GEOTECHNICAL INVESTIGATION
JANITELL BRIDGE
FOUNTAIN CREEK
EL PASO COUNTY, COLORADO

Prepared for:

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CTL|T Project No. CS19402-125

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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.

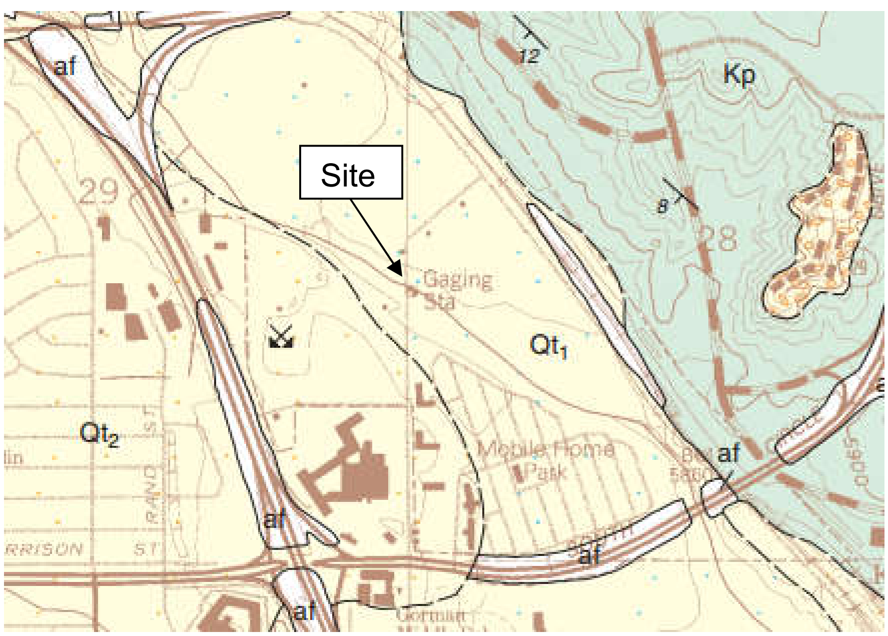
The bank on the north side of Fountain Creek in the vicinity of the bridge is about 30 feet above the current creek bed, with shale exposed in the lower few feet. Upstream and through the bridge, the south bank is about 5 feet in height and is comprised of alluvial sand and gravel deposits. The west 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:

$$e = 0.64(\frac{s}{b})^{0.34} \; for \; 1 \leq \frac{s}{b} \leq 3.75, and \; e = 1.0, \; for \; \frac{s}{b} \geq 3.75 \qquad \textit{(Equation 5.39)}$$

Leading piers:

$$e = 0.7(\frac{s}{b})^{0.26} \; for \; 1 \leq \frac{s}{b} \leq 4.0, and \; e = 1.0, for \; \frac{s}{b} \geq 4.0 \qquad \textit{(Equation 5.40)}$$

Trailing piers:

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

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:

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

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

$Ø$ = 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.

