JANITELL RD CENTERLINE

VERTICAL SCALE: 1"=8'

LEGEND:

- EXISTING RIPRAP
- EXISTING RIPRAP/SOIL TO BE REMOVED
- NEW SOIL RIPRAP
- SEEDING, TOPSOIL, AND COIR MAT

CONSTRUCTION NOTES:

1. **ABUTMENT PROTECTION**
   REMOVE EXISTING RIPRAP
   INSTALL CLASS VH RIPRAP
   D_{50}=24"
   EMBEDMENT INTO EXISTING BEDROCK=1'

2. **PIER PROTECTION**
   INSTALL TYPE VH RIPRAP
   D_{50}=24"
   EMBEDMENT INTO EXISTING BEDROCK=1'

3. **OVERFLOW CHANNEL**
   INSTALL CLASS M RIPRAP
   (SLOPES UNDER BRIDGE)
   D_{50}=12"
   EMBEDMENT INTO EXISTING BEDROCK=1'

4. **OVERFLOW CHANNEL**
   INSTALL SEEDING, TOPSOIL, AND COIR MAT
   (ALONG OVERFLOW CHANNEL WHERE THERE IS NO RIPRAP)
   THICKNESS=6"

NOTES:

1. THICKNESS OF RIPRAP SHALL BE NO LESS THAN SHOWN AND NO MORE THAN TWO (2) INCHES GREATER THAN THICKNESS SHOWN.
2. PROFILE IS OF JANITELL RD CENTERLINE. SEE SOIL RIPRAP CROSS SECTIONS (SHEETS 27–38) FOR ADDITIONAL INFORMATION REGARDING THE RIPRAP IMPROVEMENTS ALONG THE NORTH BANK AND OVERFLOW CHANNEL.
3. THE CONTRACTOR SHALL BENCH THE PROPOSED CUT TO OSHA STANDARDS. THE CONTRACTOR SHALL DETERMINE THE APPROPRIATE MEANS AND METHODS TO ENSURE SLOPE STABILITY DURING CONSTRUCTION ACTIVITY AS WELL AS LONG TERM STABILITY AFTER WORK IS COMPLETE. NO SEPARATE PAYMENT WILL BE MADE.
4. THE ELEVATION OF THE TOP OF EXISTING BEDROCK IS APPROXIMATE. FOR ADDITIONAL INFORMATION, SEE THE GEOTECHNICAL INVESTIGATION FOR THE JANITELL ROAD BRIDGE, PREPARED BY CTL THOMPSON INC. ON SEPTEMBER 28, 2021.
5. SOIL RIPRAP EMBEDMENT INTO EXISTING BEDROCK SHALL BE A MINIMUM OF 1' WHERE SOIL RIPRAP IS CALLED OUT ON THE PLANS.

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Denver, Colorado 80237
303-771-6868 Job No. 152064.01

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No Revisions:
Revised:
Void:

JANITELL RD BRIDGE SCOUR PROTECTION			
SOIL RIPRAP PROFILE			
Designer:	MES	Structure Numbers	EPC 0377-00.50A
Detailer:	LMW		
Sheet Subset:	PROF	Subset Sheets:	1 of 2

Project No./Code

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Alfred Benesch & Company
7979 E. Tufts Avenue, Suite 800
Denver, CO 80237
www.benesch.com
P 303-771-6868

December 7, 2022

Alissa Werre
El Paso County Project Manager
Department of Public Works
3275 Akers Drive
Colorado Springs, Colorado 80922

RE: **Janitell Bridge Scour Mitigation, PO#8114482**
No-Rise Certification Letter

I certify that I am a duly qualified registered Professional Engineer or Architect licensed in the state of Colorado.

I certify that the proposed project Janitell Bridge Scour Mitigation (PO#8114482) as detailed on construction drawings Janitell Road Bridge Over Fountain Creek Scour Protection Project will result in zero rise in the FEMA designated 100-year flood heights, and no increase in the 100-year discharge and no increase in the 100-year floodplain width, at published and unpublished cross sections of the current FEMA floodplain of Fountain Creek as shown on FEMA map 08041C0741G. This certification is intended as proof of meeting the requirements set forth in the Pikes Peak Regional Building Code RBC313.20.1.

I offer the following documentation in accordance with standard Engineering practice to support my findings:
a) HEC-RAS 4.1.0 Hydraulics Model, Effective Model 2013 with Corrective and Final
b) Excerpts from FEMA Flood Insurance Study for El Paso County, effective date December 7, 2018

The 2018 FIS converted older hydraulic model elevations from NGVD29 to NAVD88. These older models were converted from NGVD29 to NAVD88 using an average conversion factor of 3.5 feet (adding 3.5 feet to NGVD29 base flood elevations (BFE)). To confirm that the 2013 effective model was the effective model, the WSEL's of the effective HEC-RAS model river station cross sections that correspond to the lettered FIS/FIRM cross section were compared. They are shown in Table 1. The BFE at NAVD88 of these cross sections from the 2018 FIS are shown as well. The conversion factor (CF) for each cross section was calculated by subtracting the 2013 HEC-RAS BFE from the 2018 FIS BFE. The average CF is 3.5.

River Station	FIS Cross Section	2013 HEC-RAS Model BFE (NGVD29) (100-Year WSEL) (ft)	2018 FIS BFE (NAVD88) (100-Year WSEL) (ft)	Conversion Factor (ft)
15913	DM	5849.07	5852.6	3.53
15241	DL	5845.13	5848.6	3.47
14376	DK	5842.2	5845.7	3.5
13898	DJ	5841.91	5845.4	3.49
13598	DI	5835.89	5839.4	3.51
12358	DH	5830.76	5834.3	3.54

Table 1: FEMA cross sections effected by project.

The original FEMA cross sections and centerline alignment from the 2013 HEC-RAS model was exported to the combined current topography. Cross sections DK thru DI were cut with the corrected topography and converted to NAVD29 by subtracting 3.5 feet from the cross section elevation points to import back into the HEC-RAS model to create a new model called Corrected Effective (CorrEff). After reviewing the CorrEff model and determining the limits of the scour mitigation project, additional cross sections were added. See Table 2 for information about the additional cross sections. This new model is called CorrEff/ACS (Additional Cross Sections). Table 3 compares the BFE's from the different plans.

		1	2	3	4	1-3	1-4	3-4	
River Station	FIS Cross Section	Effective 2013 FIS Model BFE (ft)	CorrEff Model BFE (ft)	CorrEff/ ACS Model BFE (ft)	CorrFinal Model BFE (ft)	Eff-CorrEff/ ACS (ft)	Eff-Final (ft)	CorrEff/ ACS-Final (ft)	Cross Section Geometry
15913	DM	5849.07	5849.05	5849.03	5849.03	-0.04	-0.04	0	2013
15241	DL	5845.13	5845.87	5845.6	5845.5	0.47	0.37	-0.1	2013
14715		5844.6	5845.64	5845.29	5845.15	0.69	0.55	-0.14	2013
14512				5844.89	5844.89			0	Addl
14376	DK	5842.2	5840.66	5840.66	5840.66	-1.54	-1.54	0	Corrected
14235				5838.36	5837.59			-0.77	Addl
14065		5841.91	5839.61	5838.25	5837.53	-3.66	-4.38	-0.72	Corrected
13989				5837.85	5837.49			-0.36	Addl
13898	DJ	5841.93	5839.83	5838.56	5837.83	-3.37	-4.1	-0.73	Corrected
13870	Bridge								-
13849		5841.52	5839.06	5837.72	5837.34	-3.8	-4.18	-0.38	Corrected
13782				5837.2	5837.2			0	Addl
13598	DI	5835.89	5835.34	5835.34	5835.34	-0.55	-0.55	0	2013
12887		5833.86	5833.86	5833.86	5833.86	0	0	0	2013
12358	DH	5830.76	5830.76	5830.76	5830.76	0	0	0	2013

Table 2: Base flood elevations for the three models at the FEMA cross sections affected by project.

Column 1-3 takes the difference in BFE between the Effective plan and the Corrected Effective with Additional Cross sections plan. At the upper and lower limits, cross sections 15913 and 12887, the BFE's are within 0.5 feet of each other.

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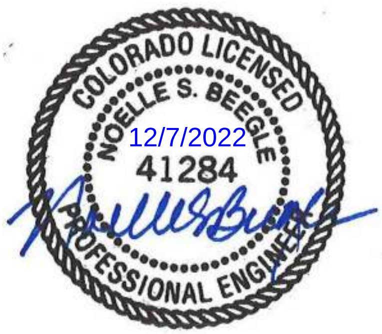
FEMA states that for a stream that has a detailed study, an effective tie-in is obtained when the base flood elevations are within 0.5 foot of the effective elevations.

With a working Corrected Effective/ACS plan, the revised channel design cross sections were entered into the Final plan. Column 3-4 shows the change in BFE from the CorrEff/ACS plan and the Final plan. The BFE drops slightly through the project cross sections but ties into the overall plan outside the project. The project does not cause a rise in the WSEL for the BFE.

Sincerely,

Noelle S. Beegle. PE, CFM



Appendix

1. As-Built Drawings-Selected Sheets

FEDERAL AID REGION NO.	DIVISION	PROJECT NO.	SHEET NO.	TOTAL SHEETS
VIII	COLORADO	BRO 0004(3)	1	

AS CONSTRUCTED

NO REVISIONS | REVISED 5/25/92 | VOID

REVISIONS

CALL UTILITY NOTIFICATION CENTER OF COLORADO
1-800-922-1987
IN METRO DENVER 534-6700
CALL 2-BUSINESS DAYS IN ADVANCE BEFORE YOU DIG, GRADE, OR EXCAVATE FOR THE MARKING OF UNDERGROUND MEMBER UTILITIES.

STATE DEPARTMENT OF HIGHWAYS
DIVISION OF HIGHWAYS—STATE OF COLORADO

PLAN AND PROFILE OF PROPOSED
FEDERAL AID PROJECT NO. BRO 0004(3)
JANITELL ROAD BRIDGE
EL PASO COUNTY

SCALES OF ORIGINAL DRAWINGS
ON PLAN 1 IN = 50 FT.
ON PROFILE { 1 IN = 50 FT. HORIZONTAL; 1 IN = 5 FT. VERTICAL
GRADE LINE ON PROFILE IS SHOWN AS GRADE OF FINISHED ROAD

INDEX OF SHEETS

New and Revised Standards

M-620-2	Field Laboratory Class 2 (1 Sheet)	5-25-88
M-210-1	Mailbox Supports (2 Sheets)	8-04-89
M-203-12	Superelevation of curves - Streets (1 Sheet)	8-16-84

TABULATION OF LENGTH & DESIGN DATA

STATION	ROADWAY LIN. FT.		MAJOR STRUCTURE LIN. FT.
1+48 BEGIN BRO 0004(3)			
	747.00		
8+95			
STRUCTURE # EPC 377-00.50A			456.40
13+51.40			
	348.60		
17+00 END BRO 0004(3)			
TOTALS	1095.60		456.40

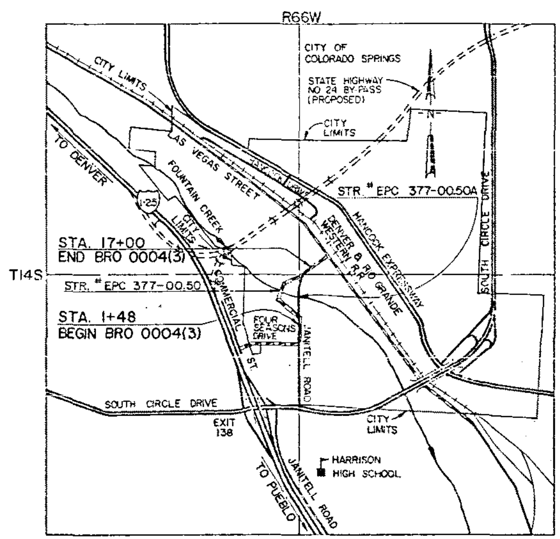
SUMMARY

	LIN. FT.		
ROADWAY MAJOR STRUCTURE	1095.60 456.40		0.208 0.086
TOTAL (NET & GROSS) LENGTH	1552.00		0.294

DESIGN DATA

MAXIMUM DEGREE OF CURVE	18°11'21"
MAXIMUM GRADE	2.1411%
MINIMUM S.S.D. HORIZONTAL	> 200
MINIMUM S.S.D. VERTICAL	> 400
MAXIMUM DESIGN SPEED	30 mph

DESIGN TRAFFIC DATA 2009
ADT = 825
DHV = 124
18K ESAL = 350,000
% TRUCKS = 15 %



VICINITY MAP
NO SCALE

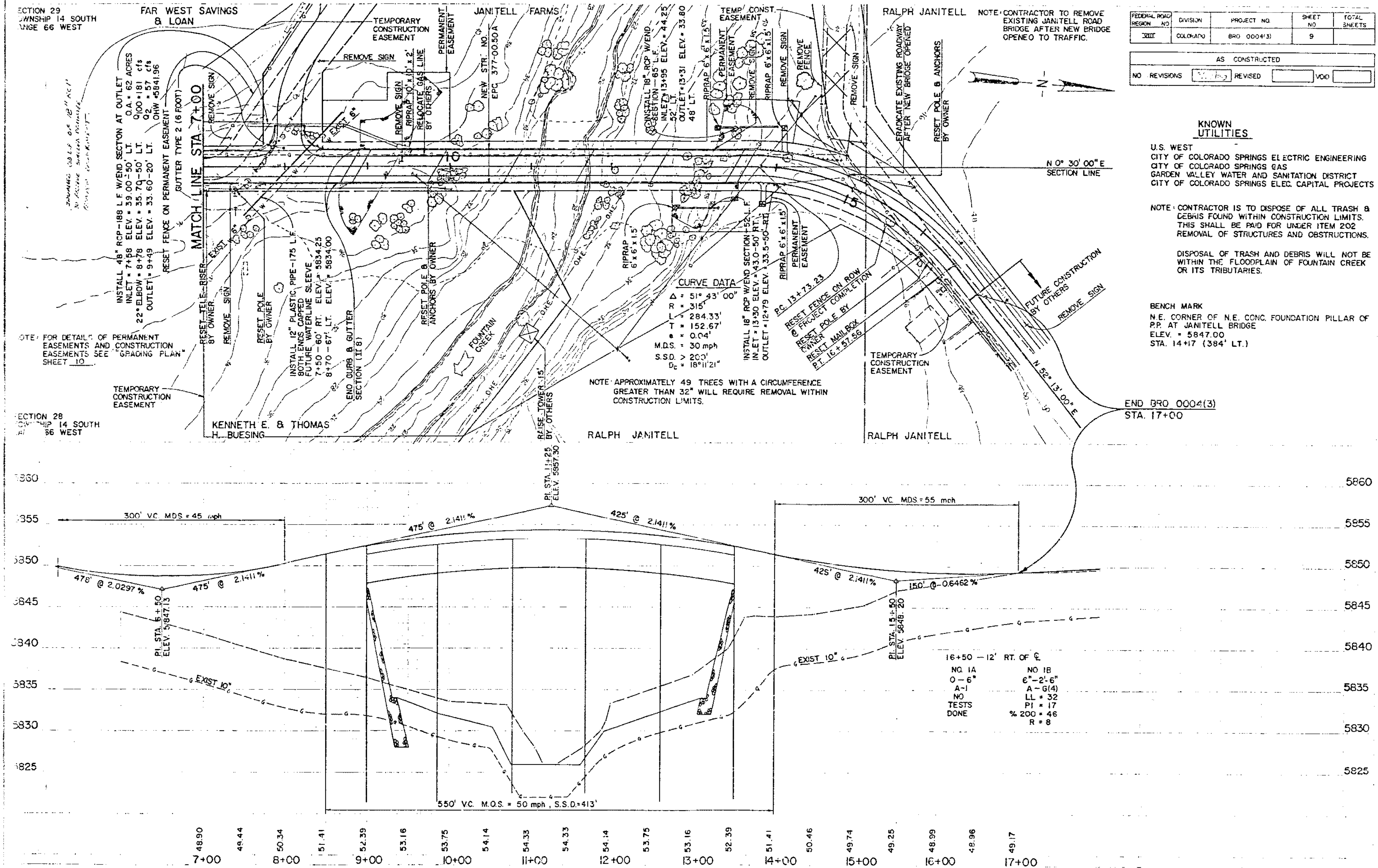
APPROVED BY ____________
DIRECTOR, DEPARTMENT OF PUBLIC WORKS

DATE ____ 19 __

AS CONSTRUCTED INFORMATION
CONTRACTOR ____
____ ENGINEER ____
(Project or Resident)
PROJECT STARTED ____
PROJECT COMPLETED ____
AS CONSTRUCTED PLANS APPROVED
____ 7/17/91
TITLE DATE

OLM / EL PASO COUNTY

Datum



FEDERAL ROAD REGION NO. VIII
DIVISION COLORADO
PROJECT NO. BRO 0004(3)
SHEET NO. 9
TOTAL SHEETS
AS CONSTRUCTED
NO REVISIONS
REVISED
VOID
KNOWN UTILITIES
U.S. WEST
CITY OF COLORADO SPRINGS ELECTRIC ENGINEERING
CITY OF COLORADO SPRINGS GAS
GARDEN VALLEY WATER AND SANITATION DISTRICT
CITY OF COLORADO SPRINGS ELEC. CAPITAL PROJECTS
NOTE: CONTRACTOR IS TO DISPOSE OF ALL TRASH & DEBRIS FOUND WITHIN CONSTRUCTION LIMITS. THIS SHALL BE PAID FOR UNDER ITEM 202 REMOVAL OF STRUCTURES AND OBSTRUCTIONS.
DISPOSAL OF TRASH AND DEBRIS WILL NOT BE WITHIN THE FLOODPLAIN OF FOUNTAIN CREEK OR ITS TRIBUTARIES.
BENCH MARK
N.E. CORNER OF N.E. CONC. FOUNDATION PILLAR OF P.P. AT JANITELL BRIDGE
ELEV. = 5847.00
STA. 14+17 (384' LT.)
NOTE: CONTRACTOR TO REMOVE EXISTING JANITELL ROAD BRIDGE AFTER NEW BRIDGE OPENED TO TRAFFIC.
N
SECTION 29
TOWNSHIP 14 SOUTH
RANGE 66 WEST
SECTION 28
TOWNSHIP 14 SOUTH
RANGE 66 WEST
FAR WEST SAVINGS & LOAN
JANITELL FARMS
RALPH JANITELL
KENNETH E. & THOMAS H. BUESING
TEMPORARY CONSTRUCTION EASEMENT
PERMANENT EASEMENT
TEMP. CONST. EASEMENT
REMOVE SIGN
REMOVE FENCE
RIPRAP 10' x 10' x 2'
RELOCATE GAS LINE BY OTHERS
NEW STR. NO. EPC 377-00.50 A
INSTALL 18" RCP W/END SECTION – 65 L.F.
INLET = 13+95 ELEV. = 44.25 52' LT.
OUTLET = 13+31 ELEV. = 33.80 48' LT.
RIPRAP 6' x 6' x 1.5'
ERADICATE EXISTING ROADWAY AFTER NEW BRIDGE OPENED
RESET POLE & ANCHORS BY OWNER
N 0° 30' 00" E SECTION LINE
INSTALL 48" RCP-188 L.F. W/END SECTION AT OUTLET
INLET = 7+58 ELEV. = 39.00-50' LT.
22° ELBOW = 8+78 ELEV. = 35.70-50' LT.
OUTLET = 9+49 ELEV. = 33.60-20' LT.
D.A. = 62 ACRES
Q_{100} = 181 cfs
Q_2 = 57 cfs
OHW = 5841.96
RESET FENCE ON PERMANENT EASEMENT
GUTTER TYPE 2 (6 FOOT)
MATCH LINE STA. 7+00
EXIST. 6"
RESET TELE. RISER BY OWNER
RESET POLE BY OWNER
INSTALL 12" PLASTIC PIPE – 175 L.F. BOTH ENDS CAPPED FUTURE WATERLINE SLEEVE
7+50 – 60' RT. ELEV. = 5834.25
8+70 – 67' LT. ELEV. = 5834.00
END CURB & GUTTER SECTION (II 8)
RESET POLE & ANCHORS BY OWNER
FOUNTAIN CREEK
RAISE TOWER 15' BY OTHERS
CURVE DATA
Δ = 51° 43' 00"
R = 315'
L = 284.33'
T = 152.67'
e = 0.04'
M.D.S. = 30 mph
S.S.D. > 200'
D_c = 18° 11' 21"
INSTALL 18" RCP W/END SECTION – 52 L.F.
INLET = 13+30 ELEV. = 43.0-50' RT.
OUTLET = 12+79 ELEV. = 33.5-50' RT.
P.C. 13+73.23
RESET FENCE ON ROW @ PROJECT COMPLETION
RESET POLE BY OWNER
RESET MAILBOX
P.T. 16+57.56
FUTURE CONSTRUCTION BY OTHERS
N 52° 13' 00" E
NOTE: FOR DETAILS OF PERMANENT EASEMENTS AND CONSTRUCTION EASEMENTS SEE "GRADING PLAN" SHEET 10.
NOTE: APPROXIMATELY 49 TREES WITH A CIRCUMFERENCE GREATER THAN 32" WILL REQUIRE REMOVAL WITHIN CONSTRUCTION LIMITS.
END BRO 0004(3)
STA. 17+00
OHE
10
15
PI. STA. 11+25
ELEV. 5857.30
300' V.C. MDS = 45 mph
475' @ 2.1411 %
425' @ 2.1411 %
300' V.C. MDS = 55 mph
478' @ 2.0297 %
475' @ 2.1411 %
425' @ 2.1411 %
150' @ -0.6462 %
PI. STA. 6+50
ELEV. 5847.13
PI. STA. 15+50
ELEV. 5848.20
EXIST. 10"
550' V.C. M.D.S. = 50 mph, S.S.D. = 413'
16+50 – 12' RT. OF ℄
NO. 1A
0 – 6"
A-1
NO TESTS DONE
NO. 1B
6"-2'-6"
A – 6(4)
LL = 32
PI = 17
% 200 = 46
R = 8
5860
5855
5850
5845
5840
5835
5830
5825
48.90
49.44
50.34
51.41
52.39
53.16
53.75
54.14
54.33
54.33
54.14
53.75
53.16
52.39
51.41
50.46
49.74
49.25
48.99
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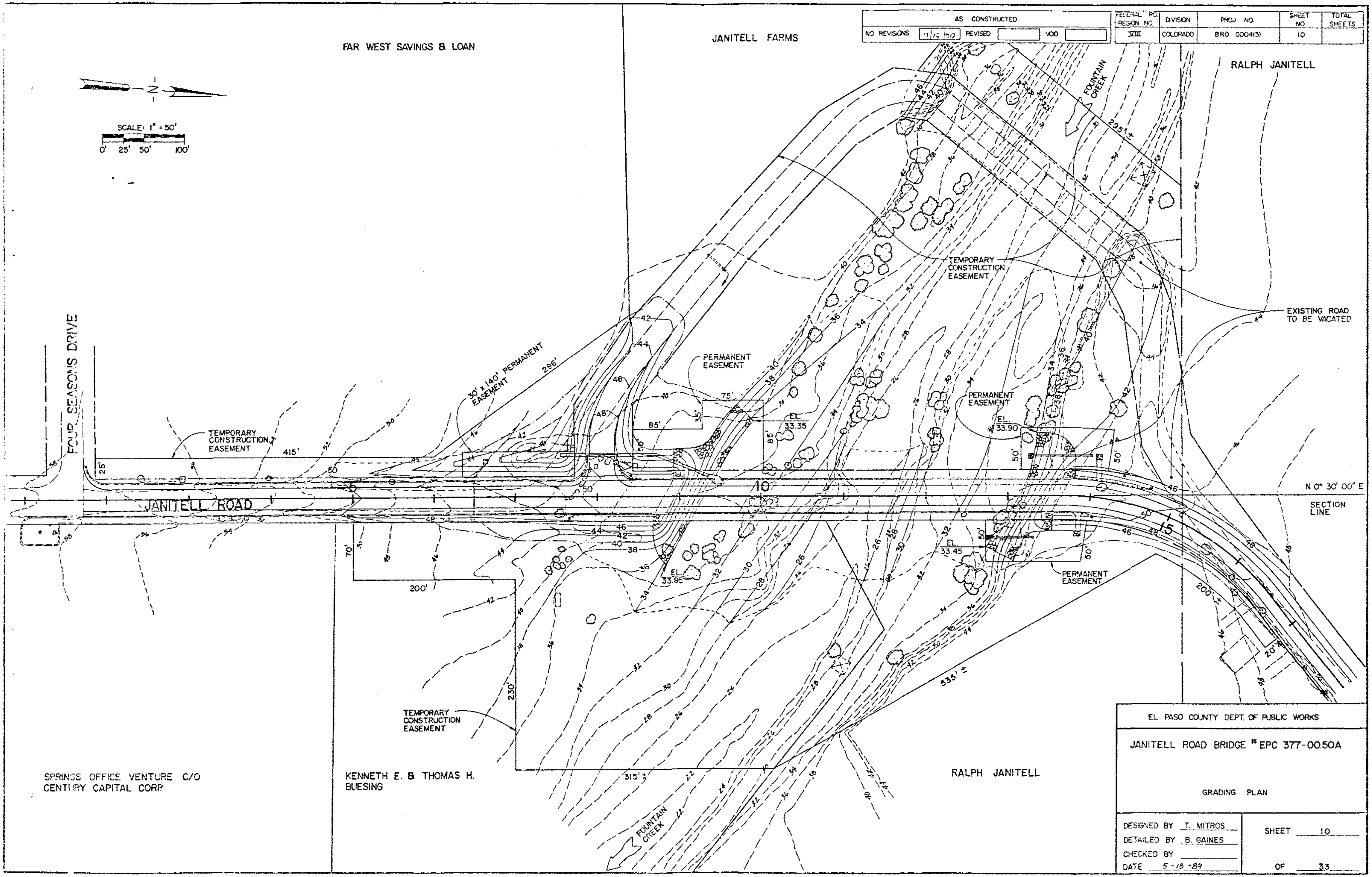
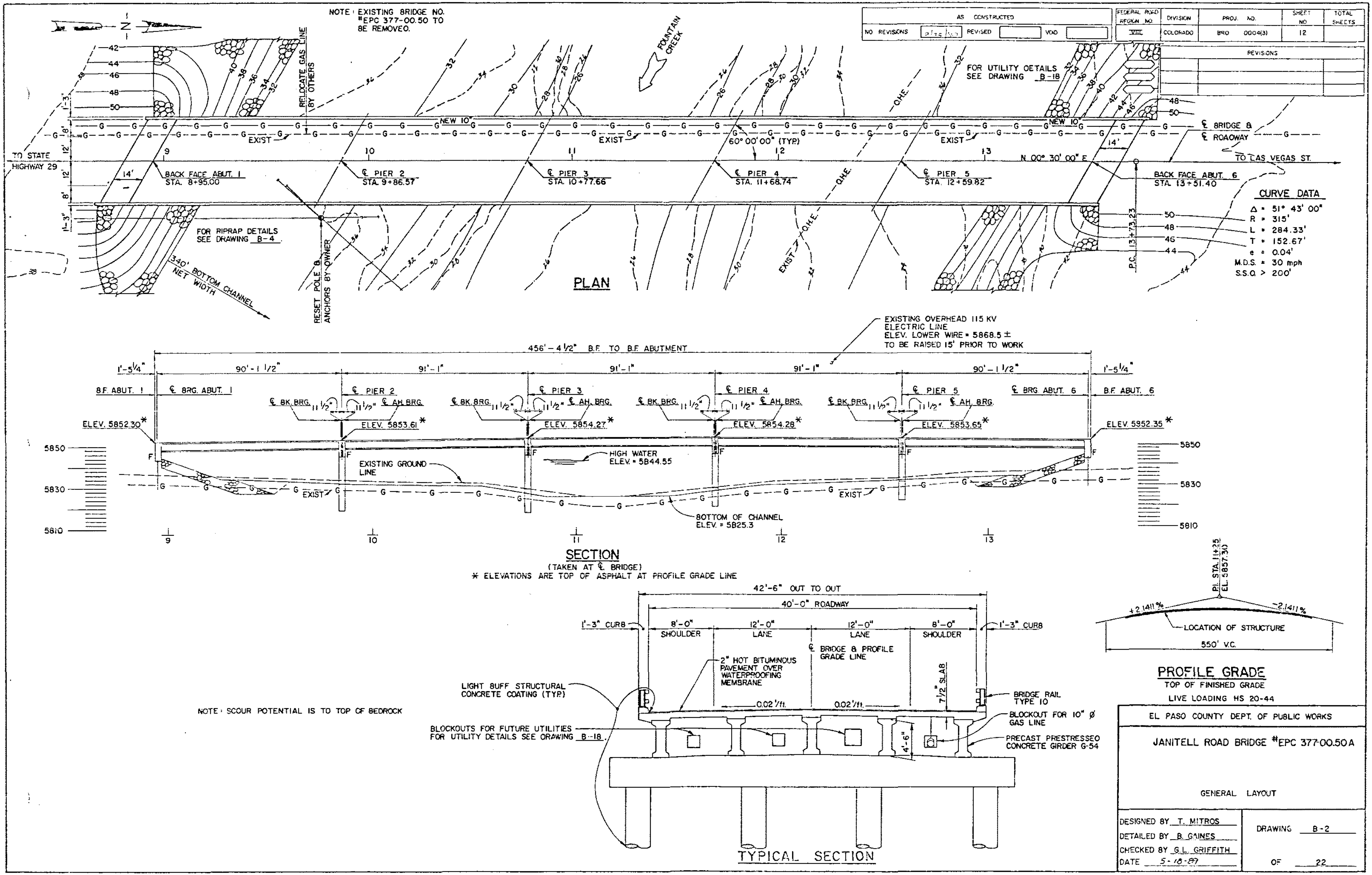
17+00


AS CONSTRUCTED
NO REVISIONS 11/15/90 REVISED VOID
FEDERAL RD. REGION NO. XIII
DIVISION COLORADO
PROJ. NO. BRO 0004(3)
SHEET NO. 10
TOTAL SHEETS
FAR WEST SAVINGS & LOAN
JANITELL FARMS
RALPH JANITELL
N
SCALE: 1" = 50'
0' 25' 50' 100'
FOUNTAIN CREEK
295' ±
TEMPORARY CONSTRUCTION EASEMENT
EXISTING ROAD TO BE VACATED
PERMANENT EASEMENT
30' x 140' PERMANENT EASEMENT
296'
75'
85'
EL. 33.35
EL. 33.90
EL. 33.45
EL. 33.95
50'
415'
25'
FOUR SEASONS DRIVE
JANITELL ROAD
N 0° 30' 00" E
SECTION LINE
200'
70'
230'
20'
535' ±
315' ±
SPRINGS OFFICE VENTURE C/O CENTURY CAPITAL CORP.
KENNETH E. & THOMAS H. BUESING
EL PASO COUNTY DEPT. OF PUBLIC WORKS
JANITELL ROAD BRIDGE # EPC 377-0050A
GRADING PLAN
DESIGNED BY T. MITROS
DETAILED BY B. GAINES
CHECKED BY
DATE 5-18-89
SHEET 10
OF 33

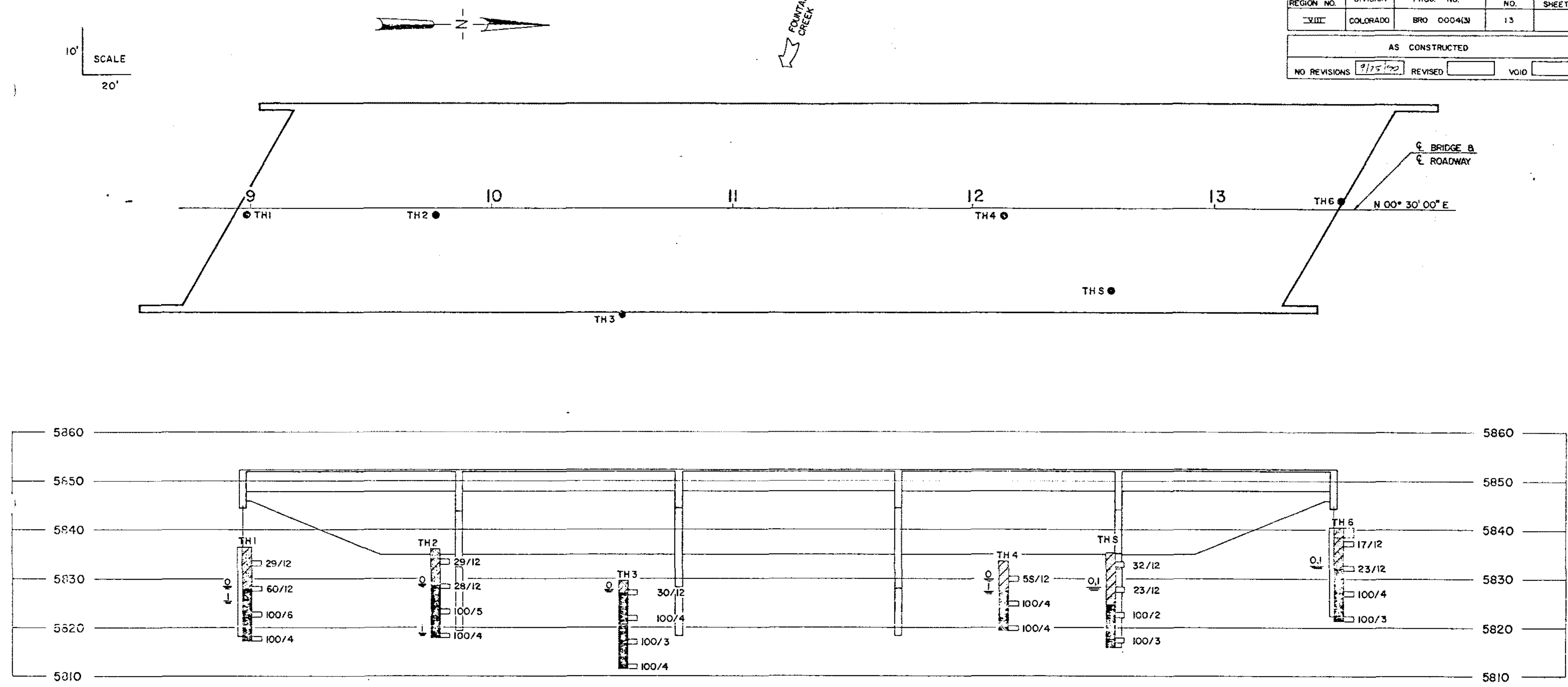


NOTE: EXISTING BRIDGE NO. #EPC 377-00.50 TO BE REMOVED.
AS CONSTRUCTED
NO REVISIONS
REVISED
VOID
FEDERAL ROAD REGION NO. VIII
DIVISION COLORADO
PROJ. NO. BRO 0004(3)
SHEET NO. 12
TOTAL SHEETS
REVISIONS
FOUNTAIN CREEK
RELOCATE GAS LINE BY OTHERS
FOR UTILITY DETAILS SEE DRAWING B-18
NEW 10"
EXIST
60° 00' 00" (TYP.)
TO STATE HIGHWAY 29
BACK FACE ABUT. 1 STA. 8+95.00
℄ PIER 2 STA. 9+86.57
℄ PIER 3 STA. 10+77.66
℄ PIER 4 STA. 11+68.74
℄ PIER 5 STA. 12+59.82
N 00° 30' 00" E
BACK FACE ABUT. 6 STA. 13+51.40
℄ BRIDGE & ℄ ROADWAY
TO LAS VEGAS ST.
P.C. 13+73.23
CURVE DATA
Δ = 51° 43' 00"
R = 315'
L = 284.33'
T = 152.67'
e = 0.04'
M.D.S. = 30 mph
S.S.O. > 200'
FOR RIPRAP DETAILS SEE DRAWING B-4
340' BOTTOM CHANNEL NET WIDTH
RESET POLE & ANCHORS BY OWNER
O.H.E.
PLAN
EXISTING OVERHEAD 115 KV ELECTRIC LINE ELEV. LOWER WIRE = 5868.5 ± TO BE RAISED 15' PRIOR TO WORK
456'-4 1/2" B.F. TO B.F. ABUTMENT
1'-5 1/4"
90'-1 1/2"
91'-1"
91'-1"
91'-1"
90'-1 1/2"
1'-5 1/4"
B.F. ABUT. 1
℄ BRG. ABUT. 1
℄ PIER 2
℄ PIER 3
℄ PIER 4
℄ PIER 5
℄ BRG ABUT. 6
B.F. ABUT. 6
℄ BK. BRG.
℄ AH. BRG.
11 1/2"
ELEV. 5852.30*
ELEV. 5853.61*
ELEV. 5854.27*
ELEV. 5854.28*
ELEV. 5853.65*
ELEV. 5952.35*
5850
5830
5810
EXISTING GROUND LINE
HIGH WATER ELEV. = 5844.55
BOTTOM OF CHANNEL ELEV. = 5825.3
SECTION
(TAKEN AT ℄ BRIDGE)
* ELEVATIONS ARE TOP OF ASPHALT AT PROFILE GRADE LINE
42'-6" OUT TO OUT
40'-0" ROADWAY
1'-3" CURB
8'-0" SHOULDER
12'-0" LANE
12'-0" LANE
8'-0" SHOULDER
℄ BRIDGE & PROFILE GRADE LINE
2" HOT BITUMINOUS PAVEMENT OVER WATERPROOFING MEMBRANE
0.02'/ft.
7 1/2" SLAB
4'-6"
LIGHT BUFF STRUCTURAL CONCRETE COATING (TYP.)
BLOCKOUTS FOR FUTURE UTILITIES FOR UTILITY DETAILS SEE DRAWING B-18
BRIDGE RAIL TYPE 10
BLOCKOUT FOR 10" Ø GAS LINE
PRECAST PRESTRESSED CONCRETE GIRDER G-54
TYPICAL SECTION
NOTE: SCOUR POTENTIAL IS TO TOP OF BEDROCK
P.I. STA. 11+25 EL. 5857.30
+2.1411%
-2.1411%
LOCATION OF STRUCTURE
550' V.C.
PROFILE GRADE
TOP OF FINISHED GRADE
LIVE LOADING HS 20-44
EL PASO COUNTY DEPT. OF PUBLIC WORKS
JANITELL ROAD BRIDGE #EPC 377-00.50A
GENERAL LAYOUT
DESIGNED BY T. MITROS
DETAILED BY B. GAINES
CHECKED BY G.L. GRIFFITH
DATE 5-18-89
DRAWING B-2
OF 22

FEDERAL ROAD REGION NO.	DIVISION	PROJ. NO.	SHEET NO.	TOTAL SHEETS
VIII	COLORADO	BRO 0004(3)	13	

AS CONSTRUCTED

NO REVISIONS 9/15/90 REVISED VOID



SUMMARY OF TEST RESULTS

SAMPLE LOCATION: HOLE	SAMPLE LOCATION: DEPTH (FEET)	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (PCF)	GRADATION: GRAVEL (%)	GRADATION: SAND (%)	PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC-ITY INDEX(%)	UNCONFINED COMPRESSIVE STRENGTH (PSF)	SOIL OR BEDROCK TYPE
1	3	3.5	110.7	35	57	8				GRAVELLY SAND
	8	15.7	115.0			98	41	18		CLAYSTONE
	12	11.8	116.0						27,360	CLAYSTONE
3	2	12.3	109.6							CLAYSTONE
4	8	12.9	117.6			98	39	16		CLAYSTONE
	13	10.2	118.0						27,960	CLAYSTONE
6	3	1.4	108.2	28	68	4				GRAVELLY SAND
	8	21.0	102.4			96	46	25		CLAYSTONE
	18	11.3	115.9						18,940	CLAYSTONE

TYPE OF MATERIAL

- SAND (SM-SC), SILTY TO CLAYEY, FINE TO COARSE GRAINED, GRAVEL-LY, WITH OCCASIONAL COBBLES AND SLIGHTLY SILTY LENSES, MEDIUM DENSE, MOIST TO WET, BROWN TO DARK BROWN.
- SAND (SP-SM), CLEAN TO SILTY, FINE TO COARSE GRAINED, GRAVEL-LY, MEDIUM TO VERY DENSE, MOIST TO WET, BROWN.
- CLAYSTONE BEDROCK, FIRM TO VERY HARD, SLIGHTLY MOIST TO MOIST, GRAY.
- DRIVE SAMPLE, 2-INCH I.D. CALIFORNIA LINER SAMPLE.
- 29/12 DRIVE SAMPLE BLOW COUNT, INDICATES THAT 29 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE CALIFORNIA SAMPLER 12 INCHES.
- DISTURBED BULK SAMPLE.
- INDICATES P.V.C. PIPE INSTALLED IN HOLE TO DEPTH SHOWN.
- 0.1 DEPTH TO WATER LEVEL AND NUMBER OF DAYS AFTER DRILLING MEASUREMENT WAS TAKEN.

