Geotechnical Investigation

New Railroad Bridges Near Agency Creek

and Warren Slough

Clatsop County, Oregon

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Prepared for

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At your request, GRI conducted a geotechnical investigation for the new railroad bridges planned near Agency Creek and Warren Slough in Clatsop County, Oregon. The general locations of the sites are shown on the Vicinity Map, Figure 1. The purpose of the investigation was to evaluate the subsurface conditions at the locations of the proposed improvements and provide our conclusions and recommendations for design and construction of the new bridges. The investigation included a review of existing geologic information for the area, exploration borings, laboratory testing, and engineering studies and analyses. This report describes the work accomplished and provides our conclusions and recommendations for use in the design and construction of the proposed improvements.

Unless otherwise noted, all elevations referenced in this report are based on the North American Vertical Datum of 1988 (NAVD 88).

1 PROJECT DESCRIPTION

We understand the project will include a new opening excavated into the existing railroad embankment at each site for the purpose of improved fish habitat. Two new bridges will support the existing railroad over the new openings. We understand precast concrete bridge structures on the order of 40 feet in length will be considered to provide openings with minimum bed widths of 15 feet to 20 feet. Preliminary estimates of pile loads for the bridge sites are about 200 kips per pile in compression with no uplift. We have assumed final rail grades will remain unchanged and minimal fill will be required and limited to backfilling behind the new bridge abutments. The new excavations are planned to extend to about elevations 0 feet to 4 feet, about 8 feet to 12 feet below the existing site grades.

We understand the project will be designed in accordance with the American Railway Engineering and Maintenance-of-Way Association (AREMA) *Manual for Railway Engineering* (AREMA, 2019), which requires a static factor of safety of 2.5 for driven piles. Based on our conversation with HPSI, we understand HP14x89 H-piles and PP16x0.5 closed-ended pipe piles are being considered for bridge foundations. We understand the bridge will be seismically designed to resist ground shaking per the current codes and AREMA guidelines but will not be designed to resist potential seismic ground deformations from liquefaction or lateral spreading. Similarly, the soil conditions at the project site will not be evaluated for additional geologic seismic hazards that require a site-specific site-response analysis.

The project will also include several levee breaches. The preliminary plans indicate spoils from levee breaches will be distributed on the floodplain near the toe of the existing railroad embankment. The planned thickness of spoils is currently unavailable, but we have assumed the thickness of these spoils will be less than 1 foot to 2 feet to limit settlement in the vicinity of the railbed. If significant thickness of fill is placed, GRI should be notified and the settlement impacts should be further evaluated.

2 SITE DESCRIPTION

2.1 Site Conditions

The proposed Agency Creek bridge site is about 1 mile to the west (downstream) of where the railroad intersects with Waterhouse Road. Based on our review of preliminary plans developed by Inter-Fluve, the top of the railroad embankment is about 10 feet wide and at about elevation 12 feet. The embankment fill slopes down to the floodplain at about elevations 5 feet to 6 feet and 0 feet to 3 feet on the south and north sides, respectively. The site is about 800 feet south of the bank of the Knappa Slough channel of the Columbia River, and the adjacent floodplain to the north of the embankment is about elevation 5 feet to 8 feet. The railroad embankment typically consists of fragmental rock and riprap. The embankment is in disrepair and has trees growing on it. Three existing 24-inch-diameter, corrugated-metal pipe culverts oriented in a generally north-south direction are embedded in the embankment at the site of the planned bridge span. The floodplain is heavily vegetated with large brush and small trees.

The Warren Slough bridge site is located on the existing railroad embankment about 0.75 mile to the east (upstream) of where the railroad intersects with Waterhouse Road and is about 250 feet north of Ziak-Gnat Creek Road. The top of the railroad embankment fill is at about elevation 12 feet and slopes down to the floodplain at about elevation 8 feet to 10 feet on the north side of the embankment. On the south side of the embankment, the adjacent ground surface is about elevation 5 feet; however, there is a ditch or channel that runs the length of the embankment and fills with water during high tide or during high river flows. The bridge site is about 1,000 feet east of Warren Slough and about 0.5 mile south of the Knappa Slough channel of the Columbia River. The embankment typically consists of fragmental rock and riprap. The railroad embankment is in disrepair, has trees and abundant blackberry bushes growing on it, and is not readily passable on foot. Two existing 36-inch-diameter, concrete pipe culverts oriented in a generally north-south direction are embedded in the embankment at the site of the planned bridge span. The floodplain is heavily vegetated with large brush and small trees.

2.2 Geologic Setting

The site is located on the floodplain of the Columbia River. Available geologic literature and the results of this investigation indicate the sites are mantled by Quaternary-age alluvium consisting of unconsolidated floodplain deposits of silt and sand. The unconsolidated saturated silt and sand are susceptible to liquefaction and lateral spreading. The alluvium is underlain by Miocene-age sandstone mapped as the Gnat Creek

Formation (Niem, 1985). Immediately south of the rail line at both locations, the elevation rises sharply, and exposures of sandstone are present along the face of the hills.

3 SUBSURFACE CONDITIONS

3.1 General

Subsurface materials and conditions at the Agency Creek site were investigated on April 26 and 27, 2021, with one boring, designated B-1 advanced to a depth of about 120.3 feet. Subsurface materials and conditions at the Warren Slough site were investigated on April 28 and 29, 2021, with two borings, designated B-2 and B-2A, advanced to depths of about 100.7 feet and 33.5 feet, respectively. Logs of the borings are provided on Figures 1A through 3A. The approximate locations of the borings are shown on the Site Plans, Figures 2 and 3.

Soil samples collected from the explorations were returned to our laboratory for further examination and physical testing. The field-investigation and laboratory-testing programs completed for this investigation are described in Appendix A. The terms used to describe the materials encountered in the explorations are defined in Tables 1A and 2A and on the attached legend. Table 3A in Appendix A provides a summary of the results of the laboratory testing completed.

3.2 Soils

The borings indicate that beneath the railroad base fill, the project sites are mantled primarily with alluvial silt and sand, which is in turn underlain by sandstone. The depth from the base of the railroad base fill to the top of the sandstone varies significantly between each site.

3.2.1 Agency Creek

For the purpose of discussion, the materials disclosed by boring B-1 have been grouped into the following categories based on their physical characteristics and engineering properties:

a. Railroad BASE (Fill)
b. Organic SILT (Alluvium)
c. SILT and SAND (Alluvium)
d. SANDSTONE (Gnat Creek Formation)

The following paragraphs provide a description of these materials.

a. Railroad BASE (Fill)

Railroad base fill consisting of relatively clean, angular, gravel- to cobble-sized rock fragments was encountered at the ground surface. The fill extends to a depth of about 6.3

feet. Based on our observations, we estimate the relative density of the fill is loose to medium dense.

b. Organic SILT (Alluvium)

Alluvial organic silt was encountered beneath the railroad base fill to a depth of about 15 feet at the location of boring B-1. The organic silt is dark brown and gray and contains varying percentages of clay and fine- to coarse-grained sand ranging from trace to some clay and a trace of sand to sandy. The organic silt contains abundant quantities of wood and fine roots. Based on standard penetration test (SPT) N-values and Torvane shear-strength values, the relative consistency of the organic silt ranges from very soft to soft. The natural moisture content of the organic silt ranges from about 75% to 126%, with the high moisture contents associated with the presence of abundant organics. The organic silt is highly compressible based on a one-dimensional consolidation test completed on a relatively undisturbed sample of organic silt at a depth of about 10.7 feet.

c. SILT and SAND (Alluvium)

Alluvial silt and sand were encountered beneath the organic silt to a depth of about 100 feet. The silt and sand are typically gray. Scattered organics consisting of plant and wood debris were observed throughout the alluvial silt and sand. Alluvial silt was encountered from about 16 feet to 25 feet, 28 feet to 40 feet, and 75 feet to 100 feet. Alluvial sand was encountered from about 15 feet to 16 feet, 25 feet to 28 feet and 40 feet to 75 feet.

The silt contains varying percentages of clay and fine- to coarse-grained sand ranging from a trace of clay to clayey and a trace of sand to sandy. Based on SPT N-values and Torvane shear-strength values, the relative consistency of the silt ranges from very soft to stiff and is typically very soft to soft to a depth of about 80 feet. The silt between about 80 feet and 100 feet is medium stiff to stiff. The natural moisture content of the silt ranges from about 29% to 65%.

The alluvial sand is silty, fine to coarse grained, and contains varying percentages of clay, ranging from trace to some clay. Based on SPT N-values, the relative density of the sand is very loose to medium dense and is typically very loose to loose. The natural moisture content of the sand ranges from about 23% to 55%.

d. SANDSTONE (Gnat Creek Formation)

The alluvium is underlain by Sandstone rock of the Gnat Creek Formation at a depth of about 100 feet below the top of the railroad embankment. The sandstone is gray, predominantly decomposed, laminated, and extremely soft to very soft (R0 to R1) on the rock hardness scale. SPT N-values in the sandstone are 50 blows for 3 inches to 4 inches of sampler penetration. Boring B-1 was terminated in the sandstone at a depth of about 120.3 feet.

3.2.2 Warren Slough

For the purpose of discussion, the materials disclosed by borings B-2 and B-2A have been grouped into the following categories based on their physical characteristics and engineering properties:

a. Railroad BASE (Fill)
b. Organic SILT (Alluvium)
c. SILT and SAND (Alluvium)
d. SANDSTONE (Gnat Creek Formation)

The following paragraphs provide a description of these materials.

a. Railroad BASE (Fill)

Railroad base fill consisting of relatively clean, angular, gravel- to cobble-sized rock fragments was encountered at the ground surface. The fill extends to a depth of about 3 feet. Based on our observations, we estimate the relative density of the fill is medium dense.

b. Organic SILT (Alluvium)

Alluvial organic silt was encountered beneath the railroad base fill to a depth of about 20 feet at the location of boring B-2. The organic silt is dark brown to brown and contains variable fine- to coarse-grained sand content, ranging from some to sandy, variable clay content, ranging from trace to some clay, and trace to some subrounded gravel. The organic silt contains abundant quantities of wood and fine roots. Based on SPT N-values, the relative consistency of the organic silt is very soft. The natural moisture content of the organic silt is greater than 100%, with the high moisture contents associated with the presence of abundant organics. Wood debris, absent of soil, was encountered between depths of about 15 feet and 20 feet.

c. SILT and SAND(Alluvium)

Alluvial silt and sand were encountered beneath the organic silt. The silt and sand alluvium extends to the maximum depth explored in boring B-2A (33.5 feet) and to a depth of about 48 feet in boring B-2.

The silt is dark brown to gray and contains varying percentages of clay and fine- to coarse-grained sand, ranging from a trace of clay to clayey and a trace of sand to sandy. Based on SPT N-values and Torvane shear-strength values, the relative consistency of the silt ranges from very soft to medium stiff. The natural moisture content ranges from about 40% to 77%. Boring B-2A was terminated in clayey silt at depth of about 33.5 feet. The silt is slightly overconsolidated and highly compressible based on a one-dimensional consolidation test completed on a relatively undisturbed sample of silt recovered in boring B-2A at a depth of about 26.3 feet.

The alluvial silty sand encountered at depths of between 45 feet and 48 feet in boring B-2 is gray mottled rust, fine to medium grained, and contains a trace of clay. Based on an SPT N-value, the relative density of the sand is loose. The natural moisture content of the silty sand is about 23%.

d. SANDSTONE (Gnat Creek Formation)

The alluvium is underlain by Sandstone rock of the Gnat Creek Formation at a depth of about 48 feet below the top of the railroad embankment. The sandstone grades from gray mottled rust to light gray, is predominantly decomposed, and is extremely soft to very soft (R0 to R1) on the rock hardness scale. SPT N-values in the sandstone are 50 blows for 2 inches to 5 inches of sampler penetration. Boring B-2 was terminated in the sandstone at a depth of about 100.6 feet.

