

Geotechnical Investigation

New Railroad Bridge and Culverts Near Wolf Bay and Aldrich Point

Clatsop County, Oregon

September 9, 2020

(ISSUED 3/22/2021)

Prepared for

Matt Cox, PE

Inter-Fluve, Inc.

501 Portway Avenue, Suite 101

Hood River, OR 97031

Prepared by



9750 SW Nimbus Avenue

Beaverton, OR 97008-7172

(503) 641-3478 | www.gri.com

TABLE OF CONTENTS

1	PROJECT DESCRIPTION	1
2	SITE DESCRIPTION	2
	2.1 Site Conditions.....	2
	2.2 Geologic Setting.....	2
3	SUBSURFACE CONDITIONS.....	3
	3.1 General.....	3
	3.2 Soils	3
	3.3 Groundwater.....	5
4	CONCLUSIONS AND RECOMMENDATIONS.....	6
	4.1 General.....	6
5	BRIDGE	6
	5.1 Seismic Design Criteria	6
	5.2 Foundation Support.....	7
	5.3 Lateral Earth Pressures on Abutment Walls.....	10
6	CULVERTS.....	10
	6.1 General.....	10
7	DESIGN REVIEW AND CONSTRUCTION SERVICES.....	13
8	LIMITATIONS	14
9	REFERENCES.....	15

TABLES

Table 5-1:	Summary of Seismic Design Parameters.....	7
Table 5-2:	Estimated Nonseismic Allowable Axial Capacity of Driven HP14X89 and PP16X0.5-in. Closed-Ended Pipe Piles – Wolf Bay East Site.....	8
Table 5-3:	Boring B-1 LPILE Input Parameters.....	9
Table 5-4:	Boring B-2 LPILE Input Parameters.....	9
Table 5-5:	Lateral Pile Group Analysis	10

APPENDICES

Appendix A: Field Explorations and Testing, and Laboratory Testing

FIGURES

Figures 1-2: Vicinity Maps

Figures 3-4: Site Plans

Figure 5: Surcharge-Induced Lateral Pressure

Figure 6: Earth Pressures for Cantilever Shoring

Figure 7: Earth Pressures for Temporary Braced Shoring

At your request, GRI conducted a geotechnical investigation for the above-referenced project. The general locations of the sites are shown on the Vicinity Maps, Figures 1 and 2. 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 bridge and culverts. 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 likely include a new bridge and culvert at the Wolf Bay site and a new culvert at the Aldrich Point site to provide fish passage and improve habitat in areas of existing railroad embankment. The existing railroad embankment will be excavated and removed at each project location prior to construction of the new structures. We understand a precast concrete bridge structure up to about 40 feet long supported by driven piles is being considered for one of the Wolf Bay sites.

As currently planned, the easternmost site at Wolf Bay is the preferred bridge location. Based on preliminary information provided to the project team by Hanson Professional Services, Inc. (HPSI), the structural engineer, we understand design axial pile loads for the bridge are about 200 kips per pile. 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 factor of safety of 2.5 for driven piles. Based on our conversation with HPSI, we understand the preferred pile type for bridge support is HP14x89 H-piles, although pipe piles such as PP16x0.5 will be considered. At this time, we have assumed 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.

Three 10-foot-diameter corrugated metal pipe (CMP) culverts placed side by side are currently planned for the western Wolf Bay site located nearest the existing bridge. At the Aldrich Point site, we understand the new culvert system will likely consist of two 10-foot-diameter CMP culverts placed side by side. Installation of the culverts will require temporary excavations to about elevation -2 feet, which will require cuts of up to about 17 feet below top of rail embankment and 3 feet below the native floodplain. We have

assumed final rail grades will remain unchanged and minimal fill will be required and limited to backfilling behind the new bridge abutment and culverts.