LEGEND

1. TEST HOLES WERE DRILLED ON SEPTEMBER 8, 1967 WITH A 4-INCH DIAMETER CONTINUOUS FLIGHT POWER AUGER.
2. LOCATIONS OF TEST HOLES WERE MEASURED APPROXIMATELY BY PACING FROM FEATURES SHOWN ON THE SITE PLAN PROVIDED.
3. ELEVATIONS OF TEST HOLES WERE OBTAINED FROM A PROFILE SHEET PROVIDED BY EL PASO COUNTY ENGINEERING DEPARTMENT.
4. THE TEST HOLES LOCATIONS AND ELEVATIONS SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.
5. THE LINES BETWEEN MATERIALS SHOWN ON THE TEST HOLE LOGS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN MATERIAL TYPES AND THE TRANSITIONS MAY BE GRADUAL.
6. WATER LEVEL READINGS SHOWN ON THE LOGS WERE MADE AT THE TIME AND UNDER CONDITIONS INDICATED. FLUCTUATIONS IN THE WATER LEVEL MAY OCCUR WITH TIME.

EL PASO COUNTY DEPT. OF PUBLIC WORKS

JANITELL ROAD BRIDGE #EPC 377-00.50A

ENGINEERING GEOLOGY

SOIL TESTS CHEN & ASSOC.
DETAILED BY B. GAINES
CHECKED BY T. MITROS
DATE 5-18-89

DRAWING B-3

OF 22

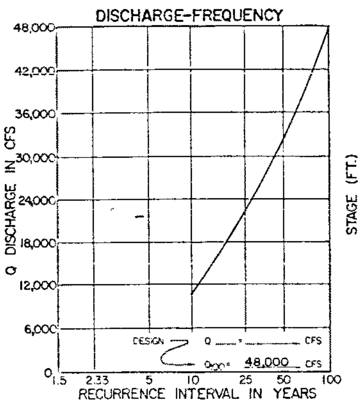
BRIDGE HYDRAULIC INFORMATION

FEDERAL ROAD REGION NO.	DIVISION	PROJECT NO.	SHEET NO.
VIII	COLORADO	BRO 0004(3)	14

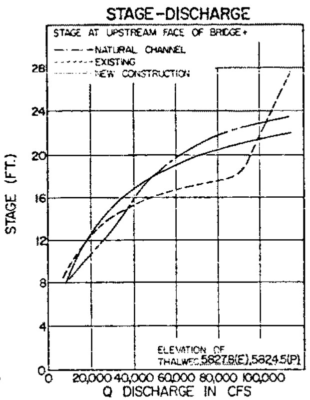
AS CONSTRUCTED

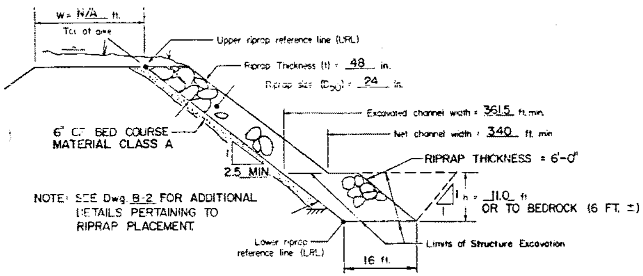
NO REVISIONS 9/3/1982 REVISED VOID

DISCHARGE-FREQUENCY



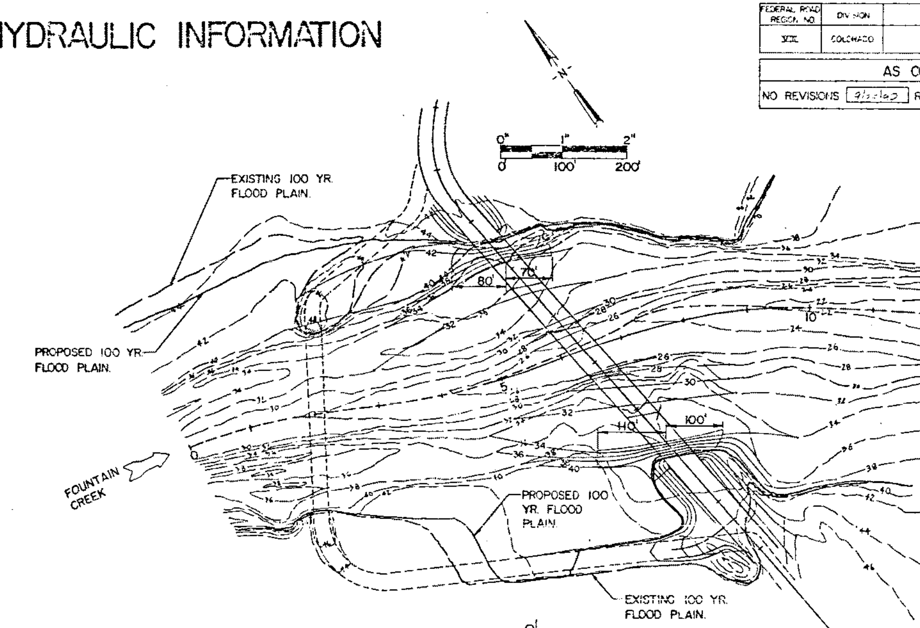
STAGE-DISCHARGE





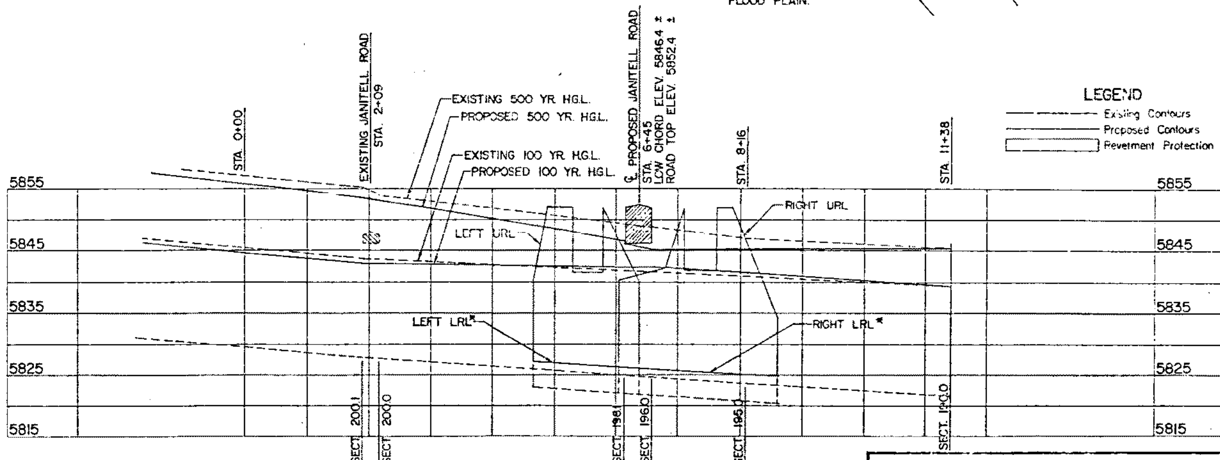
NOTE: SEE Dwg. B-2 FOR ADDITIONAL DETAILS PERTAINING TO RIPRAP PLACEMENT.

TYPICAL RIPRAP DETAIL



SEE SHEET 10 FOR ADDITIONAL DETAILS PERTAINING TO RIPRAP PLACEMENT.

LEGEND
- Existing Contours
- Proposed Contours
- Revetment Protection



PROFILE OF WATER SURFACE & RIPRAP REFERENCE LINES

* LEFT & RIGHT LOWER RIPRAP REFERENCE LINE SHALL BE DEPENDENT UPON THE ELEVATION OF CLAYSTONE BEDROCK.

Drainage Area 420 Sq. Mi.

CHANNEL DESCRIPTION

Bottom Material – Cohesive☐ Non Cohesive☒
Bottom Material Size – Clay☐ Silt☒ Sand☒ Gravel☒ Cobbles☐ Other SLIGHTLY SILTY, GRAVELLY SAND
Stream Form – Straight☒ Meandering☐ Braided☐
Mannings "n" for Design – Channel 0.030 Overbank 0.040
Debris – Brush☒ Trees/Logs☐ Ice☐ Other☐

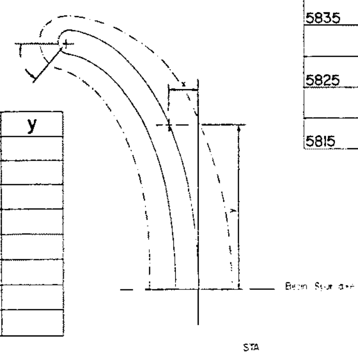
COMPARISON OF HYDRAULICS

	Velocity	Freeboard	Max. Backwater
Natural Channel	19.0 fps.	ft.	ft.
Existing	17.9 fps.	2.3 ft.	2.8 ft.
Proposed	12.3 fps.	3.8 MIN. ft.	3.0 ft.

*AT PROPOSED BRIDGE LOCATION DURING DESIGN DISCHARGE

COMMENTS

x	y



TYPICAL SPUR DIKE DETAIL

EL PASO COUNTY DEPT. OF PUBLIC WORKS

JANITELL RD. BRIDGE #EPC 377-0050A

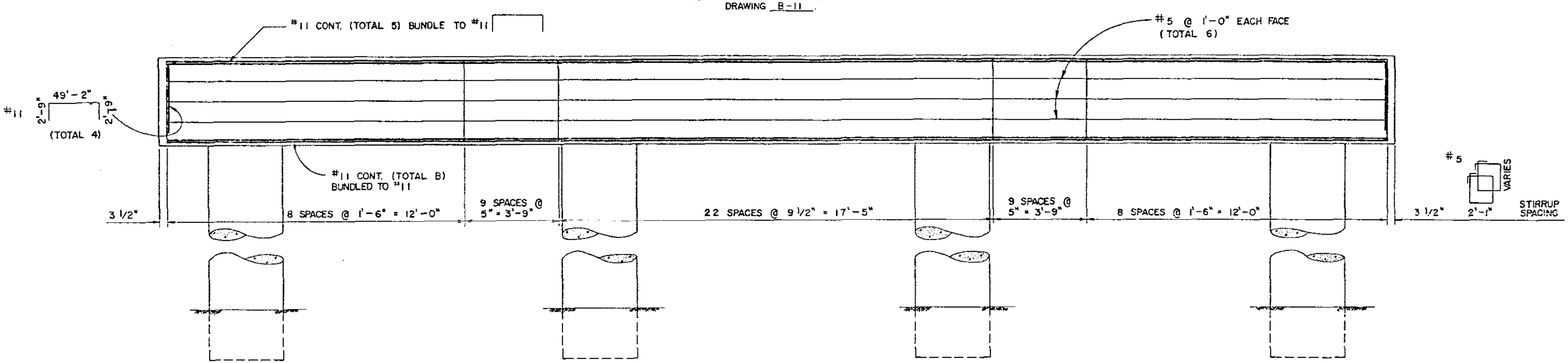
BRIDGE HYDRAULIC INFORMATION

ACROSS FOUNTAIN CREEK	STRUCTURE NUMBERS EPC 377-00.50A
DESIGNED BY A.B. MORRICE	DATE 5-18-89
DETAILED BY P.D. KERSEY	DRAWING B-4
CHECKED BY G.L. GRIFFITH	OF 22

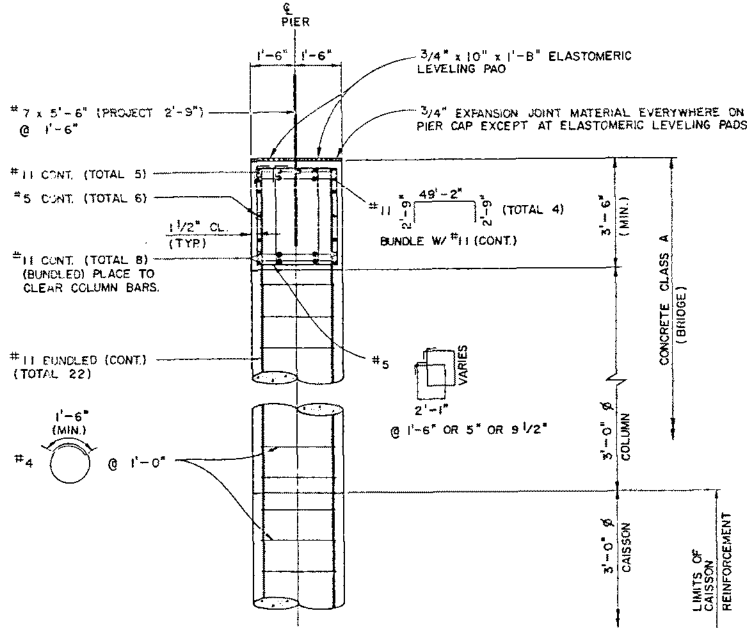
AS CONSTRUCTED						FEDERAL ROAD REGION NO.	DIVISION	PROJ. NO.	SHEET NO.	TOTAL SHEETS
NO REVISIONS	9/25/90	REVISED		VOID		VIII	COLORADO	BRO 0004(3)	22	

REVISIONS

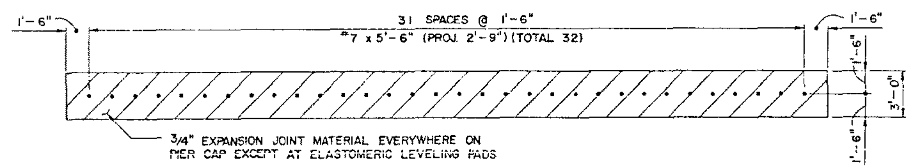
NOTE: FOR ELEVATIONS OF CAP SEE DRAWING B-11.



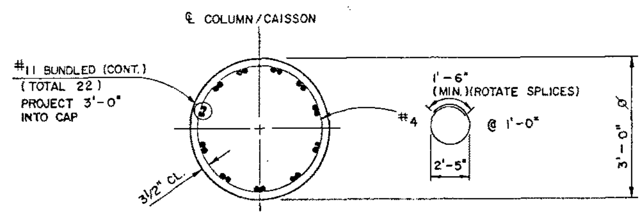
CAP REINFORCING



TYPICAL SECTION



DOWELL DETAIL-PIER



SECTION THRU COLUMN/CAISSON

NOTE:
COLUMN/CAISSON BARS SHALL BE CUT TO LENGTH AFTER PLACEMENT. IF SPLICES ARE REQUIRED, THEY SHALL BE STAGGERED 6'-10" MIN. LAP.

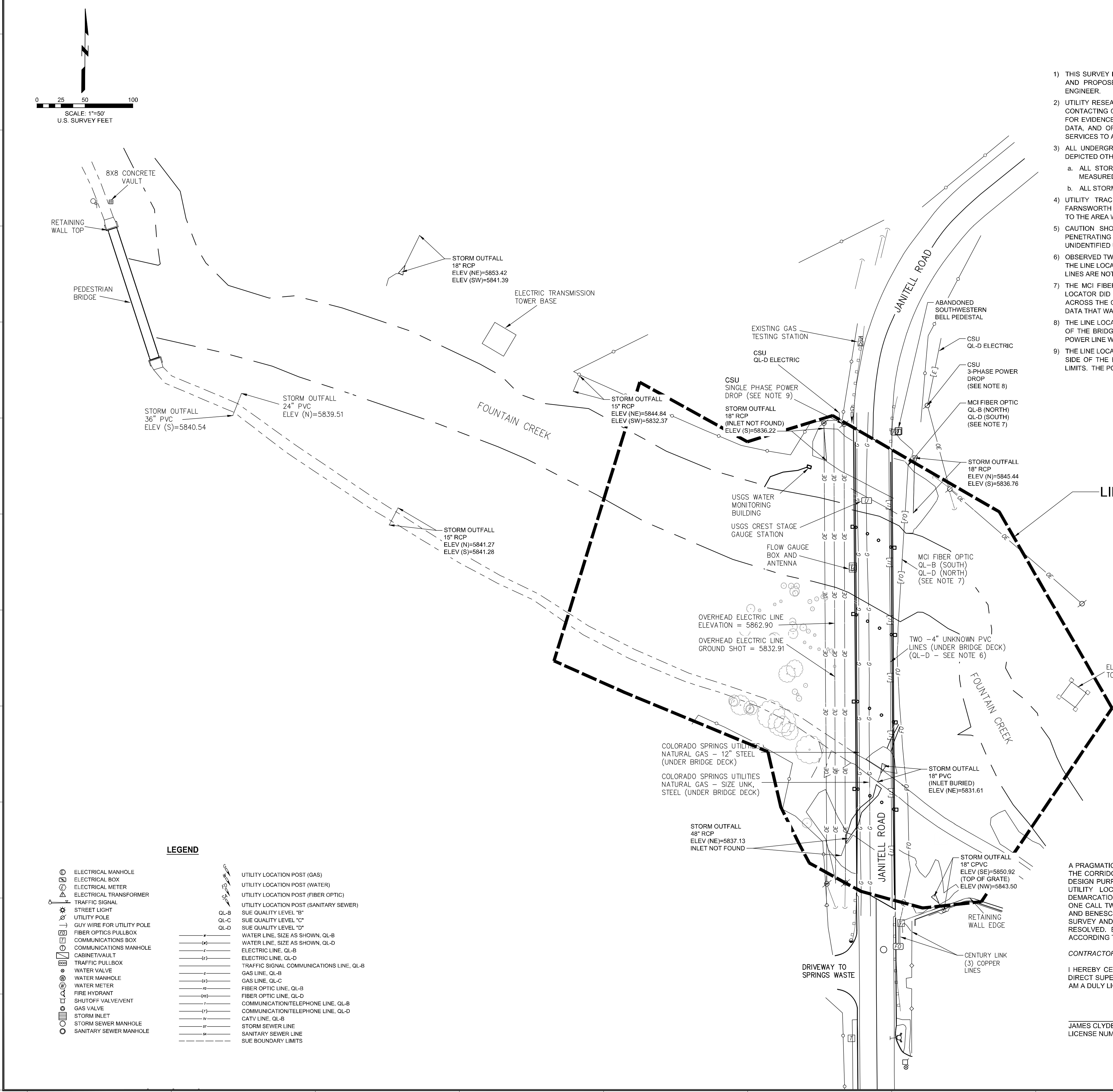
EL PASO COUNTY DEPT. OF PUBLIC WORKS

JANITELL ROAD BRIDGE #EPC 377-00.50A

PIER DETAILS

DESIGNED BY T. MITROS	DRAWING B-12
DETAILED BY B. GAINES	
CHECKED BY G.L. GRIFFITH	
DATE 5-18-89	OF 22

2. Subsurface Utility Engineering Exhibit



SUBSURFACE UTILITY ENGINEERING (SUE) NOTES

1) THIS SURVEY INCLUDES INFORMATION TO SUPPORT THE CREATION OF A DESIGN PLAN DEPICTING EXISTING UTILITIES AND PROPOSED EXCAVATIONS PREPARED UNDER THE RESPONSIBLE CHARGE OF A LICENSED PROFESSIONAL ENGINEER.
2) UTILITY RESEARCH WAS PERFORMED PER ASCE 38-02 SUE QUALITY LEVEL D (QL-D) SPECIFICATIONS. THIS INCLUDED CONTACTING COLORADO 811 FOR A LIST OF KNOWN UTILITIES IN THE AREA, MAKING A SURFACE REVIEW OF THE SITE FOR EVIDENCE OF OTHER UTILITIES, AND CONTACTING ANY APPARENT UTILITY OWNERS. UTILITY SYSTEM MAPS, GIS DATA, AND ORAL EVIDENCE OF UTILITIES WERE OBTAINED FOR ALL FOUND EVIDENCE OF UTILITIES AND KNOWN SERVICES TO ADJACENT PROPERTIES.
3) ALL UNDERGROUND UTILITY LINES, AS NOTED IN THE LEGEND, ARE QUALITY LEVEL B (QL-B) EXCEPT FOR THOSE DEPICTED OTHERWISE AND AS NOTED BELOW:
   a. ALL STORM AND SANITARY SEWERS ARE QUALITY LEVEL A (QL-A) AT MANHOLES/OUTFALLS THAT COULD BE MEASURED WITHOUT ENTRY AND HAVE NOTED INVERT ELEVATIONS.
   b. ALL STORM AND SANITARY SEWER LINES BETWEEN MANHOLES/OUTFALLS ARE QUALITY LEVEL C (QL C).
4) UTILITY TRACING WAS PERFORMED BY KINETIC ENERGY SERVICES, LLC. FIELD SURVEYING PERFORMED BY FARNSWORTH GROUP IN MARCH AND APRIL, 2021. UTILITY DEPICTIONS AND QUALITY LEVEL DESIGNATIONS APPLY ONLY TO THE AREA WITHIN THE SUE BOUNDARY.
5) CAUTION SHOULD BE USED BY THE CONTRACTOR WHEN EXCAVATING WITHIN THE PROJECT AREA. GROUND PENETRATING RADAR AND OTHER DETECTION METHODS WERE USED TO FIND UNIDENTIFIED UTILITIES. NO UNIDENTIFIED UTILITIES WERE LOCATED WITH THESE METHODS.
6) OBSERVED TWO - 4" PVC LINES RUNNING ALONG THE LENGTH OF THE BRIDGE AND DIRECTLY UNDER THE BRIDGE DECK. THE LINE LOCATOR WAS UNABLE TO LOCATE THE TWO PVC LINES DUE TO A LACK OF LOCATE METHOD. THESE TWO PVC LINES ARE NOTED AS QL-D AND ARE MARKED IN THE APPROXIMATE LOCATION BASED ON FIELD OBSERVATIONS.
7) THE MCI FIBER OPTIC LINE WAS LOCATED RUNNING PARALLEL ALONG THE EAST SIDE OF THE BRIDGE. THE LINE LOCATOR DID NOT LOCATE THE MCI LINE ACROSS THE CREEK. A PORTION OF THE MCI FIBER OPTIC LINE RUNNING ACROSS THE CREEK IS NOTED AS QL-D AND IS MARKED IN THE APPROXIMATE LOCATION BASED ON THE QL-B SURVEY DATA THAT WAS PROCESSED.
8) THE LINE LOCATOR LOCATED A CSU 3-PHASE POWER LINE FROM A POWER DROP LOCATED AT THE NORTHEAST SIDE OF THE BRIDGE. THE 3-PHASE LINE WAS LOCATED RUNNING NORTH AND OUT OF THE SUE BOUNDARY LIMITS. THE POWER LINE WAS NOT SURVEYED DUE TO FENCED AREA RESTRICTIONS.
9) THE LINE LOCATOR LOCATED A CSU SINGLE PHASE POWER LINE FROM A POWER DROP LOCATED AT THE NORTHWEST SIDE OF THE BRIDGE. THE SINGLE PHASE LINE WAS LOCATED RUNNING NORTH AND OUT OF THE SUE BOUNDARY LIMITS. THE POWER LINE WAS NOT SURVEYED DUE TO FENCED AREA RESTRICTIONS.

SUBSURFACE UTILITY ENGINEER'S CERTIFICATE

A PRAGMATIC EFFORT HAS BEEN MADE TO SYSTEMATICALLY DESIGNATE AND DEPICT BURIED UTILITIES WITHIN THE CORRIDOR TO THE EXTENT PRACTICAL FOR THE AUTHORIZED PROJECT BUDGET. THESE PLANS ARE FOR DESIGN PURPOSES ONLY AND REFLECT SUBSURFACE UTILITY CONDITIONS AT THE TIME SURVEYED. EXISTING UTILITY LOCATIONS DEPICTED ON THE PLANS DO NOT SUPERSEDE COLORADO STATE ONE CALL DEMARCATIONS OF BURIED UTILITIES OR RELIEVE THE CONTRACTOR FROM THE LEGAL REQUIREMENT TO CALL ONE CALL TWO WORKING DAYS PRIOR TO CONSTRUCTION. FARNSWORTH GROUP, KINETIC ENERGY SYSTEMS, AND BENESCH SHOULD BE NOTIFIED OF ANY DISCREPANCIES BETWEEN THE UTILITY DESIGNATING / LOCATING SURVEY AND ONE CALL MARKINGS, AND THE CONTRACTOR SHALL USE CAUTION UNTIL DISCREPANCIES ARE RESOLVED. EXISTING UTILITIES ARE LOCATED TO A QUALITY LEVEL B, EXCEPT WHERE NOTED HEREIN, ACCORDING TO ASCE 38-02.

CONTRACTOR SHALL CALL THE UTILITY NOTIFICATION SERVICE BEFORE EXCAVATING AS REQUIRED BY LAW.

I HEREBY CERTIFY THAT THIS UTILITY SUBSURFACE UTILITY ENGINEERING PLAN WAS PREPARED UNDER MY DIRECT SUPERVISION AND IT ACCURATELY DEPICTS THE KNOWN SUBSURFACE UTILITY CONDITIONS AND THAT I AM A DULY LICENSED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF COLORADO.

JAMES CLYDE CUNDALL, PE
LICENSE NUMBER 41831

COLORADO REGISTERED PROFESSIONAL ENGINEER
JAMES CLYDE CUNDALL
41831
07/29/21

Farnsworth GROUP
5775 MARK DABLING BLVD., SUITE 190
COLORADO SPRINGS, COLORADO 80919
(719) 590-9194 / info@f-w.com
www.f-w.com
Engineers | Architects | Surveyors | Scientists

ISSUE:

#	DATE:	DESCRIPTION:
	7/22/2021	UTILITY LOCATES/SUE EXHIBIT

PRELIMINARY
NOT FOR CONSTRUCTION

PROJECT:
ALFRED BENESCH & COMPANY

JANITELL ROAD BRIDGE IMPROVEMENTS

DATE:	7/28/2021
DESIGNED:	LAW
DRAWN:	CDO
REVIEWED:	GDK
FIELD BOOK NO.:	N/A

SHEET TITLE:
SUE EXHIBIT

JANITELL ROAD BRIDGE IMPROVEMENTS

SHEET NUMBER:
1
1

PROJECT NO.: 0210332.00

3. Geotechnical Investigation: Geotechnical Report

CTL | THOMPSON
INCORPORATED

GEOTECHNICAL INVESTIGATION
JANITELL BRIDGE
FOUNTAIN CREEK
EL PASO COUNTY, COLORADO

Prepared for:

Alfred Benesch & Company
7979 E. Tufts Avenue, Suite 800
Denver, Colorado 80237

Attention: Noelle Beegle

CTL|T Project No. CS19402-125

September 28, 2021

TABLE OF CONTENTS

SCOPE

This report presents the results of our Geotechnical Investigation for a Scour Evaluation of the Janitell Road Bridge over Fountain Creek in Colorado Springs, Colorado. The purpose of this investigation was to evaluate subsurface conditions at the site in order to develop geotechnical information to aid in the evaluation of remediation measures for addressing ongoing bridge foundation scour. This report summarizes the results of our field and laboratory investigations and presents discussions and parameters for evaluating remediation measures and the lateral capacity analysis of the bridge piers. We believe the investigation was completed in general accordance with our proposal (CTL|T Proposal No. CS-21-0021) dated February 8, 2021, 2008. Evaluation of the subsurface conditions for support of future structures was beyond the scope of this investigation.

The report was prepared based upon conditions disclosed by our exploratory borings, results of laboratory tests, engineering analyses, and our experience. The following section summarizes the report. More detailed descriptions of subsurface conditions and laboratory test results are presented in the report.

SUMMARY

1. The surficial conditions encountered in our borings drilled within the creek consisted of about up to 6 to 7 feet of slightly clayey to very clayey sand and gravel overlying shale bedrock. The boring drilled on the north bank of the creek, encountered about 16 feet of similar soils over the shale bedrock.
2. Groundwater occurred at depths of 2 to 8 feet below the ground surface in the three borings located below the bridge and was not encountered in the boring located on the north bank.
3. Scour protection of the piers and northern bank can be accomplished through the proposed cutoff wall around the piers and/or drop structure downstream of the bridge. The drop structure is expected to be more effective in reducing further scour of the creek channel.

SITE CONDITIONS

The investigated site is located where Janitell Road crosses over Fountain Creek in El Paso County, within the southern portions of Colorado Springs, Colorado. The general location of the site is shown in Fig. 1. Fountain Creek flows through the site in a generally west to east direction, although the general trend of the creek is to the south. Janitell Road crosses the site on a multi-span, pre-stressed concrete girder bridge, extending approximately 453 feet oriented generally north and south. The bridge was constructed in 1990. The elevation of the bridge deck is approximately 25 to 30 feet above the creek.

The most recent inspection occurred in March 2018 and the bridge was reported as being in good overall condition. Evaluation of channel protection indicated the bank is beginning to slump, river control devices and embankment protection have widespread minor damage, minor stream bed movement is evident, and debris is restricting the channel slightly.

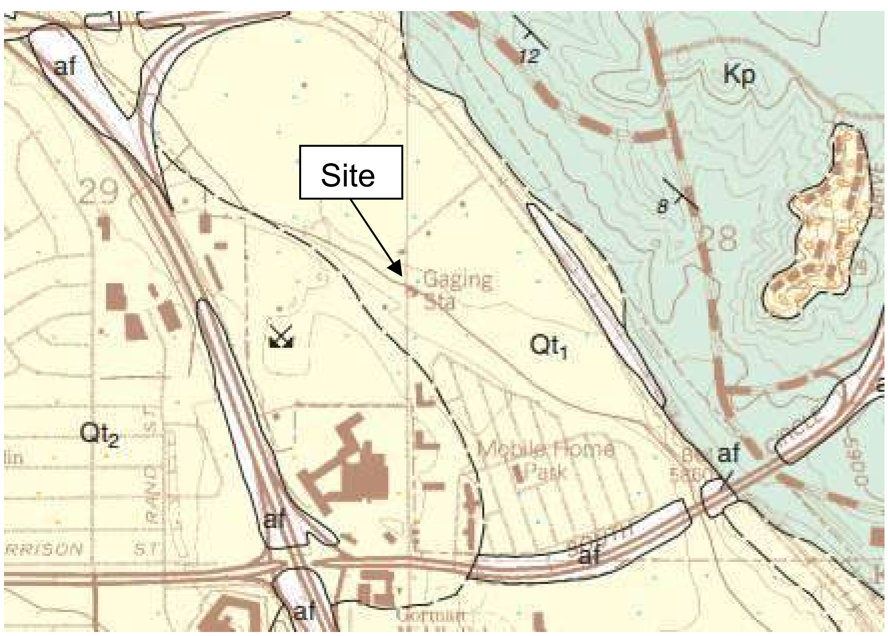
The bank on the north side of Fountain Creek in the vicinity of the bridge is about 30 feet above the current creek bed, with shale exposed in the lower few feet. Upstream and through the bridge, the south bank is about 5 feet in height and is comprised of alluvial sand and gravel deposits. The west bank eventually rises to a gravel trail before rising at the western abutment.

A gravel trail is present on the south side of the creek. The bridge abutment slopes have been armored with riprap, and some of the slopes along the north side of the creek appear to have concrete rubble to help protect against erosion. The creek channel had about 1.5 feet of water flowing during the site visits of our investigation. Appendix A provides some pictures of the bridge and surrounding area.

GEOLOGY

Previous and current observations indicate shale bedrock is exposed along most of the stream bed in this area of Fountain Creek or is within 2 to 4 feet of the stream bed. There is an intermittent thin layer of sand and gravel with scattered cobbles along the bottom of the stream over the shale bedrock. Spring Creek enters the channel to the northwest of the site at about a 45-degree angle with the stream flow. The creek flows generally straight from the confluence until just east of the Janitell Road Bridge. Southeast of the Janitell Road Bridge the creek appears to be forced to the west about 80 feet, by encroachment and narrowing of the valley. The encroachment has resulted in the development of a gravel and cobble bar on the west bank upstream and below the Janitell Road bridge. A secondary gravel bar has developed on the east bank, downstream of the sharp bend in the creek.

Geology maps of the vicinity indicate the local bedrock is Pierre Shale, which is overlain by recent alluvial deposits in the creek bed. Alluvial terrace deposits are located on the north and south banks.



Geologic Map of The Colorado Springs Quadrangle, El Paso County, Colorado

The Pierre Shale is a late Cretaceous age gray to dark gray marine shale with interbeds of siltstone and claystone. The dip of the bedrock is mapped at 8 to 12 degrees, sloping down to the southwest. The Pierre Shale locally varies from moderately hard to very soft rock depending on the extent of weathering. As the shale weathers it varies through a continuum from shale to claystone to clay.

The terrace alluvium generally consists of clayey to silty sand and gravel, with scattered sandy clay layers. The tops of the terrace are generally within about 10 to 15 feet of the current stream elevation. The surficial soils have been disturbed in the area and may contain some fill.

SUBSURFACE CONDITIONS

Subsurface conditions at the bridges were investigated by drilling four exploratory borings at the approximate locations shown in Fig. 1. The borings were drilled to depths of 20 and 25 feet below the existing ground surface. The borings were drilled using a 4-inch diameter, continuous-flight, truck-mounted power auger. The drilling operations were supervised by our field representative who logged the conditions found and obtained samples. Graphical logs of the conditions encountered in the borings, as well as the results of field penetration resistance tests, and some laboratory test data are presented in Fig. 2. Laboratory test results are presented in Figs. 3 and 4.

The three borings drilled below the bridge encountered 6 to 7 feet of slightly clayey to very clayey sand and gravel overlying shale bedrock. The surficial soils may have been fill adjacent to the trail (TH-1) or were deposited as part of the gravel bar (TH-2 and TH-3). Cobble, up to potentially small boulder material was observed as part of the gravel bar. Larger particles, over about 1.5 to 2 inches, would have been excluded from the samples.

The boring on the east bank encountered 16 feet of clayey sand. The upper portion of the soils at the east bank were likely fill; however, the presence of fill was difficult to discern in the samples. The lower 1-foot of the soil was gravelly, prior to encountering shale bedrock. Additional aspects of the soils and bedrock encountered are discussed in the following paragraphs.

Sands and Gravels

The surficial deposits are part of the alluvial terrace deposits or fills likely derived from the same. The soils are subject to erosion and deposition based on flows in the creek. A relatively flat sand/gravel bar has been forming on the south side of the creek, at the bridge location, and extends to just north of bridge pier P-4. The sands and gravels are generally not present in the main creek channel.

Bedrock

Shale bedrock was encountered in all four borings. The upper 1-foot of the shale is expected to be weathered to what is locally referred to as claystone. In this state, claystone bedding is generally not visible. The shale was generally laminated to thinly bedded, fissile, and medium to dark gray in color. We have previously tested the shale for durability using the slake durability, soundness, and LA abrasion tests, each indicating the shale is not durable. The shale has been eroded in the creek channel, near bridge pier P-5, to about 3 to 5 feet below the bedrock surface at the north bank.

Groundwater

At the time of drilling, water was measured at 2 to 8 feet below the ground surface in borings TH-1 through TH-3. The groundwater levels are expected to fluctuate with flow changes in the creek.

REMEDIATION MEASURES

We understand scour remediation measures will likely consist of a drop structure downstream of the bridge and/or a cutoff wall extending around the south side of pier P-5. Based on observations of the north bank, there has been erosion into the shale bedrock. As such, a smooth/linear change in the bedrock surface is not expected between TH-3 and TH-4. Bedrock elevations along pier P-5 are estimated to range from about 5825 to 5826, while the bedrock surface along the north bank ranges from about 5829 to 5830. This change in elevation from the pier to the bank occurs abruptly.

A cutoff wall near the piers would need to extend into the bedrock to avoid undercutting the wall. TH-3 indicated the lowest measured bedrock elevation at about 5824. Additional scour may have occurred at locations within the creek bed resulting in local variations of the bedrock surface. This bedrock elevation can also be assumed if there is scour concern for the piers adjacent to TH-3. With the poor durability of the shale, it is expected that additional scour will occur unless measures are taken to slow the water in the vicinity of the bridge. This could lead to undercutting of the proposed wall.