3.3 Groundwater

Borings B-1 through B-2A were advanced using mud-rotary methods, which do not permit the observation of groundwater conditions during drilling. For design purposes, we recommend the groundwater be assumed at the level of the Columbia River. Based on our review of the preliminary 15% plans, we understand the mean lower low water (MLLW) and mean higher high water (MHHW) elevations are 0.73 feet to 8.88 feet, respectively, at the Agency Creek site and 0.86 foot to 8.89 feet, respectively, at the Warren Slough site.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 General

The borings indicate the Agency Creek and Warren Slough sites are mantled by soft/loose alluvial material beneath the railroad base fill. Groundwater will closely follow the level of the river and is typically near the ground surface of the floodplain. In our opinion, the piles should develop axial resistance in the underlying sandstone of the Gnat Creek Formation. The depths from the top of the railroad embankment to the top of the sandstone at the Agency Creek and Warren Slough are about 100 feet and 48 feet, respectively, based on the borings.

The sites are mantled with silty soil that is moisture sensitive, which will be a key earthwork consideration during construction, particularly where excavations are required below the groundwater and river levels. The following sections of this report provide our conclusions and recommendations for the design and construction of the bridges.

5 BRIDGE DESIGN RECOMMENDATIONS

5.1 Seismic Design Criteria

We understand that due to the likely extensive impacts of seismic hazards on the entire rail line in general, the new bridges will not be designed to resist hazards associated with liquefaction, lateral spreading, or other seismic hazards associated with especially poor soil conditions. However, we understand the bridge structures will be seismically designed for inertial loading based on the site-response spectra developed in accordance with the AREMA Manual. In this regard, we have assumed the design will be based on the code-based site class and that no site-response analysis will be required for special soil conditions. Based on review of the soil conditions at the sites relative to site class definitions provided in Table 9-1-6 of the AREMA Manual, we recommend site-response coefficients for Site Class E for non-liquefied conditions be used for design of the bridges. The AREMA Manual defines three ground-motion levels (GML) to define acceptable risk/damage:

- GML 1 represents an occasional event that has a reasonable probability of being exceeded during the life of the structure. GML 1 corresponds to a Serviceability Performance Criteria Limit.

- GML 2 represents a rare event that has a low probability of being exceeded during the life of the structure. GML 2 corresponds to an Ultimate Performance Criteria Limit.

- GML 3 represents a very rare event or maximum credible event that has a very low probability of being exceeded during the life of the structure. GML 3 corresponds to a Survivability Performance Criteria Limit.

HPSI has indicated the recurrence intervals for GML 1, GML 2, and GML 3 are 100 years, 475 years, and 2,475 years, respectively. A summary of the seismic design parameters at the Agency Creek and Warren Slough sites for each of the GMLs is provided in Table 5-1. The parameters are based on the U.S. Geological Survey (USGS) 2014 database values.

Table 5-1: SUMMARY OF SEISMIC DESIGN PARAMETERS

Return Period (AREMA Level)	Recurrence Interval (R), years	Site	Seismic Parameters				
			PGA_R	S_s	F_a	S_1	F_v
GML 1 (Serviceability)	100	Agency Creek	0.12	0.11	2.50	0.03	3.50
		Warren Slough	0.13	0.11	2.50	0.03	3.50
GML 2 (Ultimate)	475	Agency Creek	0.35	0.46	1.82	0.16	3.33
		Warren Slough	0.35	0.46	1.83	0.15	3.34
GML 3 (Survivability)	2,475	Agency Creek	0.58	1.30	0.90	0.56	2.40
		Warren Slough	0.57	1.34	0.90	0.55	2.40

5.2 Foundation Support

5.2.1 General

We understand HP14x89 H-piles and PP16x0.5 closed-ended pipe piles are being considered for bridge foundations. Based on correspondence with HPSI, we understand the bridge piles will require an allowable design capacity of about 200 kips. A factor of safety of 2.5 is required per AREMA guidelines.

5.2.2 Pile Capacity

The HP14x89 piles and PP16x0.5 closed-ended pipe piles will develop their axial capacity from a combination of skin friction and end bearing in the underlying extremely soft to very soft (R0 to R1) sandstone. In our opinion, the required compressive capacity should be readily achievable for the considered pile types driven into the underlying sandstone with an adequately sized impact hammer.

Settlement of the piles driven into the extremely soft to very soft sandstone and designed based on the estimated allowable capacities provided above are anticipated to be limited to the elastic shortening of the pile plus about ¼ inch.

5.2.3 Installation Considerations.

The actual pile penetration into the underlying sandstone is difficult to predict due to variations in the weathering and hardness of R0 to R1 sandstone. However, it is our experience that an open-ended pipe or HP pile can typically achieve larger penetrations relative to closed-ended piles in extremely soft to very soft rock. To confirm the piles have achieved the required axial capacity and to evaluate the terminal driving criteria, we recommend completing Pile-Driving Analyzer (PDA) testing on at least one pile at each bridge site.

At the Warren Slough site, sandstone was encountered at a depth of about 45 feet below the base of the railbed fill. We anticipate the 500-kip ultimate pile compression capacity of the PP16X0.5-inch piles driven close-ended can be achieved at about 1- to 5-foot pile penetration into the R0 to R1 sandstone. The HP14X89 piles will drive further into the sandstone than a closed-ended pile. We anticipate H-pile penetration could be 10 feet or more, depending somewhat on variability in the degree of sandstone weathering.

Due to the depth of penetration anticipated at the Agency Creek site, it may be necessary to drive the piles in two or more sections, requiring the pile to be spliced after each section is driven. When selecting the hammer size, the contractor should consider the possibility of setup that could occur during the time required to splice the pile and subsequent driving.

If a vibratory hammer is used to drive the piles through the silt and sand, an adequately sized impact hammer will be required to develop the required capacity in the underlying sandstone. We anticipate commonly available, diesel impact pile-driving hammers will be suitable for driving the pile sections. To avoid damage to the piles during installation, driving stresses should not exceed 0.9 fy for steel piles. Piles driven into the underlying rock should be provided with commercially available tip protection. Subsection 00520.20(d) of the 2020 Oregon Department of Transportation *Standard Specifications for Construction* recommends an impact hammer with a minimum field energy of 30,000 foot-pounds be utilized to achieve a nominal pile bearing resistance of 400 kips to 500 kips. However, our opinion is that the contractor should select the hammer size based on a Wave Equation Analysis of Pile Driving (WEAP) analysis. Piles should have a minimum center-to-center spacing of at least three pile diameters.

A description of the proposed pile-driving equipment and accessories to be used for installation of the piles should be provided to the geotechnical engineer for review prior to mobilizing the equipment to the site. We recommend a geotechnical engineer from GRI observe pile installation and a continuous record of the driving resistance (blows/foot or blows/inch) for each pile driven be maintained for the full depth of penetration of each pile.

5.2.4 Lateral Pile Capacity

For lateral-loading conditions, we understand the piles will be evaluated using the computer software LPILE developed by Ensoft, Inc., of Austin, Texas. We understand the foundations will not be designed to consider seismic liquefaction, soil strength loss or lateral deformations. Our recommended non-seismic LPILE parameters to be used in lateral pile capacity analyses at the Agency Creek and Warren Slough bridge sites are tabulated below in Tables 5-2 and 5-3, respectively.

Table 5-2: AGENCY CREEK (B-1) LPILE INPUT PARAMETERS

Depth, feet[(a)(b)]	Soil Type (LPILE p-y Model)	Soil and Rock Properties				
		γ', pcf	c, psf	ϕ'	k, pci	ε_{50}
0.0 to 9.0	Silt (Soft Clay)	35	250	NA	NA	0.020
9.0 to 69.0	Silty Sand (API Sand)	53	NA	30°	20	NA
69.0 to 94.0	Clayey Silt (Soft Clay)	58	1,000	NA	NA	0.010
		γ', pcf	q_u, psi	E, psi	RQD, %	K_{rm}
94.0 and below	Weak Rock	68	500	5,000	10	0.0005

Notes:

a) Depth is below the base of the existing railroad base fill.

b) Groundwater is assumed to be at the ground surface at the base of the railbed fill.

Table 5-3: WARREN SLOUGH (B-2) LPILE INPUT PARAMETERS

Depth, feet[a][b]	Soil Type (LPILE p-y Model)	Soil and Rock Properties				
		γ', pcf	c, psf	ϕ'	k, pci	ε_{50}
0.0 to 17.0	Silt (Soft Clay)	35	250	NA	NA	0.020
17.0 to 42.0	Clayey Silt (Soft Clay)	48	250	NA	NA	0.020
42.0 to 45.0	Silty Sand (API Sand)	53	NA	35°	20	NA
		γ', pcf	q_u, psi	E, psi	RQD, %	K_{rm}
45.0 and below	Weak Rock	63	500	5,000	10	0.0005

Notes:

c) Depth is below the base of the existing railroad base fill.

d) Groundwater is assumed to be at the ground surface at the base of the railbed fill.

It should be noted that LPILE provides isolated, single-pile capacities. Depending on the direction of the loading and the orientation of the piles, group effects should be considered for spacing less than five pile diameters. This reduction is often applied as a p-multiplier, which LPILE uses as a reduction for pile spacing less than five pile diameters. The following table provides a summary of p-multipliers for various center-to-center pile spacing. Ground-slope effects can be taken into consideration with the input of an appropriate slope angle.

Table 5-4: LATERAL PILE GROUP ANALYSIS

Center-to-Center Pile Spacing	Calculated P-Multipliers for Rows 1, 2, and 3+
3D	0.80, 0.40, 0.30
4D	0.90, 0.65, 0.50
5D	1.00, 0.85, 0.70

5.3 Lateral Earth Pressures on Abutment Walls

The magnitude of lateral earth pressures that develop against retaining walls will depend on the type of backfill, backslope, method of backfill placement, degree of backfill compaction, magnitude and location of adjacent surcharge loads, and degree to which the wall can yield laterally during or after placement of backfill. We anticipate the abutment walls will be relatively rigid. For static, fully drained, horizontal backfill conditions, the abutment walls can be designed to resist an at-rest lateral earth pressure computed based on an equivalent fluid having a unit weight of 55 pounds per cubic foot (pcf). Additional lateral earth pressures due to surcharge loadings may be estimated using the guidelines presented on Figure 4.

In addition to the lateral earth pressures described above, the abutment and wing walls should be designed to accommodate surcharge loading in accordance with AREMA guidelines. If abutments will be designed to resist seismic loading, the methods of Agusti and Sitar (2013) can be used to develop the seismically induced lateral earth pressures. The method applies a triangular lateral earth pressure distribution with a pressure of 0H (pounds per square foot [psf]) at the ground surface where H is the height of the wall, and a maximum pressure at the base of the wall. Using this method and assuming a GML 3 event, the maximum pressure at the base of the wall is 23H (psf). The resultant force acts at a point above the base of the wall equal to one third the wall height. This pressure assumes the backfill behind the structure is horizontal.

The above criteria assume the abutments will be fully drained and backfilled with relatively clean, granular material, i.e., medium-grained sand, sand and gravel, or well-graded gravel, with not more than 5% passing the No. 200 sieve (washed analysis). We recommend this material be compacted to about 95% of the maximum dry density as determined by ASTM International D698. Heavy compaction equipment should not operate within 5 feet of the abutment.