The proposed bridge at Wolfe Bay and the Aldrich culvert will include tidal channel that extend to the river. The proposed tidal channel bed elevation is 3 feet for both sites. The preliminary plans indicate spoils from excavation of the channels will be distributed on the floodplain near the toe of the existing railroad embankment. The planned thickness of spoils is not currently available, but we have assumed the thickness of these spoils will be less than 2 feet to 3 feet. Larger thickness of spoils, if proposed, placed adjacent to the railbed may induce settlement in the vicinity of the railbed and should be avoided or further evaluated.

2 SITE DESCRIPTION

2.1 Site Conditions

The proposed Wolf Bay culvert and bridge sites are about 70 feet and 1,100 feet east, respectively, of the existing Wolf Bay Bridge, which crosses a shallow side channel. 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 15 feet. The embankment fill slopes down to the floodplain elevation of about 3 feet near the existing bridge. The Wolf Bay channel at the existing bridge is at about elevation 0 feet to 2 feet. The railroad embankment at the eastern site is about 300 feet from the river and the adjacent floodplain elevation is about 6 feet to 8 feet. The railroad embankment typically consists of fragmental rock and riprap. The embankment is in disrepair and has trees growing in it. The floodplain is heavily vegetated with large brush and small trees.

The Aldrich Point culvert site is located on the existing railroad embankment about 2,100 feet upstream from Aldrich Point. The top of the railroad embankment fill is at about elevation 15 feet and slopes down to the floodplain at about elevation 5 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 that fills with water during high tide or during high river flows. The embankment is about 250 feet from the river. The embankment typically consists of fragmental rock and riprap. The railroad embankment is in extensive disrepair, has abundant trees growing in it, and is not readily passable on foot. The floodplain is heavily vegetated with large brush and small trees.

2.2 Geologic Setting

The site lies approximately 25 kilometers inland from the surface expression of the Cascadia Subduction Zone (CSZ), an active plate boundary along which remnants of the Farallon Plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American continent.

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 clay, silt, sand, and gravel. The unconsolidated saturated clay, silt, and sand are susceptible to liquefaction and lateral spreading. The alluvium is underlain by Miocene-age siltstone mapped as the Astoria Formation. Columbia River Basalt is also present at the surface near Aldrich Point.

3 SUBSURFACE CONDITIONS

3.1 General

Subsurface materials and conditions at the Wolf Bay sites were investigated February 24 through 27 2020, with two borings, designated B-1 and B-2. The borings were advanced to depths of about 100.3 feet and 100.4 feet below the surface of the railbed, respectively. Logs of the borings are provided on Figures 1A and 2A. The approximate locations of the borings are shown on the Site Plan, Figure 3.

Conventional geotechnical drill rig access to the Aldrich point site was not feasible. Subsurface materials and conditions at the Aldrich Point site were investigated on March 12, 2020, with a hand-augered boring, designated HA-1, and a Wildcat dynamic cone penetration (DCP) exploration, designated DCP-1. The hand-augered boring and DCP were advanced to depths of 10 feet (depth at which the auger hole was terminated due to caving) and 16.5 ft, respectively. A log of the hand-augered boring is provided on Figure 3A, and a log of the DCP is provided on Figure 4A. The approximate locations of the explorations are shown on the Site Plan, Figure 4.

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. At the Wolf Bay site, the alluvium includes gravel and is underlain by siltstone, the upper portion of which is decomposed to the consistency of soil. The depth from the base of the railroad base fill to the top of the siltstone varies significantly between the borings, ranging from about 31 feet at the west site (B-1) to 89 feet at the east site (B-2).