At the proposed drop structure, the bedrock is expected to be at a similar elevation (5824) to TH-3 near the existing channel, with the same caveat concerning additional scour in the creek bed. The bedrock appears to be exposed on the northern bank where the elevation increases quickly to about 5839. The bedrock surface is expected to gradually rise towards the south where it was encountered at an elevation of about 5828.5 at boring TH-1.

Cutoff walls such as sheet piles, if used for the wall itself or part of the drop structure, are expected to need pre-excavation to allow installation into the bedrock. Excavation into the shale, for trenches or keyways, can be completed with conventional heavy-duty equipment, although rock teeth may be required to expedite the work.

Dewatering during construction is expected. Most of the dewatering effort is expected to be accomplished through diversion of the surficial flows. Seepage is expected through the surficial granular soils; however, the bedrock is expected to be relatively impermeable, and limited flow is expected through the bedrock with most water coming through fissures in the rock. Working during a cold and dry time of the year, such as late fall or early winter, when there is less water flow in the creek may be appropriate.

LATERALLY-LOADED PIERS

Lateral load analysis of piers can be performed with the software analysis package LPILE by Ensoft, Inc. We believe this method of analysis is typically appropriate for piers with a pier length to diameter ratio of seven or greater. Suggested criteria for LPILE analysis are presented in the following Table. We have provided values for the sands and gravels, based on the materials being relatively rounded due to the action of the stream. Clay values may be appropriate where new drop structures slow the water around the piers allowing for deposition of finer particles. It may be that a combination of materials will be deposited so we recommend determining the more conservative analysis between the two materials. Other models, such as “Silt” may be appropriate; however, without knowing what mixture of materials may be deposited, it becomes more difficult to determine strengths using a combination of cohesion and friction angles for the unknown materials.

SOIL INPUT DATA FOR "LPILE"

Soil Type	Sands and Gravels	Natural Clay	Shale Bedrock
Recommended p-y Curve Model	Sand	Soft Clay	Weak Rock
Density (pci)	0.063	0.060	0.075
Friction Angle (degrees)	25	-	-
k_s (pci)	20	-	-
k - Static (pci)	-	1000	-
k - Cyclic (pci)	-	-	-
E50	-	0.02	-
c (psi)	-	2	-
Compressive Strength (psi)	-	-	300
Young's Modulus, E (psi)	-	-	0.5×10^6
K_{rm}	-	-	0.0001
RQD (%)	-	-	70

Other analysis procedures require input of a horizontal modulus of subgrade reaction (K_h). We believe the following formulas listed in the table below are appropriate for calculating horizontal modulus of subgrade reaction (K_h) values.

HORIZONTAL MODULUS OF SUBGRADE REACTION

Soil Type	Sands	Clays	Bedrock
Modulus of Subgrade Reaction, K_h (tcf)	$K_h = \frac{15 \times Z}{d}$	$K_h = \frac{20}{d}$	$K_h = \frac{300}{d}$

Where z = depth (ft); d = pier diameter (ft).

Closely-Spaced Pier Reduction Factors

For axial loading, no reduction is needed for a minimum spacing of three diameters (center to center). At one diameter (piers touching), the skin friction reduction factor for both piers would be 0.5. End pressure values would not be reduced provided the bases of the piers are at similar elevations. Interpolation can be used between one and three diameters.

For lateral loading, no reduction is needed for piers in-line with the direction of lateral loads with a minimum spacing of six diameters (center-to-center) based upon the larger pier. If a closer spacing is required, the modulus of subgrade reaction for initial and trailing piers should be reduced. At a spacing of three diameters, the effective modulus of subgrade reaction of the first pier can be estimated by multiplying the given modulus by 0.6; for trailing piers in a line at three-diameter spacing, the factor is 0.4. Linear interpolation can be used for spacing between three and six diameters.

Reductions to the modulus of subgrade reaction can be accomplished in LPILE by inputting the appropriate modification factors for p-y curves. Reducing the modulus of subgrade reaction in trailing piers will result in greater computed deflections on these piers. In practice, a grade beam can force deflections of all piers to be equal. Load-deflection graphs can be generated for each pier by using the appropriate p-multiplier values. The sum of the piers lateral load resistance at selected deflections can be used to develop a total lateral load versus deflection graph for the system of piers.

For lateral loads perpendicular to the line of piers, a minimum spacing of three diameters can be used with no capacity reduction. At one diameter (piers touching) the piers should be analyzed as one unit. Interpolation can be used for intermediate conditions.

The above method has been used by our firm for years with success, but sometimes results in overly conservative values. We believe the prediction equations proposed by Reese and Van Impe[1] result in more practical solutions for group efficiency. They were formulated by fitting curves to data representing group efficiency versus pile spacing. No differentiation was made between soil type, pile diameter, or penetration. The data indicates that for side-by-side piers, group efficiency becomes unity at spacing of about 4 pier diameters. For in-line piers, the lead piers were found to have efficiency of unity with spacing of about 4 diameters, and the trailing piers were unity efficiency with spacing of 7 diameters. The equations for solving group efficiency for side-by-side, leading and trailing piers are shown below, where the variable "s" is the pile spacing and "b" is the pile diameter.

Side-by-side piers:

$$e = 0.64(\frac{s}{b})^{0.34} \ for\ 1 \le \frac{s}{b} \le 3.75, and\ e = 1.0,\ for\ \frac{s}{b} \ge 3.75$$ *(Equation 5.39)*

Leading piers:

$$e = 0.7(\frac{s}{b})^{0.26} \ for\ 1 \le \frac{s}{b} \le 4.0, and\ e = 1.0, for\ \frac{s}{b} \ge 4.0$$ *(Equation 5.40)*

Trailing piers:

$$e = 0.48(\frac{s}{b})^{0.38} \ for\ 1 \le \frac{s}{b} \le 7.0, and\ e = 1.0, for\ \frac{s}{b} \ge 7.0$$ *(Equation 5.41)*

For piers that are skewed at an angle (i.e. between in-line and side-by-side), the group efficiency is taken as a modification to shadow and edge effects. The efficiency can be estimated by:

$\mathrm{e} = (e_i^2 \cos^2 \emptyset + e_s^2 \sin^2 \emptyset)^2$; where e_i = efficiency of pile in-line,

e_s = efficiency of pier side-by-side, and

$\emptyset$ = angle between piers (Reese & Wang, 1996)

[1]"Single Piles and Pile Groups Under Lateral Loading," Authored by Lymon C. Reese and William F. Van Impe, 2001; Section 5.7.5, Pages 158 and 159

LIMITATIONS

Our borings were located to obtain a reasonably accurate indication of subsurface foundation conditions. The borings are representative of conditions encountered at the exact boring location only. Variations in subsurface conditions not indicated by the borings are possible.

We believe this investigation was conducted with that level of skill and care normally used by geotechnical engineers practicing in this area at this time. No warranty, express or implied, is made.

If we can be of further service in discussing the contents of this report or in the analysis of the influence of subsoil conditions on design of the structures from a geotechnical engineering point-of-view, please call.

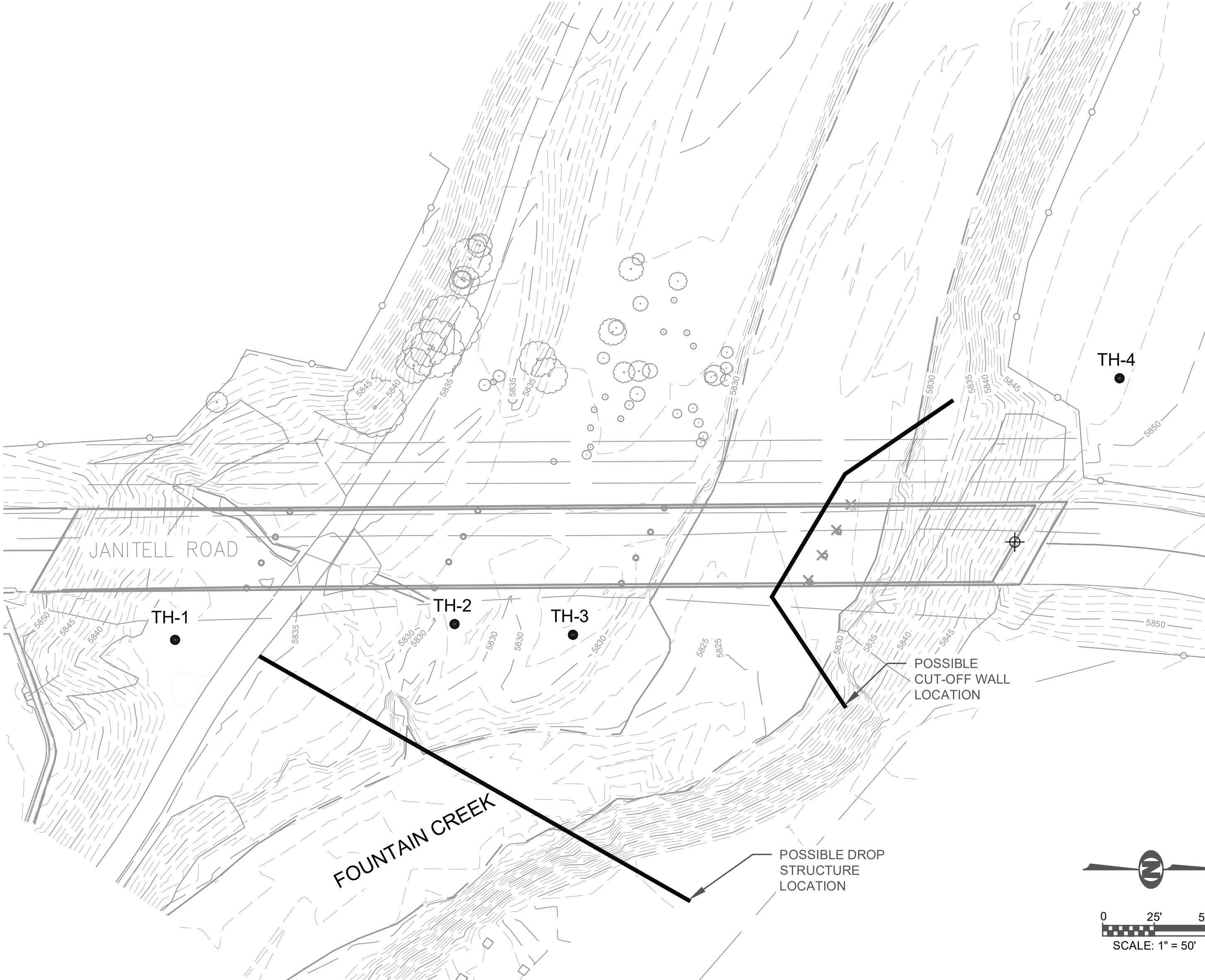
CTL | THOMPSON, INC.

Timothy A. Mitchell, P.E.
Principal Engineer

Reviewed by:

Jeffrey M. Jones, P.E.
Associate Engineer

TAM:JMJ:tam



VICINITY MAP

(NOT TO SCALE)

LEGEND:

TH-1 ● APPROXIMATE LOCATION OF EXPLORATORY BORING.

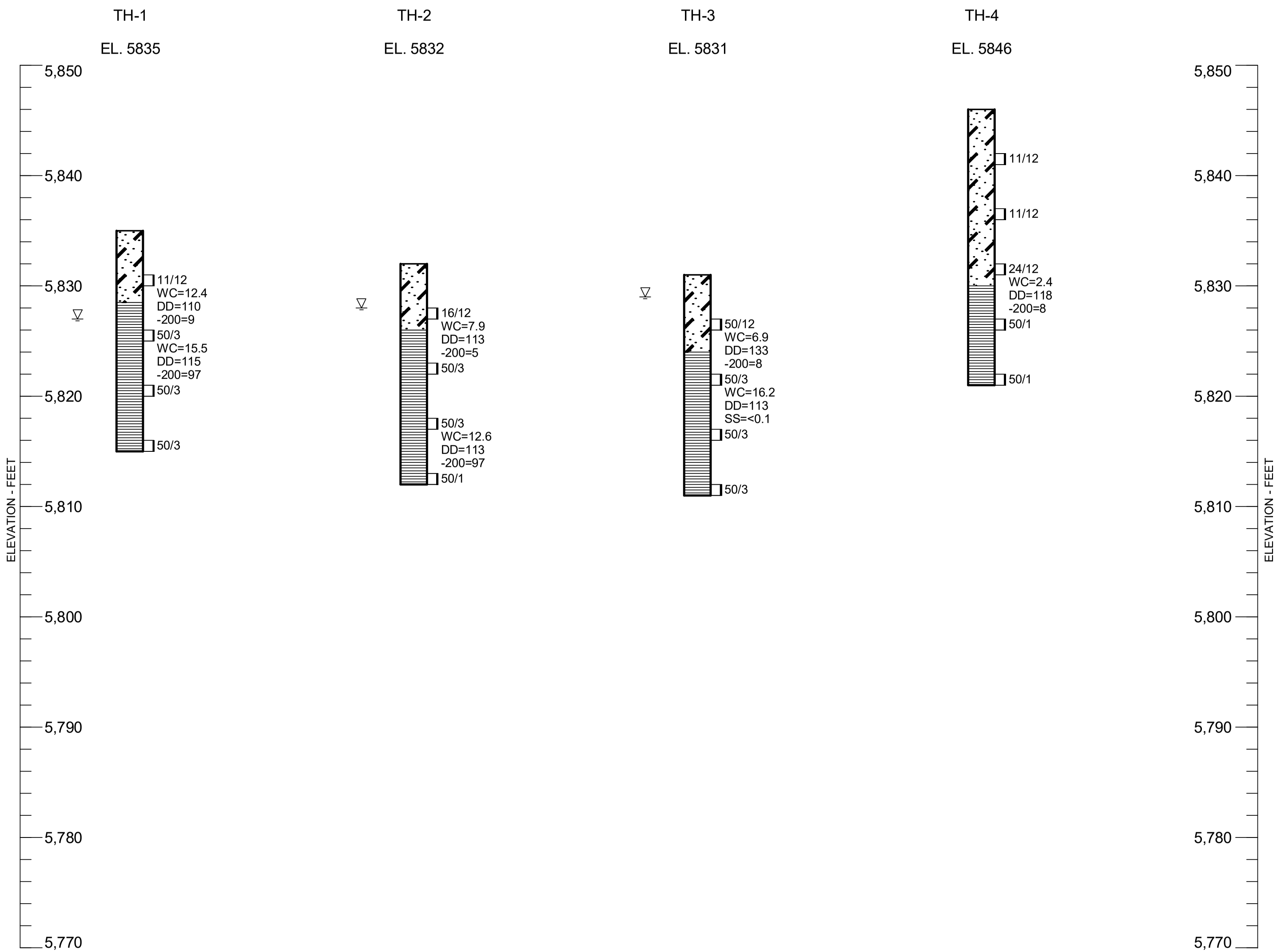
⌖ BENCHMARK. THE CONCRETE AT THE NORTH SIDE OF EXPANSION JOINT AT THE CENTERLINE WAS ESTIMATED TO BE 5855 PER THE TOPOGRAPHY SHOWN.

NOTE:
BASE DRAWING WAS PROVIDED BY BENESCH (PROJECT NO. 0210332.00, DATED 4/19/2021

Location of Exploratory Borings

FIG. 1

BENESCH
JANITELL BRIDGE
CTL|T PROJECT NO. CS19402-125



LEGEND:

SAND, SLIGHTLY CLAYEY TO VERY CLAYEY, GRAVELLY, COBBLES, DENSE, MOIST TO WET, GRAY, LIGHT BROWN. (SP-SC, SC ,GW, GP)

SHALE, VERY HARD, SLIGHTLY MOIST, GRAY.

DRIVE SAMPLE. THE SYMBOL 11/12 INDICATES 11 BLOWS OF A 140-POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE A 2.5-INCH O.D. SAMPLER 12 INCHES.

GROUNDWATER LEVEL MEASURED AT TIME OF DRILLING.

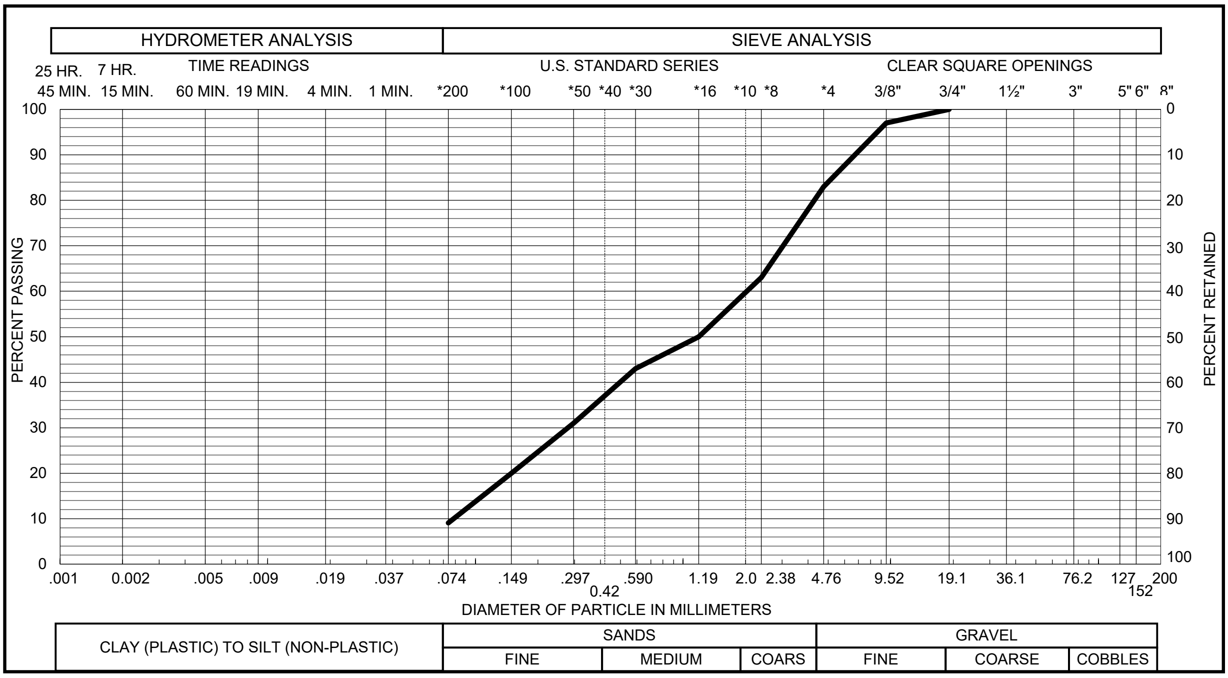
NOTES:

1. THE BORINGS WERE DRILLED USING A 4-INCH DIAMETER, CONTINUOUS-FLIGHT AUGER AND A CME 45, TRUCK-MOUNTED DRILL RIG.
2. THESE LOGS ARE SUBJECT TO THE EXPLANATIONS, LIMITATIONS, AND CONCLUSIONS AS CONTAINED IN THIS REPORT.
3. ELEVATIONS SHOWN WERE DETERMINED BY CTL|THOMPSON, INC. USING A LEVEL AND THE BENCHMARK SHOWN ON FIG. 1.
4. WC - INDICATES MOISTURE CONTENT. (%)
   DD - INDICATES DRY DENSITY. (PCF)
   -200 - INDICATES PASSING NO. 200 SIEVE. (%)
   SS - INDICATES WATER-SOLUBLE SULFATE CONTENT. (%)

BENESCH
JANITELL BRIDGE
CTL|T PROJECT NO. CS19402-125

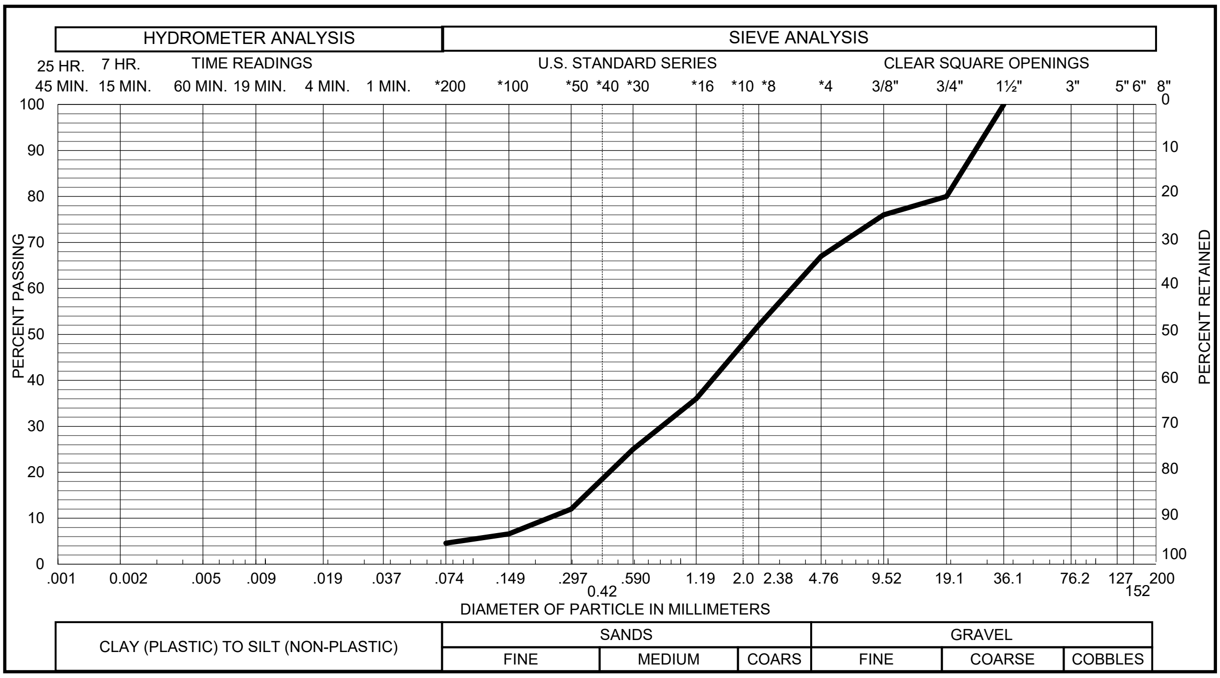
Summary Logs of Exploratory Borings

FIG. 2



Sample of SAND, SLIGHTLY CLAYEY, WITH GRAVEL (SP-SC)
From TH - 1 AT 4 FEET

GRAVEL 17 % SAND 74 %
SILT & CLAY 9 % LIQUID LIMIT %
PLASTICITY INDEX %



Sample of SAND, SLIGHTLY CLAYEY, WITH GRAVEL (SP-SC)
From TH - 2 AT 4 FEET

GRAVEL 33 % SAND 62 %
SILT & CLAY 5 % LIQUID LIMIT %
PLASTICITY INDEX %

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JANITELL BRIDGE
CTL|T PROJECT NO. CS19402-125
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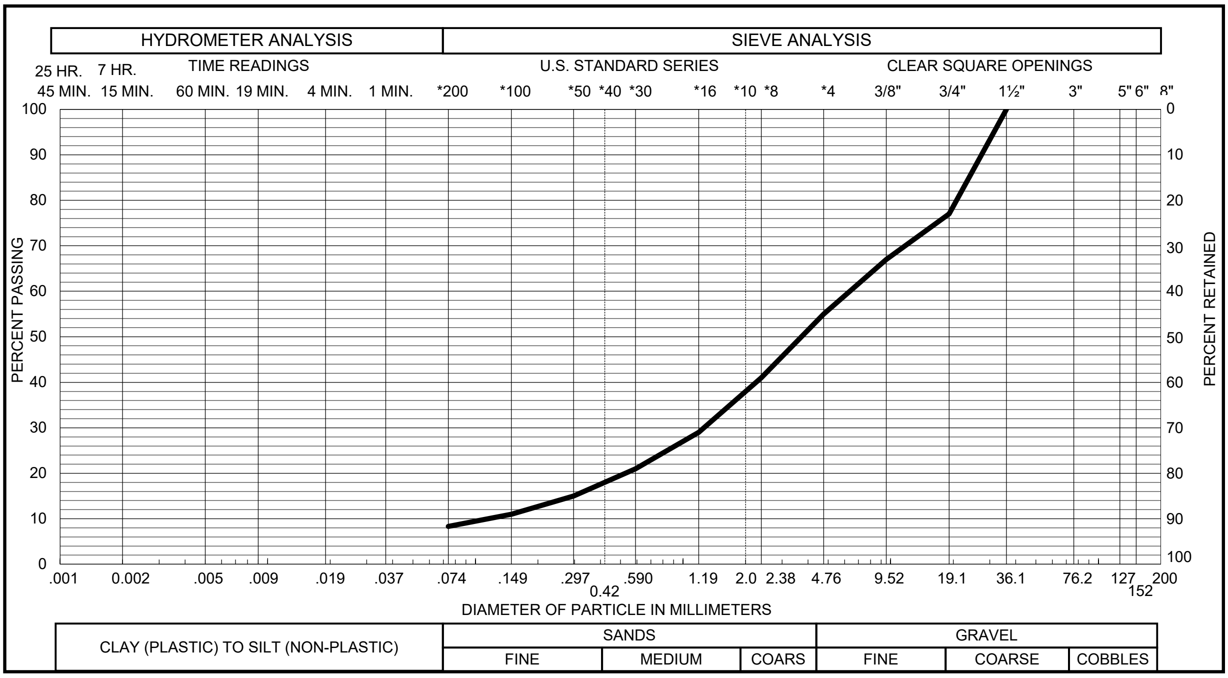
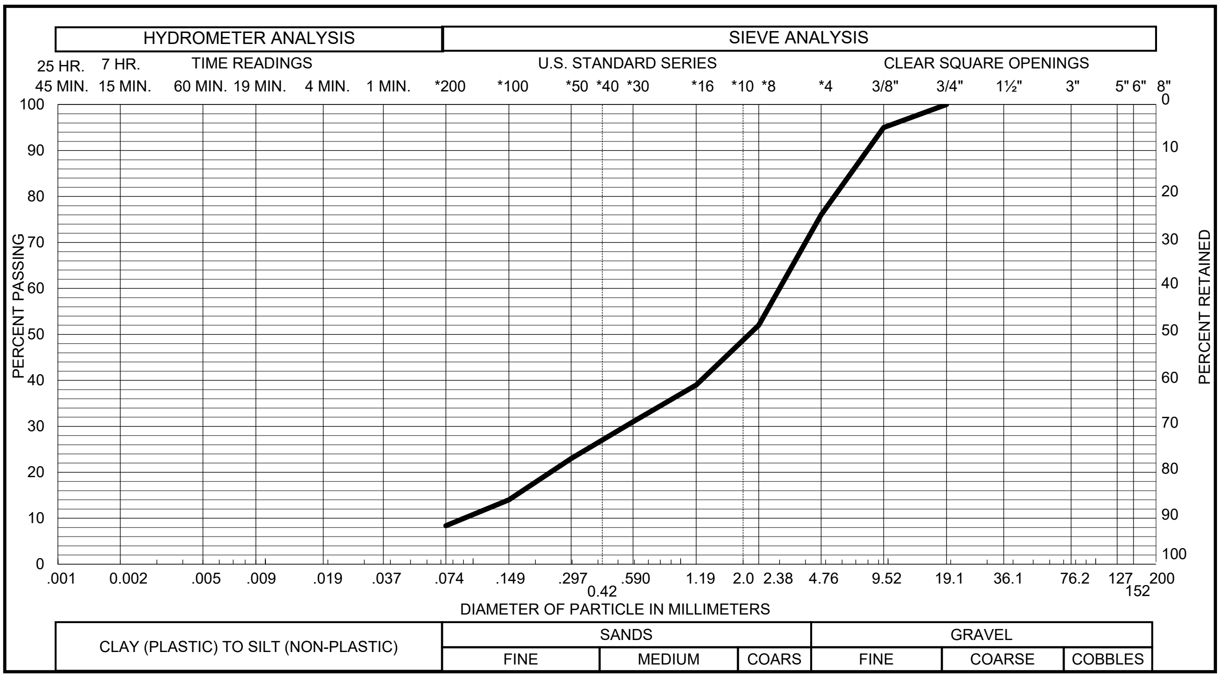
Gradation Test Results

FIG. 3



Sample of SAND, SLIGHTLY CLAYEY, WITH GRAVEL (SP-SC) GRAVEL 45 % SAND 47 %
From TH - 3 AT 4 FEET SILT & CLAY 8 % LIQUID LIMIT %
PLASTICITY INDEX %



Sample of SAND, SLIGHTLY CLAYEY, WITH GRAVEL (SP-SC) GRAVEL 24 % SAND 68 %
From TH - 4 AT 14 FEET SILT & CLAY 8 % LIQUID LIMIT %
PLASTICITY INDEX %

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CTL|T PROJECT NO. CS19402-125

Gradation Test Results

FIG. 4

APPENDIX A

SITE PHOTOGRAPHS

1. Upstream from the bridge.

2. Downstream from the bridge.

3. Confluence of Spring Creek northwest of the bridge.

4. Erosion of the shale bedrock northwest of the bridge, near Spring Creek.

5. Janitell Bridge from upstream.

6. Gravel/sand bar, trail, and south abutment.

7. Gravel/sand bar looking north towards pier P-4 and P-5.

8. Gravel/sand bar at edge of creek at pier P-4.

9. West side of pier P-5.

10. Pier P-5 looking north.

11. East side of pier P-5.

12. Measurment of western pier at P-5; approximately 9 feet below previous ground elevation.

13. Measurment of middle-western pier at P-5; approximately 8.5 feet below previous ground elevation.

14. Measurment of middle-eastern pier at P-5; approximately 8.5 feet below previous ground elevation.

15. Measurment of eastern pier at P-5; approximately 9.3 feet below previous ground elevation.

3. Geotechnical Investigation: Correspondence

Beegle, Noelle

From:	Mitchell, Timothy <TMitchell@CTLThompson.com>
Sent:	Friday, June 25, 2021 2:39 PM
To:	Beegle, Noelle; Sabo, John
Cc:	Epp, William; Fitzhugh, Shawn
Subject:	RE: El Paso County CTL-Thompson-Janitell
Attachments:	CS19402.000-125 FIG 1.pdf; CS19402.000-125 FIG 2.pdf; CS19402-125 GRADATION FIG 3.pdf; CS19402-125 GRADATION FIG 4.pdf

Please see the attached. I was having some changes done to the logs. It appears that the last revision caused the test hole designations to drop off. They are numbered 1 to 4 from left to right.

Based on observations of the north bank, there has been erosion into the shale bedrock. Therefore, I would not expect a smooth/ linear change in the bedrock surface between TH-3 and TH-4 – there will be a drop at the existing creak bank. A cutoff wall near the piers would need to extend into the bedrock, based on the bedrock elevation of TH-3, which is about 5819. Additional scour may have occurred at locations within the creek bed, and I believe any surficial soils remaining will be in a thin layer.. With diversion of the water, this bedrock elevation should also be used if there is concern for the piers adjacent to TH-3. I believe this option may work well for protecting the most at risk piers and abutment on the north, but the addition of the cutoff wall on its own could shift the area of concern to the south. Initial thoughts on excavation are included below.

For the drop structure, I would expect the bedrock to be at a similar elevation to TH-3 near the existing channel, with the same caveat concerning additional scour in the creek bed. Once again the bedrock appears to be exposed on the northern bank, while it is expected to gradually rise towards the south. Cutoff walls such as sheet piles, if used for the wall itself or part of the drop structure, are expected to need pre-excavation to allow installation into the bedrock. Excavation into the shale, for trenches or keyways, can be completed with conventional heavy duty equipment, although rock teeth may be required to expedite the work.

I will continue to get the reporting wrapped up on this, but if you have specific thoughts on the type of drop structure and/or cutoff walls planned, please let me know so I can address related geotechnical. Also, please let me know if there are questions on the items attached or discussed above.

Tim

Timothy A. Mitchell, P.E.
Principal Engineer | Division Manager
Colorado Springs and Pueblo
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Colorado Springs, Colorado 80918
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tmitchell@ctlthompson.com
www.ctlt.com [ctlt.com]
Licensed States: CO, UT, VA

Beegle, Noelle

From: Mitchell, Timothy <TMitchell@CTLThompson.com>
Sent: Thursday, July 15, 2021 8:52 AM
To: Beegle, Noelle; Sabo, John
Cc: Epp, William; Fitzhugh, Shawn
Subject: RE: El Paso County CTL-Thompson-Janitell

The CAD information should be being sent out. The following is recommended for Laterally loaded piers:

Laterally-Loaded Piers

Lateral load analysis of piers can be performed with the software analysis package LPILE by Ensoft, Inc. We believe this method of analysis is typically appropriate for piers with a pier length to diameter ratio of seven or greater. Suggested criteria for LPILE analysis are presented in the following Table. We have provided values for the sands and gravels, based on the materials being relatively rounded due to the action of the stream. Clay values may be appropriate where new drop structures slow the water around the piers allowing for deposition of finer particles. It may be that a combination of materials will be deposited so we recommend determining the more conservative analysis between the two materials. Other models, such as "Silt" may be appropriate; however, without knowing what mixture of materials may be deposited, it becomes more difficult to determine strengths using a combination of cohesion and friction angles for the unknown materials.

SOIL INPUT DATA FOR "LPILE"

Soil Type	Sands and Gravels	Natural Clay	Shale Bedrock
Recommended p-y Curve Model	Sand	Soft Clay	Weak Rock
Density (pci)	0.063	0.060	0.075
Friction Angle (degrees)	25	-	-
k_s (pci)	20	-	-
k - Static (pci)	-	1000	-
k - Cyclic (pci)	-	-	-
E50	-	0.02	-
c (psi)	-	2	-
Compressive Strength (psi)	-	-	300
Young's Modulus, E (psi)	-	-	0.5×10^6
K_{rm}	-	-	0.0001
RQD (%)	-	-	70

Other analysis procedures require input of a horizontal modulus of subgrade reaction (K_h). We believe the following formulas listed in the table below are appropriate for calculating horizontal modulus of subgrade reaction (K_h) values.

HORIZONTAL MODULUS OF SUBGRADE REACTION

Soil Type	Sands	Clays	Bedrock
Modulus of Subgrade Reaction, K_h (tcf)	$K_h = \frac{15 \times Z}{d}$	$K_h = \frac{20}{d}$	$K_h = \frac{300}{d}$

Where z = depth (ft); d = pier diameter (ft).

Closely-Spaced Pier Reduction Factors

For axial loading, no reduction is needed for a minimum spacing of three diameters (center to center). At one diameter (piers touching), the skin friction reduction factor for both piers would be 0.5. End pressure values would not be reduced provided the bases of the piers are at similar elevations. Interpolation can be used between one and three diameters.

For lateral loading, no reduction is needed for piers in-line with the direction of lateral loads with a minimum spacing of six diameters (center-to-center) based upon the larger pier. If a closer spacing is required, the modulus of subgrade reaction for initial and trailing piers should be reduced. At a spacing of three diameters, the effective modulus of subgrade reaction of the first pier can be estimated by multiplying the given modulus by 0.6; for trailing piers in a line at three-diameter spacing, the factor is 0.4. Linear interpolation can be used for spacing between three and six diameters.

Reductions to the modulus of subgrade reaction can be accomplished in LPILE by inputting the appropriate modification factors for p-y curves. Reducing the modulus of subgrade reaction in trailing piers will result in greater computed deflections on these piers. In practice, a grade beam can force deflections of all piers to be equal. Load-deflection graphs can be generated for each pier by using the appropriate p-multiplier values. The sum of the piers lateral load resistance at selected deflections can be used to develop a total lateral load versus deflection graph for the system of piers.

For lateral loads perpendicular to the line of piers, a minimum spacing of three diameters can be used with no capacity reduction. At one diameter (piers touching) the piers should be analyzed as one unit. Interpolation can be used for intermediate conditions.

The above method has been used by our firm for years with success, but sometimes results in overly conservative values. We believe the prediction equations proposed by Reese and Van Impe[1] result in more practical solutions for group efficiency. They were formulated by fitting curves to data representing group efficiency versus pile spacing. No differentiation was made between soil type, pile diameter, or penetration. The data indicates that for side-by-side piers, group efficiency becomes unity at spacing of about 4 pier diameters. For in-line piers, the lead piers were found to have efficiency of unity with spacing of about 4 diameters, and the trailing piers were unity efficiency with spacing of 7 diameters. The equations for solving group efficiency for side-by-side, leading and trailing piers are shown below, where the variable “s” is the pile spacing and “b” is the pile diameter.

Side-by-side piers:

$$e = 0.64\left(\frac{s}{b}\right)^{0.34} for\ 1 \le \frac{s}{b} \le 3.75, and\ e = 1.0,\ for\ \frac{s}{b} \ge 3.75$$ *(Equation 5.39)*

Leading piers:

$$e = 0.7\left(\frac{s}{b}\right)^{0.26} for\ 1 \le \frac{s}{b} \le 4.0, and\ e = 1.0, for\ \frac{s}{b} \ge 4.0$$ *(Equation 5.40)*

Trailing piers:

$$e = 0.48(\frac{s}{b})^{0.38}\ for\ 1 \le \frac{s}{b} \le 7.0, and\ e = 1.0, for\ \frac{s}{b} \ge 7.0 \qquad \textit{(Equation 5.41)}$$

For piers that are skewed at an angle (i.e. between in-line and side-by-side), the group efficiency is taken as a modification to shadow and edge effects. The efficiency can be estimated by:

$\mathrm{e} = (e_i^2 \cos^2 \emptyset + e_s^2 \sin^2 \emptyset)^2$; where e_i = efficiency of pile in-line,
e_s = efficiency of pier side-by-side, and
$\emptyset$ = angle between piers (Reese & Wang, 1996)

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Licensed States: CO, UT, VA

From: Beegle, Noelle <NBeegle@benesch.com>
Sent: Monday, July 12, 2021 8:50 AM
To: Mitchell, Timothy <TMitchell@CTLThompson.com>; Sabo, John <JSabo@benesch.com>
Cc: Epp, William <WEpp@benesch.com>; Fitzhugh, Shawn <Sfitzhugh@CTLThompson.com>
Subject: RE: El Paso County CTL-Thompson-Janitell

Tim,
Can you provide the completed report? We need an L-pile table to analyze the existing piers. Also, please provide the boring locations in CAD so we can add them to our drawings.

Please let me know when you can complete this request. We are running up against a deadline.