6 DESIGN REVIEW AND CONSTRUCTION SERVICES

GRI should review geotechnical aspects of construction plans and specifications for this project as they are being developed. In addition, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we are of the opinion a representative from GRI should observe construction operations dealing with earthwork and pile installation. Our construction-phase services will allow for timely design changes if site conditions are encountered that differ from those described in this report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

7 LIMITATIONS

This report has been prepared to assist the owner and engineer in the design of this project. The scope is limited to the specific project and location described herein. Our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the new culverts and bridge. In the event that any changes in the design and location of the modifications as outlined in this report are planned, we should be given the opportunity to review the changes and to modify or reaffirm the conclusions and recommendations of this report in writing.

The analyses and recommendations submitted in this report are based on the data obtained from the subsurface explorations made at the locations shown on Figures 2 and 3 and from the other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between exploration locations and that groundwater levels will fluctuate with time. This report does not reflect any variations that may occur between these explorations. The nature and extent of variations may not become evident until construction. If, during construction, subsurface conditions differ from those described in this report or appear to be present beneath or beyond the limits of earthwork, we should be advised at once so that we can observe these conditions and reconsider our recommendations where necessary.

Submitted for GRI,

REGISTERED PROFESSIONAL ENGINEER
56553PE
OREGON
July 9, 2001
MATTHEW S. SHANAHAN

RENEWS: 06/2024

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This document has been submitted electronically.

8 REFERENCES

Agusti, G. C., and Sitar, N., 2013, Seismic earth pressures on retaining structures in cohesive soils, University of California, Berkeley, UCB GT 13-02.

American Railway Engineering and Maintenance-of-Way Association (AREMA), 2019, AREMA manual for railway engineering.

Niem, A.R., and Niem, W., 1985, Geologic map of the Astoria Basin, Clatsop and northernmost Tillamook Counties, northwest Oregon: Portland, Oreg., Oregon Dept. of Geology and Mineral Industries Oil and Gas Investigation Map OGI-14, Plate 1, scale 1:100,000.

Oregon Department of Transportation, 2021, Standard specifications for construction.

GRI

INTER-FLUVE
AGENCY CREEK/WARREN SLOUGH
RAILROAD BRIDGES

VICINITY MAP

JULY 2021 | JOB NO. 6474-A | FIG. 1

BORING COMPLETED BY GRI
(APRIL 26-27, 2021)

0 300 600
Feet

GRI

INTER-FLUVE, INC.
AGENCY CREEK AND WARREN SLOUGH

SITE PLAN
AGENCY CREEK

JULY 2021 JOB NO. 6474-A FIG. 2

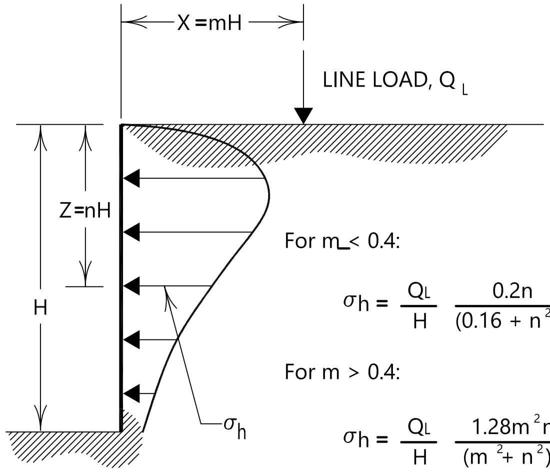
BORING COMPLETED BY GRI
(APRIL 28-29, 2021)

0 300 600
Feet

GRI
INTER-FLUVE, INC.
AGENCY CREEK AND WARREN SLOUGH

SITE PLAN
WARREN SLOUGH

JULY 2021 JOB NO. 6474-A FIG. 3



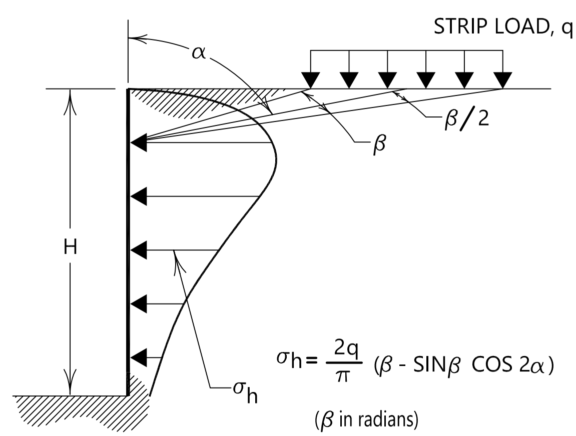
For $m \leq 0.4$:

$$\sigma_h = \frac{Q_L}{H} \frac{0.2n}{(0.16 + n^2)^2}$$

For $m > 0.4$:

$$\sigma_h = \frac{Q_L}{H} \frac{1.28m^2 n}{(m^2 + n^2)^2}$$

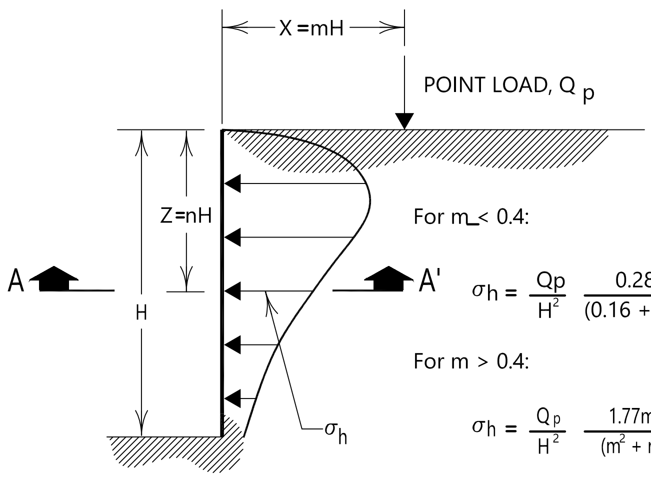
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{\pi} (\beta - \sin\beta \ \cos 2\alpha)$$

(β in radians)

STRIP LOAD PARALLEL TO WALL

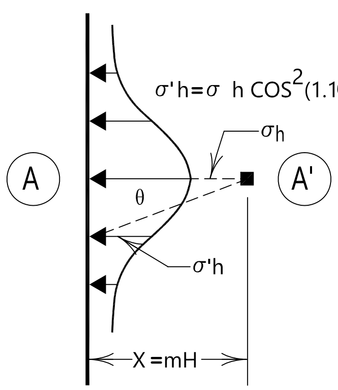


For $m \leq 0.4$:

$$\sigma_h = \frac{Q_p}{H^2} \frac{0.28n^2}{(0.16 + n^2)^3}$$

For $m > 0.4$:

$$\sigma_h = \frac{Q_p}{H^2} \frac{1.77m^2 n^2}{(m^2 + n^2)^3}$$



DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.

2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

GRI

INTER-FLUVE, INC.
AGENCY CREEK AND WARREN SLOUGH

SURCHARGE-INDUCED LATERAL PRESSURE

JULY 2021 | JOB NO. 6474-A | FIG. 4

APPENDIX A

Field Explorations and Testing and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND TESTING AND LABORATORY TESTING

A.1 FIELD EXPLORATIONS

Subsurface materials and conditions at the Agency Creek site were investigated between April 26 and 27, 2021, with one boring, designated B-1. Subsurface materials and conditions at the Warren Slough site were investigated on April 28 and 29, 2021, with two borings, designated B-2 and B-2A. The approximate locations of the borings are shown on the Site Plans, Figures 2 and 3. Boring 2A was completed approximately 5 feet northeast of boring B-2. The explorations were observed by an experienced member of GRI's engineering staff.

Borings B-1 and B-2 were advanced to depths of about 120.3 feet and 100.6 feet, respectively, and boring B-2A was advanced to a depth of about 33.5 feet. The borings were completed with mud-rotary drilling techniques using a GeoProbe 7822 DT track-mounted drill rig provided and operated by Western States Soil Conservation, Inc., of Hubbard, Oregon. Disturbed and undisturbed samples were typically obtained at 2.5-foot intervals of depth in the upper 20 feet and at 5-foot intervals below this depth. Disturbed samples were obtained using a standard split-spoon sampler. The standard penetration test (SPT) was conducted by driving the sampler into the soil a distance of 18 inches using a 140-pound hammer dropped 30 inches. The number of blows required to drive the SPT sampler the last 12 inches is known as the standard penetration resistance, or SPT N-value. The SPT N-values provide a measure of the relative density of granular soils, such as sand, and the relative consistency, or stiffness, of cohesive soils, such as silt. The split-spoon samples were carefully examined in the field and saved in airtight jars for further examination and physical testing in our laboratory.

Relatively undisturbed samples were collected by pushing a 3-inch-outside-diameter Shelby tube into the undisturbed soil a maximum distance of 24 inches using the hydraulic ram of the drill rig. The soil exposed in the end of each Shelby tube was examined and classified in the field. After classification, each tube was sealed with rubber caps and returned to our laboratory for further examination and testing.

Logs of the borings are provided on Figures 1A through 3A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth at which the materials and/or characteristics of the materials change. To the right of the descriptive summary the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, SPT N-values are shown graphically, along with natural moisture content values, dry unit weight, percentage of material passing the No. 200 sieve, and Torvane shear-strength values. The dry unit weight of selected soil samples is included in the far-right column of the logs. The terms used to describe the soil and rock are defined in Tables 1A and 2A.

A.2 LABORATORY TESTING

A.2.1 General

All samples obtained from the borings were returned to our laboratory, where the physical characteristics of the samples were noted, and the field classifications modified where necessary. The laboratory-testing program included determinations of natural moisture content and washed sieve analyses. Additional testing included Torvane shear strength, dry unit weight determination, one-dimensional consolidation, and triaxial compression testing. A summary of laboratory test results is provided in Table 3A. The following paragraphs describe the testing program in more detail.

A.2.2 Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM International (ASTM) D2216. The results are shown on Figures 1A through 3A and in Table 3A.

A.2.3 Washed-Sieve Analyses

Washed-sieve analyses were performed for selected soil samples obtained from the borings to assist in their classification. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the sieve is oven-dried and reweighed, and the percentage of material (by weight) that passed the No. 200 sieve is calculated. Test results are tabulated below and shown on Figures 1A through 3A and in Table 3A.

A.2.4 Undisturbed Unit Weight

The unit weight, or density, of undisturbed soil samples was determined in the laboratory in substantial conformance with ASTM D2937. The results are summarized on Figures 1A through 3A and in Table 3A.

A.2.5 Torvane Shear Strength

The approximate undrained shear strength of relatively undisturbed fine-grained soil samples was determined using a Torvane shear device. The Torvane is a handheld apparatus with vanes that are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of the Torvane shear strength tests are summarized on Figures 1A through 3A.

A.2.6 One-Dimensional Consolidation

One-dimensional consolidation testing was performed in accordance with ASTM D2435 on two relatively undisturbed soil samples obtained in borings B-1 and B-2A at depths of about 10.7 feet and 26.3 feet, respectively. The test provides data on the compressibility of fine-grained soils. Test results are shown on Figures 4A and 5A in the form of curves showing effective stress versus percent strain. The initial dry unit weight and moisture content of the samples are also shown on the figure.

A.2.7 Triaxial Compression

Four anisotropically consolidated, undrained (CU) triaxial compression test with pore pressure measurements was performed on selected samples of soil from borings B-1 and B-2A. The samples were prepared from relatively undisturbed, 2.85-inch-inside-diameter Shelby tube samples. The triaxial test results will be included with the final report.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per ft
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows per ft	Torvane or Undrained Shear Strength, tsf
Very Soft	0 - 2	less than 0.125
Soft	2 - 4	0.125 - 0.25
Medium Stiff	4 - 8	0.25 - 0.50
Stiff	8 - 15	0.50 - 1.0
Very Stiff	15 - 30	1.0 - 2.0
Hard	over 30	over 2.0

Grain-Size Classification

Boulders:
>12 in.