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Associate Engineer

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Sep 28 2021 3:49 PM

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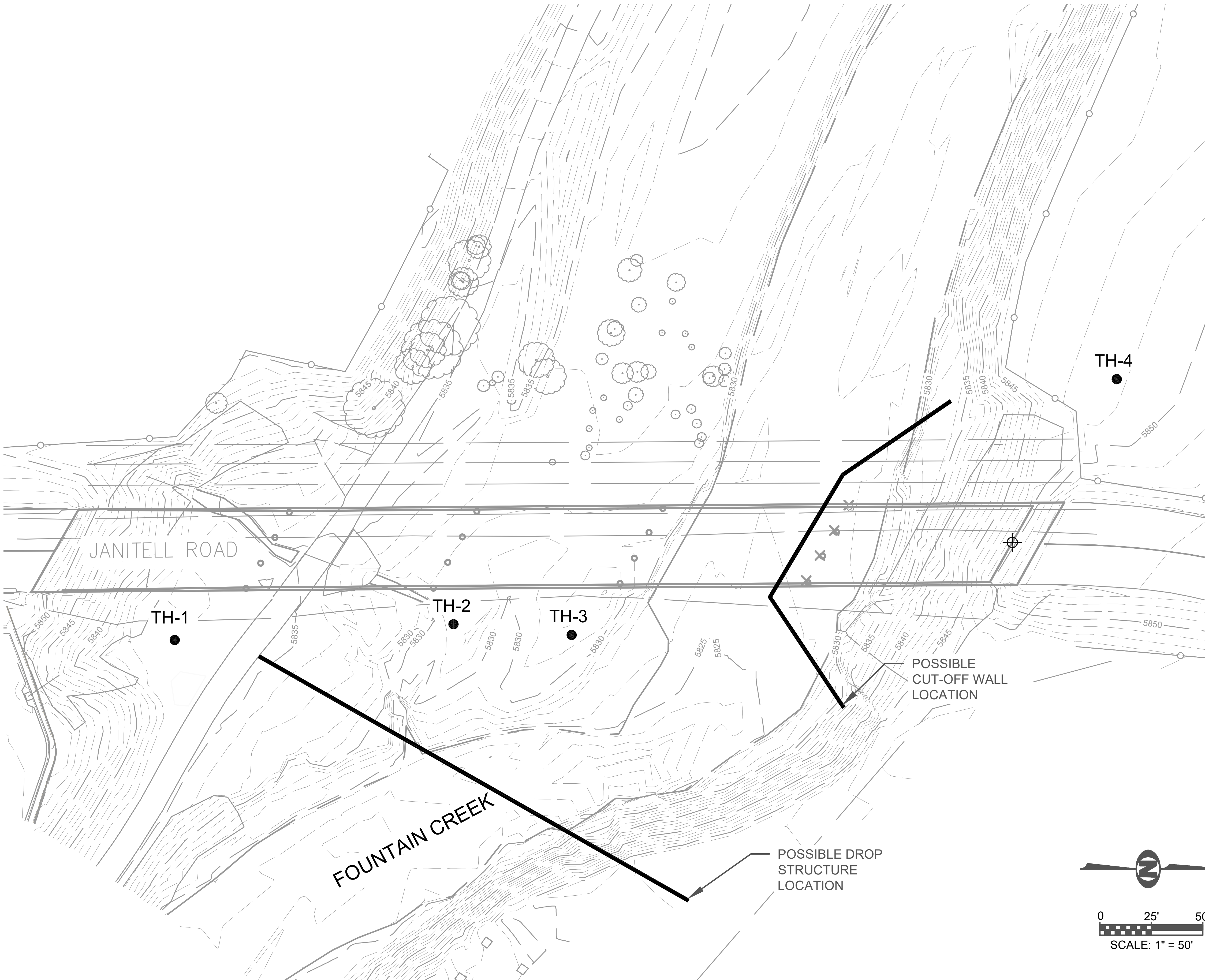

VICINITY MAP

(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.

N

0 25' 50'

SCALE: 1" = 50'