Noelle Beegle, PE, CFM
Project Manager

nbeegle@benesch.com
direct: 720-473-7582 **mobile:** 303-499-6991 **office:** 303-771-6868

Beegle, Noelle

From: Mitchell, Timothy <TMitchell@CTLThompson.com>
Sent: Wednesday, August 4, 2021 10:15 PM
To: Beegle, Noelle
Cc: Murphy, John; Bechtold, Daniel; Fitzhugh, Shawn; Sabo, John; Epp, William
Subject: RE: Janitell Bridge Boring Clarification

Based on the elevations measured today, the bedrock surface at the piers along Pier 5 ranges from 5826.3 down to 5825.7 feet. There was some gravel and cobble near a couple of the piers, so bedrock elevations were obtained where the water was moving faster nearby. I would suggest using a bedrock elevation of 5825.5 for analyzing the piers. The measurements down from the previous ground surface were about 8.3 to 9.2 feet. As noted, some were measured to gravels and cobble.

It appears the locations of the borings were off resulting in a elevation bust for TH-3 (actual elevation = 5831 feet). The other elevations appear to be within reason.

The measured bedrock surface at the north bank was about 5830 on the west side to 5829 on the east side, indicating there has been erosion of about 3.5 to 4 feet of the bedrock near Pier 5.

We use the centerline of the road at the north side of the joint at abutment 6 as our benchmark. We assigned that an elevation of 5855 feet. We also took a shot down to the top of Pier 5 on the east side of the pier to help verify the elevations, which came out to about 5850.7. The original plans indicated an elevation of about 5847.8, so I believe that was close with the change in the Datum. As we are not surveyors, these values should be considered approximate. Pictures of places surveyed can be found here:

https://ctlt.box.com/s/cr163iz8iehnnteph8pbx722qpy4s55g [ctlt.box.com] Unfortunately James' camera did not do the best at capturing the rod. The flow in the creek does pick up quickly, even with small rain events to the north. Getting back from the piers was more challenging than getting out to them.

Tim

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CTL|THOMPSON YEARS 50 FOUNDED IN 1971

From: Beegle, Noelle <NBeegle@benesch.com>
Sent: Wednesday, August 4, 2021 8:52 AM
To: Mitchell, Timothy <TMitchell@CTLThompson.com>; Touchberry, James <jtouchberry@ctlthompson.com>
Cc: Murphy, John <JMurphy@benesch.com>; Bechtold, Daniel <DBechtold@benesch.com>; Fitzhugh, Shawn <Sfitzhugh@CTLThompson.com>; Sabo, John <JSabo@benesch.com>; Epp, William <WEpp@benesch.com>
Subject: Re: Janitell Bridge Boring Clarification

Thanks Tim.

If possible, please ask them to wade out and measure the distance from existing creek bed to the location where the column changed into caisson. That should help us tie to the as builts better.

Noelle Beegle, PE
Alfred Benesch & Co
C: 303-499-6991
Noelle Beegle, PE, CFM
Project Manager

nbeegle@benesch.com
direct: 720-473-7582 **mobile:** 303-499-6991 **office:** 303-771-6868

---

75 years benesch

From: Mitchell, Timothy <TMitchell@CTLThompson.com>
Sent: Tuesday, August 3, 2021 11:58:24 PM
To: Beegle, Noelle <NBeegle@benesch.com>; Touchberry, James <jtouchberry@ctlthompson.com>
Cc: Murphy, John <JMurphy@benesch.com>; Bechtold, Daniel <DBechtold@benesch.com>; Fitzhugh, Shawn <Sfitzhugh@CTLThompson.com>; Sabo, John <JSabo@benesch.com>; Epp, William <WEpp@benesch.com>
Subject: RE: Janitell Bridge Boring Clarification

It would seem highly unlikely that these piers are currently stable with little to no embedment into bedrock so we believe there must be discrepancy between the as-built data and the Geotech findings. Could you provide some insight on this issue? I will have James, who drilled, out to the site with me tomorrow (Wednesday) to verify elevations. The elevations we used were based on the elevations shown on the figure. We will get things tied to a specific point tomorrow to narrow down what elevations we are at. The depths within the borings are well defined as they are a significant change in materials.

James, get the level, tripod, and rod so we can get this done. It will need to be late morning / early afternoon for me, and we can coordinate tomorrow.

Are your elevation based on the NAVD88 datum? See above. Elevations were based on Benesch supplied information. We will tie this into an easily established point tomorrow. Do you have a preferred benchmark?

In your email, you said that you would not expect a linear transition from TH3 (Pier 4) to TH4 (North Abutment) so would Pier 5 have a similar bedrock elevation as TH3 or would it be higher? I expect the elevation to be similar to the elevation of TH-3 (Pier 4). This is based on the observed erosion of the bedrock along the northern shore (attached picture). We will verify the elevation of the bedrock surface at the northern streambank, and get some elevations at the pier.

Our requested boring locations had shown spots adjacent to Pier 5. Could you fill us in as to why these borings were not obtained in your investigation? The requested boring locations are within the stream channel. My email from May 10 briefly discussed this. There was no access obtained for the property to the northeast, and there is generally not access to the south of the north abutment on the north side of the creek. We drilled where we could on the south side of the channel.

Is it possible to get back out there and complete more borings? It is essential that we understand the embedment. What is your schedule for doing this work? If it is not possible in the next few weeks we may have to try and schedule another firm. We can get more borings; however, the locations they are accessible are limited, based on access rights and the stream channel. With the erosion extending below the observed bedrock elevation at the stream bank, the bedrock surface at the piers is generally expected to be the elevation of the stream channel. We will get some shots on this area to verify the data we have at which point we can better determine what additional information is required. I believe the elevation data obtained tomorrow will provide the needed information for the current bedrock elevation at the pier.

To verify some additional information, what were the contours in your CAD figure based on? Was there a specific survey of the creek channel?

Tim

Timothy A. Mitchell, P.E.
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Colorado Springs and Pueblo
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tmitchell@ctlthompson.com
www.ctlt.com [ctlt.com]
Licensed States: CO, UT, VA

CTL|THOMPSON YEARS 50 FOUNDED IN 1971

From: Beegle, Noelle <NBeegle@benesch.com>
Sent: Tuesday, August 3, 2021 3:20 PM
To: Mitchell, Timothy <TMitchell@CTLThompson.com>
Cc: Murphy, John <JMurphy@benesch.com>; Bechtold, Daniel <DBechtold@benesch.com>; Fitzhugh, Shawn <Sfitzhugh@CTLThompson.com>; Sabo, John <JSabo@benesch.com>; Epp, William <WEpp@benesch.com>
Subject: Janitell Bridge Boring Clarification
Importance: High

Tim,

We have encountered some issues based on the information we have available while running our pier analysis. See the attached Location of Borings Exhibit you provided. Your investigation included borings nearest Pier 4 and Abutment 6 and the results from TH3 (nearest Pier 4) show the bedrock at 5819 ft which we assumed would be the same for Pier 5. The as-builts for the project indicate that the bottom of shaft elevation is 5818.4 (NAVD29) at Pier 4 and 5815.9 ft (NAVD29) at Pier 5. Those elevations adjusted to NAVD88 would become 5821.4 ft for Pier 4 and 5818.9 ft for Pier 5.

That would conclude that the existing shaft at Pier 4 is no longer embedded in bedrock at Pier 4 and is only embedded 0.1ft at Pier 5. It would seem highly unlikely that these piers are currently stable with little to no embedment into bedrock so we believe there must be discrepancy between the as-built data and the Geotech findings. Could you provide some insight on this issue?

Additionally, we have a few items we would like to confirm; Are your elevation based on the NAVD88 datum? In your email, you said that you would not expect a linear transition from TH3 (Pier 4) to TH4 (North Abutment) so would Pier 5 have a similar bedrock elevation as TH3 or would it be higher? Our requested boring locations had shown spots adjacent to Pier 5. Could you fill us in as to why these borings were not obtained in your investigation? Is it possible to get back out there and complete more borings? It is essential that we understand the embedment. What is your schedule for doing this work? If it is not possible in the next few weeks we may have to try and schedule another firm.

Noelle Beegle, PE, CFM
Project Manager

nbeegle@benesch.com
direct: 720-473-7582 **mobile:** 303-499-6991 **office:** 303-771-6868
7979 E. Tufts Avenue, Suite 800, Denver, CO 80237

This message has originated from an **External Source**. Please use proper judgment and caution when opening attachments, clicking links or responding to this email.

4. Non-Destructive Testing Report

Olson ENGINEERING

Corporate Office:
12401 W. 49th Avenue
Wheat Ridge, CO 80033 USA
phone: 303.423.1212
fax: 303.423.6071

March 9, 2022

Benesch
7979 E. Tufts Ave., Suite 800
Denver, CO 80237

Attn: Noelle Beegle
Tel: 720-473-7582
Email: nbeegle@benesch.com

RE: Nondestructive Evaluation (NDE) Investigation Report
Sonic Echo/Impulse Response Test Results
Integrity and Length Evaluation of 12 Concrete Drilled Piers
Janitell Road Bridge Over Fountain Creek
Colorado Springs,
Olson Job No. 7192A

Ms. Beegle,

This report presents the results of a Nondestructive Evaluation (NDE) investigation of 12 concrete drilled piers supporting the Janitell Road Bridge over Fountain Creek in Colorado Springs, Colorado. The testing was requested to evaluate the depth and integrity of the pier's drilled concrete foundations. The Pier 5 foundation was the main focus of the investigation because of scour visible on the foundation elements. A summary of our investigation findings and conclusion is presented below followed by SE/IR example results and discussion and the test method description.

EXECUTIVE SUMMARY

The fieldwork was performed by Mr. Mason Timm, Project Engineer and Mr. Nick Gerace, Staff Physicist of our firm on February 8th and March 1st, 2022. The Sonic Echo/Impulse Response (SE/IR) test method was used to evaluate the integrity of the twelve concrete pier footings in general accordance with ASTM D5882. The SE/IR results indicate Sound conditions for all tested foundations, with overall embedded depths ranging from 11.5 to 26 feet. The embedded depths (depth below current grade level) at Pier 3 ranged from 19.3 to 26.0 ft, Pier 4 from 16.8 to 19.6, and Pier 5 from 11.5 to 16.4 ft.

SONIC ECHO/IMPULSE RESPONSE (SE/IR) TEST RESULTS

The results of the testing are presented below in Table I. A compressional wave velocity of 12,500 feet per second (fps) was assumed to compute the reflector depths for the tested pier footings. This velocity is typical of normal strength mix designs for concrete foundations. It is estimated that predicted depths are accurate to within 10% of actual values. The results table includes the pier and column numbers, the raw echo depth, and the echo depth corrected to local grade at the time of testing. Also included in the last column is a notation on the relative strength or clarity of the apparent tip echo. "Weak" echoes typically indicate less clear tip echoes that likely indicate the shaft is well-bonded to hard bedrock at the tip. As seen in the results table, there was a wide range of apparent tip depths identified in the results, with some results clearer than others. In general, the foundations for Pier 3 extended the deepest from current grade level (as expected) while the foundations of Pier 5 were the shortest when measured from current grade level.

Table I – Janitell Road Bridge SE/IR Test Results

File	Pier	Column		SE/IR Echo Depth (ft)	Receiver Elevation Above Grade (ft)	Embedded Shaft Depth (ft)	Echo Quality
s_1	3	1	1=east	26.8	0.8	26.0	V. Good
s_4	3	2		19.8	0.5	19.3	Good
s_5	3	3		20.4	0.5	19.9	Moderate
s_7	3	4		26.2	0.3	26.0	Good
s_9	4	1	1=east	17.8	0.3	17.5	Weak
s_11	4	2		19.6	0.8	18.8	Good
s_14	4	3		17.9	0.3	17.6	Moderate
s_15	4	4		16.8	0.1	16.7	Weak
s_18	5	1	1=east	17.1	1.0	16.1	Moderate
s_19	5	2		17.4	1.0	16.4	Weak
s_21	5	3		15.3	1.0	14.3	Good
s_23	5	4		12.5	1.0	11.5	Good

All collected SE/IR data records were acquired by a pair of accelerometer receivers (which were then integrated to velocity) mounted to the side of the column above grade with epoxy. Each column was then impacted with a 1-lb or 3-lb instrumented hammer with a hard black plastic tip to impart the compressional wave energy into a strike block mounted to the side of the column via a "Hilti" style concrete anchor. Reported echo depths are measured from the location of the mounted receiver downward to the foundation tip (actual heights are in the results table) which are adjusted in the table to reflect embedment depth below current grade level. Figure 1 presents a photograph that shows the SE/IR test setup on the eastern column of Pier 3.

Figure 1: Photograph of the data collection setup using the Olson Instruments Freedom Data PC, black tip 3-lb impulse hammer, and two accelerometer receiver mounted to the side of the column above the foundation.

Figure 2 presents the SE results for the East foundation of Pier 3. The SE/IR results indicate Sound pier conditions for the South pier, with a strong, clear tip echo at 26.8 feet below the receiver. As noted in the results table, the receiver was located 0.8 ft above local grade, giving an embedded depth of 26.0 feet for this shaft. No notable shallow echoes that would indicate a concrete anomaly were observed in the data.

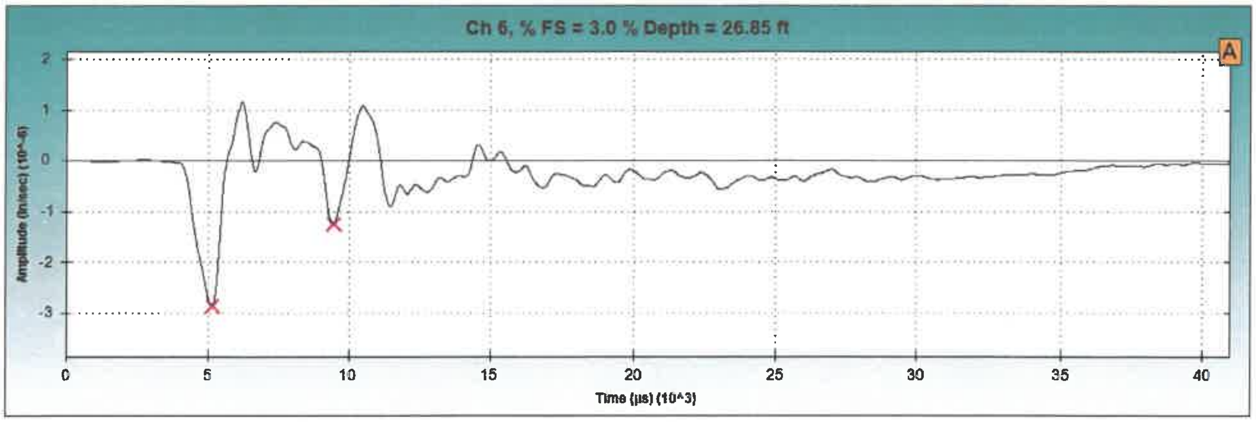


Figure 2: Sonic Echo (SE) data for the S Pier (File SEIR6) showing a bottom echo depth at 26.8 feet, with no significant echoes shallower than the bottom echo.

Note that the SE/IR method is typically effective at identifying anomalies that result in a change in the cross-sectional area of the foundation of 20-25% or more. For example, a 48" diameter pier has an area of 1810 in^2; if a full circumference necking defect was present, a diameter reduction to 41.5" would result in a 25% reduction in cross-sectional area. Alternatively, an enclosed defect would need a cross-sectional area of 452 in^2, which is equivalent to a circle with a diameter of 24" or about a 21" x 21" square.

SONIC ECHO (SE) AND IMPULSE RESPONSE (IR) METHODS

Sonic Echo (SE) Test Method. The SE method is a low strain integrity test conducted from the top of the shaft as shown in Figure 3. Test equipment typically includes a 1 or 3-lb impulse hammer, receiver(s) (accelerometers) mounted on the exposed top or upper side of the shaft, and a PC-based or hand-held data acquisition/analysis system. The impulse hammer has a built-in load cell that can measure the force and duration of the impact. The test involves hitting the foundation top with the hammer to generate energy that travels to the bottom of the foundation. The wave reflects off irregularities (cracks, necks, bulbs, soil intrusions, voids, etc.) and/or the bottom of the foundation and travels back along the foundation to the top. The receiver measures the vibration response of the foundation to each impact. The signal analyzer processes and displays the hammer and receiver outputs. Foundation length and integrity of concrete are evaluated by identifying and analyzing the arrival times, direction, and amplitude of reflections measured by the receivers in time. The echo depth (D) is calculated by multiplying the reflection time (t) by the compression wave velocity (V) and dividing this quantity by 2 to account for the fact that the wave has gone down and reflected back (i.e. $D = V*t/2$).

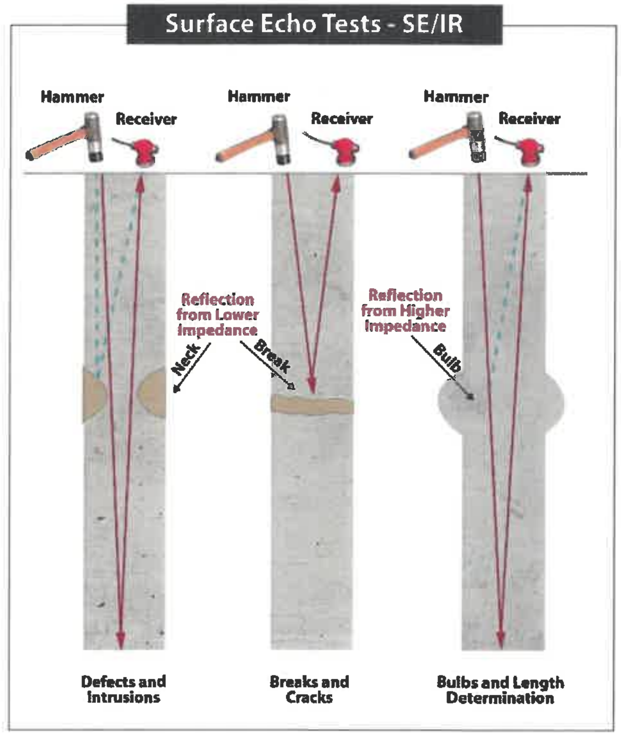


Figure 3: Sonic Echo/Impulse Response Test

Impulse Response (IR) Test Method. The IR method is also an echo test and uses the same test equipment as the SE method. The test procedures are similar to the SE test procedures, but the data processing is different. The IR method involves frequency domain data processing, i.e., the vibrations of the foundation measured by the receivers are processed with Fast Fourier Transform (FFT) algorithms to generate transfer functions for analyses. The coherence of the impulse hammer impact and accelerometer receiver response data versus frequency is calculated to indicate the data quality. A coherence near 1.0 indicates good quality data. For shafts in air or in relatively soft soils, the coherence will typically only be near 1.0 at frequencies for which the mobility is non-zero. In the IR records the linear transfer function amplitude is in inches/second/pound force on the vertical axis (mobility) and frequency in Hz on the horizontal axis. Because of the rod-like shape of a deep foundation, reflections are indicated by equally spaced resonant peaks that correspond to modes of vibration associated with the depth of the reflector. The inverse of the SE reflection time, t, is equal to the change in frequency, Δf, between the resonant peaks in the IR mobility plot. The reflector depth is then calculated as:

$$D = V/(2*\Delta f).$$

SE/IR Analyses. Analysis of the length determination and the integrity evaluation of a foundation for both the SE and IR methods is based on the identification and evaluation of reflections. However, test results are analyzed in the time domain for the SE and in the frequency domain for the IR method. The reflections are shown as resonant frequency peaks in the frequency domain for IR test data. The two methods complement each other because the identifications of reflections are sometimes clearer in either the time or the frequency domain.

The SE and IR test methods are sensitive to changes in the shaft impedance (shaft concrete area * velocity * mass density where mass density equals unit weight divided by gravity), which cause the reflections of the compression wave energy. Compression wave energy (hammer impact energy) reflects differently from increased shaft impedance than from decreased shaft impedance. This phenomenon allows the type of reflector to be identified as follows. Soil intrusions, honeycomb, breaks, cracks, cold joints, poor quality concrete and similar defects (referred to herein as a neck) are identified as reflections that correspond to a decrease in the shaft impedance. Increases in the shaft cross-section or the competency of surrounding materials such as bedrock and stiffer soil strata (referred to herein as a bulb) are identified as reflections corresponding to increases in the shaft impedance. A decrease in impedance is indicated by a downward initial break of a reflection event in an SE record and frequency peaks positioned in a record such that a peak could be extrapolated to be near 0 Hz in the mobility plot. Conversely, an increase in shaft impedance is identified by an upward initial break for an SE reflector and frequency peaks positioned in an IR record such that a trough could be extrapolated to be near 0 Hz in the mobility plot.

When length to diameter ratios exceed 20:1 to 30:1 for shafts in stiffer soils/bedrock, the attenuation of compression wave energy is high and bottom echoes are weak or unidentifiable in SE/IR test results. The term "weak bottom echo" is used to indicate that a bottom echo is seen, but that it is barely visible above the normal background noise. Note that it is possible for a perfectly sound shaft to have a weak bottom echo, as a weak echo can be caused by either a shallow reflector blocking energy from the shaft bottom, or from wave energy coupling into the bedrock at the shaft bottom rather than reflecting back up. Thus, the strength of the bottom echo is used as a secondary, minor consideration as to shaft condition.

CLOSURE

The field portion of this NDT investigation was performed in accordance with generally accepted testing procedures. If we can provide additional information or services on this project, or additional information becomes available that would impact the findings of this investigation, please notify our office.

Respectfully submitted,

Olson Engineering, Inc.

Dennis Sack, P.E.
Principal Engineer

COLORADO REGISTERED
DENNIS A. SACK
43353
3-9-22
PROFESSIONAL ENGINEER

(1 pdf copy e-mailed)

5. Structural Evaluation Memo

Alfred Benesch & Company
7979 E. Tufts Avenue, Suite 800
Denver, CO 80237
www.benesch.com
P 303-771-6868

August 9, 2021

El Paso County
El Paso County Transportation
3275 Akers Drive
Colorado Springs, CO 80922

Attn: Alissa Werre, PE

RE: **Existing Pier Analysis at Janitell Road Bridge over Fountain Creek**

Dear Ms. Werre,

Alfred Benesch & Company (Benesch) is pleased to provide this memorandum of the analysis of the existing pier located in the waterway of Fountain Creek.

Benesch performed a structural analysis of Pier 5. The purpose of the analysis was to assess the existing structural stability of the pier in its current condition. The analysis is based on as-built drawings provided by El Paso County dated 7/17/1991 and a recent geotechnical investigation performed by CTL Thompson.

The Janitell Bridge over Fountain Creek is considered scour critical. There is significant scour at Pier 5 due to channel migration to the north cutting the toe of the slope below the north abutment. The existing pier is comprised of a four 36-inch diameter concrete columns and drilled shafts that are approximately 28-feet in height from the top of the column to the bottom of the shaft. The depth of scour noted in the 2020 inspection report was 9ft. From CTL Thompson's field investigation, they noted that the top of bedrock was located at elevation 5825.5ft (NAVD88). As-built drawings show the bottom of the shaft to be at 5815.9ft NAVD29 which converted to NAVD88 would be 5818.9ft. That information indicates that the shafts at Pier 5 are embedded 6.6 feet in bedrock.

Benesch performed a structural analysis of Pier 5 per current AASHTO LRFD Bridge Design Specification standards. Our calculations analyzed the design loads for the superstructure and substructure of the bridge to determine the forces on the existing shafts. Those forces were input into a pile analysis software along with the soil parameters provided by CTL Thompson to determine the stability of the pier. Our calculations confirmed that the strength and serviceability of the scoured shafts is sufficient in its current state.

Sincerely,

John Murphy, PE

COLORADO LICENSED
JOHN E. MURPHY
56764
08/09/2021
PROFESSIONAL ENGINEER

CC: Noelle Beegle, PE
File

6. FEMA FIS/FIRM References

NOTES TO USERS

This map is for use in administering the National Flood Insurance Program. It does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size. The **community map repository** should be consulted for possible updated or additional flood hazard information.

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

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

Boundaries of the **floodways** were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the Flood Insurance Study report for this jurisdiction.

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

The **projection** used in the preparation of this map was Universal Transverse Mercator (UTM) zone 13. The **horizontal datum** was NAD83, GRS80 spheroid. Differences in datum, spheroid, projection or UTM zones zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

Flood elevations on this map are referenced to the **North American Vertical Datum of 1988 (NAVD88)**. These flood elevations must be compared to structure and ground elevations referenced to the same **vertical datum**. For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at http://www.ngs.noaa.gov/ or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, N/NGS12
National Geodetic Survey
SSMC-3, #9202
1315 East-West Highway
Silver Spring, MD 20910-3282

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

Base Map information shown on this FIRM was provided in digital format by El Paso County, Colorado Springs Utilities, City of Fountain, Bureau of Land Management, National Oceanic and Atmospheric Administration, United States Geological Survey, and Anderson Consulting Engineers, Inc. These data are current as of 2006.

This map reflects more detailed and up-to-date **stream channel configurations and floodplain delineations** than those shown on the previous FIRM for this jurisdiction. The floodplains and floodways that were transferred from the previous FIRM may have been adjusted to conform to these new stream channel configurations. As a result, the Flood Profiles and Floodway Data tables in the Flood Insurance Study Report (which contains authoritative hydraulic data) may reflect stream channel distances that differ from what is shown on this map. The profile baselines depicted on this map represent the hydraulic modeling baselines that match the flood profiles and Floodway Data Tables if applicable, in the FIS report. As a result, the profile baselines may deviate significantly from the new base map channel representation and may appear outside of the floodplain.

Corporate limits shown on this map are based on the best data available at the time of publication. Because changes due to annexations or de-annexations may have occurred after this map was published, map users should contact appropriate community officials to verify current corporate limit locations.

Please refer to the separately printed **Map Index** for an overview map of the county showing the layout of map panels; community map repository addresses; and a Listing of Communities table containing National Flood Insurance Program dates for each community as well as a listing of the panels on which each community is located.

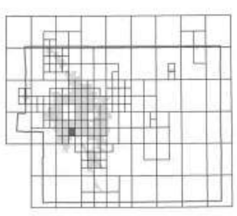
Contact **FEMA Map Service Center** (MSC) via the FEMA Map Information eXchange (FMIX) 1-877-336-2627 for information on available products associated with this FIRM. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. The MSC may also be reached by Fax at 1-800-358-9620 and its website at http://www.msc.fema.gov/.

If you have **questions about this map** or questions concerning the National Flood Insurance Program in general, please call **1-877-FEMA MAP** (1-877-336-2627) or visit the FEMA website at http://www.fema.gov/business/nfip.

El Paso County Vertical Datum Offset Table

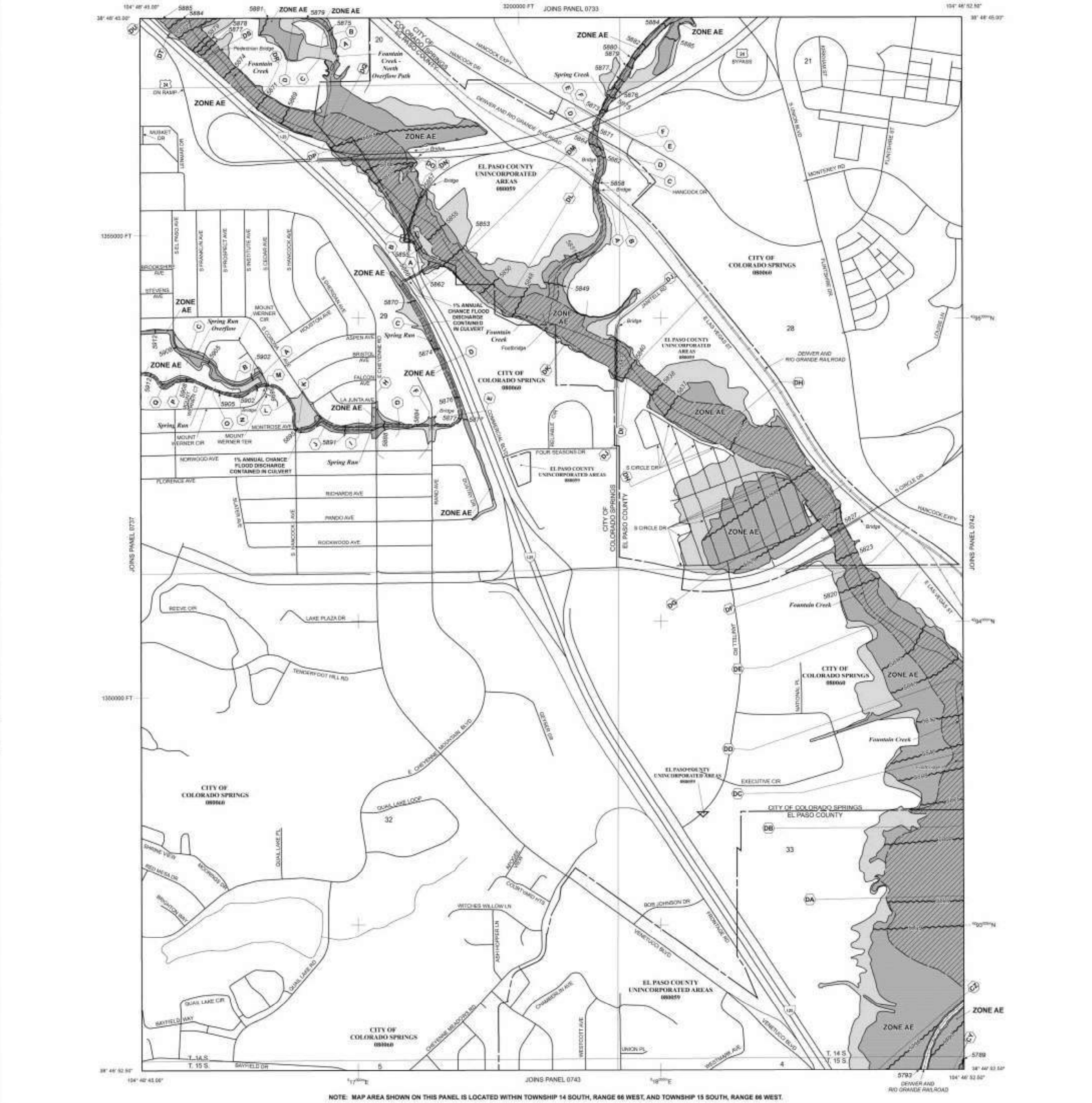
Flooding Source	Vertical Datum Offset (ft)
REFER TO SECTION 3.3 OF THE EL PASO COUNTY FLOOD INSURANCE STUDY FOR STREAM BY STREAM VERTICAL DATUM CONVERSION INFORMATION	

Panel Location Map



This Digital Flood Insurance Rate Map (DFIRM) was produced through a Cooperating Technical Partner (CTP) agreement between the State of Colorado Water Conservation Board (CWCB) and the Federal Emergency Management Agency (FEMA).

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



NOTE: MAP AREA SHOWN ON THIS PANEL IS LOCATED WITHIN TOWNSHIP 14 SOUTH, RANGE 66 WEST, AND TOWNSHIP 15 SOUTH, RANGE 66 WEST.

LEGEND

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

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood 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 Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

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

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 Areas.

	Floodplain boundary
	Floodway boundary
	Zone D Boundary
	CBRS and OPA boundary
	Boundary dividing Special Flood Hazard Areas of different Base 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)
	1000-meter Universal Transverse Mercator grid ticks, zone 13
6000000 FT	5000-foot grid ticks: Colorado State Plane coordinate system, central zone (FIPSZONE 0502), 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
MARCH 17, 1997

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL
DECEMBER 7, 2018 - to update corporate limits, to change Base Flood Elevations and Special Flood Hazard Areas, to update map format, to add roads and road names, and to incorporate previously issued Letters of Map Revision.

For community map revision history prior to countywide mapping, refer to the Community 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 insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

MAP SCALE 1" = 500'

250 0 500 1000 FEET

150 0 150 300 METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0741G

FIRM
FLOOD INSURANCE RATE MAP
EL PASO COUNTY, COLORADO
AND INCORPORATED AREAS

PANEL 741 OF 1300

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
COLORADO SPRINGS, CITY OF	080060	0741	G
EL PASO COUNTY	080059	0741	G

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

MAP NUMBER 08041C0741G

MAP REVISED DECEMBER 7, 2018

Federal Emergency Management Agency

Table 4. Summary of Discharges (cont.)

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (Cubic Feet Per Second) 10-Year	50-Year	100-Year	500-Year
Fountain Creek					
Near Colorado Springs, CO	103	1,300	2,800	3,700	6,500
At Colorado Springs, CO	392	7,900	14,300	18,000	29,400
At Janitell, CO	413	11,800	18,800	22,400	32,200
At Security, CO	495	11,900	21,000	25,800	39,500
At Fountain, CO	681	14,700	29,900	39,400	71,300
Near Pinon, CO	849	10,700	24,200	33,300	66,800
Near Pueblo, CO	926	14,400	30,200	39,600	70,000
Franceville Tributary to Jimmy Camp Creek					
At confluence with Jimmy Camp Creek	4.1	1,700	2,800	3,500	4,300
Haegler Ranch Tributary 1					
At Eastonville Road	0.96	--[1]	--[1]	80	--[1]
Haegler Ranch Tributary 1A					
At Eastonville Road	0.07	--[1]	--[1]	70	--[1]
Haegler Ranch Tributary 2					
At Eastonville Road	0.96	--[1]	--[1]	40	--[1]
Haegler Ranch Tributary 2					
At confluence with Geick Ranch West Tributary	1.47	--[1]	--[1]	592	--[1]
Haegler Ranch Tributary 3					
At approximately 2,300 feet upstream of the confluence with Haegler Ranch Tributary 4	1.09	--[1]	--[1]	505	--[1]
Haegler Ranch Tributary 4					
At approximately 3,700 feet upstream of the confluence with Haegler Ranch Tributary 3	0.60	--[1]	--[1]	130	--[1]
Jackson Creek					
At Assembly Road	2.44	--[1]	--[1]	1,313	--[1]

[1]Data not available

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER-SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
Fountain Creek (cont.)								
CS	133,500	504	3,076	8.3	5756.5	5756.5	5756.5	0.0
CT	134,500	311	2,259	11.3	5761.0	5761.0	5761.1	0.1
CU	135,500	404	1,829	14.0	5765.3	5765.3	5765.3	0.0
CV	136,215	450	2,504	10.2	5769.0	5769.0	5769.2	0.2
CW	137,191	303	3,547	7.2	5779.1	5779.1	5779.1	0.0
CX	138,368	410	2,200	11.6	5780.8	5780.8	5780.8	0.0
CY	139,949	344	1,949	11.5	5789.9	5789.9	5790.3	0.4
CZ	140,498	282	2,384	9.4	5796.8	5796.8	5796.8	0.0
DA	141,417	1,038	4,175	5.4	5800.2	5800.2	5800.3	0.1
DB	142,188	747	2,526	8.9	5802.4	5802.4	5802.5	0.1
DC	142,985	243	2,234	10.0	5809.5	5809.5	5809.8	0.3
DD	143,753	367	3,299	6.8	5814.3	5814.3	5815.1	0.8
DE	144,976	166	1,761	12.7	5817.0	5817.0	5817.9	0.9
DF	145,626	191	1,482	15.1	5821.3	5821.3	5821.3	0.0
DG	146,486	308	3,403	6.6	5831.2	5831.2	5831.2	0.0
DH	147,821	426	2,229	10.1	5834.3	5834.3	5834.3	0.0
DI	149,061	186	1,456	15.4	5839.4	5839.4	5839.4	0.0
DJ	149,360	374	4,250	5.3	5845.4	5845.4	5845.4	0.0
DK	149,839	335	2,253	9.9	5845.7	5845.7	5845.7	0.0
DL	150,705	160	1,571	11.5	5848.6	5848.6	5848.6	0.0
DM	151,374	215	1,991	9.0	5852.6	5852.6	5852.8	0.2
DN	152,135	340	2,433	7.4	5857.3	5857.3	5857.5	0.2
DO	152,434	327	1,412	12.8	5858.0	5858.0	5858.0	0.0
DP	152,978	754	3,736	4.8	5864.6	5864.6	5864.8	0.2

[1]Feet Above El Paso – Pueblo County Line.

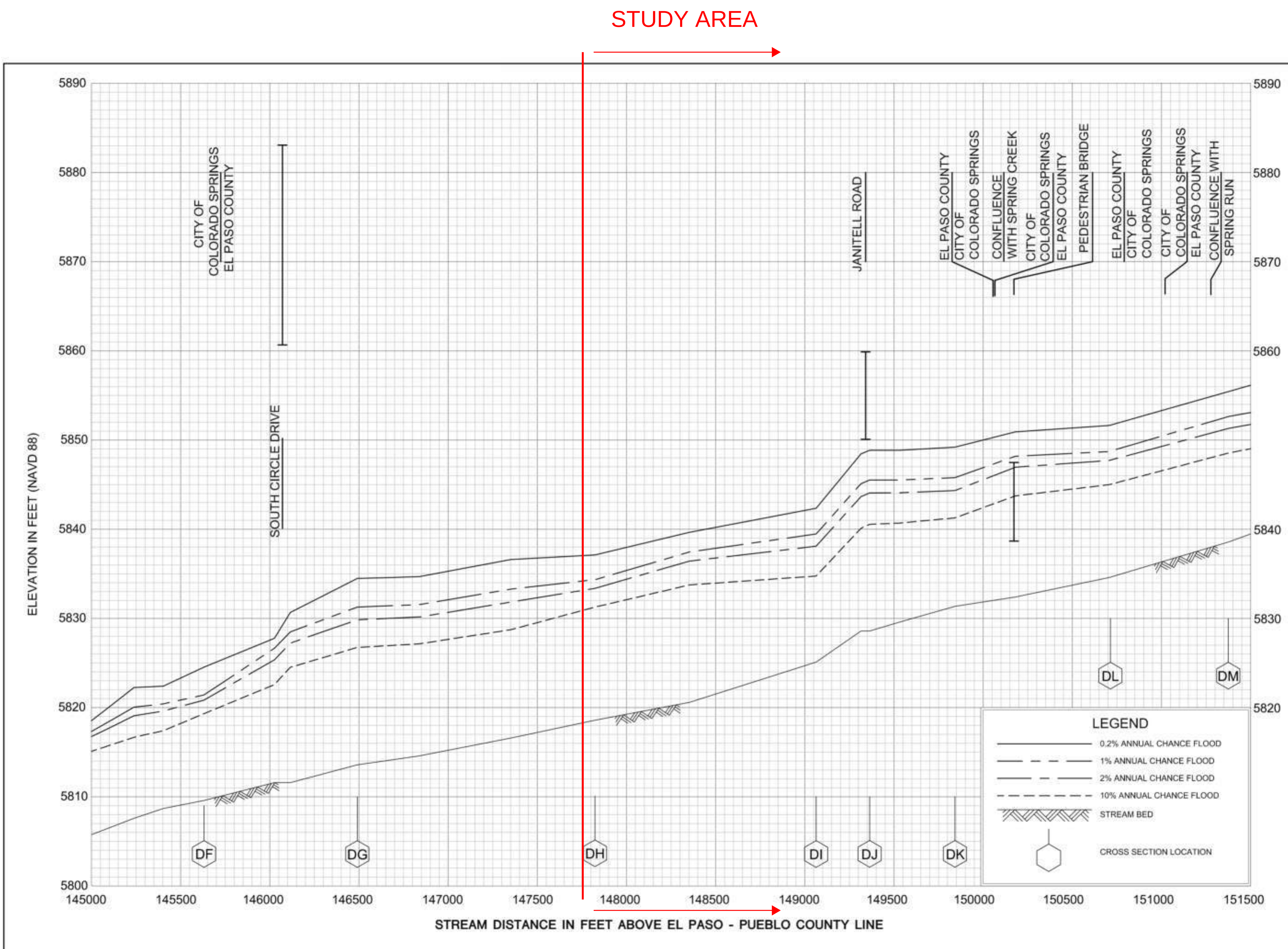
TABLE 8

FEDERAL EMERGENCY MANAGEMENT AGENCY

EL PASO COUNTY, CO
AND INCORPORATED AREAS

FLOODWAY DATA

FOUNTAIN CREEK



FLOOD PROFILES

FOUNTAIN CREEK

FEDERAL EMERGENCY MANAGEMENT AGENCY

EL PASO COUNTY, CO

(AND INCORPORATED AREAS)

Table 7. Vertical Datum Conversion Factors (cont.)