Cobbles:
3-12 in.

Gravel:
¼ - ¾ in. (fine)
¾ - 3 in. (coarse)

Sand:
No. 200 - No. 40 sieve (fine)
No. 40 - No. 10 sieve (medium)
No. 10 - No. 4 sieve (coarse)

Silt/Clay:
Pass No. 200 sieve

Modifier for Subclassification

Adjective	Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
	Percentage of Other Material (By Weight)	
trace:	5 - 15 (sand, gravel)	5 - 15 (sand, gravel)
some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
trace:	<5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
some:	5 - 12 (silt, clay)	
silty, clayey:	12 - 50 (silt, clay)	

Table 2A

GUIDELINES FOR CLASSIFICATION OF ROCK

Relative Rock Weathering Scale

Term	Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 in. into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominantly Decomposed	Rock mass is more than 50% decomposed. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to soil with hand pressure.

Relative Rock Hardness Scale

Term	Hardness Designation	Field Identification	Approximate Unconfined Compressive Strength
Extremely Soft	R0	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	< 100 psi
Very Soft	R1	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocket knife and scratched with fingernail.	100 - 1,000 psi
Soft	R2	Can be peeled by a pocket knife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.	1,000 - 4,000 psi
Medium Hard	R3	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.	4,000 - 8,000 psi
Hard	R4	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.	8,000 - 16,000 psi
Very Hard	R5	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	> 16,000 psi

RQD and Rock Quality

Relation of RQD and Rock Quality	
RQD (Rock Quality Designation), %	Description of Rock Quality
0 - 25	Very Poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Terminology for Planar Surface		
Bedding	Joints and Fractures	Spacing
Laminated	Very Close	< 2 in.
Thin	Close	2 in. – 12 in.
Medium	Moderately Close	12 in. – 36 in.
Thick	Wide	36 in. – 10 ft
Massive	Very Wide	> 10 ft

Table 3A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-1	S-1	5.0	7.0	111	--	--	--	--	GRAVEL
	S-2	7.5	4.5	126	--	--	--	--	ASPHALT
	S-3	10.5	1.5	68	--	--	--	52	ASPHALT
	S-3	10.9	1.1	80	54	--	--	--	ASPHALT
	S-4	12.0	0.0	75	--	--	--	76	ASPHALT
	S-5	15.0	-3.0	36	--	--	--	16	Silty SAND
	S-6	17.5	-5.5	38	80	--	--	60	Sandy SILT
	S-7	19.5	-7.5	65	--	--	--	98	Sandy SILT
	S-8	25.5	-13.5	29	--	--	--	15	Silty SAND
	S-8	26.3	-14.3	33	87	--	--	--	Silty SAND
	S-9	27.0	-15.0	32	--	--	--	18	Silty SAND
	S-10	30.0	-18.0	43	--	--	--	50	Sandy SILT
	S-11	35.0	-23.0	41	--	--	--	--	Sandy SILT
	S-12A	40.0	-28.0	20	--	--	--	47	Silty SAND
	S-13	45.0	-33.0	23	--	--	--	25	Silty SAND
	S-14	50.9	-38.9	27	--	--	--	46	Silty SAND
	S-14	51.5	-39.5	24	--	--	--	39	Silty SAND
	S-15	52.0	-40.0	31	--	--	--	40	Silty SAND
	S-16	55.0	-43.0	28	--	--	--	--	Silty SAND
	S-17	60.0	-48.0	40	--	--	--	47	Silty SAND
	S-18	65.0	-53.0	55	--	--	--	--	Silty SAND
	S-19	70.8	-58.8	26	--	--	--	24	Silty SAND
	S-19	71.5	-59.5	26	96	--	--	--	Silty SAND
	S-20	72.0	-60.0	39	--	--	--	29	Silty SAND
	S-21	75.0	-63.0	52	--	--	--	--	SILT
	S-22	80.0	-68.0	37	--	--	--	77	SILT
	S-23	85.0	-73.0	41	--	--	--	--	SILT
	S-24	91.3	-79.3	37	--	--	--	82	SILT
	S-24	91.4	-79.4	29	94	--	--	--	SILT
	S-25	91.5	-79.5	29	--	--	--	--	SILT
	S-26	95.0	-83.0	39	--	--	--	--	SILT
	S-27	100.0	-88.0	22	--	--	--	31	SANDSTONE
	S-28	105.0	-93.0	26	--	--	--	--	SANDSTONE
	S-29	110.0	-98.0	26	--	--	--	--	SANDSTONE
	S-30	115.0	-103.0	25	--	--	--	--	SANDSTONE
	S-31	120.0	-108.0	24	--	--	--	--	SANDSTONE
B-2	S-1	5.0	7.0	367	--	--	--	--	ASPHALT
	S-2	7.5	4.5	133	--	--	--	--	ASPHALT
	S-3	10.0	2.0	151	--	--	--	--	ASPHALT
	S-4	12.5	-0.5	206	--	--	--	62	ASPHALT

Table 3A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-2	S-7	20.0	-8.0	62	--	--	--	88	Clayey SILT
	S-8	25.0	-13.0	77	--	--	--	99	Clayey SILT
	S-9	30.0	-18.0	69	--	--	--	77	Clayey SILT
	S-11	37.0	-25.0	63	--	--	--	89	Clayey SILT
	S-12	40.3	-28.3	63	--	--	--	71	SILT
	S-12	40.6	-28.6	59	62	--	--	--	SILT
	S-12	41.1	-29.1	71	--	--	--	79	SILT
	S-13	42.0	-30.0	40	--	--	--	63	SILT
	S-14	45.0	-33.0	23	--	--	--	37	Silty SAND
	S-15	50.0	-38.0	23	--	--	--	--	SANDSTONE
	S-16	55.0	-43.0	24	--	--	--	--	SANDSTONE
	S-17	60.0	-48.0	25	--	--	--	--	SANDSTONE
	S-18	65.0	-53.0	24	--	--	--	--	SANDSTONE
	S-19	70.0	-58.0	25	--	--	--	--	SANDSTONE
	S-20	75.0	-63.0	24	--	--	--	--	SANDSTONE
	S-21	80.0	-68.0	24	--	--	--	--	SANDSTONE
	S-22	85.0	-73.0	25	--	--	--	--	SANDSTONE
	S-23	90.0	-78.0	25	--	--	--	--	SANDSTONE
	S-24	95.0	-83.0	25	--	--	--	--	SANDSTONE
	S-25	100.0	-88.0	24	--	--	--	--	SANDSTONE
B-2A	S-2	27.0	-15.0	67	--	--	--	--	Clayey SILT
	S-4	32.0	-20.0	71	--	--	--	--	Clayey SILT

BORING AND TEST PIT LOG LEGEND

SOIL SYMBOLS

Symbol	Typical Description
	LANDSCAPE MATERIALS
	FILL
	GRAVEL; clean to some silt, clay, and sand
	Sandy GRAVEL; clean to some silt and clay
	Silty GRAVEL; up to some clay and sand
	Clayey GRAVEL; up to some silt and sand
	SAND; clean to some silt, clay, and gravel
	Gravelly SAND; clean to some silt and clay
	Silty SAND; up to some clay and gravel
	Clayey SAND; up to some silt and gravel
	SILT; up to some clay, sand, and gravel
	Gravelly SILT; up to some clay and sand
	Sandy SILT; up to some clay and gravel
	Clayey SILT; up to some sand and gravel
	CLAY; up to some silt, sand, and gravel
	Gravelly CLAY; up to some silt and sand
	Sandy CLAY; up to some silt and gravel
	Silty CLAY; up to some sand and gravel
	PEAT

BEDROCK SYMBOLS

Symbol	Typical Description
	BASALT
	MUDSTONE
	SILTSTONE
	SANDSTONE

SURFACE MATERIAL SYMBOLS

Symbol	Typical Description
	Asphalt concrete PAVEMENT
	Portland cement concrete PAVEMENT
	Crushed rock BASE COURSE

SAMPLER SYMBOLS

Symbol	Sampler Description
	2.0 in. O.D. split-spoon sampler and Standard Penetration Test with recovery (ASTM D1586)
	Shelby tube sampler with recovery (ASTM D1587)
	3.0 in. O.D. split-spoon sampler with recovery (ASTM D3550)
	Grab Sample
	Rock core sample interval
	Sonic core sample interval
	Push probe sample interval

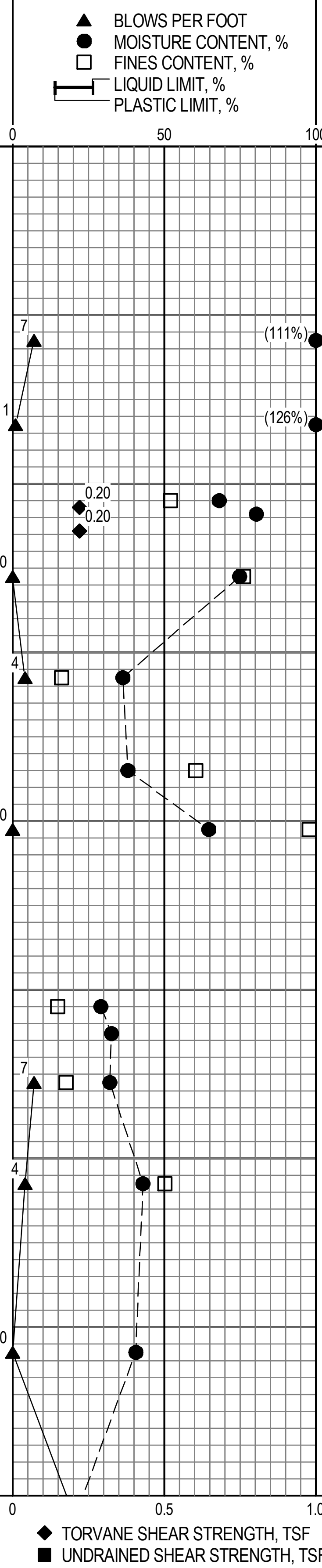
INSTALLATION SYMBOLS

Symbol	Symbol Description
	Flush-mount monument set in concrete
	Concrete, well casing shown where applicable
	Bentonite seal, well casing shown if applicable
	Filter pack, machine-slotted well casing shown where applicable
	Grout, vibrating-wire transducer cable shown where applicable
P	Vibrating-wire pressure transducer
	1-in.-diameter solid PVC
	1-in.-diameter hand-slotted PVC
	Grout, inclinometer casing shown where applicable

FIELD MEASUREMENTS

Symbol	Typical Description
	Groundwater level during drilling and date measured
	Groundwater level after drilling and date measured
	Rock/sonic core or push probe recovery (%)
	Rock quality designation (RQD, %)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: 12.0 ft [±] (NAVD88)						
0		Railway BASE at ground surface (Fill)						
5					S-1		5 6 1	
		Organic SILT, some fine- to coarse-grained sand to sandy, trace clay, dark brown, very soft to soft, contains abundant wood debris and fine roots (Alluvium) ---some clay, dark brown and gray below 7.5 ft ---trace to some fine-grained sand, gray below 10 ft	5.8 / 6.3		S-2		0 1 0	
10					S-3			Dry Density = 54 pcf
					S-4		0 0 0	
15		Silty SAND, gray, very loose to loose, fine grained, contains organics (Alluvium)	-3.0 / 15.0		S-5		1 2 2	
		Sandy SILT, trace clay, gray, soft, contains organics (Alluvium)	-4.0 / 16.0		S-6			Dry Density = 80 pcf
20					S-7		0 0 0	
25		Silty SAND, gray, very loose, fine grained, contains organics (Alluvium)	-13.0 / 25.0		S-8			Dry Density = 87 pcf
		Sandy SILT, trace to some clay, gray, soft, contains organics (Alluvium)	-16.0 / 28.0		S-9		0 3 4	
30					S-10		0 1 3	
35		---some clay, dark brown to gray, very soft at 35 ft			S-11		0 0 0	
40								