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

Location of Exploratory Borings

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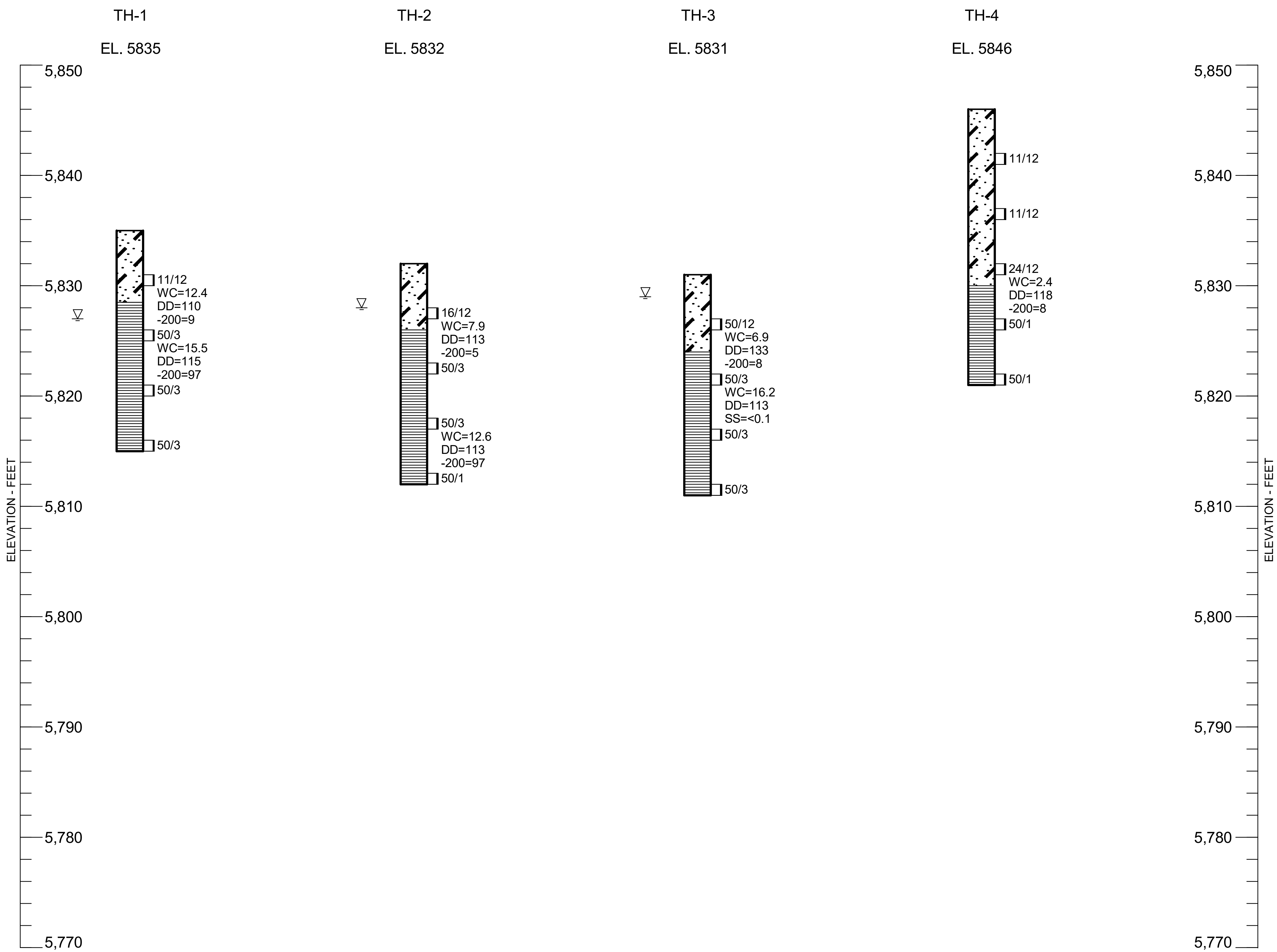
FIG. 1



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.

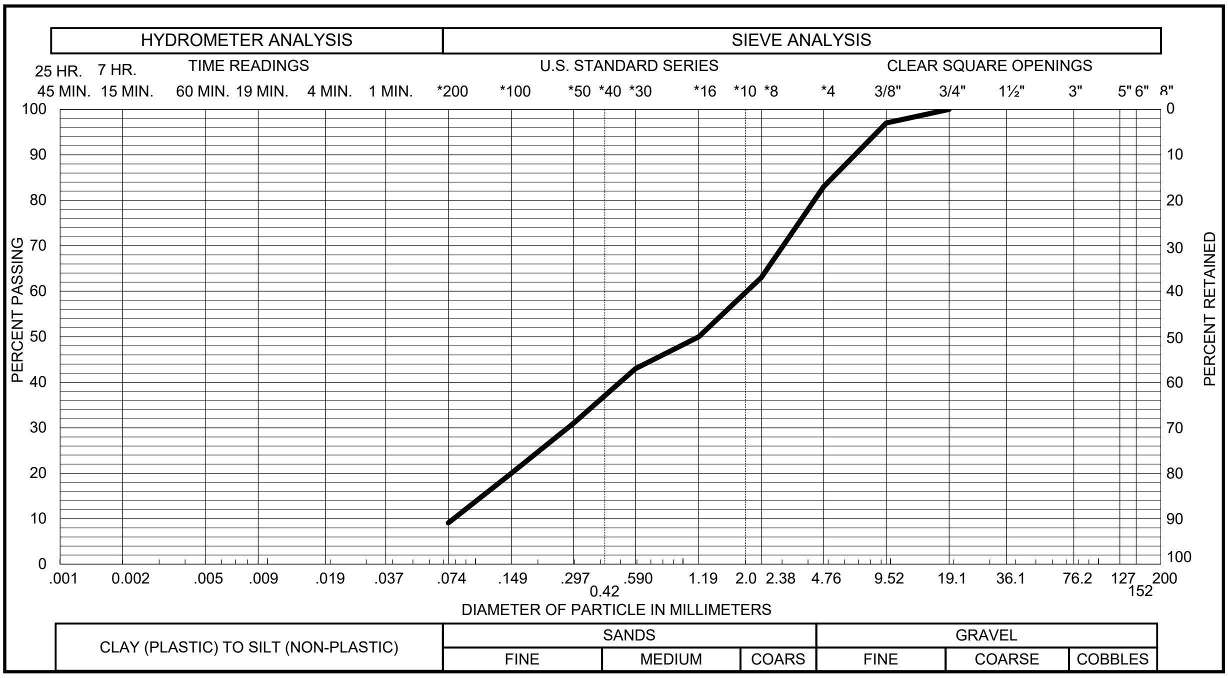
GROUNDWATER LEVEL MEASURED AT TIME OF DRILLING.