Stream/Reach	Minimum Conversion	Maximum Conversion	Average Conversion	Maximum Offset	Downstream Station	Upstream Station
Dry Creek	3.5	3.8	3.6	0.1	Entire Reach	
East Cherry Creek	4.1	4.1	4.1	0.0	Entire Reach	
East Tributary to Black Squirrel Creek	3.9	3.9	3.9	0.0	Entire Reach	
East Tributary to Black Squirrel Creek – West Fork Bennett Ranch Basin	3.5	3.8	3.6	0.2	Entire Reach	
Ellicott Consolidated[1]	N/A	N/A	N/A	N/A		
Ellicott Consolidated – East Tributary[1]	N/A	N/A	N/A	N/A		
Fairfax Creek	3.6	4.0	3.8	0.1	Entire Reach	
Fisher's Canyon	3.1	3.5	3.3	0.2	Entire Reach	
Fisher's Canyon – Above Loomis Avenue	3.1	3.5	3.3	0.2	Entire Reach	
Fisher's Canyon – South Branch	3.1	3.5	3.3	0.2	Entire Reach	
Fountain Creek (Lower)	3.1	3.5	3.3	0.2	El Paso – Pueblo County Boundary	Downstream of Sand Creek
Fountain Creek (Middle)	3.4	3.7	3.5	0.2	Upstream of Sand Creek	Downstream of Monument Creek
Franceville Tributary – Jimmy Camp Creek	3.3	3.5	3.4	0.2	Entire Reach	
Haegler Ranch Tributary 2 (Pond D and E)	3.5	3.8	3.6	0.2	Entire Reach	

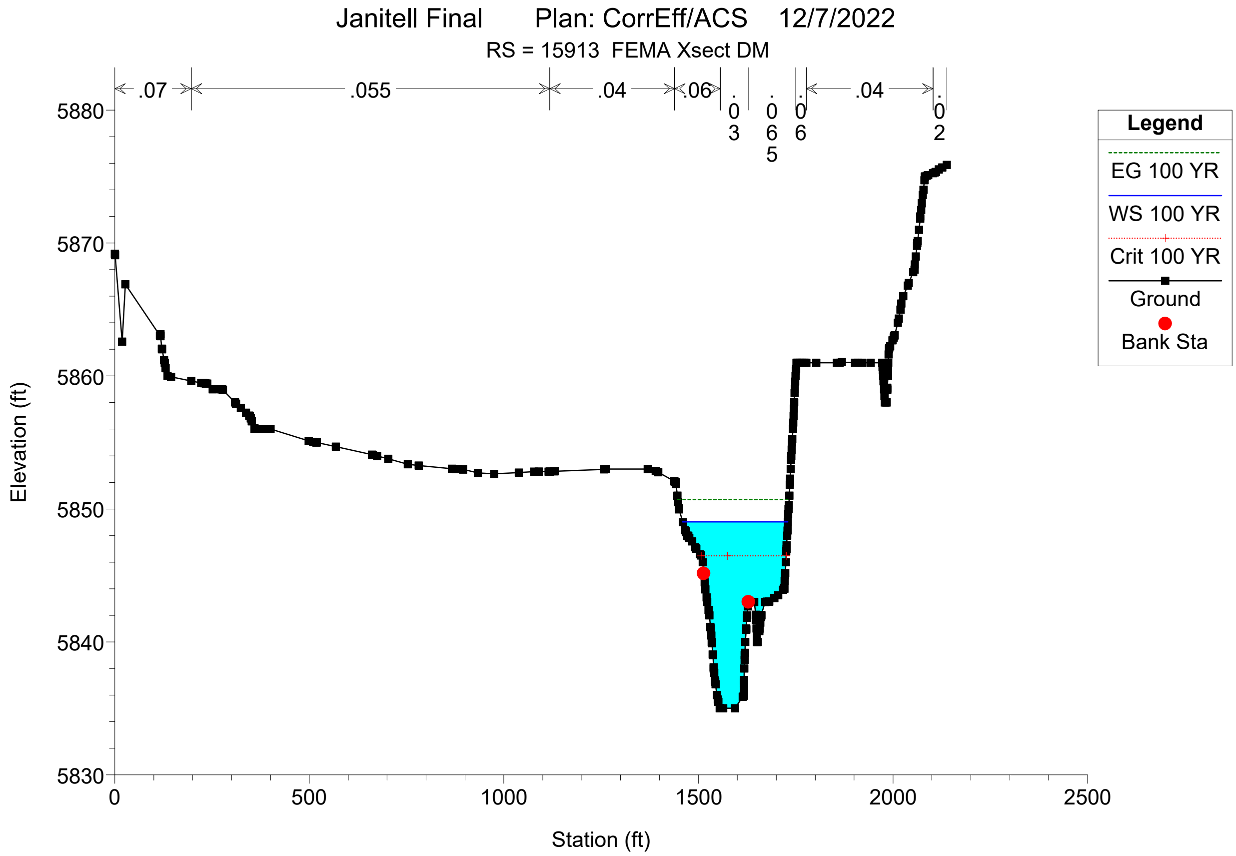
[1] Detailed study performed in NAVD88.
[2] Stream not included as part of map modernization; vertical datum is NGVD29.
[3] Downstream portion of stream not included as part of map modernization; vertical datum is NGVD29.
[4] The portion of Monument Creek between Cross Section O and downstream of Cross Section AW not included as part of map modernization; vertical datum is NGVD29.
[5] Upstream portion of stream not included as part of map modernization; vertical datum is NGVD29.

7.HEC-RAS Model Tables and Cross Sections

HEC-RAS Output table
All Plans
100-Year Profile

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)	
Main DS	15913	100 YR	FC_Mar 201	18000	5835	5849.07	5846.48	5850.73	0.004031	11.15	2032.61	270.62	0.58
Main DS	15913	100 YR	CORR	18000	5835	5849.05	5846.48	5850.72	0.004052	11.17	2028.38	270.44	0.58
Main DS	15913	100 YR	CorrEffACS	18000	5835	5849.03	5846.48	5850.7	0.004087	11.21	2021.65	270.14	0.58
Main DS	15913	100 YR	CorrFinal	18000	5835	5849.03	5846.48	5850.7	0.004086	11.21	2021.78	270.15	0.58
Main DS	15241	100 YR	FC_Mar 201	18000	5831	5845.13	5842.64	5847.03	0.007817	11.19	1749.88	304.27	0.63
Main DS	15241	100 YR	CORR	18000	5831	5845.87	5842.64	5847.41	0.00592	10.17	1977.11	306.86	0.55
Main DS	15241	100 YR	CorrEffACS	18000	5831	5845.6	5842.64	5847.26	0.006544	10.53	1893.78	305.92	0.58
Main DS	15241	100 YR	CorrFinal	18000	5831	5845.5	5842.64	5847.21	0.006795	10.67	1862.88	305.57	0.59
Main DS	14715	100 YR	FC_Mar 201	18000	5828.82	5844.6	5838.97	5845.1	0.001529	6.33	3749.36	571.53	0.32
Main DS	14715	100 YR	CORR	18000	5828.82	5845.64	5838.97	5846	0.001041	5.51	4350.81	586.39	0.27
Main DS	14715	100 YR	CorrEffACS	18000	5828.82	5845.29	5838.97	5845.69	0.001181	5.77	4146.75	582.51	0.28
Main DS	14715	100 YR	CorrFinal	18000	5828.82	5845.15	5838.97	5845.57	0.001243	5.88	4067.22	580.89	0.29
Main DS	14512	100 YR	CorrEffACS	18000	5825.74	5844.89	5837.38	5845.39	0.000622	6.13	3533.8	379.84	0.28
Main DS	14512	100 YR	CorrFinal	18000	5825.74	5844.89	5837.38	5845.39	0.000622	6.13	3533.8	379.84	0.28
Main DS	14376	100 YR	FC_Mar 201	22400	5827.77	5842.2	5839.21	5844.08	0.004324	11.35	2318.62	392.72	0.59
Main DS	14376	100 YR	CORR	22400	5827.77	5840.66	5840.66	5844.8	0.006861	16.37	1399.54	189.48	0.96
Main DS	14376	100 YR	CorrEffACS	22400	5827.77	5840.66	5840.66	5844.8	0.006861	16.37	1399.54	189.48	0.96
Main DS	14376	100 YR	CorrFinal	22400	5827.77	5840.66	5840.66	5844.8	0.006861	16.37	1399.54	189.48	0.96
Main DS	14235	100 YR	CorrEffACS	22400	5823.51	5838.36	5835.96	5840.46	0.003987	11.69	1956.23	227.47	0.66
Main DS	14235	100 YR	CorrFinal	22400	5823.51	5837.59	5835.96	5840.08	0.005213	12.72	1788.72	211.7	0.74
Main DS	14065	100 YR	FC_Mar 201	22400	5826	5841.91	5836.46	5842.91	0.002086	8	2801.07	285.52	0.42
Main DS	14065	100 YR	CORR	22400	5822.38	5839.61	5834.97	5840.72	0.001741	8.46	2649.01	253.49	0.46
Main DS	14065	100 YR	CorrEffACS	22400	5822.38	5838.25	5834.97	5839.72	0.002693	9.7	2309.3	249.01	0.56
Main DS	14065	100 YR	CorrFinal	22400	5822.57	5837.53	5834.13	5839.07	0.003587	9.95	2250.92	245.98	0.58
Main DS	13989	100 YR	CorrEffACS	22400	5822.5	5837.85	5834.87	5839.5	0.001606	11.84	2561.3	282.97	0.55
Main DS	13989	100 YR	CorrFinal	22400	5822.69	5837.49	5833.48	5838.79	0.001412	10.46	2609.72	280.25	0.5
Main DS	13898	100 YR	FC_Mar 201	22400	5825	5841.93	5834.84	5842.38	0.001157	5.49	4249.09	376.15	0.28
Main DS	13898	100 YR	CORR	22400	5822.29	5839.83	5832.85	5840.23	0.000435	5.11	4488.1	412.12	0.26
Main DS	13898	100 YR	CorrEffACS	22400	5822.29	5838.56	5832.85	5839.07	0.00064	5.75	3969.86	403.77	0.31
Main DS	13898	100 YR	CorrFinal	22400	5822.52	5837.83	5833.82	5838.5	0.000814	8.56	3565.25	398.35	0.39
Main DS	13870			Bridge									
Main DS	13849	100 YR	FC_Mar 201	22400	5825	5841.52	5834.49	5841.97	0.001212	5.39	4183.93	370.34	0.28
Main DS	13849	100 YR	CORR	22400	5821.81	5839.06	5832.14	5839.46	0.010435	5.04	4429.92	414.32	0.26
Main DS	13849	100 YR	CorrEffACS	22400	5821.81	5837.72	5832.14	5838.24	0.016069	5.76	3884.32	403.77	0.32
Main DS	13849	100 YR	CorrFinal	22400	5821.75	5837.34	5832.85	5838.04	0.001146	7.02	3520.97	400.3	0.37
Main DS	13782	100 YR	CorrEffACS	22400	5818.63	5837.2	5831.62	5837.98	0.001064	7.43	3367.93	351.75	0.37
Main DS	13782	100 YR	CorrFinal	22400	5820.12	5837.2	5832.67	5837.96	0.000901	7.06	3263.15	350.7	0.39
Main DS	13598	100 YR	FC_Mar 201	22400	5821.52	5835.89	5835.89	5840.25	0.005246	17.39	1456.65	189.17	0.85
Main DS	13598	100 YR	CORR	22400	5819.12	5835.34	5831.89	5837.52	0.003475	11.85	1890.58	170.24	0.63
Main DS	13598	100 YR	CorrEffACS	22400	5819.12	5835.34	5831.89	5837.52	0.003475	11.85	1890.58	170.24	0.63
Main DS	13598	100 YR	CorrFinal	22400	5819.12	5835.34	5831.89	5837.52	0.003475	11.85	1890.58	170.24	0.63
Main DS	12887	100 YR	FC_Mar 201	22400	5817	5833.86	5828.25	5834.87	0.003018	8.19	2939.44	588.15	0.42
Main DS	12887	100 YR	CORR	22400	5817	5833.86	5828.25	5834.87	0.003018	8.19	2939.44	588.15	0.42
Main DS	12887	100 YR	CorrEffACS	22400	5817	5833.86	5828.25	5834.87	0.003018	8.19	2939.44	588.15	0.42
Main DS	12887	100 YR	CorrFinal	22400	5817	5833.86	5828.25	5834.87	0.003018	8.19	2939.44	588.15	0.42
Main DS	12358	100 YR	FC_Mar 201	22400	5815	5830.76	5827.87	5832.66	0.00562	11.23	2229.73	425.8	0.61
Main DS	12358	100 YR	CORR	22400	5815	5830.76	5827.87	5832.66	0.00562	11.23	2229.73	425.8	0.61
Main DS	12358	100 YR	CorrEffACS	22400	5815	5830.76	5827.87	5832.66	0.00562	11.23	2229.73	425.8	0.61
Main DS	12358	100 YR	CorrFinal	22400	5815	5830.76	5827.87	5832.66	0.00562	11.23	2229.73	425.8	0.61

Corrected Effective with Additional Cross Sections



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WS 100 YR
Crit 100 YR
Ground
Bank Sta
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Station (ft)
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5880
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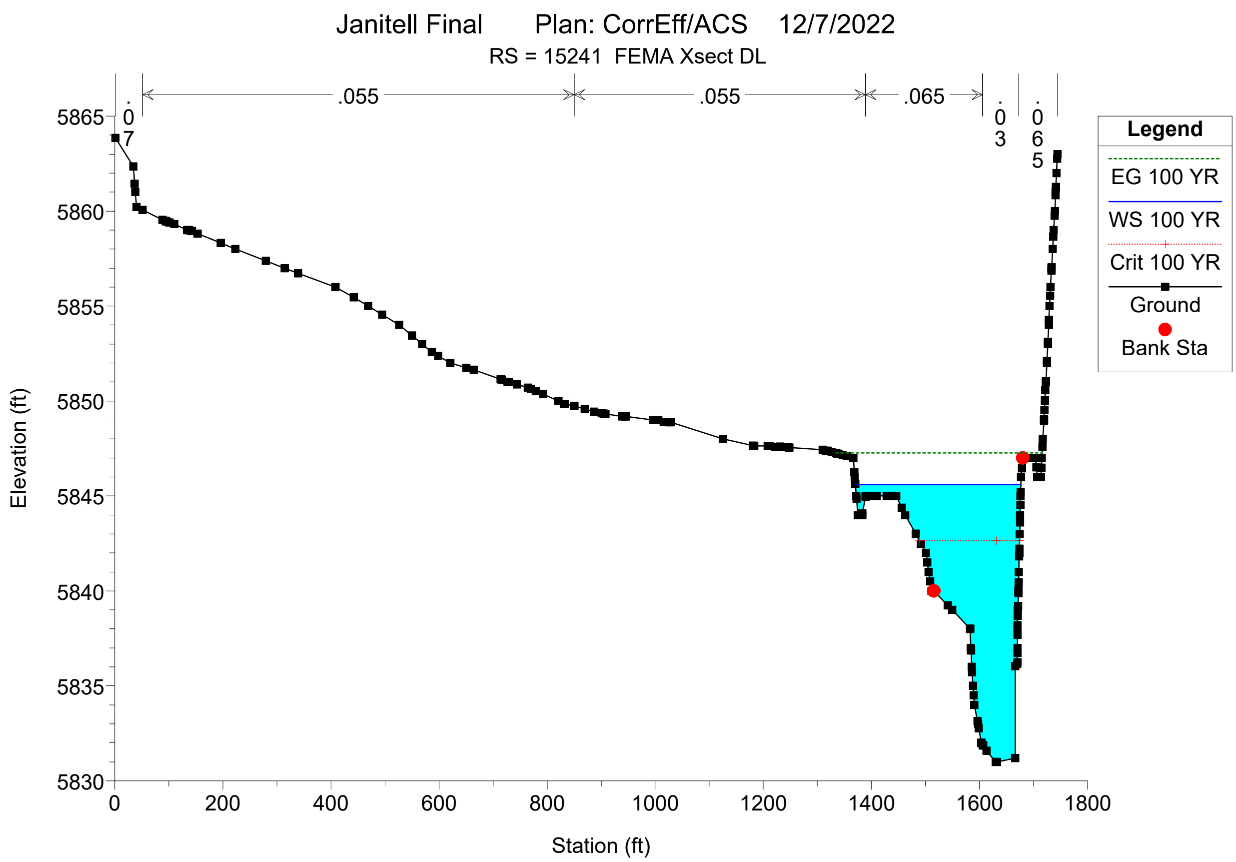
1500
2000
2500



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Crit 100 YR
Ground
Bank Sta
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Station (ft)
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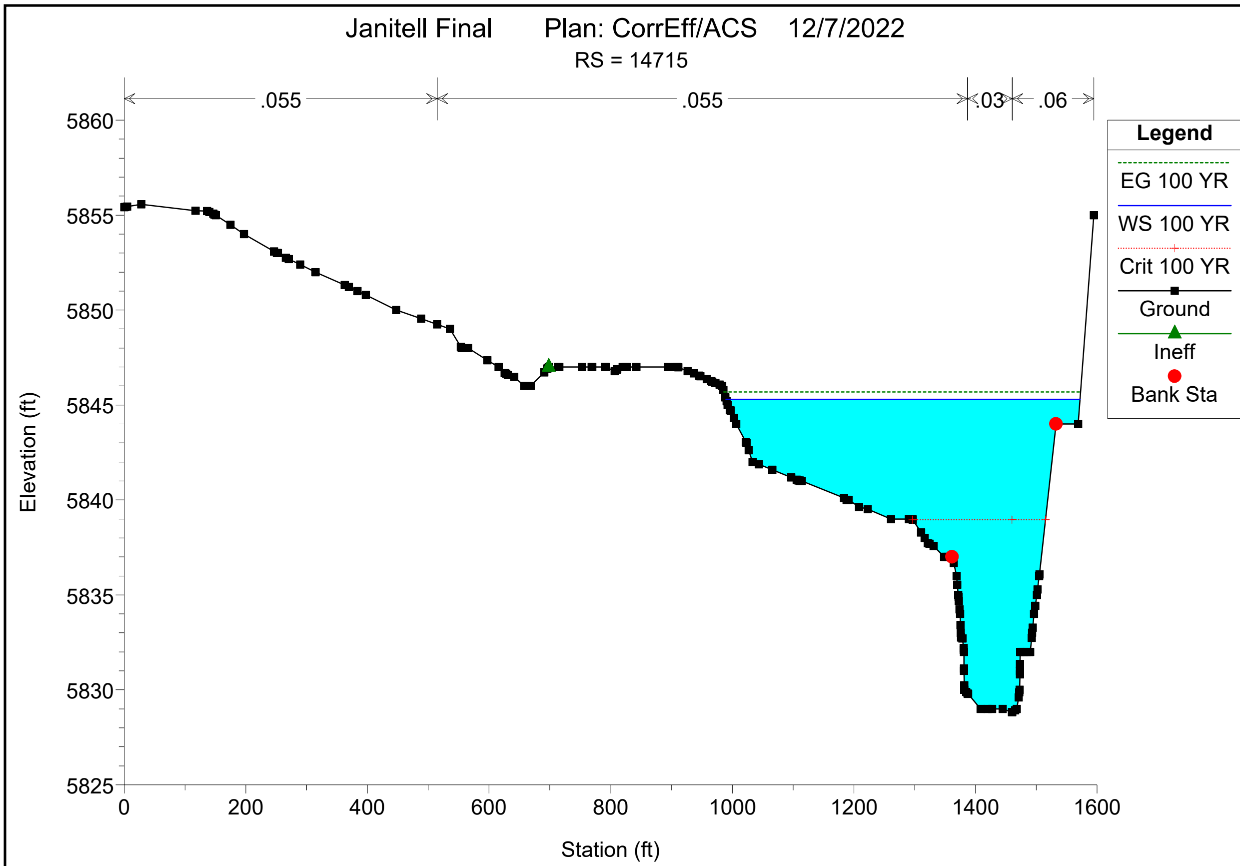
1600
1800



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WS 100 YR
Crit 100 YR
Ground
Ineff
Bank Sta
Elevation (ft)
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5830
5825
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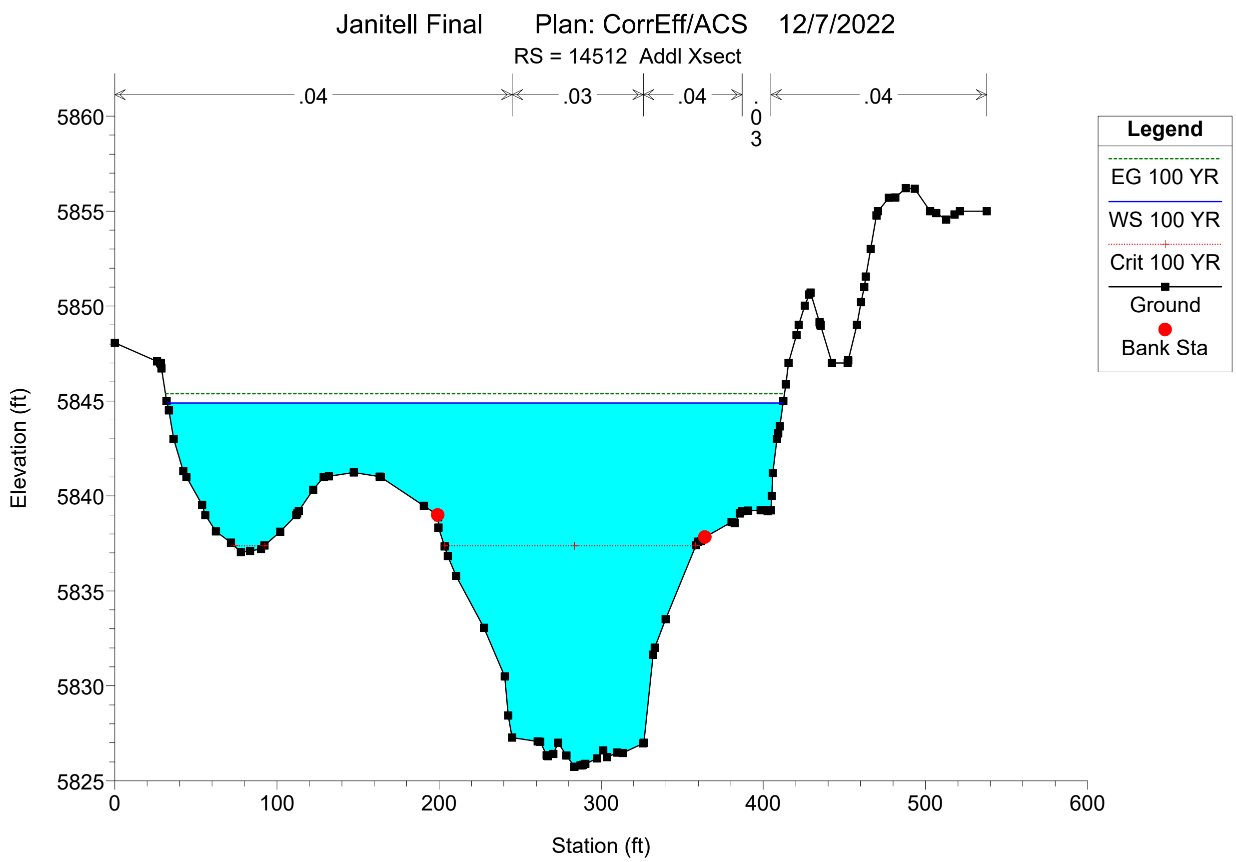
200
400
600
800
1000
1200
1400
1600
Station (ft)



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3
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WS 100 YR
Crit 100 YR
Ground
Bank Sta
Elevation (ft)
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5855
5850
5845
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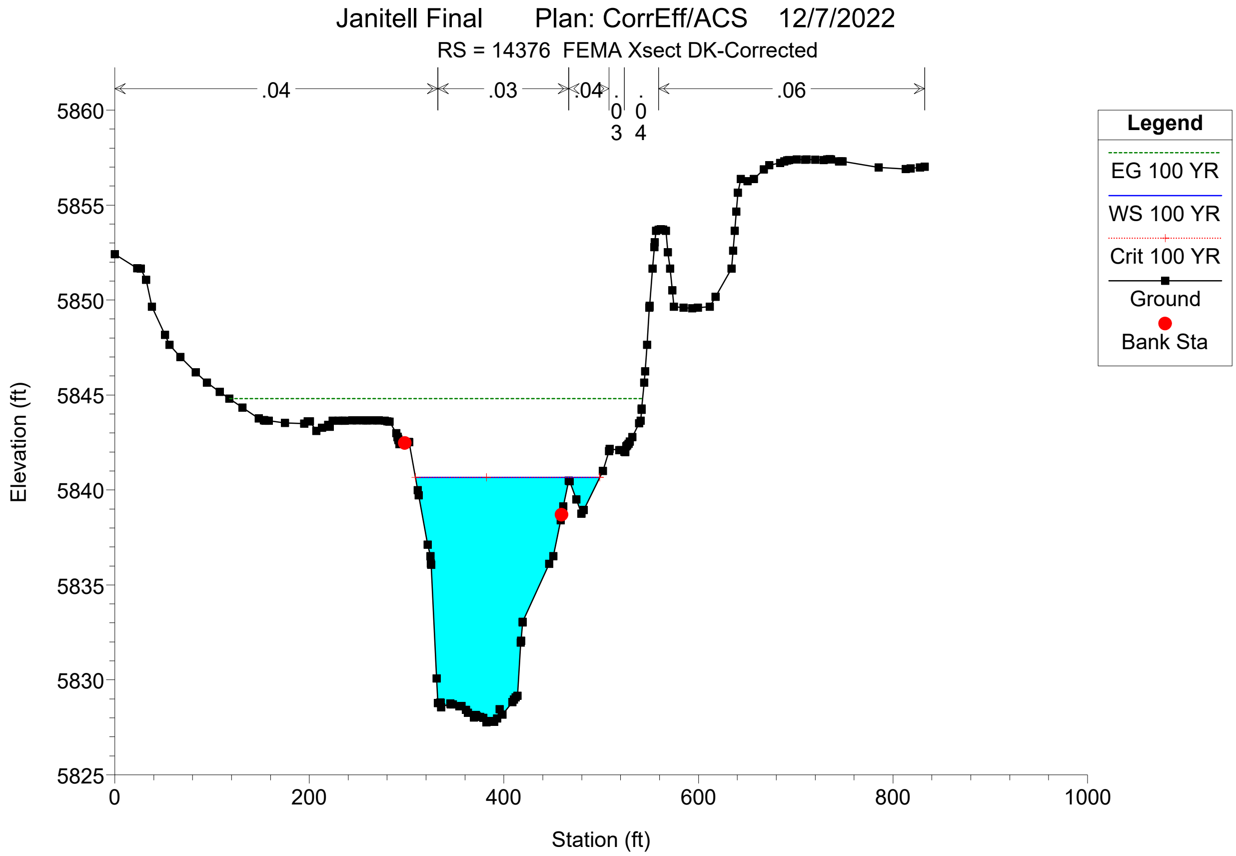
100
200
300
400
500
600
Station (ft)



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.03
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WS 100 YR
Crit 100 YR
Ground
Bank Sta
Elevation (ft)
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Station (ft)
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600
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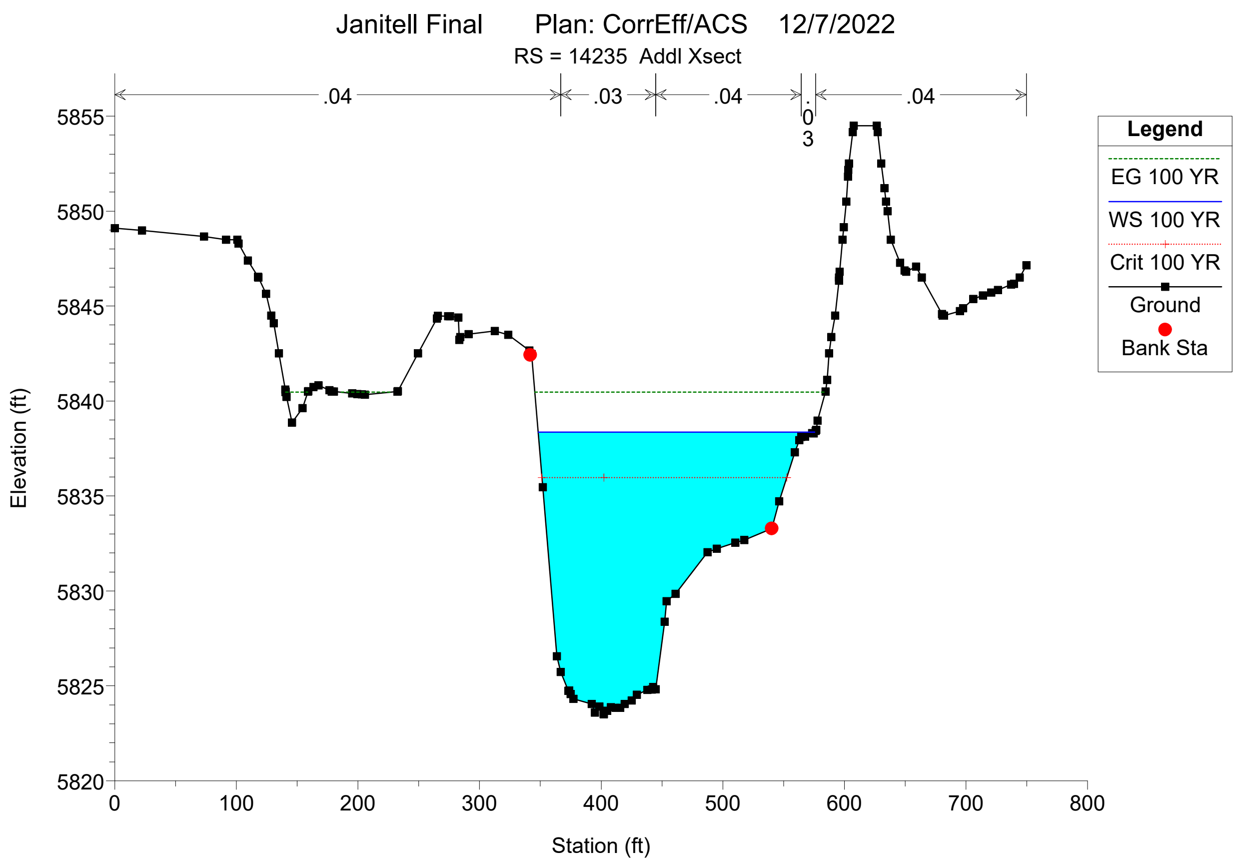