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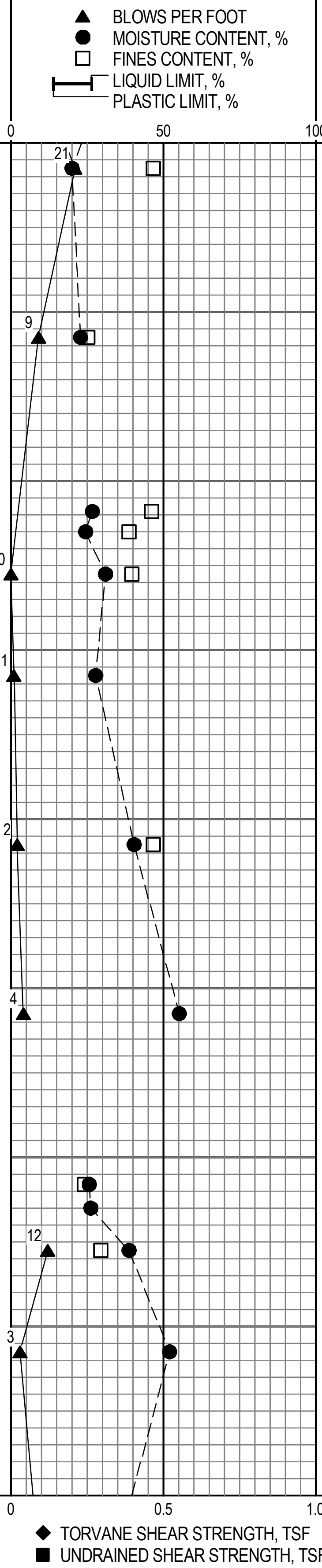
Logged By: T. Wilcox	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 04/26/21	**Coordinates:** Not Available
Drilling Method: Mud Rotary **Equipment:** Geoprobe 7822 DT **Hole Diameter:** 4 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 80%
Note: See Legend for Explanation of Symbols	

GRI

BORING B-1 (Agency Creek)

JULY 2021 JOB NO. 6474-A FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: 12.0 ft [±] (NAVD88)						
		Silty SAND, some clay, gray, medium dense, fine to coarse grained (Alluvium)			S-12A		5 12 9	
45		---trace to some clay, loose, fine-grained sand below 45 ft			S-13		4 5 4	
50					S-14		0	
		---very loose below 51.5 ft			S-15		0 0 0	
55		---contains wood and plant debris below 55 ft			S-16		0 0 1	
60					S-17		0 0 2	
65		---very loose to loose, contains abundant wood debris at 65 ft			S-18		1 1 3	
70		---medium dense, fine to coarse grained, contains wood debris below 70 ft			S-19			Dry Density = 96 pcf
					S-20		2 4 8	
75		Clayey SILT, some fine-grained sand, gray, soft, contains wood debris (Alluvium)	-63.0 / 75.0		S-21		1 1 2	
80								



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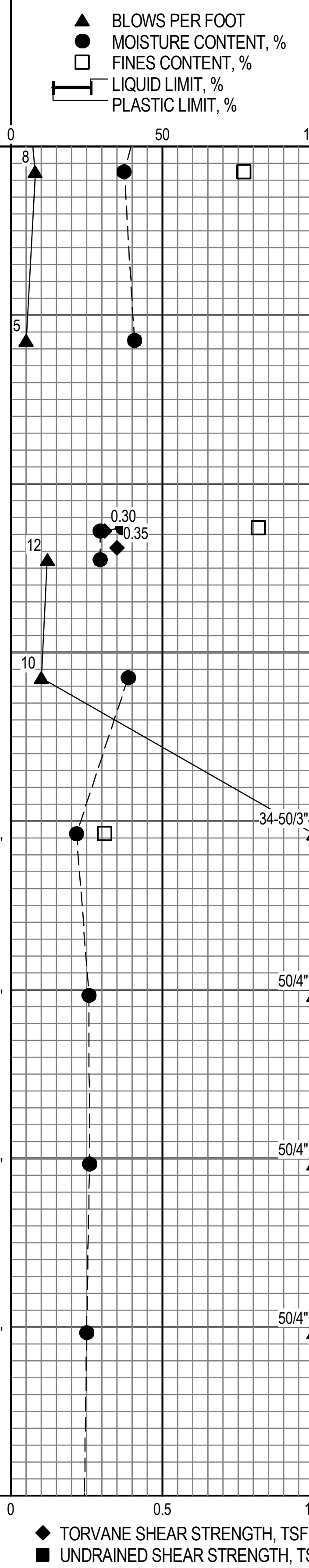
GRI BORING LOG (LAT/LONG) GRI DATA TEMPLATE.GDT 07/19/21

GRI

BORING B-1 (Agency Creek)

JULY 2021 JOB NO. 6474-A FIG. 1A

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Surface Elevation: 12.0 ft [±] (NAVD88)					
	Clayey SILT, some fine-grained sand, gray, medium stiff to stiff (Alluvium)			S-22	1 3 5	
85	---medium stiff below 85 ft			S-23	2 2 3	
90	---contains wood debris below 90 ft			S-24		
	---stiff below 91.5 ft			S-25	3 5 7	Dry Density = 94 pcf
95				S-26	1 0 10	
100	SANDSTONE, gray, predominantly decomposed, extremely soft to very soft (R0 to R1), laminated (Gnat Creek Formation)	-88.0 / 100.0		S-27	34 50/3"	
105				S-28	50/4"	
110				S-29	50/4"	
115				S-30	50/4"	
120						



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GRI BORING LOG (LAT/LONG) GRI DATA TEMPLATE.GDT 07/19/21

GRI

BORING B-1
(Agency Creek)

JULY 2021 JOB NO. 6474-A FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL Surface Elevation: 12.0 ft [±] (NAVD88)	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	▲ BLOWS PER FOOT ● MOISTURE CONTENT, % □ FINES CONTENT, % LIQUID LIMIT, % PLASTIC LIMIT, % (0 — 50 — 50/3" 00)	COMMENTS AND ADDITIONAL TESTS
		SANDSTONE, gray, predominantly decomposed, extremely soft to very soft (R0 to R1), laminated (Gnat Creek Formation)	-108.3 120.3		S-31		50/3"	50/3"	
		(4/27/2021)							
125									
130									
135									
140									
145									
150									
155									
160									

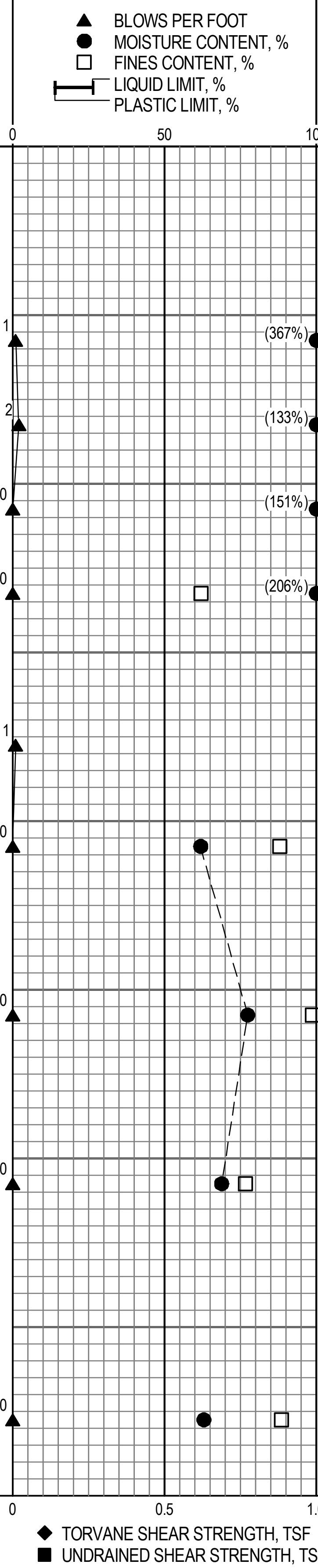
0 — 0.5 — 1.0

◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

GRI

BORING B-1
(Agency Creek)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: 12.0 ft [±] (NAVD88)						
0		Railway BASE at ground surface (Fill)						
3		Organic SILT, some fine- to coarse-grained sand to sandy, trace to some clay and subrounded gravel, dark brown to brown, very soft, contains abundant wood debris and fine roots (Alluvium)	9.0 / 3.0					Driller noted softer drilling below 3 ft
5					S-1		0 1 0	Gravel piece in sample shoe prevented more recovery (S-1)
					S-2		0 1 1	After gravel in sample shoe prevented recovery, a 3-in. California sampler was used at the same depth to recover the sample (S-2)
10					S-3		1 0 0	
					S-4		0 0 0	
15		Wood Debris (Alluvium)	-3.0 / 15.0		S-5			
					S-6		1 0 1	
20		Clayey SILT, trace fine-grained sand, dark brown and gray, very soft, contains wood debris, contains organics (Alluvium)	-8.0 / 20.0		S-7		0 0 0	
25		---gray below 25 ft			S-8		0 0 0	
30		---some fine-grained sand at 30 ft			S-9		0 0 0	
35		---trace to some clay and fine-grained sand below 35 ft			S-10			
					S-11		0 0 0	
40								



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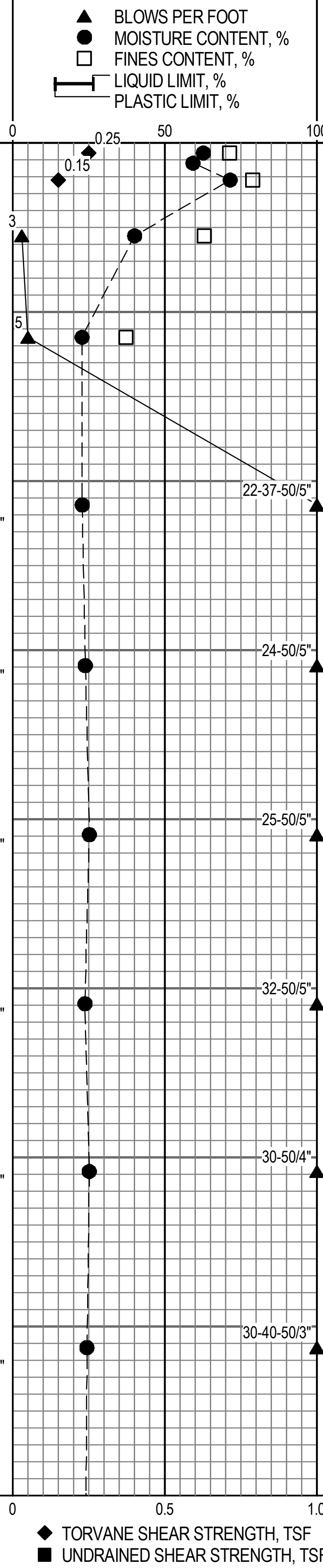
Logged By: T. Wilcox	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 04/28/21	**Coordinates:** Not Available
Drilling Method: Mud Rotary **Equipment:** Geoprobe 7822 DT **Hole Diameter:** 4 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 80%
Note: See Legend for Explanation of Symbols	

GRI

BORING B-2 (Warren Slough)