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

- a. Railroad BASE (Fill)
- b. SILT, SAND, and GRAVEL (Alluvium)
- c. SILT, Clayey SILT, and Silty CLAY (Decomposed Siltstone)
- d. SILTSTONE (Astoria Formation)

The following paragraphs provide a detailed description of these materials and a discussion of the groundwater conditions at the site.

a. Railroad BASE (Fill)

Railroad base fill consisting of relatively clean, angular, gravel- to cobble-sized rock fragments was encountered at the ground surface in borings B-1 and B-2. The fill extends to a depth of about 4 feet in boring B-1 and 1 foot in boring B-2. Based on our observations and a standard penetration test (SPT) N-value of 13 blows/foot, we estimate the relative density of the fill is medium dense.

b. SILT, SAND, and GRAVEL (Alluvium)

Interbedded alluvial silt, sand, and gravel were encountered beneath the railroad base fill in borings B-1 and B-2. The alluvium extends to depths of about 17.5 feet to 30 feet below the ground surface in borings B-1 and B-2, respectively. Hand-augered boring HA-1 encountered alluvial silt at the ground surface and was terminated in silt at a depth of about 10 feet.

The silt portion of the alluvium 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. Up to some subangular to subrounded gravel is present in zones, as well as zones of abundant organics and wood debris. A layer of gravelly silt was encountered between depths of about 3 feet and 8.5 feet in boring B-2. Based on SPT N-values of 0 blows/foot to 4 blows/foot and Torvane shear-strength values of 0.15 tons per square foot (tsf) to 0.30 tsf, the relative consistency of the silt ranges from very soft to medium stiff and is typically very soft to soft. An SPT N-value of 6 blows/foot was obtained in the gravelly silt layer in B-2; however, it is our opinion the SPT value is elevated due to the presence of gravel and is likely not representative of the consistency of the silt at that location. The natural moisture content of the silt in borings B-1 and B-2 ranges from about 20% to 74%, with higher moisture contents associated with the presence of clay and/or organics. The natural moisture content of the silt in HA-1 ranges from 62% to 119% due to high organic content.

The alluvial sand is fine to coarse grained and contains varying percentages of clay and silt, ranging from up to some clay to a trace of silt to silty. Scattered gravel is present in the unit. An SPT N-value of 3 blows/foot indicates the relative density of the sand is very loose to loose. The natural moisture content of the sand ranges from 25% to 41%.

The gravel portion of the alluvium contains varying percentages of silt and fine- to coarse-grained sand ranging from some silt to silty and a trace of sand to sandy. SPT N-values ranging from 4 blows/foot to 23 blows/foot indicate the relative density of the gravel is very loose to medium dense.

c. SILT, Clayey SILT, and Silty CLAY (Decomposed Siltstone)

Silt, clayey silt, and silty clay derived from the decomposition of the underlying siltstone were encountered beneath the alluvium in borings B-1 and B-2. The decomposed siltstone extends to depths of about 35 feet to 90 feet below the ground surface in borings B-1 and B-2, respectively. The decomposed siltstone contains trace to some fine- to coarse-grained sand and scattered shell fragments and wood debris. Based on SPT N-values between 14 blows/foot and 76 blows/foot, the relative consistency of the decomposed siltstone ranges from stiff to hard. The moisture content of the material ranges from 19% to 45%.

d. SILTSTONE (Astoria Formation)

In borings B-1 and B-2, the decomposed siltstone is underlain by siltstone rock of the Astoria Formation. The depth from the base of the railroad base fill to the top of the siltstone varies significantly between boring B-1 and B-2, ranging from 31 feet to 89 feet, respectively. The siltstone is typically brown to dark gray and moderately weathered to predominantly decomposed. In general, the siltstone is extremely soft to very soft (R0 to R1) on the rock hardness scale. SPT N-values in the siltstone ranged from 50 blows for 5 inches of sampler penetration to 50 blows for 3 inches of sampler penetration. Borings B-1 and B-2 were terminated in the siltstone at depths of 100.3 feet and 100.4 feet, respectively.