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Crit 100 YR
Ground
Bank Sta
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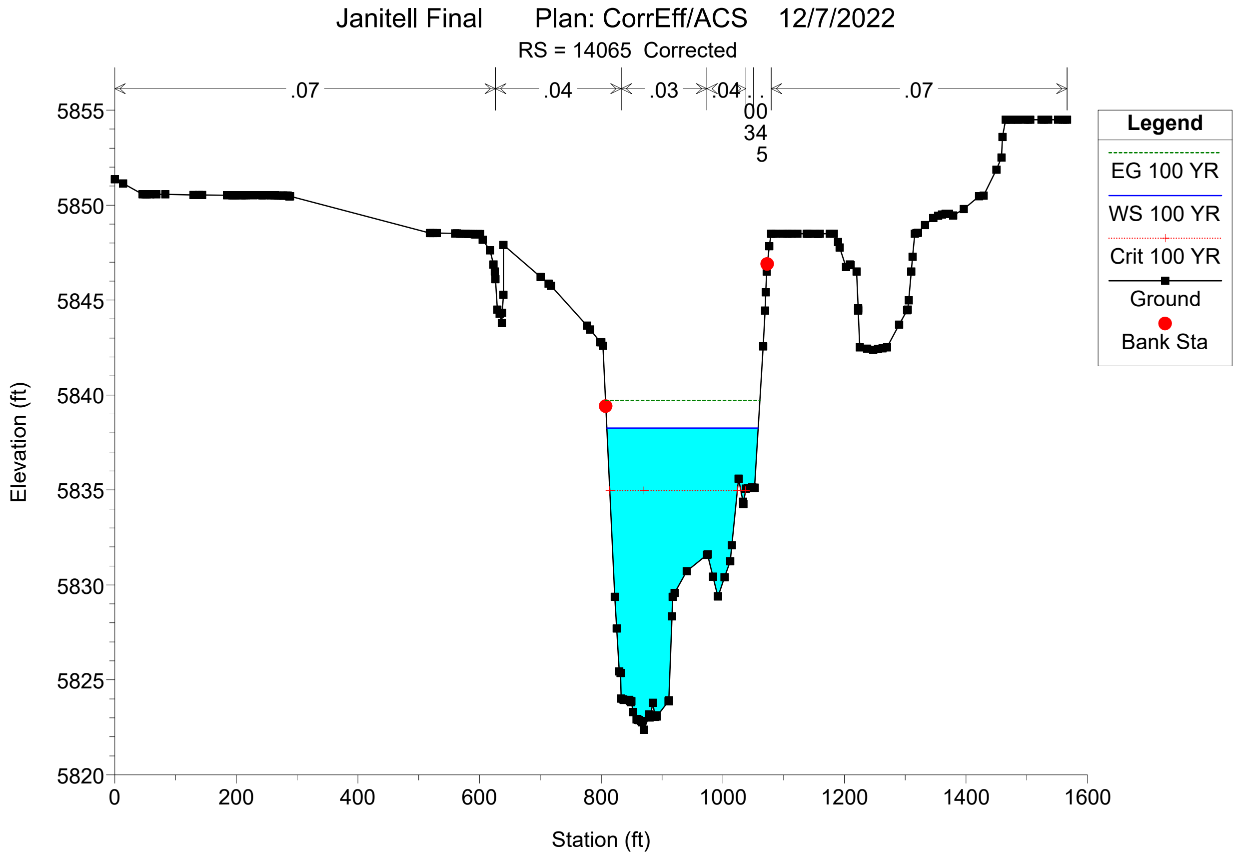
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Legend
EG 100 YR
WS 100 YR
Crit 100 YR
Ground
Bank Sta
Elevation (ft)
Station (ft)
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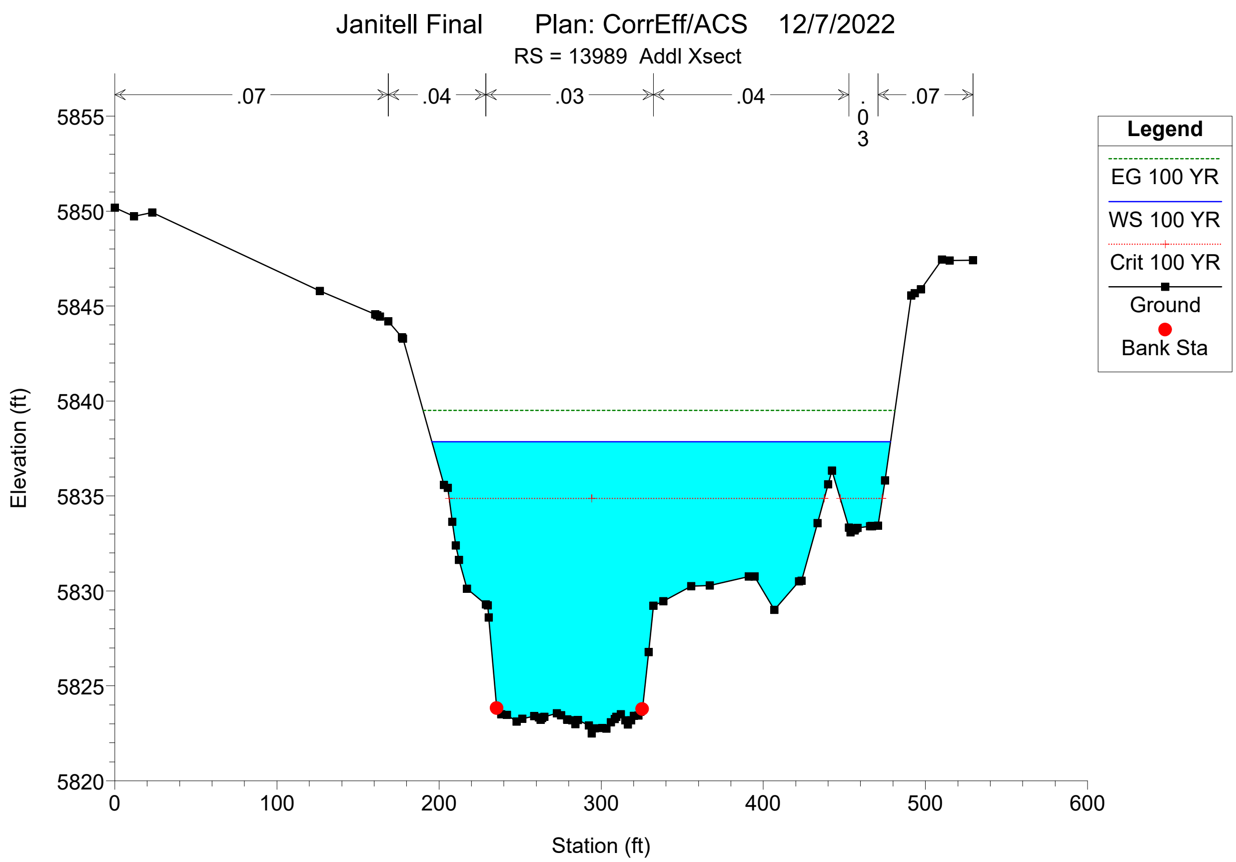
1200
1400
1600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WS 100 YR
Crit 100 YR
Ground
Bank Sta
Elevation (ft)
Station (ft)
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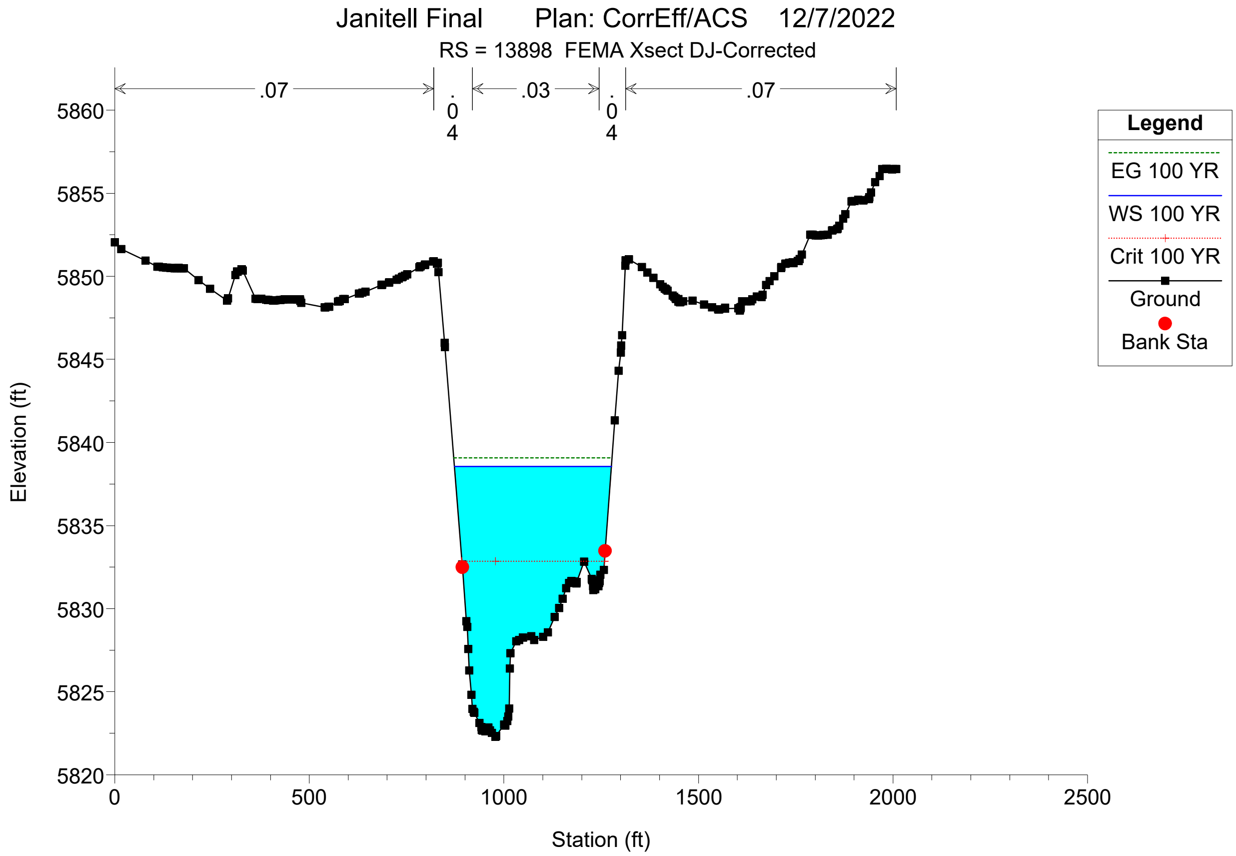
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Legend
EG 100 YR
WS 100 YR
Crit 100 YR
Ground
Bank Sta
Elevation (ft)
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5855
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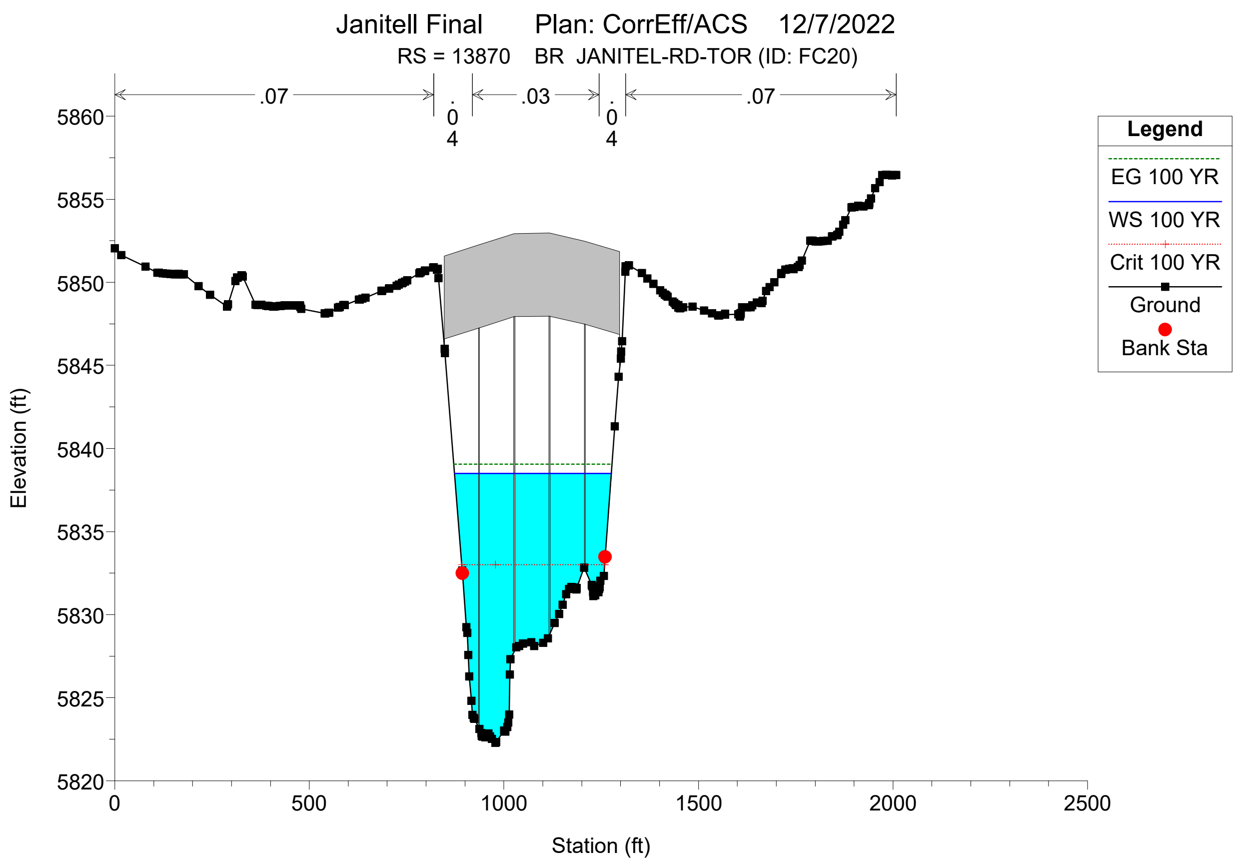
0
500
1000
1500
2000
2500
Station (ft)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Crit 100 YR
Ground
Bank Sta
Elevation (ft)
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5825
5830
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5840
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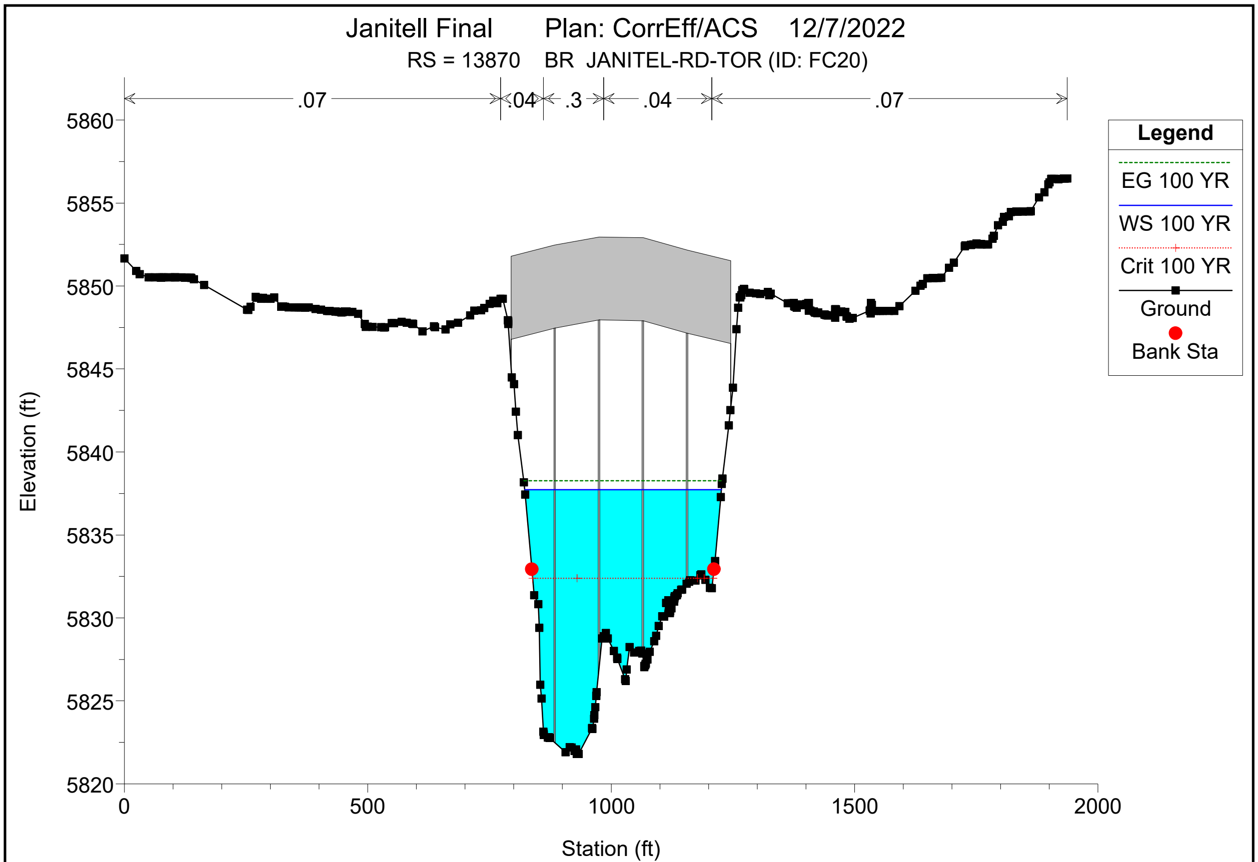
Station (ft)



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Legend
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Crit 100 YR
Ground
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Elevation (ft)
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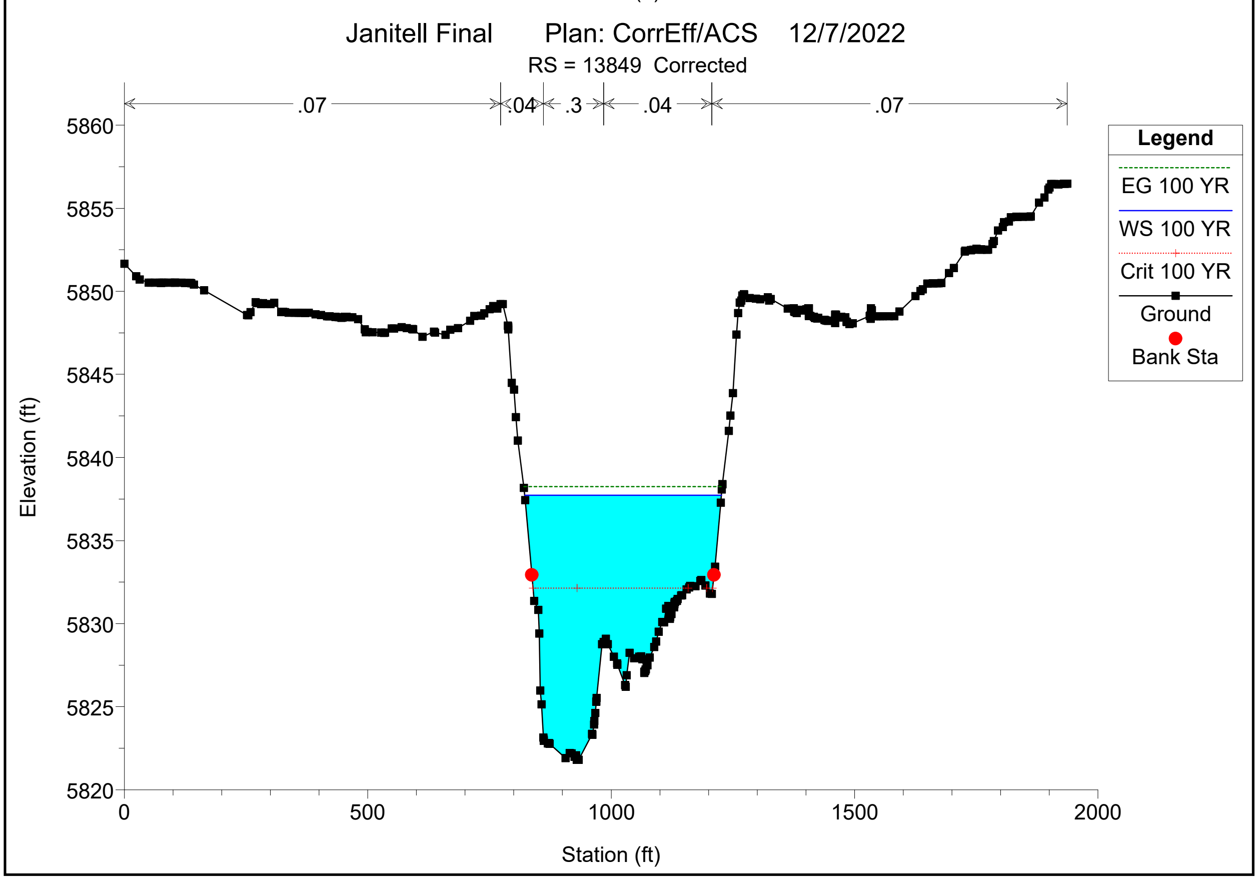
Station (ft)



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WS 100 YR
Crit 100 YR
Ground
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Elevation (ft)
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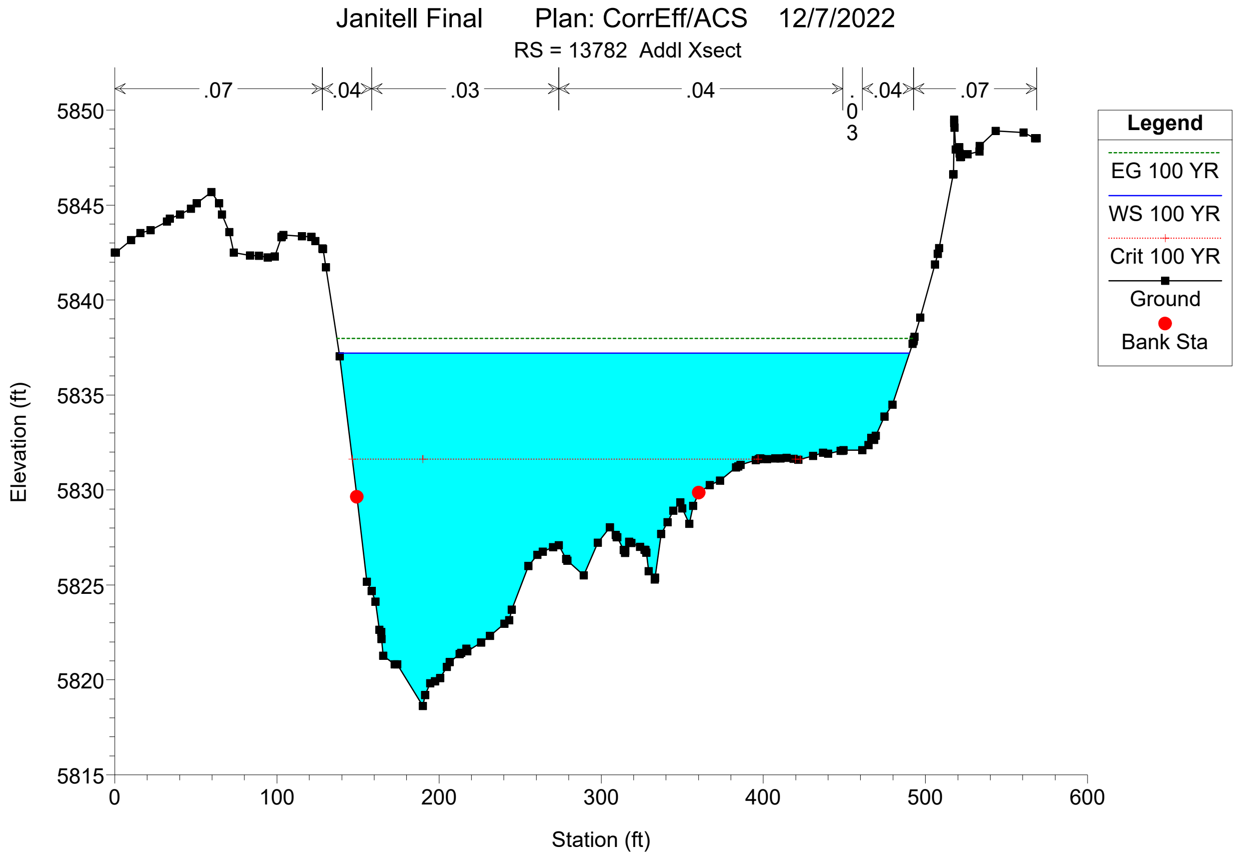
Station (ft)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Crit 100 YR
Ground
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Elevation (ft)
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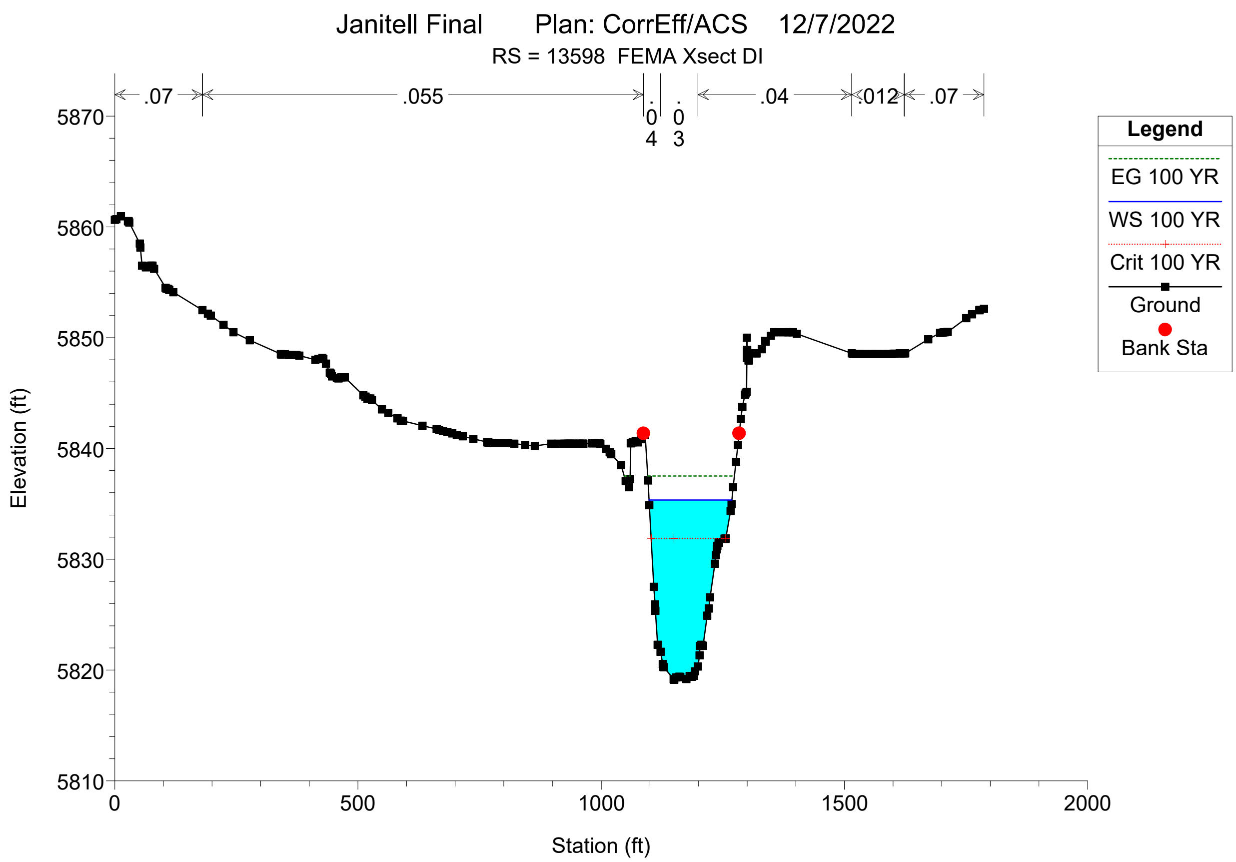
Station (ft)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Legend
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WS 100 YR
Crit 100 YR
Ground
Bank Sta
Elevation (ft)
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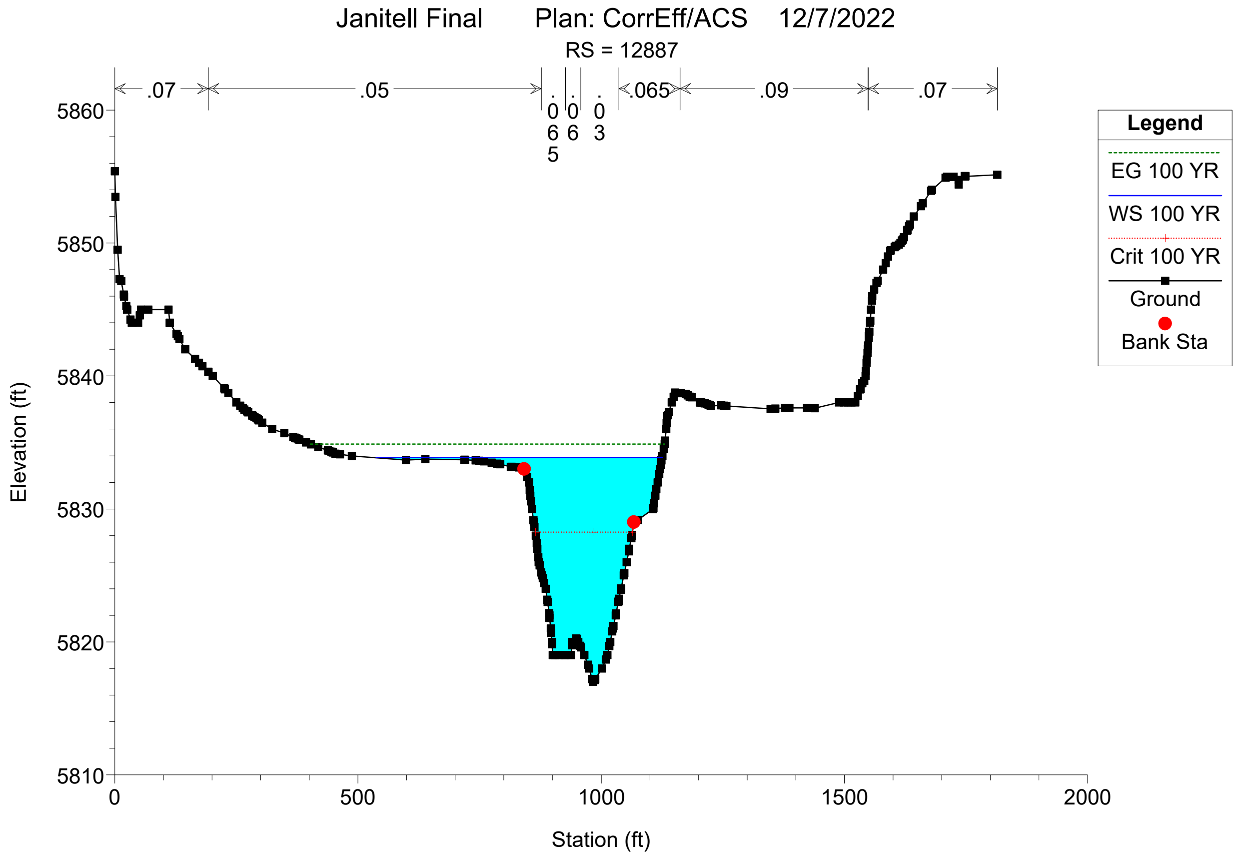
Station (ft)



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Legend
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WS 100 YR
Crit 100 YR
Ground
Bank Sta
Elevation (ft)
Station (ft)
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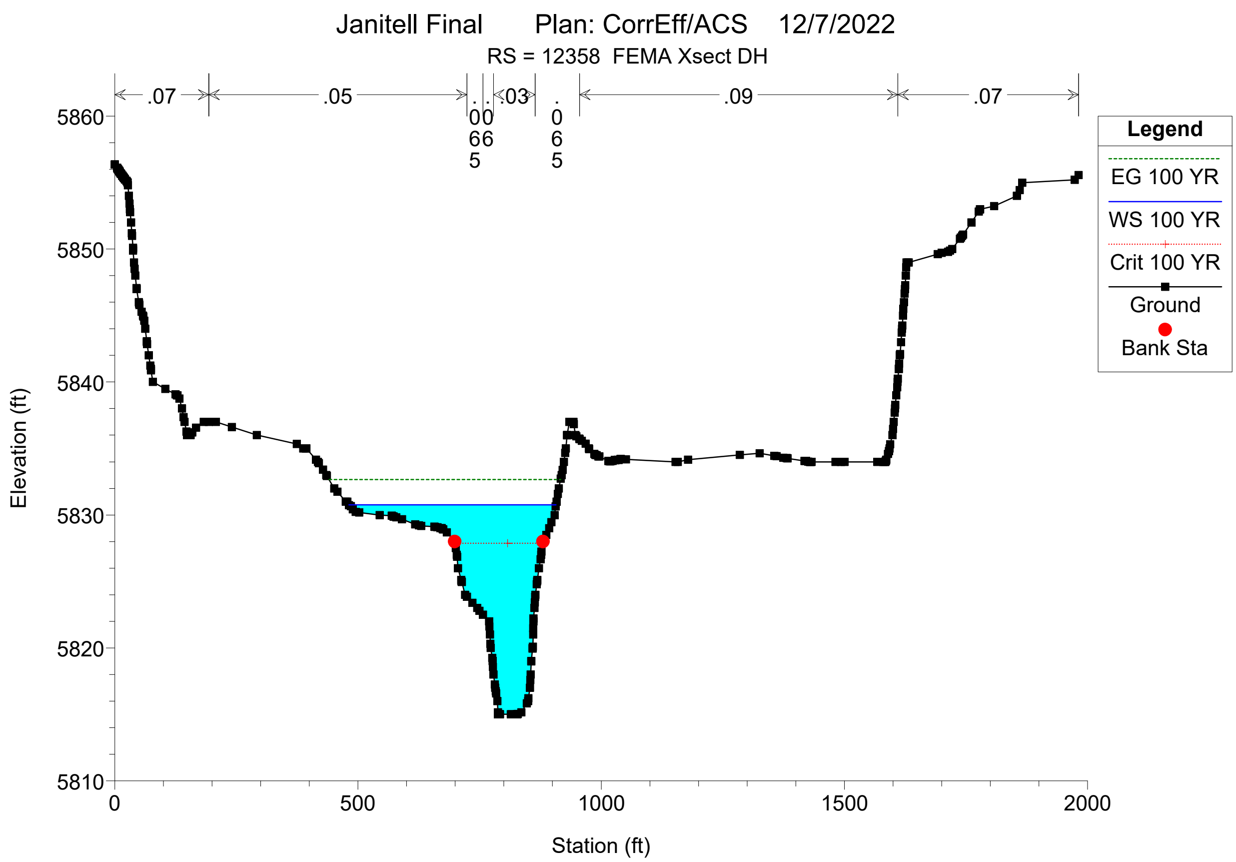

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Legend
EG 100 YR
WS 100 YR
Crit 100 YR
Ground
Bank Sta
Elevation (ft)
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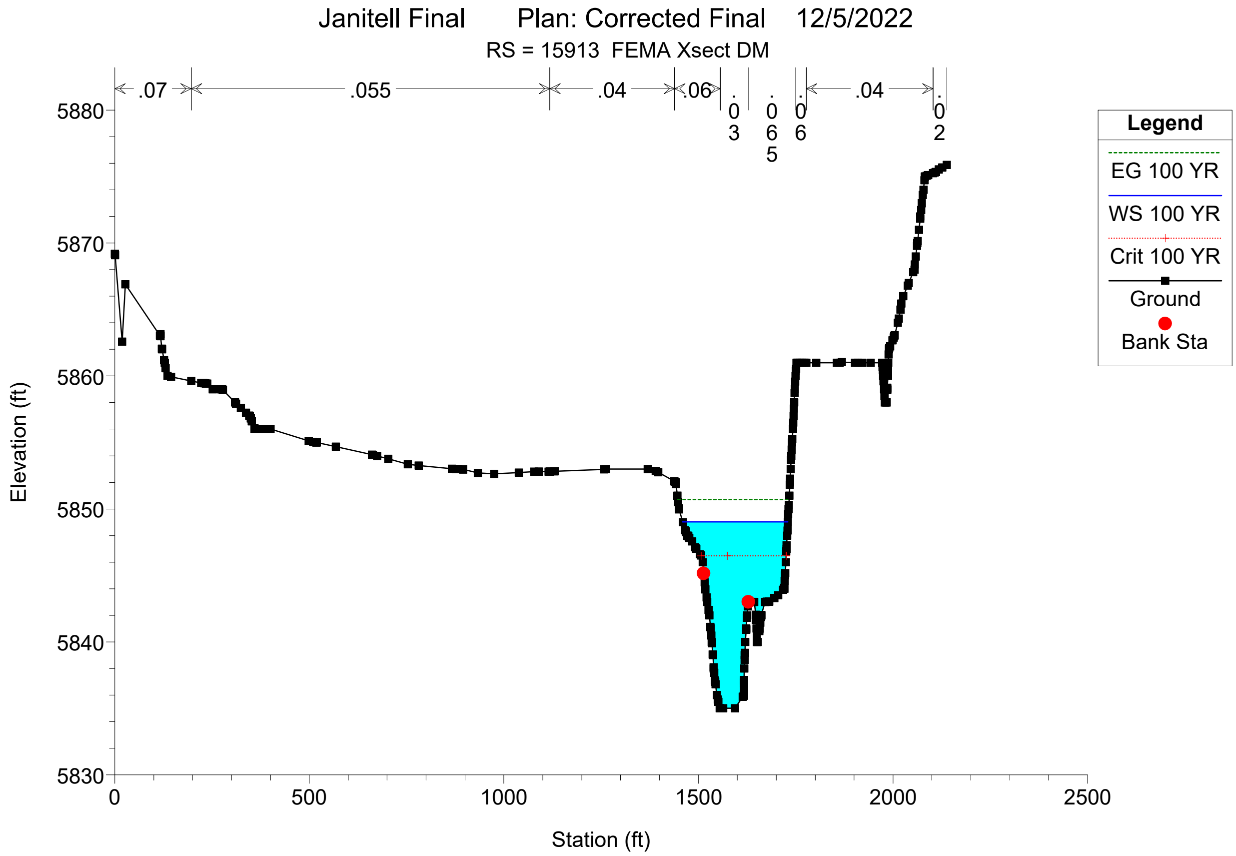
Corrected Final



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WS 100 YR
Crit 100 YR
Ground
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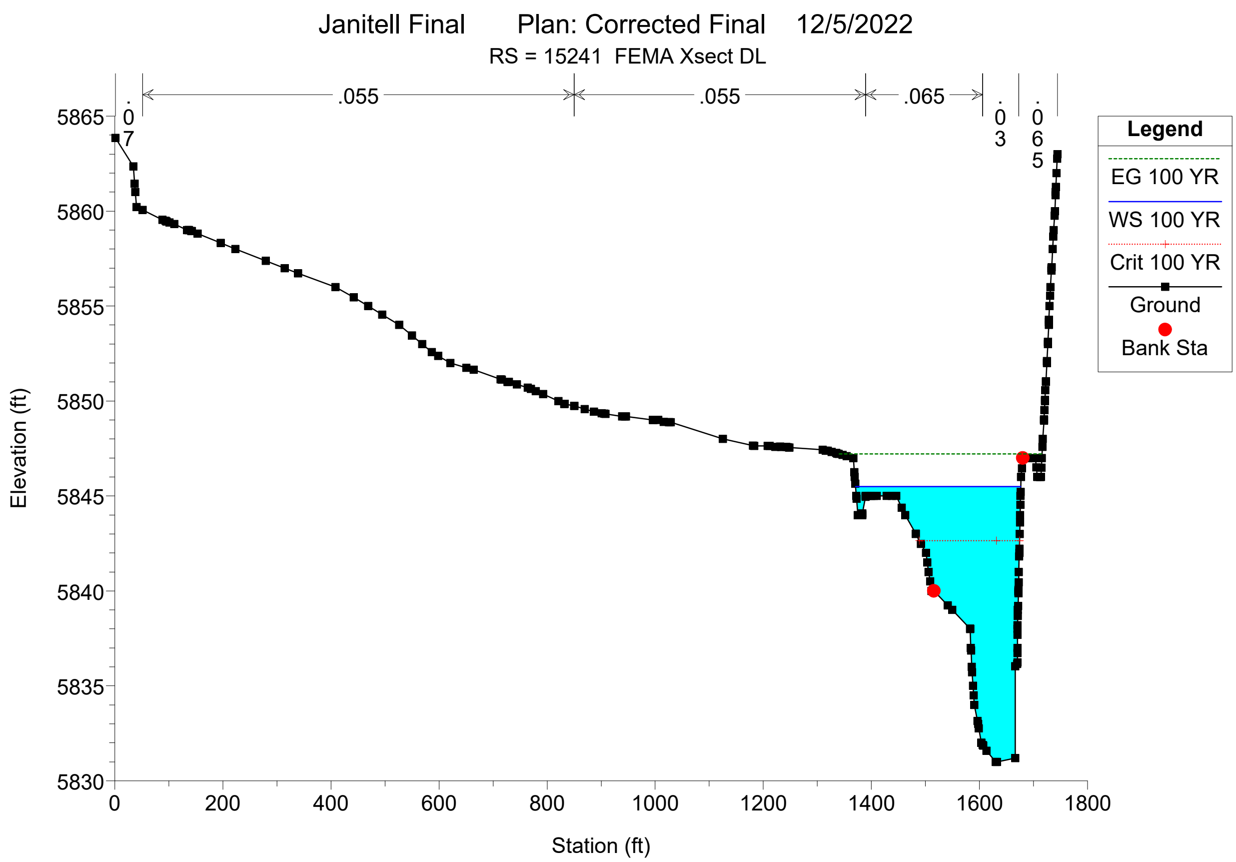
Elevation (ft)
Station (ft)



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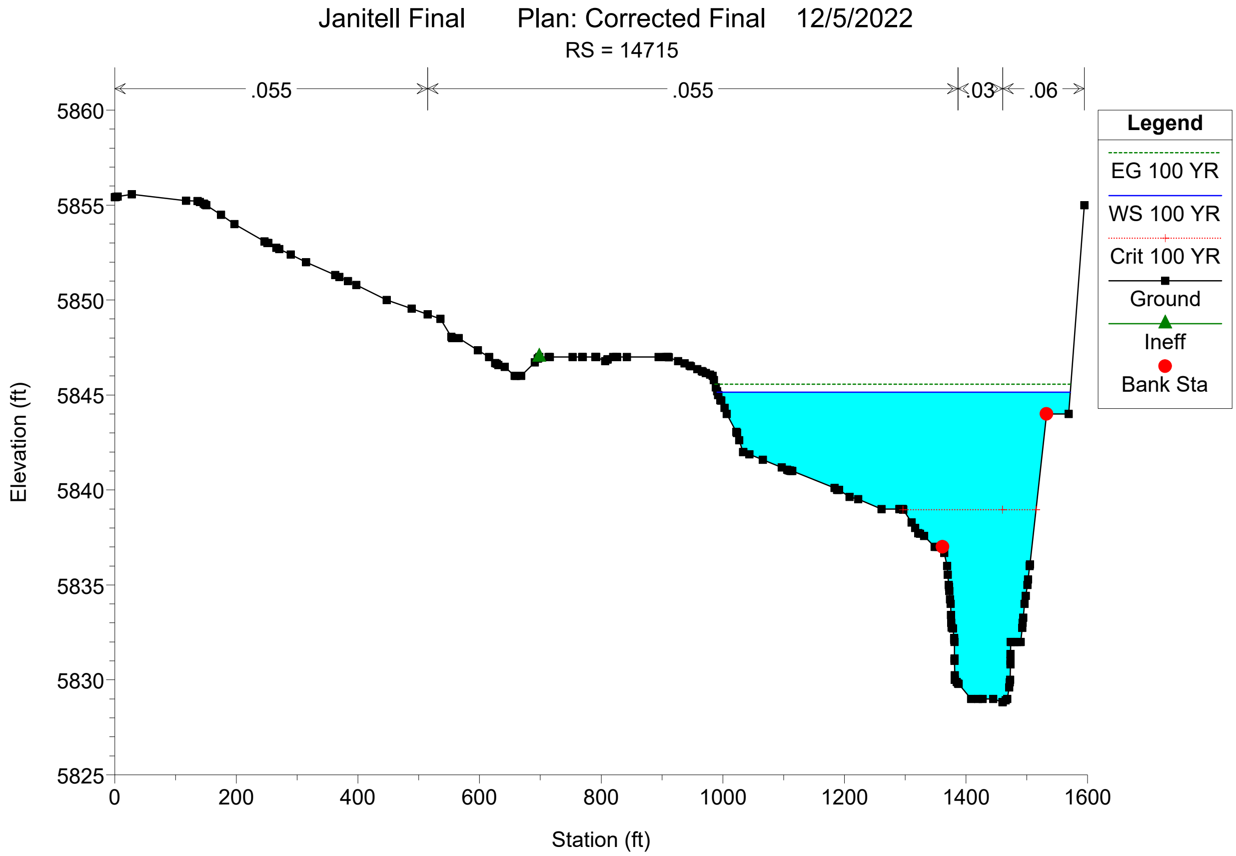
Ground
Bank Sta
Elevation (ft)
Station (ft)