JULY 2021 JOB NO. 6474-A FIG. 2A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL Surface Elevation: 12.0 ft [±] (NAVD88)	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		SILT,some fine- to medium-grained sand to sandy, trace clay, gray, soft to medium stiff, contains up to 2-in.-thick interbedded lenses of silty sand (Alluvium)			S-12			Dry Density = 62 pcf
					S-13		0 0 3	
45		Silty SAND, trace clay, gray mottled rust, loose, fine to medium grained (Alluvium)	-33.0 45.0		S-14		0 0 5	
		SANDSTONE, gray mottled rust, predominantly decomposed, extremely soft to very soft (R0 to R1), laminated (Gnat Creek Formation)	-36.0 48.0					Driller noted hard drilling below 48 ft
50					S-15		22 37 50/5"	
55					S-16		24 50/5"	
60					S-17		25 50/5"	
65					S-18		32 50/5"	
70					S-19		30 50/4"	
75					S-20		30 40 50/3"	
80								



(CONTINUED NEXT PAGE)

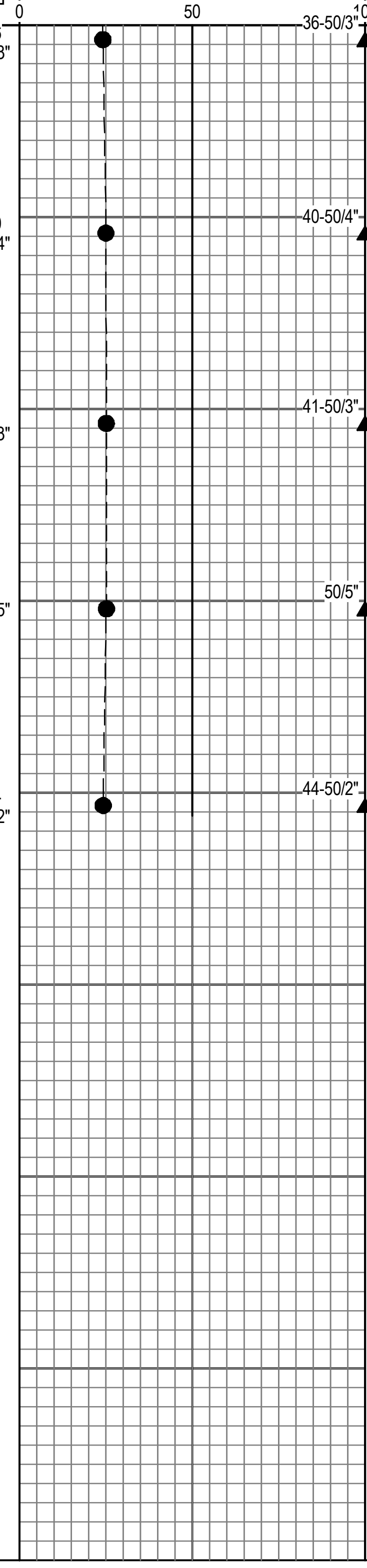
GRI BORING LOG (LAT/LONG) GRI DATA TEMPLATE.GDT 07/19/21

GRI

BORING B-2
(Warren Slough)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: 12.0 ft [±] (NAVD88)						
		SANDSTONE, gray mottled rust, predominantly decomposed, extremely soft to very soft (R0 to R1), laminated (Gnat Creek Formation)			S-21	I	36 50/3"	
85		---light gray below 85 ft			S-22	I	40 50/4"	
90					S-23	I	41 50/3"	
95					S-24	I	50/5"	
100			-88.6 / 100.6		S-25	I	44 50/2"	
		(4/29/2021)						
105								
110								
115								
120								

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %
□ FINES CONTENT, %
LIQUID LIMIT, %
PLASTIC LIMIT, %

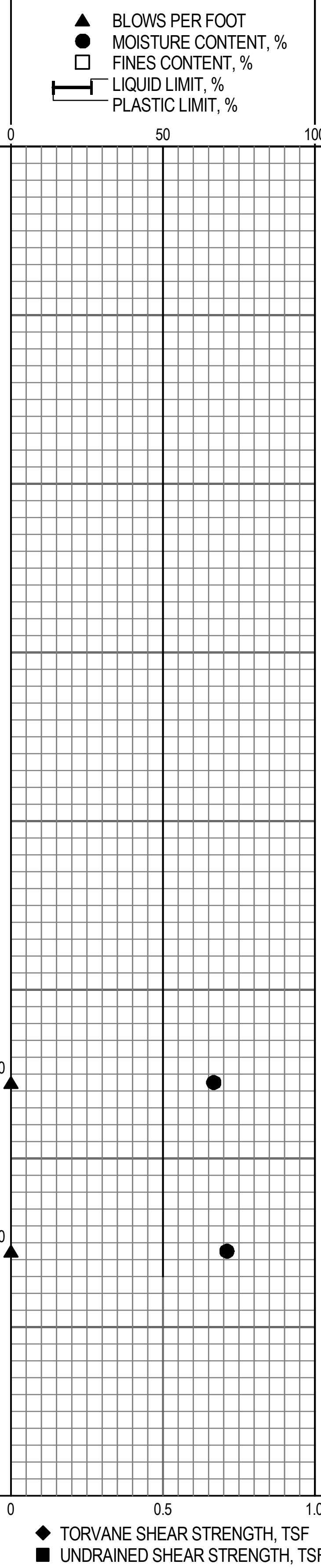


◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

GRI

BORING B-2
(Warren Slough)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: 12.0 ft [±] (NAVD88)						
0		Blind drilled to 25 feet without sampling						Boring located approximately 5 ft NE (along the railway tracks) from B-2. Blind drilled to 25 ft to take Shelby tubes. Silt, sand, and organic debris was observed in cuttings down to roughly 20 ft.
25		Clayey SILT, trace fine-grained sand, gray, very soft to soft (Alluvium)	-13.0 / 25.0		S-1			
					S-2		0 0 0 0	
30		---some sand below 30 ft			S-3			
					S-4		0 0 0 0	
33.5		(4/29/2021)	-21.5 / 33.5					

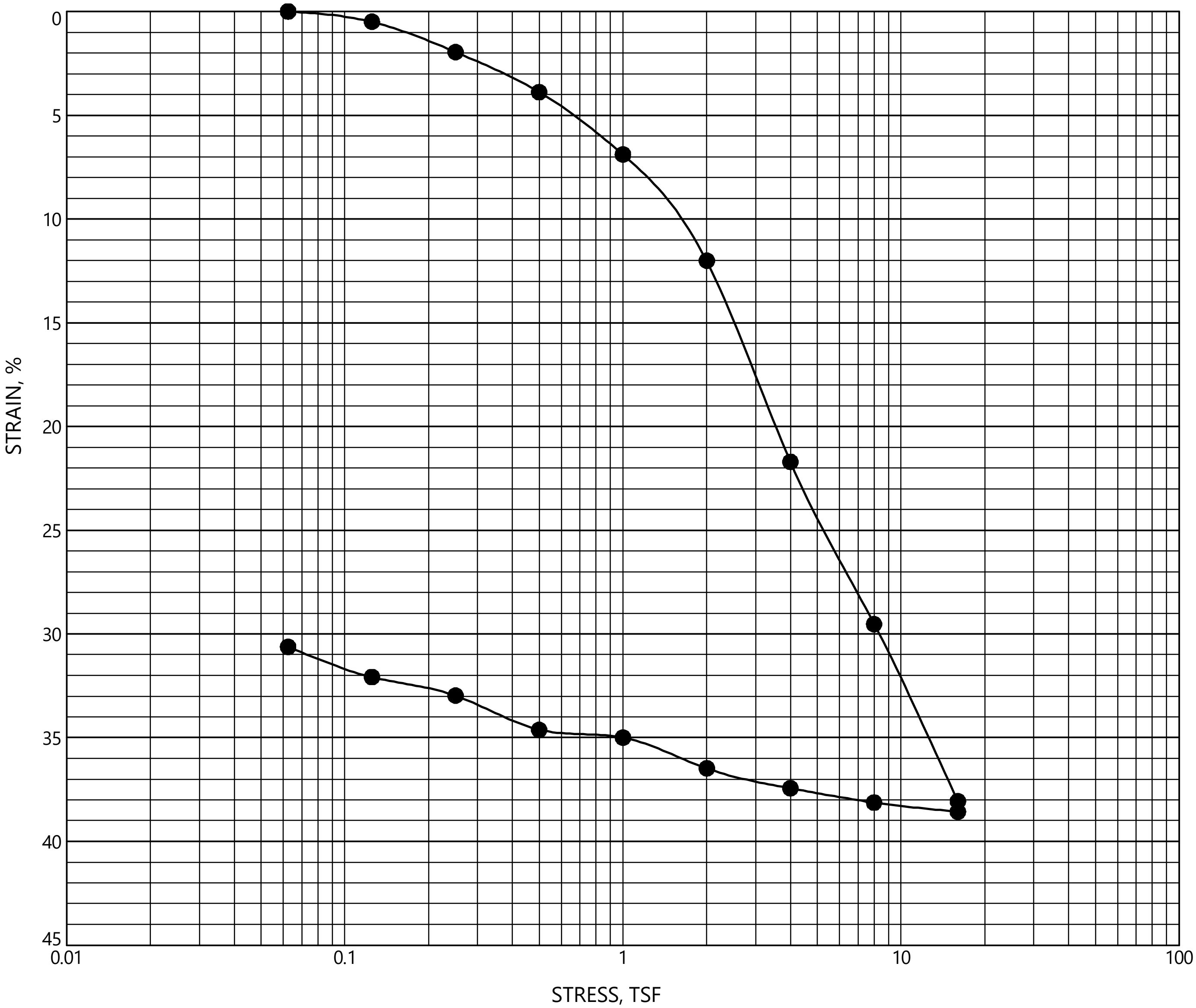


Logged By: T. Wilcox	**Drilled by:** Western States Soil Conservation, Inc.	
Date Started: 04/29/21	**Coordinates:** Not Available	
Drilling Method: Mud Rotary **Equipment:** Geoprobe 7822 DT **Hole Diameter:** 4 in.		**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 80%
Note: See Legend for Explanation of Symbols		

GRI

BORING B-2A (Warren Slough)