3.3 Groundwater

Borings B-1 and B-2 were advanced using mud-rotary methods, which do not permit the observation of groundwater conditions during drilling. Groundwater was encountered at a depth 1.5 feet below the ground surface in hand-augered boring HA-1. The groundwater level reflects the level of the Columbia River and fluctuates in response to river levels. For design purposes, we recommend the groundwater be assumed at the level of the Columbia River. Based on our review of the preliminary 30% plans, we understand the mean lower low water (MLLW) and mean higher high water (MHHW) elevations range from 0.56 foot to 8.86 feet, respectively, at Wolf Bay and from 1.34 feet to 8.94 feet, respectively, at Aldrich Point.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 General

The borings indicate the Wolf Bay and Aldrich Point sites are mantled by soft/loose alluvial material beneath the railroad base fill. The alluvium is susceptible to liquefaction and lateral spreading during a code-based earthquake. The alluvium is underlain by siltstone of the Astoria Formation; the top of which has weathered to the consistency of soil. The depth to the top of the siltstone varies significantly between the possible bridge sites at Wolf Bay.

As previously mentioned, we have assumed the improvements will not be designed to resist potential seismic ground deformations from liquefaction or lateral spreading. We anticipate axial bridge support will be provided by driven piles. High groundwater levels will result in challenging excavation conditions during installation of culverts. Silty sand is present below the groundwater level, and the potential for running sand will be an important consideration during excavation and construction of the new culverts. The following sections of this report provide our conclusions and recommendations for the design and construction of the bridge and culverts.

5 BRIDGE

5.1 Seismic Design Criteria

Based on the materials encountered in the subsurface explorations, the site can be classified as Site Class D in accordance with Section 20.3.1 of American Society of Civil Engineers (ASCE) 7-16 document, *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-16). As previously mentioned, we have assumed the improvements will not be designed to resist ground deformations from liquefaction or lateral spreading; however, we understand the seismic structural bridge design be in accordance with AREMA guidelines. 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 interval for GML 1, GML 2, and GML 3 is 100 years, 475 years, and 2,475 years, respectively. A summary of the seismic design parameters 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, years	$S_{a(T-O)}$	S_{ds}	S_{d1}
GML 1 (Serviceability)	100	0.0754	0.1667	0.0694
GML 2 (Ultimate)	475	0.3008	0.6662	0.3442
GML 3 (Survivability)	2,475	0.6811	1.4459	0.8931

5.2 Foundation Support

5.2.1 General

As currently planned, the bridge will be located at the east site at Wolf Bay where the siltstone was encountered at a depth of about 89 feet below the base of the railbed fill. We understand HP14x89 H-piles is the preferred pile type for bridge foundations; however, pipe piles such as PP16x0.5 will also be considered. The preliminary estimate of axial pile load is about 200 kips per pile and we understand a factor of safety of 2.5 is required per AREMA guidelines.

The HP14x89 piles will develop their supporting capacity from a combination of skin friction and end bearing in the underlying soft R0 to R1 siltstone rock, and piles that are driven to adequate resistance in the soft rock with a sufficiently large pile driving hammer can develop resistances that approach the structural capacity of the pile section. The PP16x0.5 piles will develop their supporting capacity from a combination of skin friction and end bearing. The indicated capacities apply to the total of all loads (dead, live, and wind or seismic) and include an estimated factor of safety of at least 2.5. Estimated allowable axial capacities for the above-mentioned pile sections at the currently proposed bridge site are tabulated below in Table 5-2.

Table 5-2: ESTIMATED NONSEISMIC ALLOWABLE AXIAL CAPACITY OF DRIVEN HP14X89 AND PP16X0.5-IN. CLOSED-ENDED PIPE PILES – WOLF BAY EAST SITE

Pile Type	Embedment Depth, ft	Allowable Axial Capacity, kips
HP14X89	90 ^{(a),(b)}	265
PP16X0.5-in.-Closed End	70	200

Notes:

- a) Assumes pile driven to minimum 12 ft embed in R0 to R1 siltstone.
- b) Siltstone approximately 89 ft below base of railbed fill at boring B-2.