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Crit 100 YR
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Elevation (ft)
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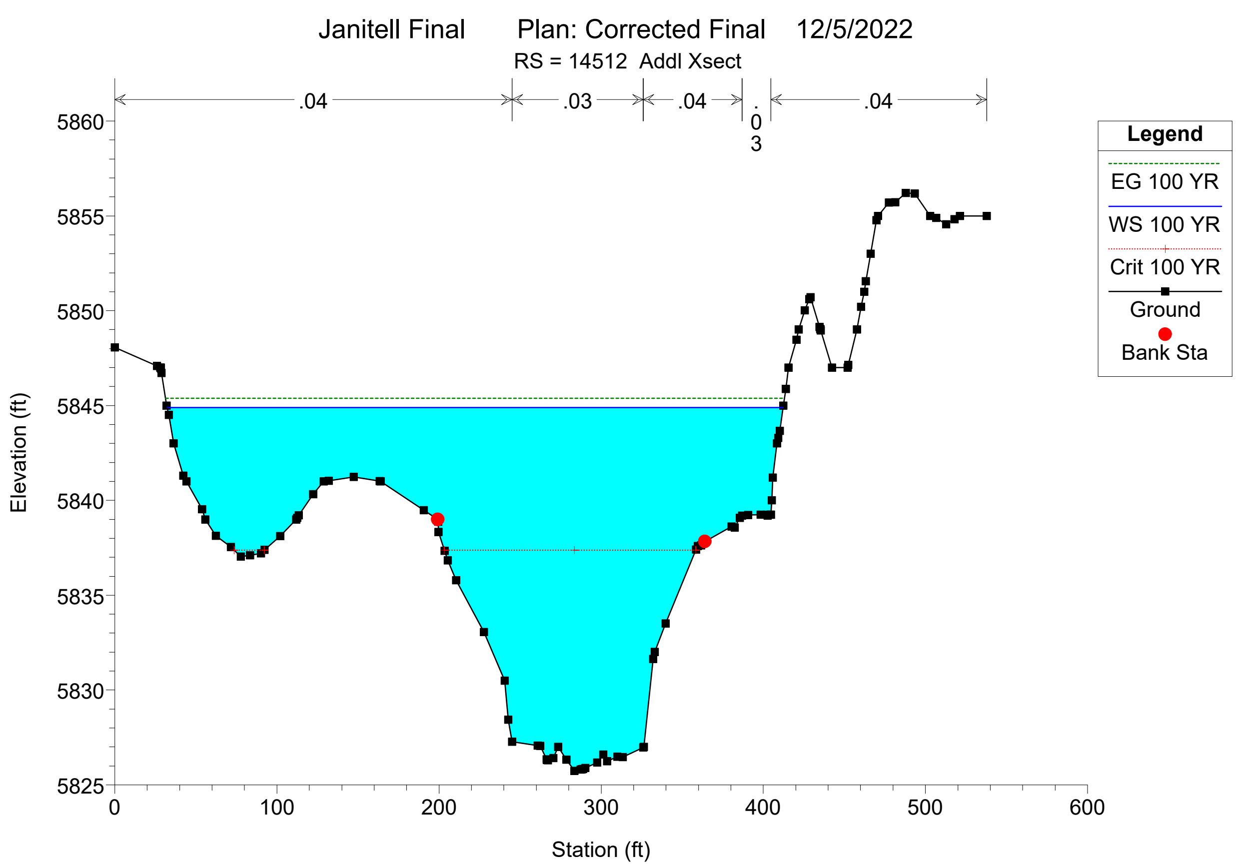

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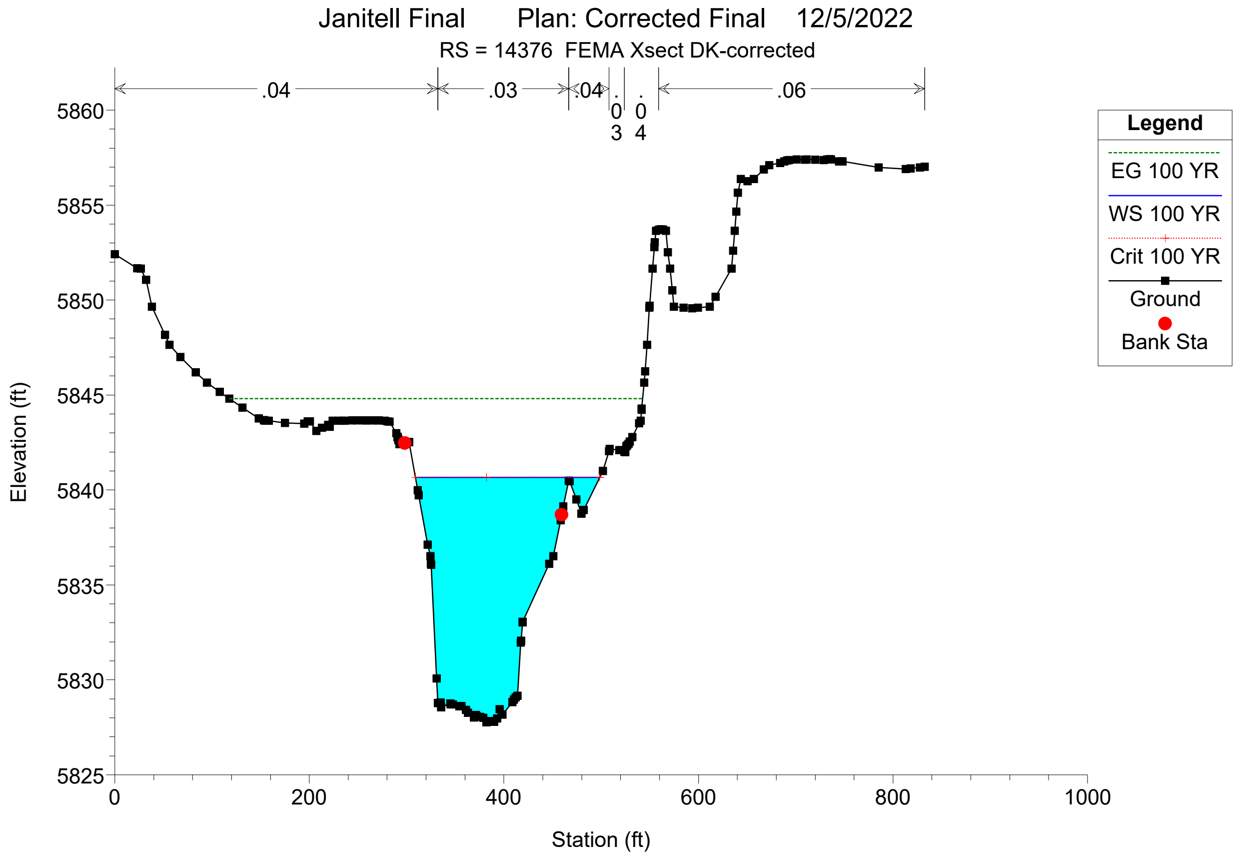
Crit 100 YR
Ground
Bank Sta
Elevation (ft)
Station (ft)



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Station (ft)
Legend
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WS 100 YR
Crit 100 YR
Ground
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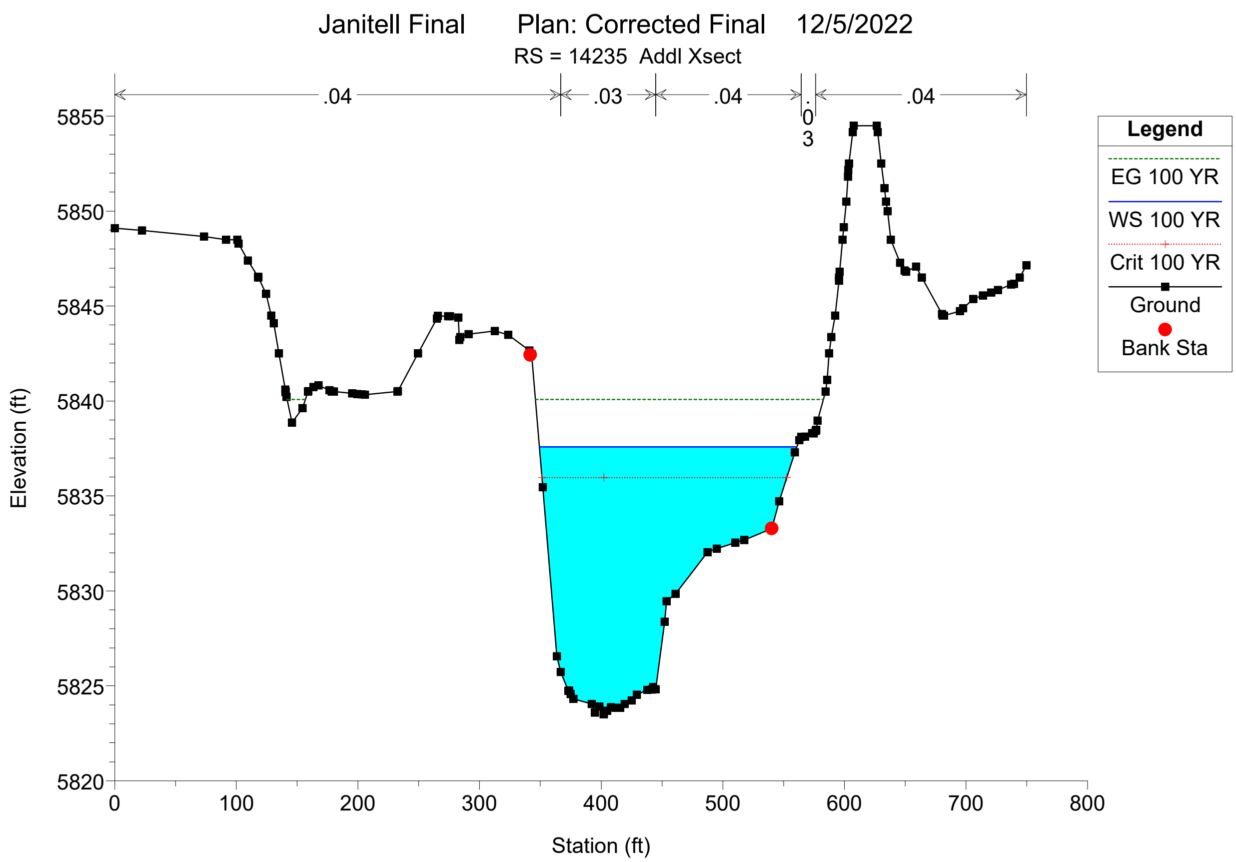

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Station (ft)
Legend
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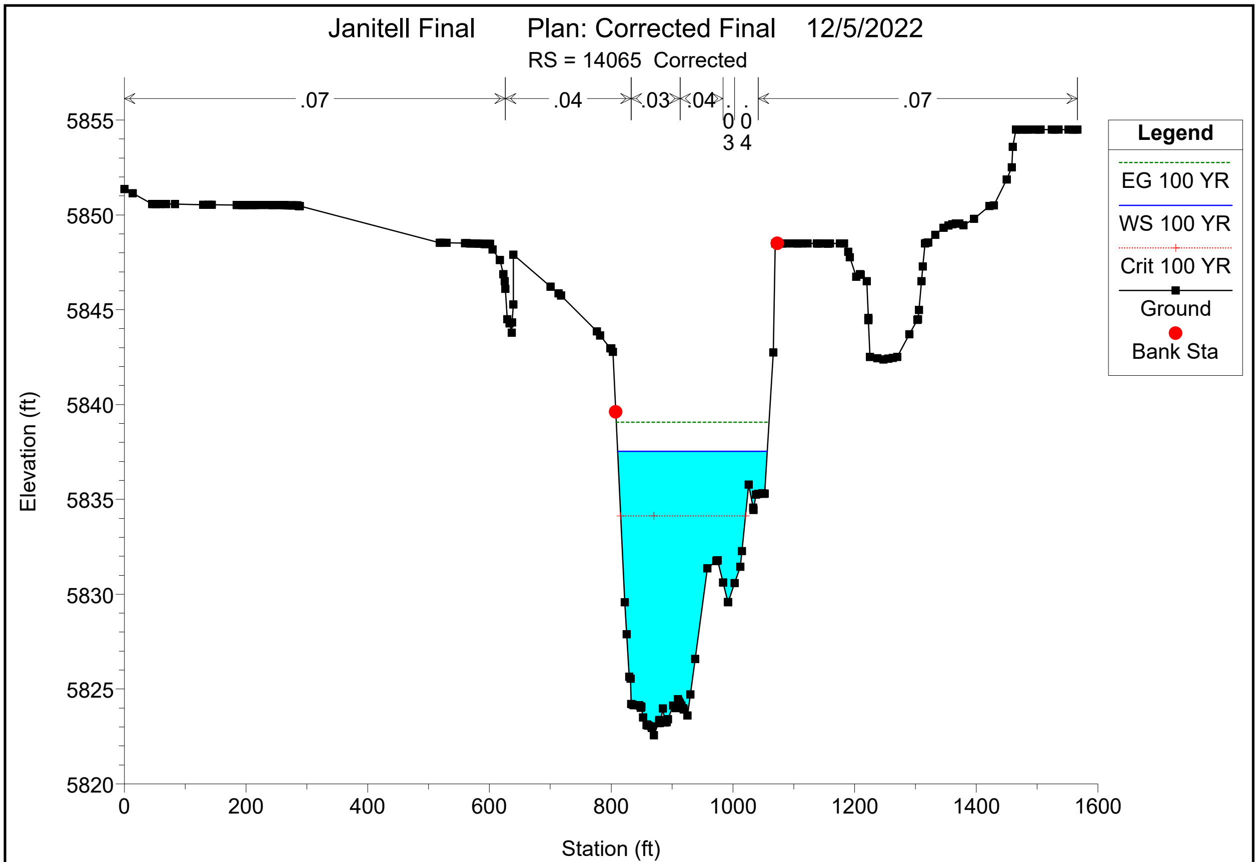
WS 100 YR
Crit 100 YR
Ground
Bank Sta



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Legend
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WS 100 YR
Crit 100 YR
Ground
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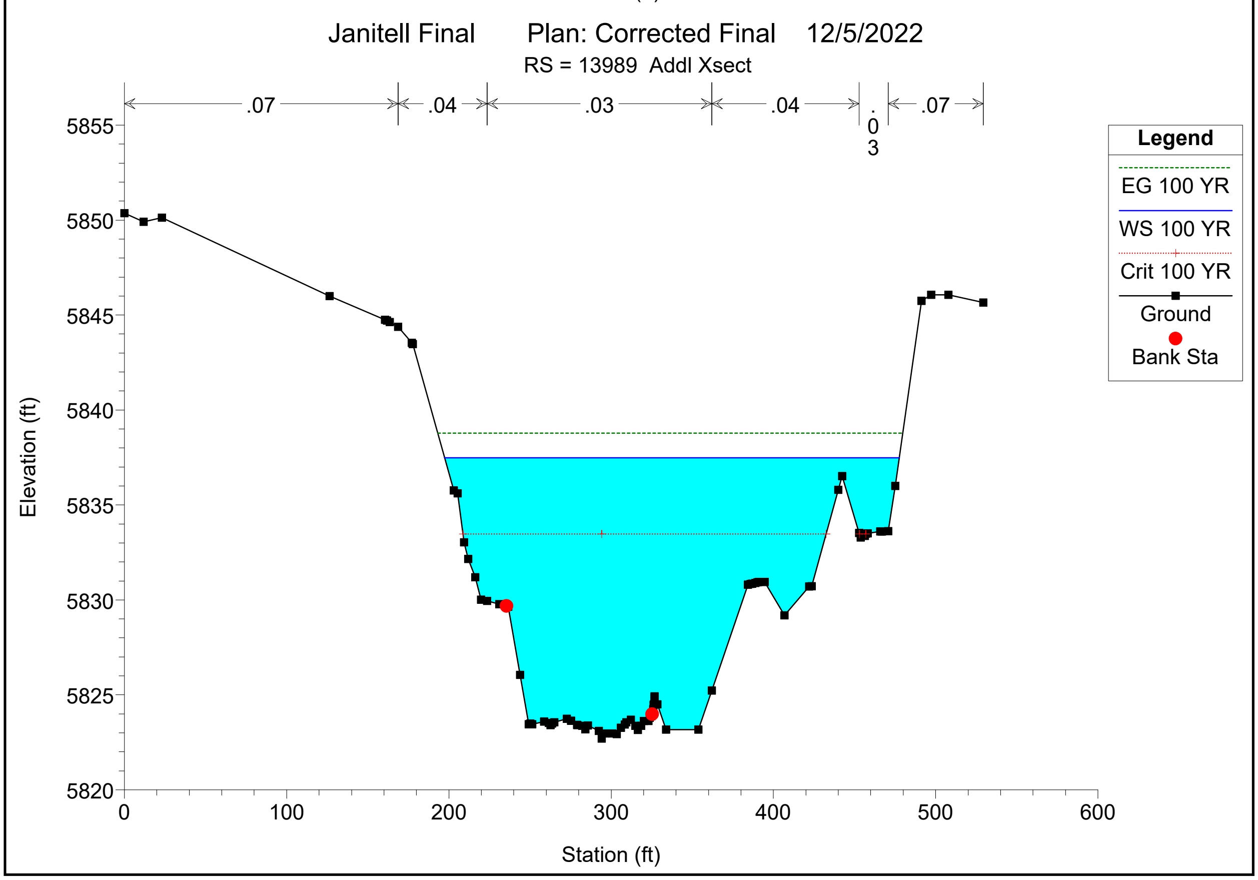
Station (ft)



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Legend
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Crit 100 YR
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Elevation (ft)
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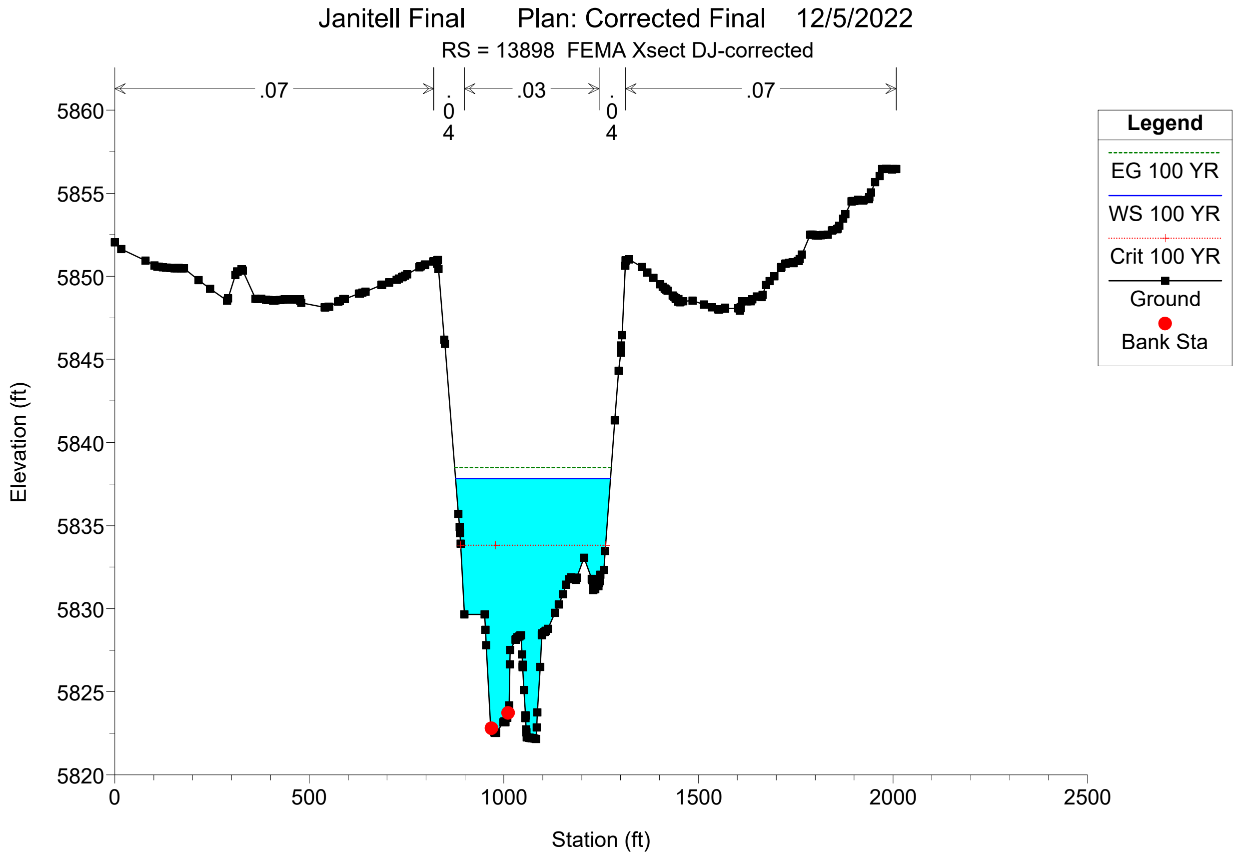
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Legend
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WS 100 YR
Crit 100 YR
Ground
Bank Sta
Elevation (ft)
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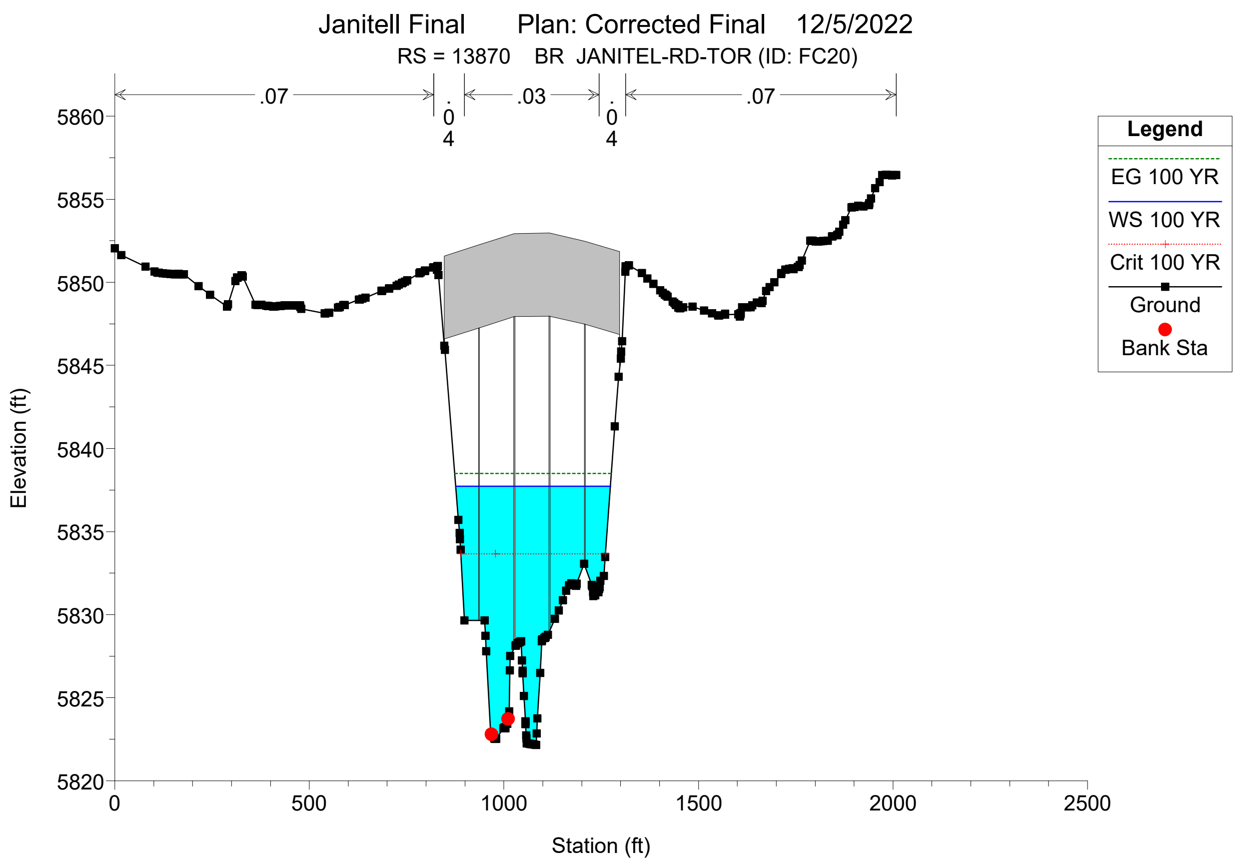
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Legend
EG 100 YR
WS 100 YR
Crit 100 YR
Ground
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Elevation (ft)
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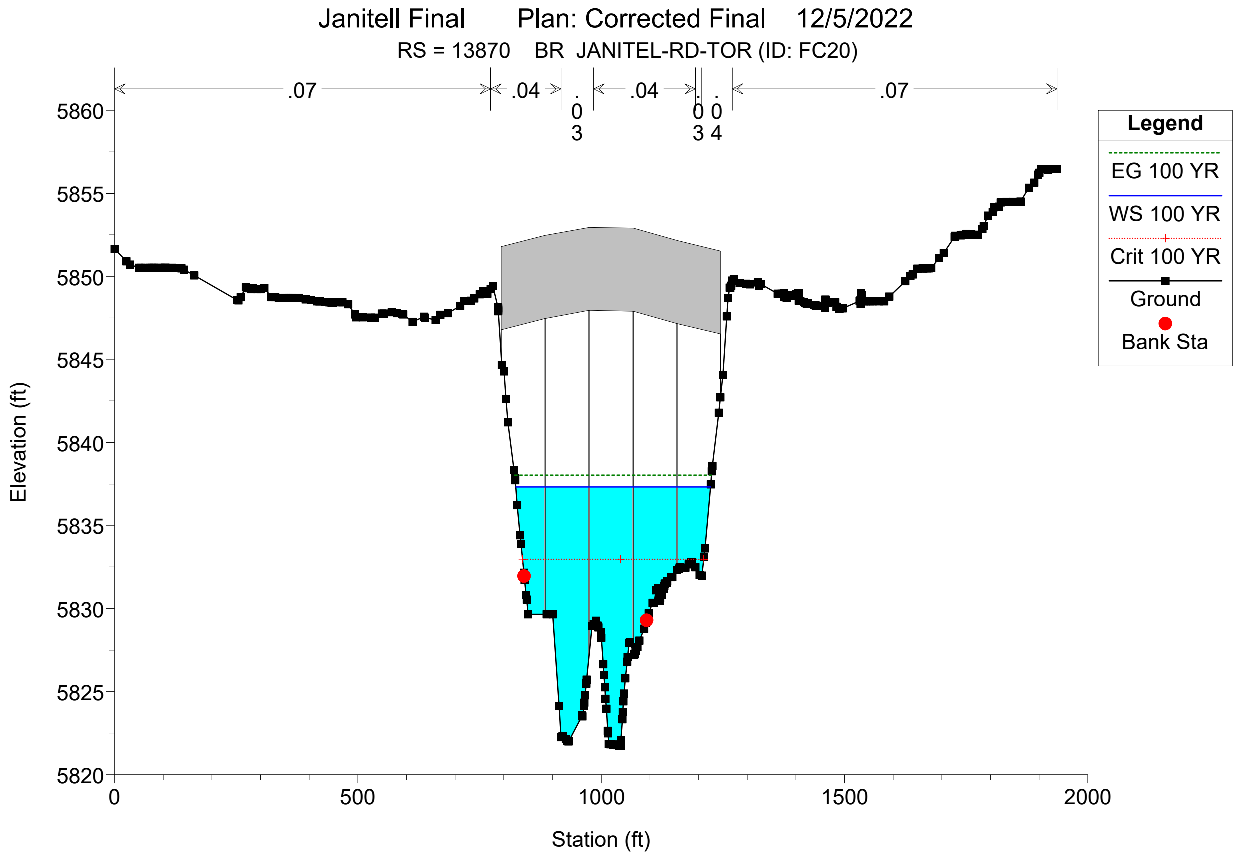

Janitell Final Plan: Corrected Final 12/5/2022
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Legend
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Crit 100 YR
Ground
Bank Sta
Elevation (ft)
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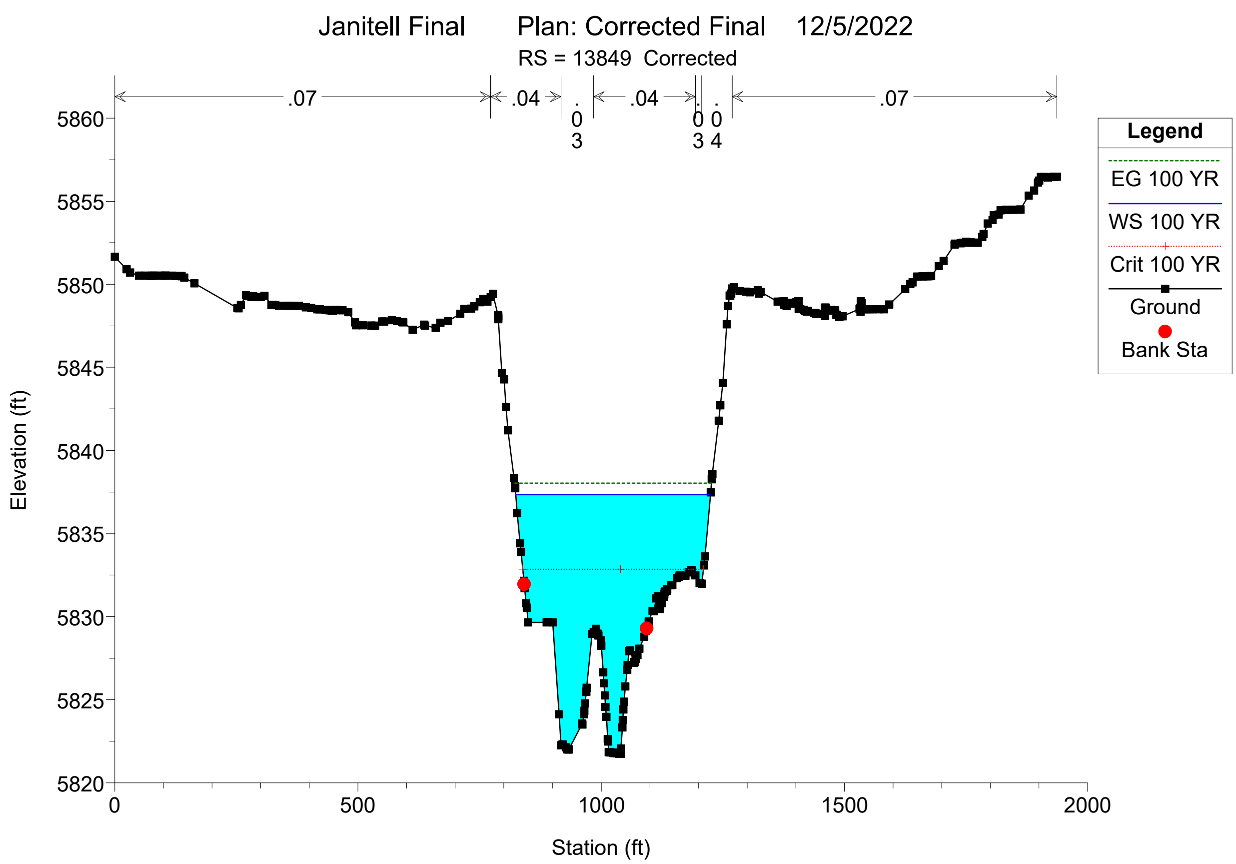

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Legend
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Crit 100 YR
Ground
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Elevation (ft)
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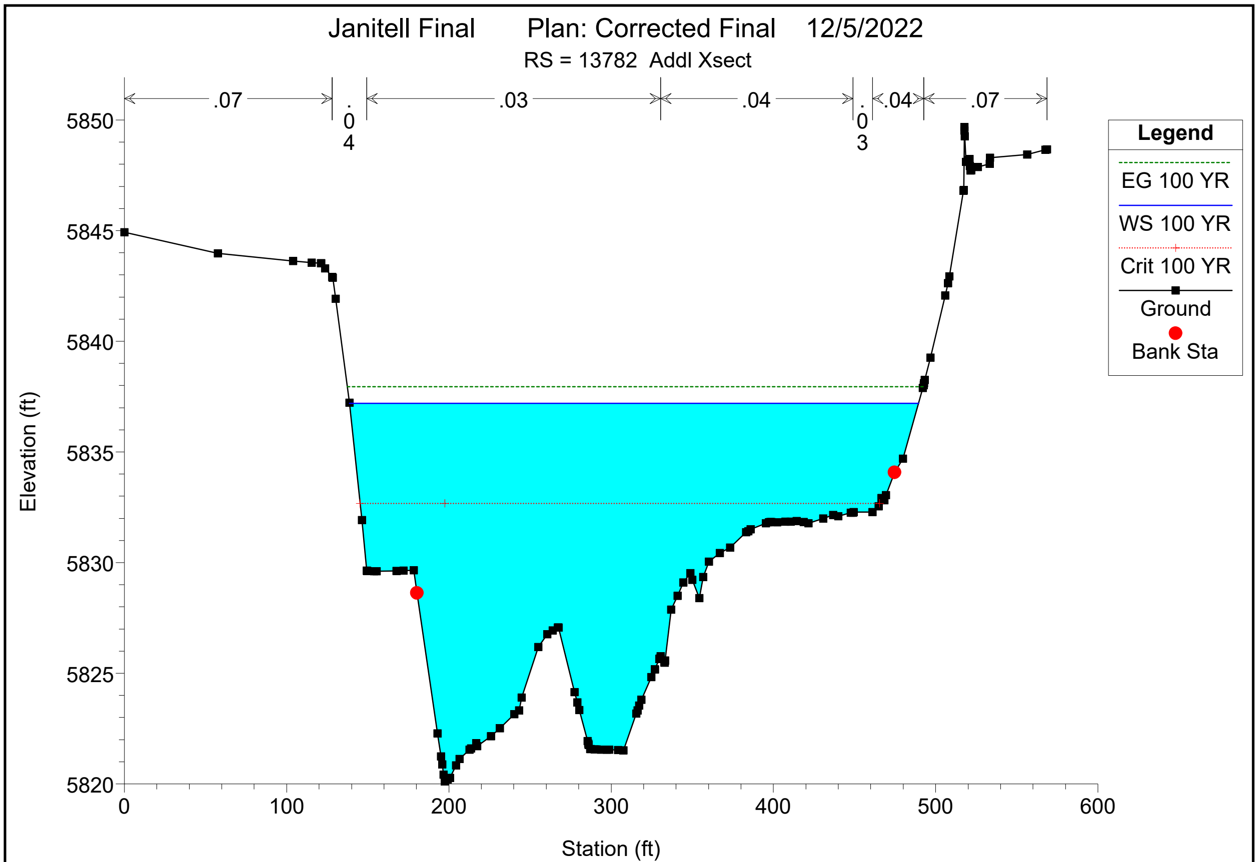
Station (ft)



Janitell Final Plan: Corrected Final 12/5/2022
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Legend
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WS 100 YR
Crit 100 YR
Ground
Bank Sta
Elevation (ft)
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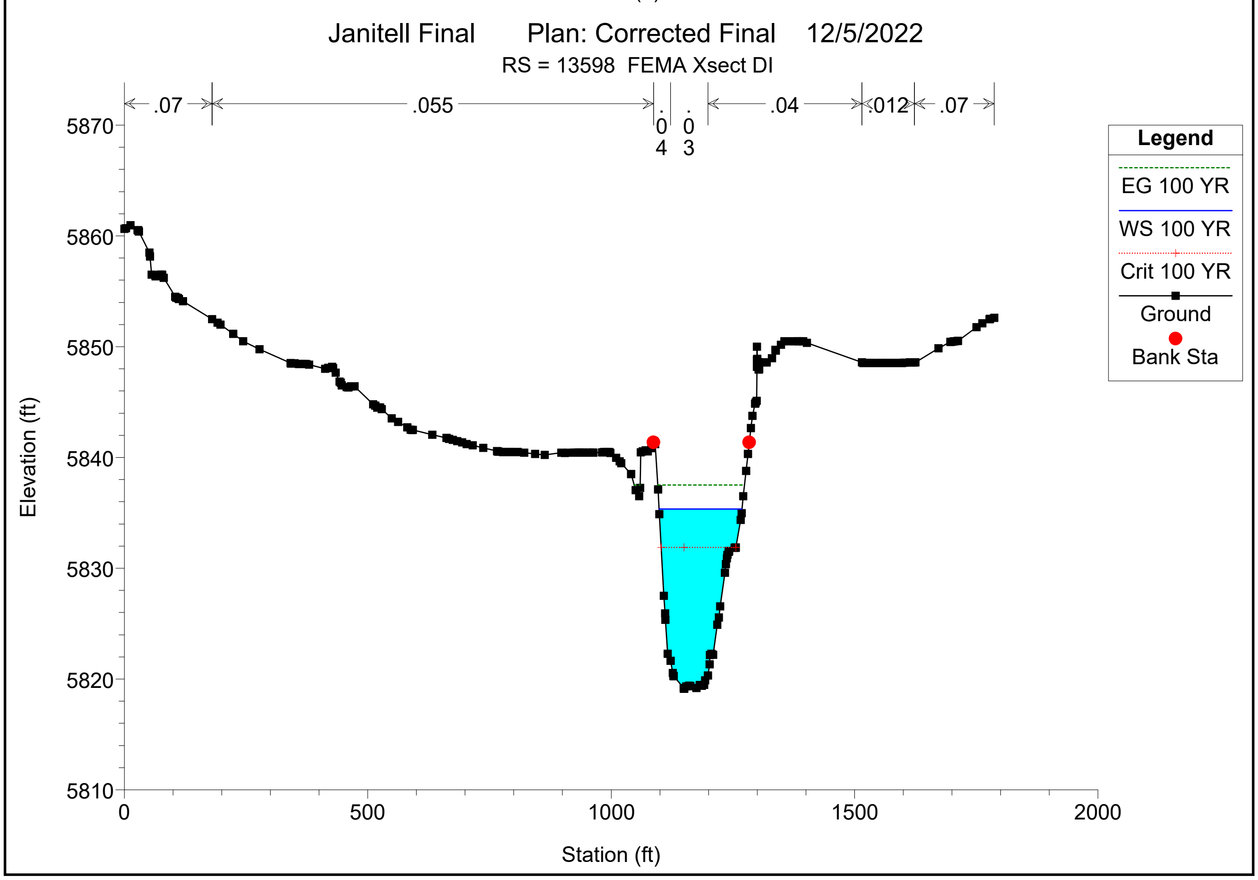