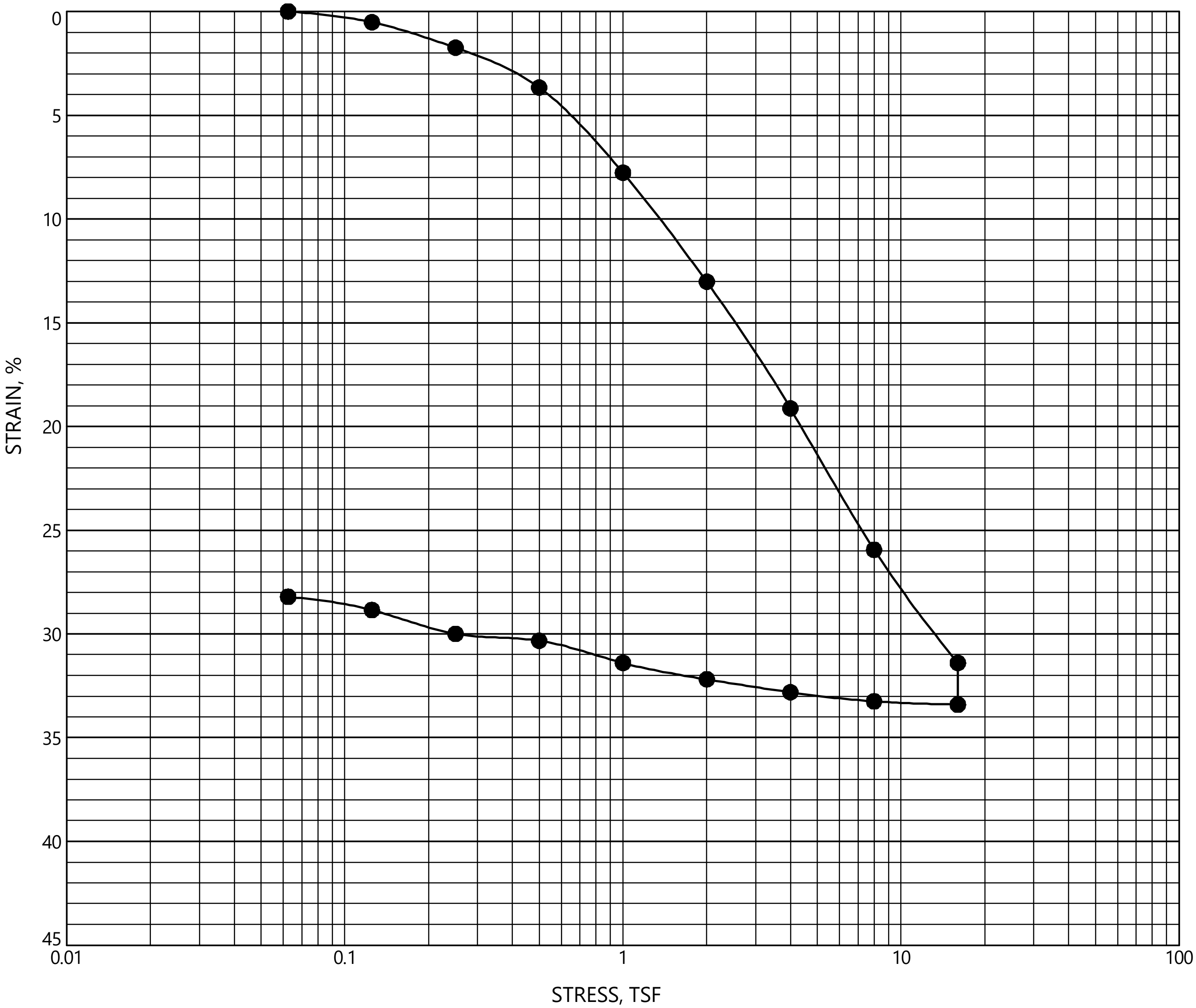
JULY 2021 JOB NO. 6474-A FIG. 3A



	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-1	S-3	10.7	SILT, trace to some clay and fine-grained sand, gray, very soft, contains abundant plant debris	51	79

GRI

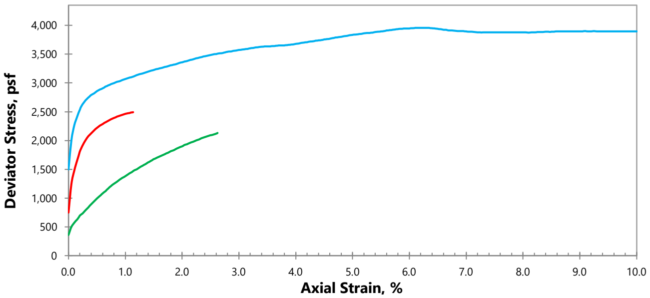
CONSOLIDATION TEST

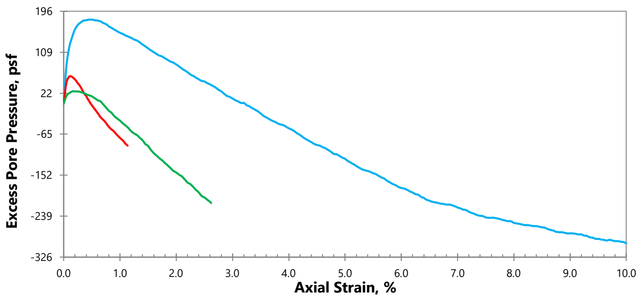


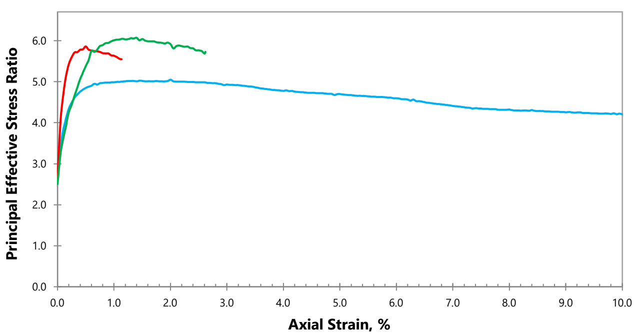
	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-2A	S-1	26.3	SILT, some clay to clayey, trace fine-grained sand, gray, very soft to soft, contains plant debris	51	82

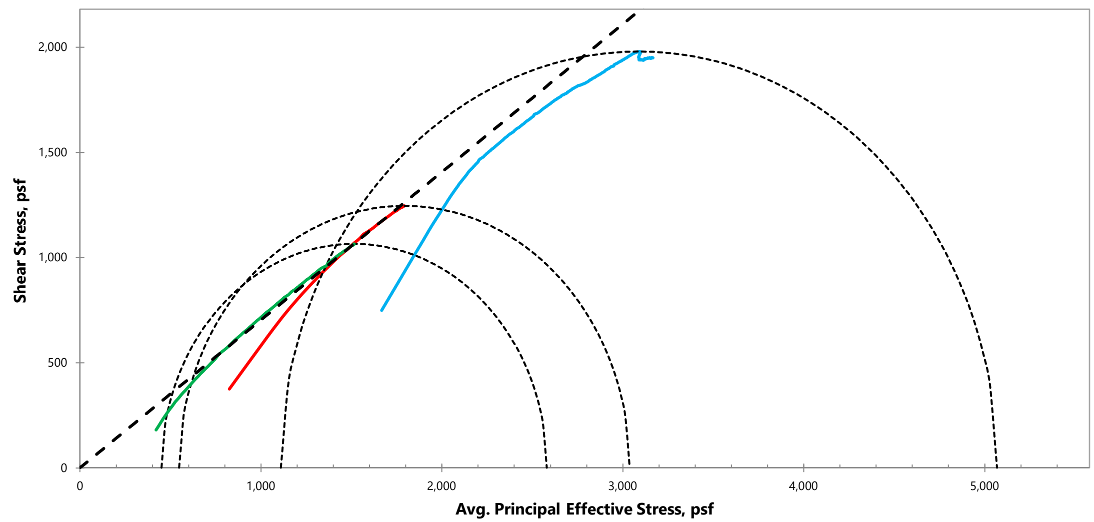
GRI

CONSOLIDATION TEST









	STAGE 1	STAGE 2	STAGE 3
TEST SYMBOL	■	■	■
BORING NO.	B-1	B-1	B-1
SAMPLE NO.	S-6	S-6	S-6
DEPTH (FT)	17.5	17.5	17.5
VERTICAL EFFECTIVE CONSOLIDATION STRESS (PSF)	600	1200	2420
SAMPLE HEIGHT (IN)	6.00	6.00	6.00
INITIAL SAMPLE DIAMETER (IN)	2.87	2.87	2.87
DRY UNIT WEIGHT (PCF)	116	116	117
INITIAL WATER CONTENT (%)	38	38	38
FINAL WATER CONTENT (%)	16	16	15.7
STRAIN RATE (%/HR)	2		

TYPE OF TEST: ■ CU □ CD

FAILURE CRITERIA: ■ MAX DEVIATOR STRESS □ MAX STRESS RATIO □ % STRAIN

TYPE OF SAMPLE: ■ UNDISTURBED □ REMOLDED

■ BACK SATURATED

SOIL CLASSIFICATION:

ESTIMATED STRENGTH PARAMETERS

c = 0 PSF

ϕ = 35.1°

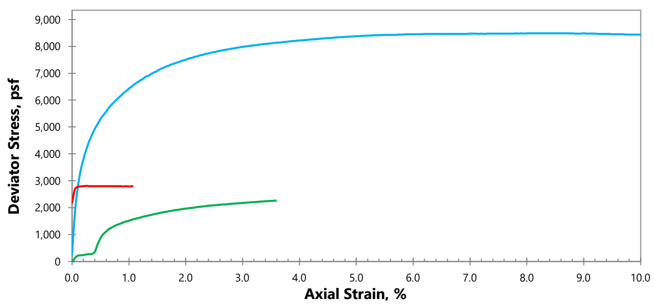
tan(ϕ) = 0.70

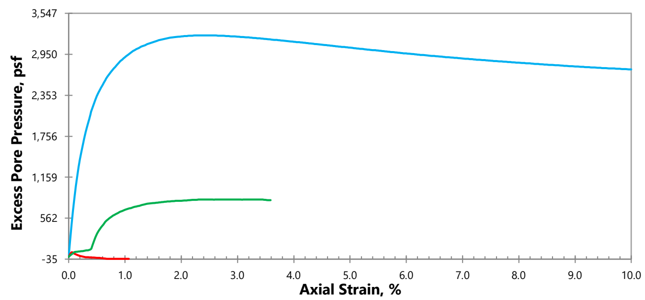
GRI

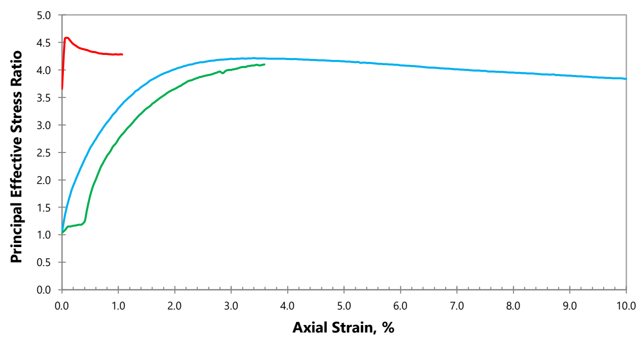
TRIAXIAL SHEAR STRENGTH TEST

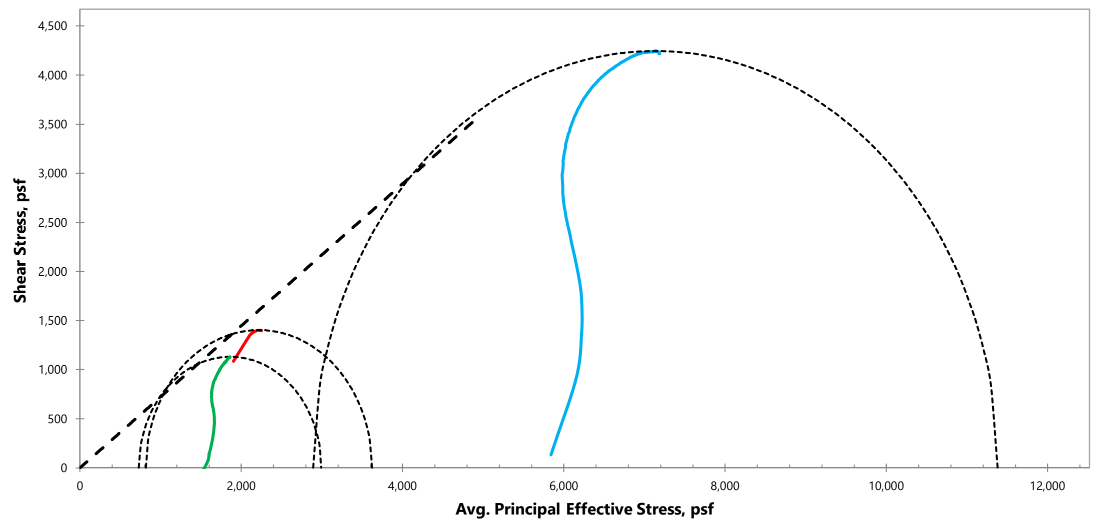
(BORING B-1, S-6)