As previously mentioned, the depth to the top of the siltstone varies significantly between the east and west sites at Wolf Bay. At the west site nearest the existing bridge, siltstone was encountered at a depth of about 31 feet below the base of the railbed. In this regard, a bridge at this location could be founded on much shorter piles driven into the soft rock.. We anticipate PP16X0.5-inch piles driven close-ended would penetrate about 1 foot to 5 feet into the R0 to R1 siltstone, or up to about 10 feet if driven open-ended, while HP14X89 piles would likely penetrate at least 5 feet to 10 feet into the siltstone. Based on information provided by the structural engineer, we understand a lower factor of safety may be acceptable for piles bearing on rock.

Piles driven into the underlying rock should be provided with commercially available tip protection. We recommend a geotechnical engineer from GRI observe pile installation to maintain pile driving logs and assist in developing terminal driving resistance and penetration criteria. A submittal of the contractor's proposed equipment, materials, and methods for installing the piles should be provided in advance of pile installation for review.

We anticipate settlement of driven piles under the service compressive loads will be less than about ¼ in., or the elastic shortening of the pile.

5.2.2 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 are provided in Tables 5-3 and 5-4.

Table 5-3: BORING B-1 LPILE INPUT PARAMETERS

Depth, ft ^{(a)(b)}	Soil Type (LPILE p-y Model)	Soil and Rock Properties				
		γ' , pcf	c, psf	ϕ'	k, pci	ϵ_{50}
0.0 to 8.0	Silt (Soft Clay)	48	500	NA	NA	0.020
8.0 to 11.0	Silty Gravel (API Sand)	58	NA	34°	20	NA
11.0 to 16.0	Silt (Soft Clay)	48	250	NA	NA	0.020
16.0 to 31.0	Silt (Stiff Clay without Free Water)	58	1,000	NA	NA	0.005
		γ' , pcf	q_u , psi	E, psi	RQD, %	K_{rm}
31.0 and below	Weak Rock	63	500	50,000	10	0.0005

Table 5-4: BORING B-2 LPILE INPUT PARAMETERS

Depth, ft ^{(a)(b)}	Soil Type (LPILE p-y Model)	Soil and Rock Properties				
		γ' , pcf	c, psf	ϕ'	k, pci	ϵ_{50}
0.0 to 11.5	Sand, Gravelly Silt, Silty Sand (Soft Clay)	46	500	NA	NA	0.020
11.5 to 19.0	Silt (Soft Clay)	48	250	NA	NA	0.020
19.0 to 22.0	Sandy Gravel (API Sand)	58	NA	34°	60	NA
22.0 to 29.0	Sandy Silt (Soft Clay)	48	250	NA	NA	0.020
29.0 to 87.0	Clayey Silt (Stiff Clay without Free Water)	58	1,000	NA	NA	0.005
		γ' , pcf	q_u , psi	E, psi	RQD, %	K_{rm}
87.0 and below	Weak Rock	63	500	50,000	10	0.0005

Notes:

- a) Depth is below base of the railroad base fill.
- b) Groundwater is assumed to be at the ground surface at the base of the rail bed 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 of the k_h value for pile spacing less than five pile diameters. The following table provides a summary of p-multipliers for various center-to-center pile spacing.

Table 5-5: 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 on the basis of 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 5.

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 $19H$ (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 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 (ASTM) D698. Heavy compaction equipment should not operate within 5 feet of the abutment.

6 CULVERTS

6.1 General

As previously discussed, CMP culverts will be installed at the Wolf Bay and Aldrich Point sites. The maximum depth of excavations necessary to construct the new culverts will be about 17 feet below top of rail embankment. Excavations for culvert foundations will

extend below the groundwater/river level, and temporary shoring and dewatering may be required. Due to the proximity of the river, dewatering will be difficult, and we understand it is preferable to complete the excavations in the wet. Due to the width of the excavation needed for culvert placement, we anticipate culvert installation may be completed in stages.