Janitell Final Plan: Corrected Final 12/5/2022
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Legend
EG 100 YR
WS 100 YR
Crit 100 YR
Ground
Bank Sta
Elevation (ft)
Station (ft)
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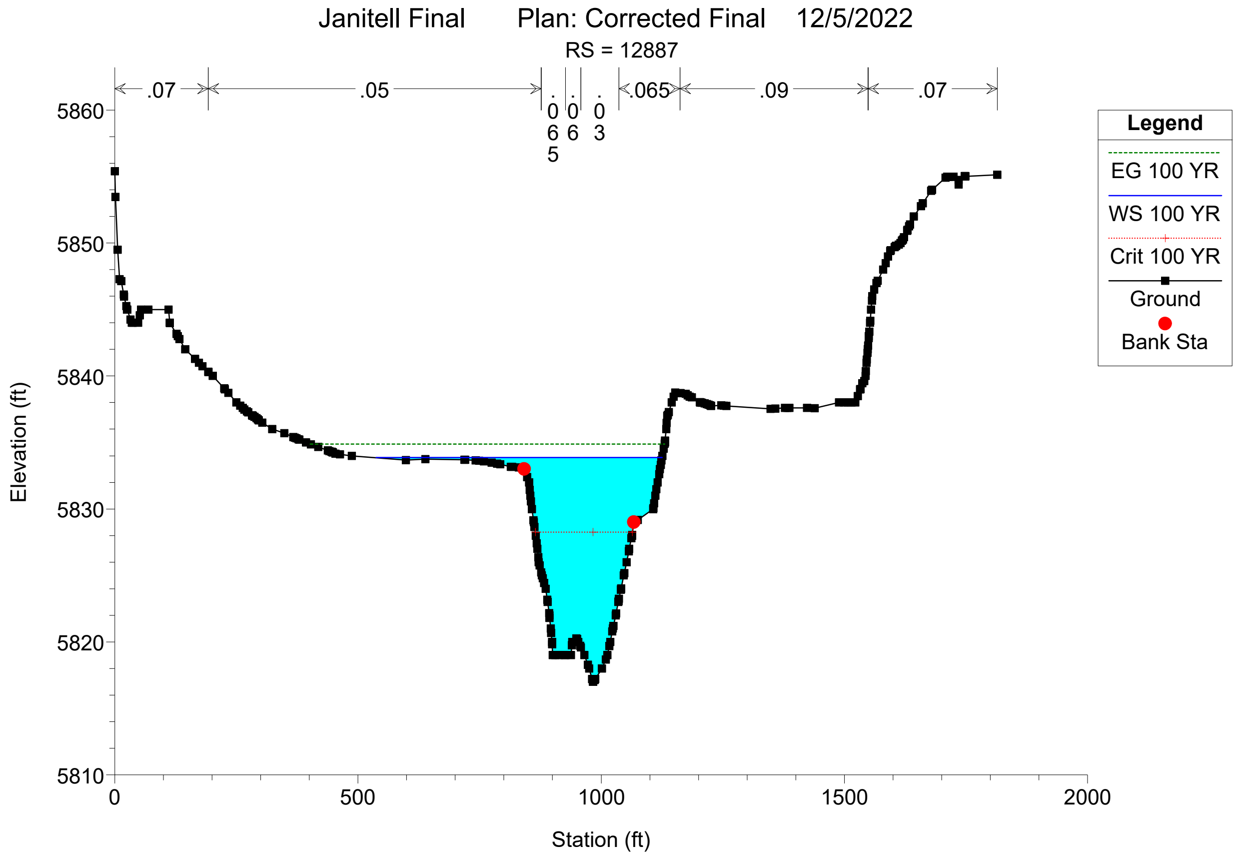

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Legend
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WS 100 YR
Crit 100 YR
Ground
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Elevation (ft)
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Station (ft)
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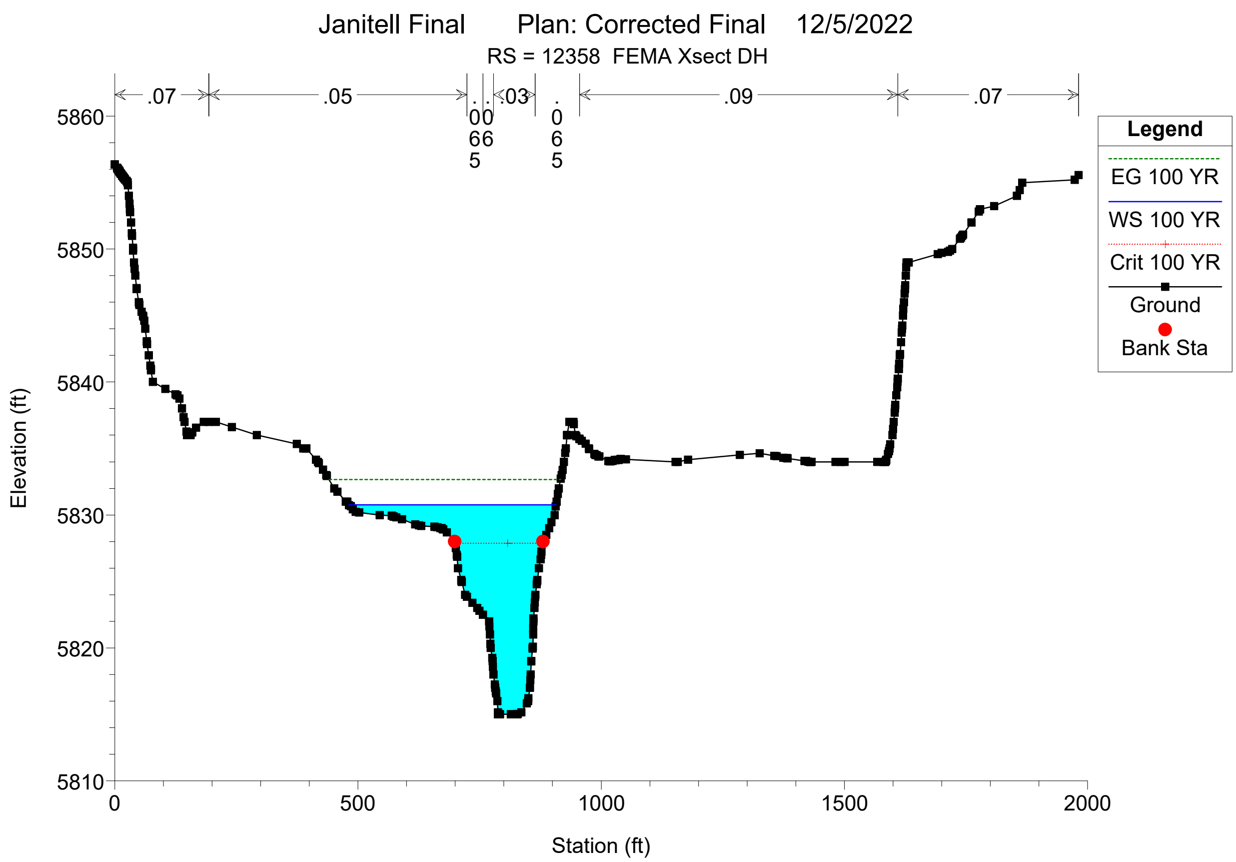

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Legend
EG 100 YR
WS 100 YR
Crit 100 YR
Ground
Bank Sta
Elevation (ft)
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Station (ft)
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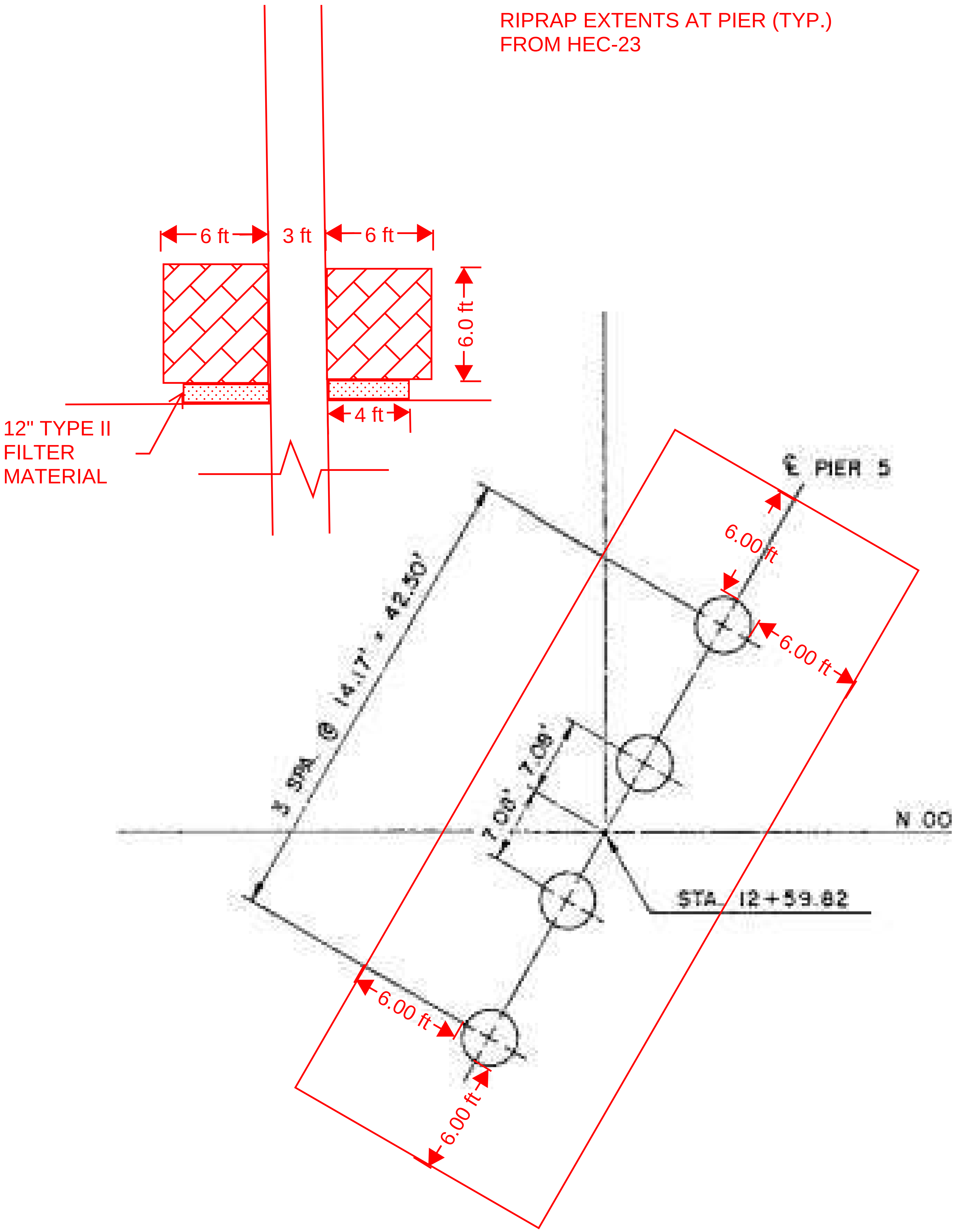
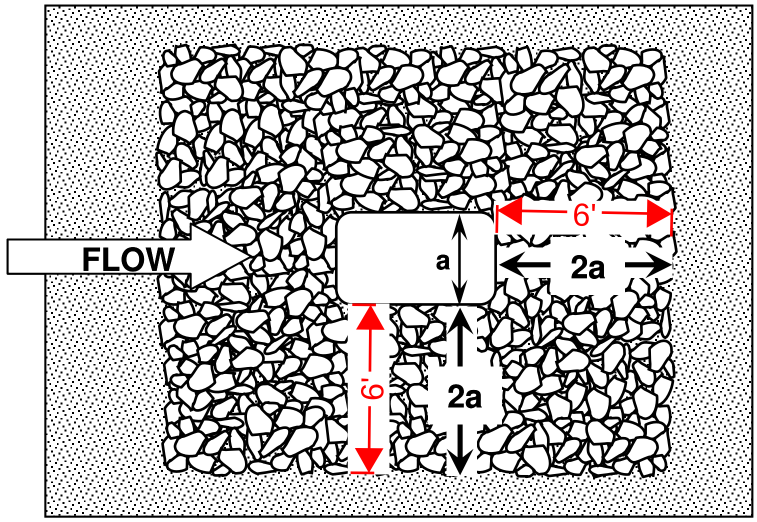
2000

8. Riprap Design Calculations

RIPRAP EXTENTS AT PIER (TYP.)
FROM HEC-23

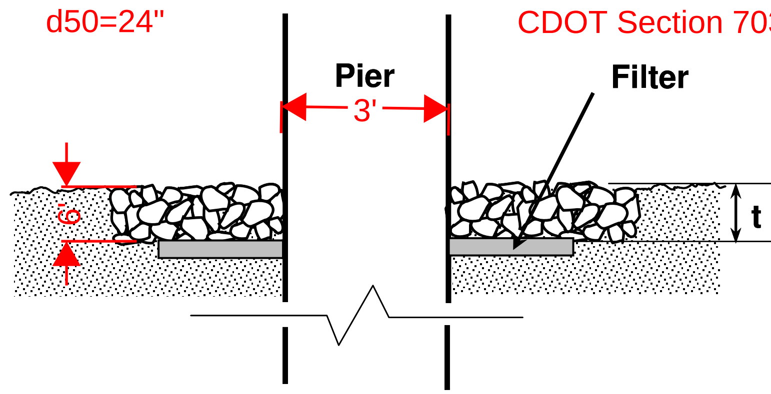




Pier width = "a" (normal to flow)   a=3'
Riprap placement = 2(a) from pier (all around)

a. Plan View

206-00510 Filter Material (CL A) CY
12" thick
CDOT Section 703.9 Filter Material



Minimum riprap thickness t = $3d_{50}$, depth of contraction scour and long-term degradation, or depth of bedform trough, whichever is greatest

Filter placement = 4/3(a) from pier (all around)   3'x4/3=4'

b. Profile

Figure 11.15. Riprap layout diagram for pier scour protection.

Abutment Riprap Design Calculations

Parameter	Value	Units	Notes
Channel Parameters			
Select Channel	<Define Local Data>		
	Channel Calculator...		
Input Parameters			
	Transfer Values From Channel Calcu...		
Structure Type	abutment		
Abutment Type	spill-through abutment		
Set-back Length	0.001	ft	The set-back length is the distance from the r
Main Channel Average Flow Depth	9.941	ft	
Flow Depth at Toe of Abutment	16.000	ft	
Total Discharge	**22400.000**	cfs	Calculations will use either total or overbank (
Overbank Discharge	3000.000	cfs	
Total Bridge Area	**4888.470**	ft^2	
Setback Area	0.001	ft^2	
Maximum Channel Velocity	**6.430**	ft/s	
Specific Gravity of Riprap	2.650		
Results			
Set-back ratio	0.000		
Characteristic Velocity	4.582	ft/s	
Froude Number at the Abutment Toe	0.202		
Abutment Coefficient	0.890		
D50	107.292	mm	
Riprap Shape	Riprap shape should be angular		
Riprap Class			
Riprap Class Name	CLASS I		
Riprap Class Order	1		
D15	114.30	mm	This value is an 'average' of the size fraction
D50	165.10	mm	This value is an 'average' of the size fraction
D85	228.60	mm	This value is an 'average' of the size fraction
D100	304.80	mm	This value is an 'average' of the size fraction
Layout			
Riprap Thickness	12.000	in	
Minimum Horizontal Extent of the Toe Apron from the Abutment Toe	25.000	ft	
Minimum Extent of "Wrap Around" beyond the Abutment Radius, along the Approach Embankment	25.000	ft	See HEC 23, Figure 14.7

Pier Riprap Design Calculations

Parameter	Value	Units	Notes
Channel Parameters			
Select Channel	<Define Local Data>		
	Channel Calculator...		
Input Parameters			
	Transfer Values From Channel Calcu...		
Velocity Input Type	average velocity at the bridge		
Channel Average Velocity (at the bridge)	6.430	ft/s	
Velocity Adjustment Factor for location in the channel	1.200		Ranges from 0.9 for a pier near the bank in a straight reach to 1.7 for a pier lo...
Pier Shape Factor	round-nose pier		
Pier Width (normal to flow)	3.000	ft	
Contraction Scour Depth	0.000	ft	
Bed Form Depth	0.000	ft	
Specific Gravity of Riprap	2.650		
Results			
Design Velocity	11.574	ft/s	
D50	266.115	mm	
Riprap Shape	Riprap shape should be angular		
Riprap Class			
Riprap Class Name	CLASS III		
Riprap Class Order	3		
D15	228.60	mm	This value is an 'average' of the size fraction range for the selected riprap class
D50	317.50	mm	This value is an 'average' of the size fraction range for the selected riprap class
D85	431.80	mm	This value is an 'average' of the size fraction range for the selected riprap class
D100	609.60	mm	This value is an 'average' of the size fraction range for the selected riprap class
Layout			
Depth of Riprap below Streambed	37.500	in	Design thickness of riprap below streambed is greatest of Contraction Scour De...
Minimum Riprap Extent	6.000	ft	See HEC 23, Figure 11.15
Filter Placement Extent	4.000	ft	See HEC 23, Figure 11.15

9. Environmental:
USACOE Permit
Descriptions
Field Observations

Nationwide Permit	Description	Statutory Authority	Limits	Pre-Construction Notification (PCN) Threshold	Delineation Required?	Applicable Waters	Changes	Other Information
NWP 3 -Maintenance	Repair , rehabilitation, or replacement of previously authorized, currently serviceable structures or fills	10/404	authorizes only minor deviations for maintenance	PCN not required	no	all waters of the U.S.	Authorize the repair, rehabilitation, or replacement of any currently serviceable structure or fill that did not require a permit at the time it was constructed. Authorize new or additional riprap to protect the structure or fill.	Does not authorize: new stream channelization or stream relocation projects. Limits stream channel modification to the minimum necessary for the maintenance activity.
NWP-14	Linear Transportation Projects	10/404	1/2 acre in non-tidal waters	>1/10 acre discharges into special aquatic sites	yes, if PCN required	all waters of the U.S.	Add "driveways " to list of examples of linear projects.	Temporary fills must be removed in their entirety and the affected areas returned to preconstruction elevations. Does not authorize storage buildings, parking lots, train stations, aircraft hangars, or other non-linear transportation features.

Beegle, Noelle

From: Karin McShea <McShea@pinyon-env.com>
Sent: Tuesday, March 30, 2021 11:09 AM
To: Alissa Werre <AlissaWerre@elpasoco.com>; Beegle, Noelle <NBeegle@benesch.com>; Lorelei Ward <lward@F-W.com>
Cc: Chase Taylor <Taylor@pinyon-env.com>
Subject: RE: [EXTERNAL] Janitell Bridge-Fountain Creek Scour

Good morning,
I can set up a call to chat about my questions, but I think the main thing is for Tony to see the site, and then we can all regroup afterwards.

The main things from an ecological perspective to discuss with Tony:

- There are only a few potential wetlands on the fringes of the creek – no other wetlands in the area.
- The ecological value of the area is fairly low right now – with elm trees and smooth brome as dominant vegetation. Some native trees and shrubs in area. (see quick summary list of species noted below).
- The side drainage under the bridge on the south side of the creek – please ask Tony if he believes this stormwater drain and erosional feature is a jurisdictional feature.
- Wetlands will need to be delineated, but will he also want other data collected? For example on the stream itself? Using the CSQT?
- I would like to hear his thoughts on what permits could be used for this work (of course depending on alterative and scale of project).

I posted photos and a PDF with maps showing the general order of the photos. I take a lot of photos – generally doing a 360 – turning to the right.
I took some panos from both sides of the bridge, and from the large power pole on the NE side of the creek (just south of sand creek confluence).

Janitell [pinyonenv1-my.sharepoint.com]

If there is anything else – please let me know.
Thanks,

Karin

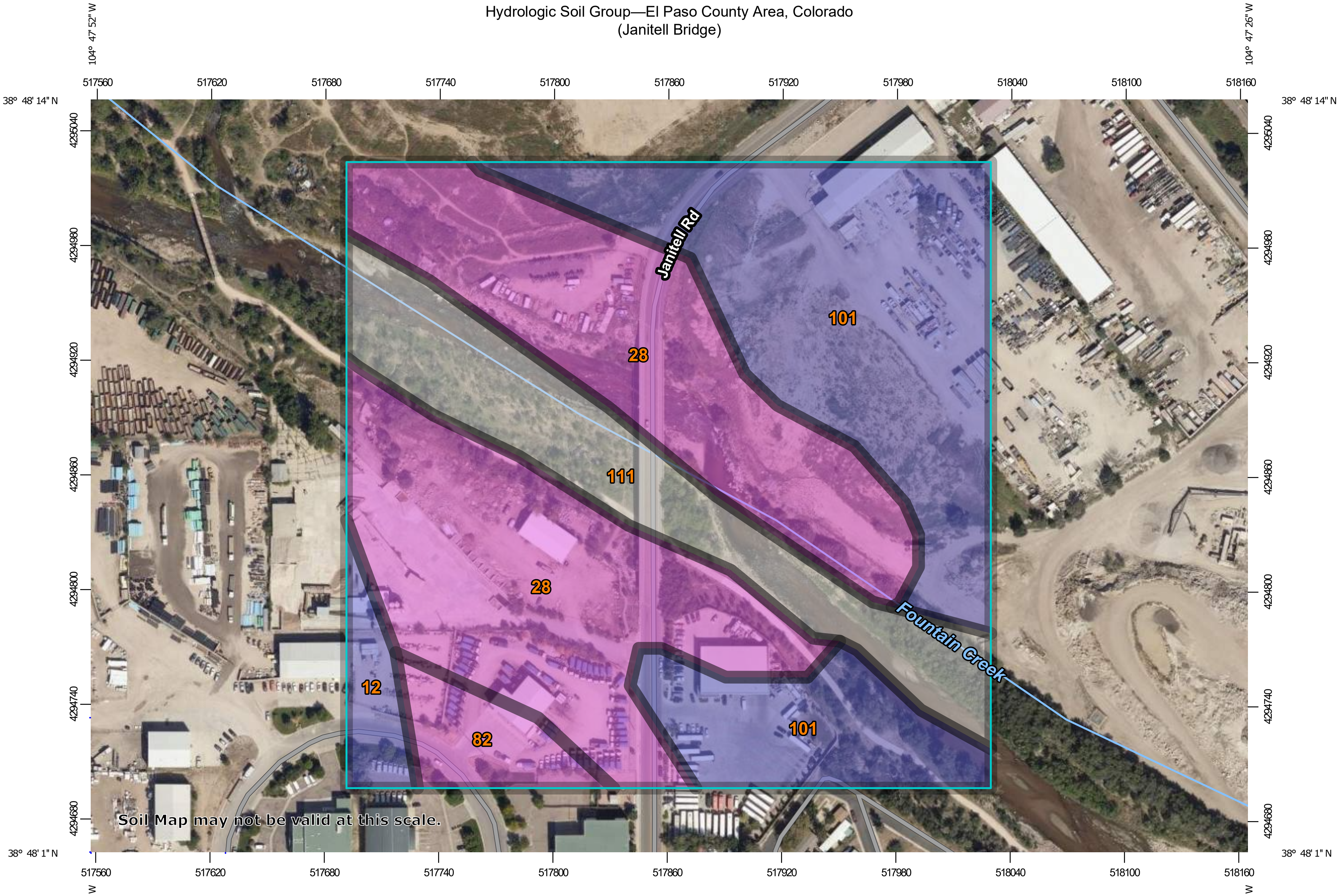
General plant list
Siberian/Chinese elm – dominant in area
Smooth brome – dominant in area
3 leaf sumac – some on hillsides
Snowberry – some on hillsides
Chokecherry – some on hillsides
Crested wheatgrass
Sonchus
Coyote willow – scattered in area – no thick stands – likely due to lack of groundwater and trampling.
Sweetclover
Spotted knapweed
Russian thistle
Kochia
Scotch thistle
Curly doc
Boxelder (not confirmed – but opposite stems indicates)
Cottonwoods – some ~20 year east of bridge. Multiple older 80 year trees in area. Did not note any saplings (ie no recruitment)
Rose – scattered near trail
Apple tree
Reed canary grass – dominant in fringe wetland
Russian olive

Karin McShea | *Technical Group Manager - Biological Resources*

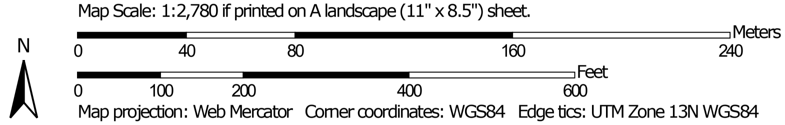
Pinyon Environmental, Inc.
P 303.980.5200 | **D** 303.468.9714 | **M** 720.441.9811
Pinyon is now offering services in northern Colorado from our new Loveland office [pinyon-env.net]!

10. USGS Soils Report

Hydrologic Soil Group—El Paso County Area, Colorado
(Janitell Bridge)



Map Scale: 1:2,780 if printed on A landscape (11" x 8.5") sheet.



Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 13N 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: El Paso County Area, Colorado
Survey Area Data: Version 19, Aug 31, 2021

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

Date(s) aerial images were photographed: Aug 19, 2018—Sep 23, 2018

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
12	Bresser sandy loam, cool, 3 to 5 percent slopes	B	0.8	2.8%
28	Ellicott loamy coarse sand, 0 to 5 percent slopes	A	12.0	43.7%
82	Schamber-Razor complex, 8 to 50 percent slopes	A	1.1	3.9%
101	Ustic Torrifluvents, loamy	B	9.4	34.0%
111	Water		4.3	15.5%
Totals for Area of Interest			**27.5**	**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.

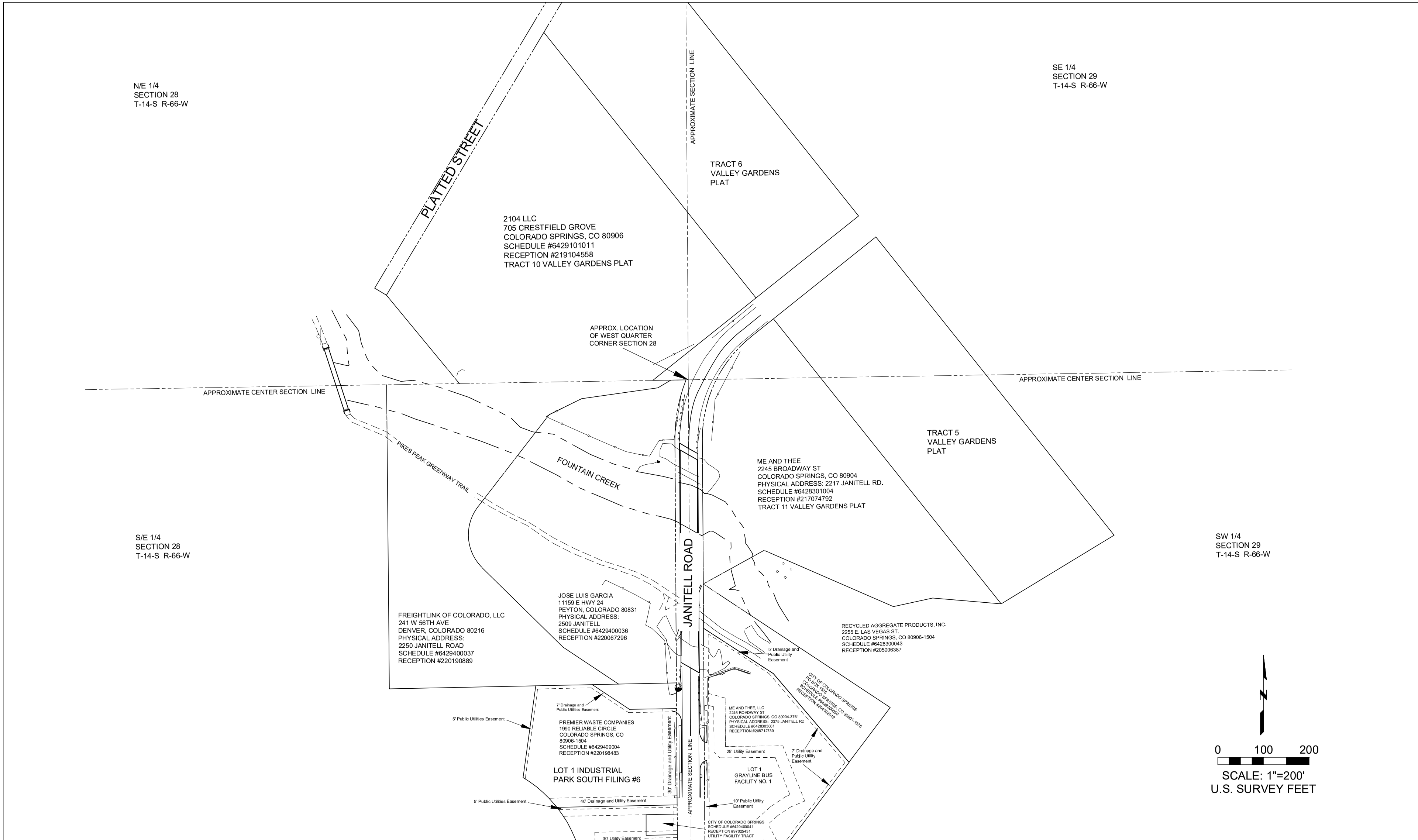
Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

11. Property Impacts



Print Date:	April 19, 2021
File Name:	PROJECT: JANITELL BRIDGE SCOUR
Horizontal Scale: 1" = 200'	Vertical Scale: N/A

Farnsworth GROUP
5775 MARK DABLING BLVD., SUITE 190
COLORADO SPRINGS, COLORADO 80919
(719) 590-9194 / info@f-w.com

benesch
7979 E. Tufts Avenue,
Suite 800
Denver, Colorado 80237
303-771-6868

Sheet Revisions

Date	Comments	Initials

EL PASO COUNTY
COLORADO
3275 Akers Drive
Colorado Springs, CO 80922
tel 719.520.6460
fax 719.520.6879

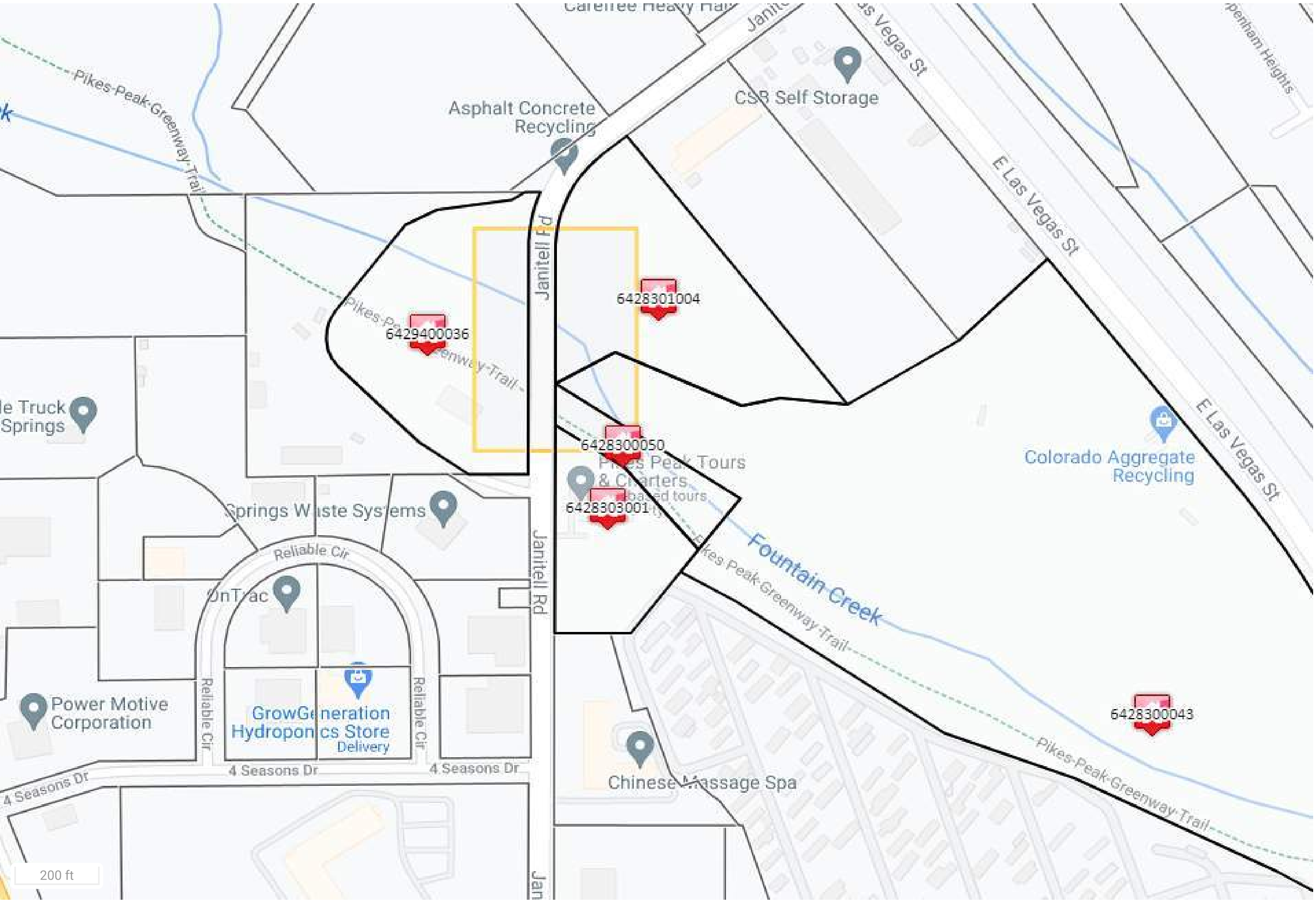
As Constructed
No Revisions:
Revised:
Void:

JANITELL BRIDGE SCOUR
OWNERSHIP MAP

Designer:	LAW	Structure Numbers	
Detailer:	CDO		
Subset:	Ownership Map	Sheets:	1 of 1

Project No./Code
#152064

Sheet Number 1



2255 E LAS VEGAS ST Market Value **$126,280**

6428300043
RECYCLED AGGREGATE PRODUCTS INC

JANITELL RD Market Value **$2,374**

6428300050
COLORADO SPRINGS CITY OF

2217 JANITELL RD Market Value **$421,660**

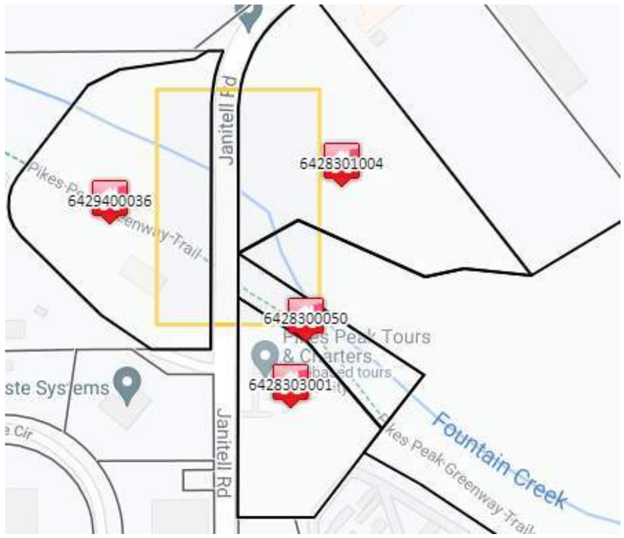
6428301004
ME & THEE LLC

2375 JANITELL RD Market Value **$1,003,104**

6428303001
ME & THEE LLC

2509 JANITELL RD Market Value **$359,562**

6429400036
GARCIA JOSE LUIS


6428301004
6429400036
6428300050
6428303001
Janitell Rd
Janitell Rd
Pikes Peak Tours & Charters
Pikes Peak Greenway Trail
Fountain Creek
Pikes Peak Greenway Trail

Schedule Number	Owner Name	Location	Market Value	Website	Acres
6428300043	RECYCLED AGGREGATE PRODUCTS INC	2255 E LAS VEGAS ST	$126,280	https://property.spatialest.com/co/elpaso/#/property/6428300043	28.99
6428300050	COLORADO SPRINGS CITY OF	JANITELL RD	$2,374	https://property.spatialest.com/co/elpaso/#/property/6428300050	1.09
6428301004	ME & THEE LLC	2217 JANITELL RD	$421,660	https://property.spatialest.com/co/elpaso/#/property/6428301004	4.84
6428303001	ME & THEE LLC	2375 JANITELL RD	$1,003,104	https://property.spatialest.com/co/elpaso/#/property/6428303001	2.31
6429400036	GARCIA JOSE LUIS	2509 JANITELL RD	$359,562	https://property.spatialest.com/co/elpaso/#/property/6429400036	4.92