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. (%)

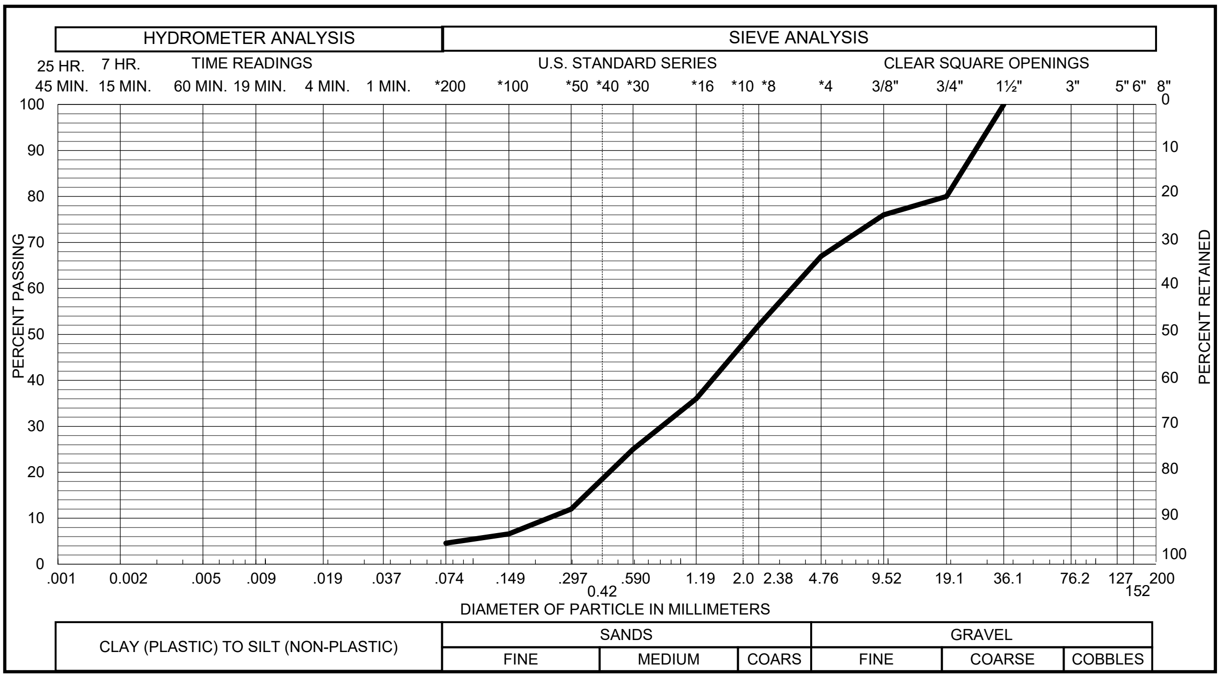
Summary Logs of Exploratory Borings

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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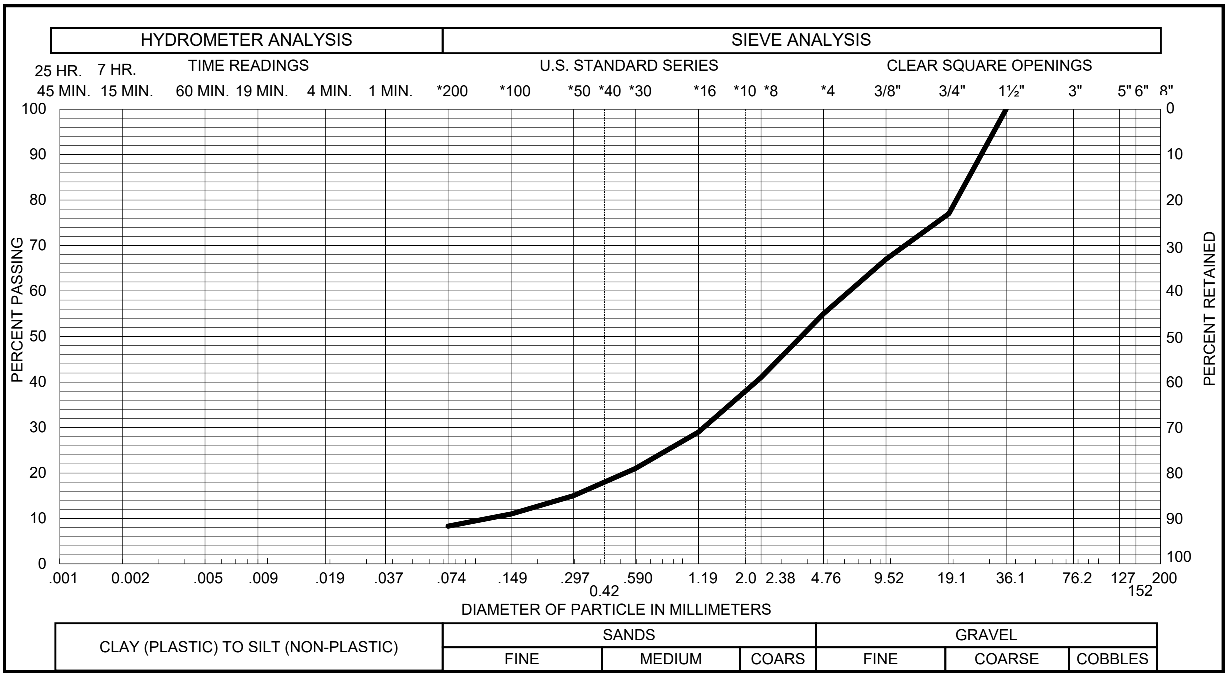