6.1.1 Excavation

We anticipate the excavations will be open cut with sloped sidewalls where practical. The method of excavation and groundwater control, and the design of the excavation support is typically the responsibility of the contractor and should conform to applicable local, state, and federal regulations. We recommend the contractor submit for review by the owner and owner's design team an excavation, shoring, and dewatering plan prepared by a professional engineer registered in Washington. The information provided below is for the use of our client and should not be interpreted to mean we are assuming responsibility for the contractor's actions or site safety. The soils disclosed by our borings should be classified as Type C soil according to the most recent Occupational Safety and Health Administration (OSHA) regulations.

The inclination of temporary excavation slopes will depend on the groundwater conditions encountered at the time of construction and the soil type. Temporary excavation slopes extending below the river level will be subject to fluctuating river and groundwater levels and should be no steeper than 2H:1V (Horizontal to Vertical) to reduce the risk of slumps and raveling. Temporary excavation slopes above the river level should be no steeper than 1.5H:1V. If significant seepage and running-soil conditions or slope instability are observed during excavation, flatter slopes may be necessary. Some minor amounts of sloughing, slumping, or running of temporary slopes should be anticipated during and shortly after excavation, particularly if there is seepage caused by river level fluctuations. A blanket of relatively clean, well-graded, crushed rock placed on the slopes may be required to reduce the risk of these conditions, particularly if seepage is observed in the slopes. We recommend the use of relatively clean, well-graded crushed rock, with maximum size of about 4 inches, for this purpose. The required thickness of the granular blanket should be evaluated based on actual conditions but could be in the range of 12 inches to 24 inches. Heavy surcharge loads should not be allowed within about 15 feet of the top of the cut.

Sheetpiles, steel sheets installed between soldier piles, or other braced shoring could also be used to shore temporary excavations. The lateral earth pressure criteria shown on Figures 6 and 7 can be used for design of temporary cantilevered sheetpiles and braced excavation support systems, respectively. Additional pressures due to surcharge loads, such as construction equipment operating adjacent to the shoring at the top of the excavation, can be computed in accordance with the criteria shown on Figure 5.

6.1.2 Groundwater Control

Groundwater levels are expected to be consistent with river levels, and challenges associated with controlling water in excavations could potentially be reduced by scheduling construction during low river levels and during periods of low tide. The appropriate method of groundwater control will depend on actual water levels at the time of construction. The use of temporary cofferdams such as sandbags, supersacks, earthen berms, sheetpiles, or soldier piles with steel sheets driven with a large hydraulic excavator or other suitable equipment may be necessary to help control the flow of water. GRI should review the contractor's proposed method of excavation and groundwater control prior to mobilization to the site. Soft silt and sandy soils are present in the floodplain at varying elevations. It may not be feasible to dewater excavations in the soft silt and sandy soils by pumping from sumps within the excavations due to bottom heave and potential for heavy seepage causing running-soil conditions. Construction dewatering using pumping wells or well points if needed should be designed by the contractor.

6.1.3 Excavation Bottom Stabilization and Culvert Bedding

Due to the need to excavate moisture-sensitive soils below the groundwater level, we recommend overexcavation of the subgrade and installation of granular stabilization material to provide level and uniform support of the culverts. It has been our experience that 1 foot to 2 feet of overexcavation will likely be needed; however, the actual depth of overexcavation would be best established based on observations at the time of construction. Stabilization material should consist of clean, free draining, angular, fragmental rock with a maximum size of up to about 4 inches, less than 5% passing the No. 4 sieve, and less than 2% passing the No. 100 sieve. Bottom stabilization material should be placed in a single lift and compacted with vibratory equipment or tamped in until well keyed. We recommend a minimum-6-inch thickness of $\frac{3}{4}$ -inch-minus granular aggregate be provided over the excavation bottom stabilization material to serve as a leveling course and "choke" the surface of the coarser rock. This material is also suitable for use as bedding for the culvert if placed in the dry.