JULY 2021 JOB NO. No. 6474-A: FIG. 6A









	STAGE 1	STAGE 2	STAGE 3
TEST SYMBOL	■	■	■
BORING NO.	B-1	B-1	B-1
SAMPLE NO.	S-14	S-14	S-14
DEPTH (FT)	51	51	51
VERTICAL EFFECTIVE CONSOLIDATION STRESS (PSF)	1500	2990	5970
SAMPLE HEIGHT (IN)	6.00	6.00	6.00
INITIAL SAMPLE DIAMETER (IN)	2.87	2.87	2.87
DRY UNIT WEIGHT (PCF)	122	116	121
INITIAL WATER CONTENT (%)	26.7	26.7	26.7
FINAL WATER CONTENT (%)	13.4	15.9	13.7
STRAIN RATE (%/HR)	2		

TYPE OF TEST: ■ CU ☐ CD

FAILURE CRITERIA: ■ MAX DEVIATOR STRESS ☐ MAX STRESS RATIO ☐ % STRAIN

TYPE OF SAMPLE: ■ UNDISTURBED ☐ REMOLDED

■ BACK SATURATED

SOIL CLASSIFICATION:

ESTIMATED STRENGTH PARAMETERS

c = 0 PSF

ϕ = 35.9°

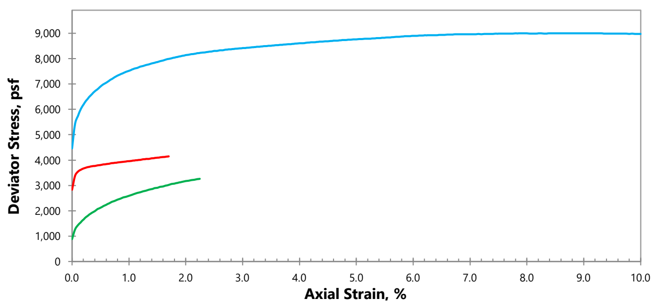
tan(ϕ) = 0.72

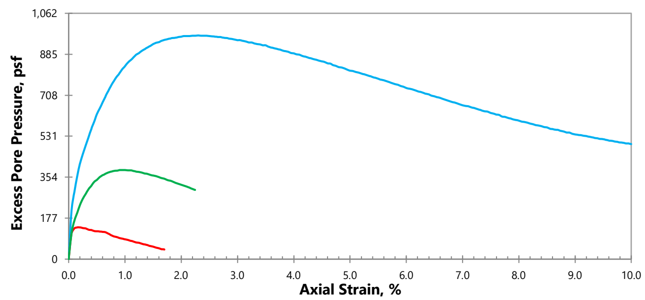
GRI

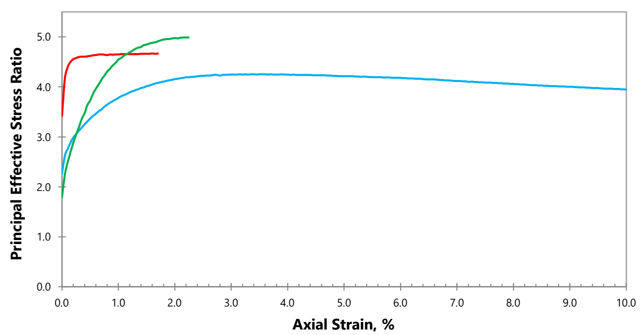
TRIAXIAL SHEAR STRENGTH TEST

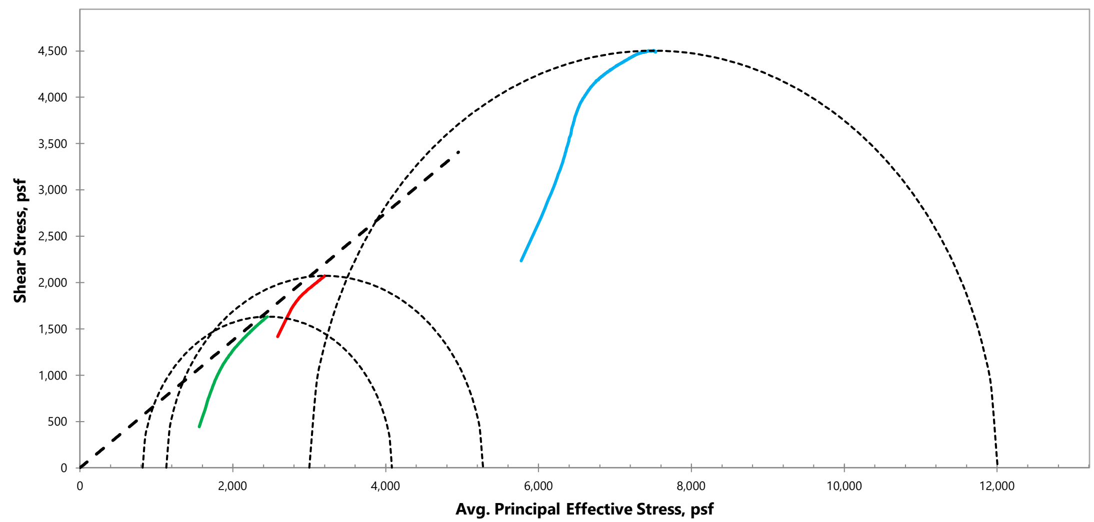
(BORING B-1, S-14)

JULY 2021 JOB NO. No. 6474-A: FIG. 7A









	STAGE 1	STAGE 2	STAGE 3
TEST SYMBOL	■	■	■
BORING NO.	B-1	B-1	B-1
SAMPLE NO.	S-19	S-19	S-19
DEPTH (FT)	70	70	70
VERTICAL EFFECTIVE CONSOLIDATION STRESS (PSF)	2000	4000	8010
SAMPLE HEIGHT (IN)	6.00	6.00	6.00
INITIAL SAMPLE DIAMETER (IN)	2.87	2.87	2.87
DRY UNIT WEIGHT (PCF)	84.7	84.9	87.4
INITIAL WATER CONTENT (%)	37	37	37
FINAL WATER CONTENT (%)	35.9	35.8	33.7
STRAIN RATE (%/HR)	2		

TYPE OF TEST: ■ CU ☐ CD

FAILURE CRITERIA: ■ MAX DEVIATOR STRESS ☐ MAX STRESS RATIO ☐ % STRAIN

TYPE OF SAMPLE: ■ UNDISTURBED ☐ REMOLDED

■ BACK SATURATED

SOIL CLASSIFICATION:

ESTIMATED STRENGTH PARAMETERS

c = 0 PSF

ϕ = 34.5°

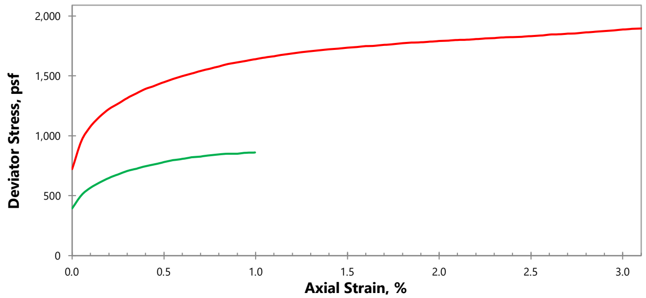
tan(ϕ) = 0.69

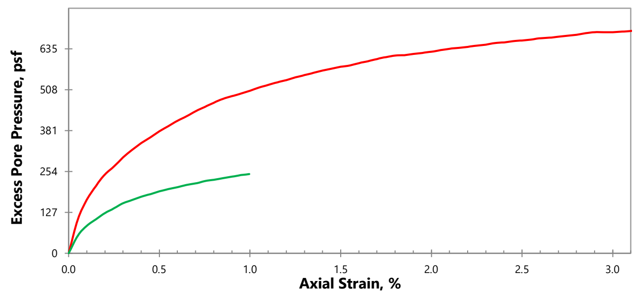
GRI

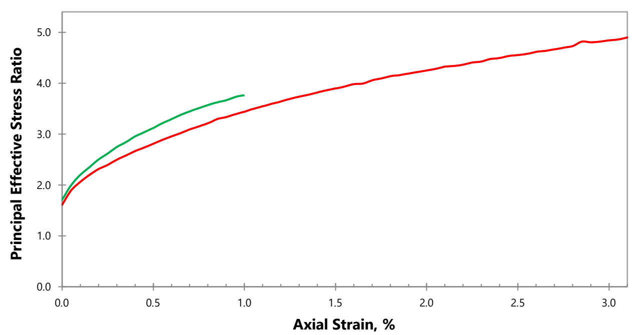
TRIAXIAL SHEAR STRENGTH TEST

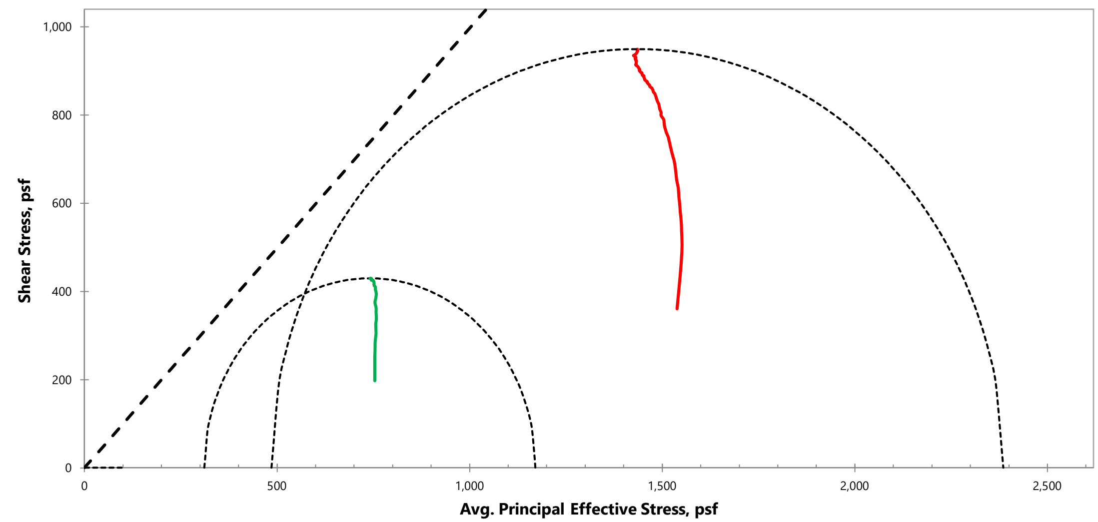
(BORING B-1, S-19)

JULY 2021 JOB NO. No. 6474-A: FIG. 8A









	STAGE 1	STAGE 2
TEST SYMBOL	■ (green)	■ (red)
BORING NO.	B-2B	B-2B
SAMPLE NO.	S-3	S-3
DEPTH (FT)	30	30
VERTICAL EFFECTIVE CONSOLIDATION STRESS (PSF)	950	1900
SAMPLE HEIGHT (IN)	6.00	6.00
INITIAL SAMPLE DIAMETER (IN)	2.87	2.87
DRY UNIT WEIGHT (PCF)	86.9	89.4
INITIAL WATER CONTENT (%)	37	37
FINAL WATER CONTENT (%)	34.1	32.1
STRAIN RATE (%/HR)	2	

TYPE OF TEST: ■ CU □ CD

FAILURE CRITERIA: ■ MAX DEVIATOR STRESS □ MAX STRESS RATIO □ % STRAIN

TYPE OF SAMPLE: ■ UNDISTURBED □ REMOLDED

■ BACK SATURATED

SOIL CLASSIFICATION:

ESTIMATED STRENGTH PARAMETERS

c = 0 PSF

ϕ = 44.9°

tan(ϕ) = 1.00

TRIAXIAL SHEAR STRENGTH TEST

(BORING B-2B, S-3)

JULY 2021 JOB NO. No. 6474-A: FIG. 9A