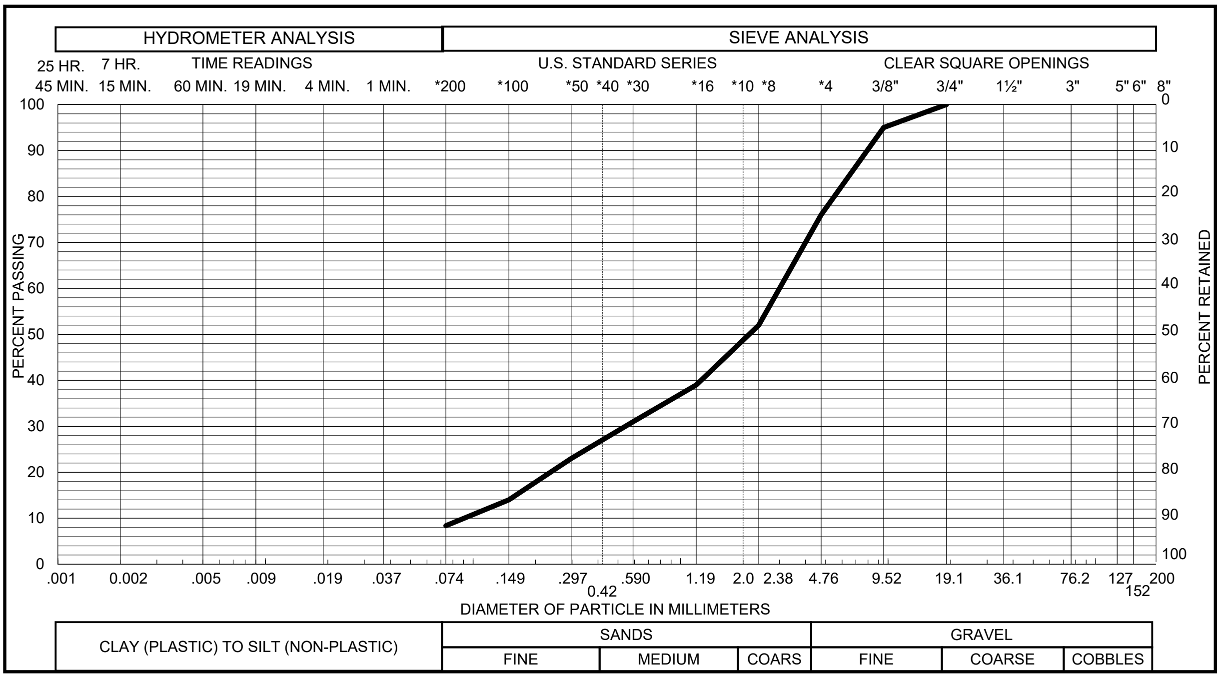
Gradation Test Results

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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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.