6.1.4 Culvert Backfill

Based on the planned depth of excavation, it is likely that some in-water placement of backfill will be required. This in-water fill should consist of clean, free-draining, angular, fragmental rock that meet the requirements of the stabilization material described above. Backfill placed in dry conditions should consist of sand or well-graded crushed rock with a maximum size of about 2 inches and less than about 5% passing the No. 200 sieve. We recommend the granular backfill material be placed in lifts and compacted until well keyed using vibratory equipment. Lift thicknesses should be proportioned to be appropriate with the type of compaction equipment used. Backfill should be placed in lifts and compacted with vibratory equipment to at least 95% of the maximum density as determined by ASTM

D698. Care should be taken to raise the level of the backfill equally on both sides of the culvert during the backfilling. We recommend finished embankment slopes at the inlets and outlets be no steeper than 2H:1V.

6.1.5 Settlement

Post-construction settlement of the ground surface can be reduced by backfilling the excavation with clean, granular structural fill as previously recommended. Inadequate removal of disturbed, soft, or loosened materials prior to installation of the stabilization/bedding material beneath the culvert may result in post-construction settlement of the culvert. Subgrade disturbance could be caused by improper excavation, insufficient groundwater control, or trafficking of an exposed and unprotected subgrade. For culverts installed as recommended above, we anticipate settlement of the culvert will be less than about 1 inch, assuming the rail grade will not be raised.

6.1.6 Earth Pressures

Design earth pressures depend on whether the structure is submerged, the drainage condition provided outside the culvert walls, and the ability of the culvert walls to yield. For lateral pressures, we anticipate the culvert will have relatively rigid walls and will be designed to resist full hydrostatic pressure. For this condition, we recommend designing the culvert walls using a hydrostatic pressure based on an equivalent fluid having a unit weight of 90 pcf. The pressure on the roof of the culverts due to the weight of the fill may be estimated assuming a bulk unit weight of about 130 pcf. Additional live-load pressures due to train traffic should also be included in the structural design of the culvert per AREMA guidelines.

6.1.7 Seismic Considerations

Based on our review of AREMA guidelines, we understand culverts are presumed to be designed to resist seismic forces, but not to resist displacements due fault rupture or ground movements caused by liquefaction or lateral spreading. If culverts will be designed to resist seismic forces, the seismic lateral earth pressures provided above for abutment walls are appropriate for use.

7 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 culvert 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

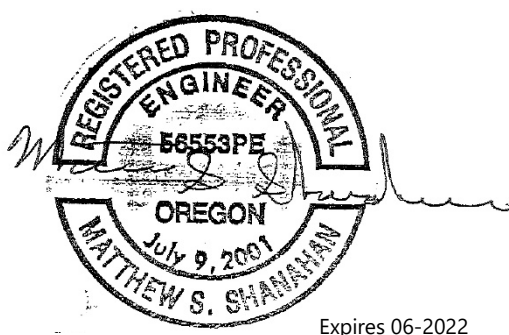
application of our recommendations to subsurface conditions that are different from those described in this report.

8 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 3 and 4 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,



Matthew S. Shanahan, PE, GE
Principal

A handwritten signature in cursive script that reads "Tamara G. Kimball".

Tamara G. Kimball, PE, GE
Senior Engineer

This document has been submitted electronically.

9 REFERENCES

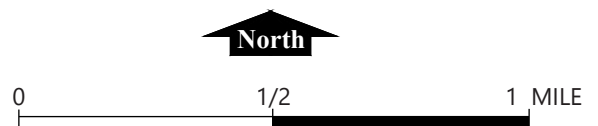
American Association of State Transportation Officials (AASHTO), 2020, LRFD bridge design specifications, 9th edition, American Association of State Highway and Transportation Officials, Washington, DC.

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 (AREMA), 2019, AREMA manual for railway engineering.



USGS TOPOGRAPHIC MAP
CATHLAMET BAY, OREG. (2017)



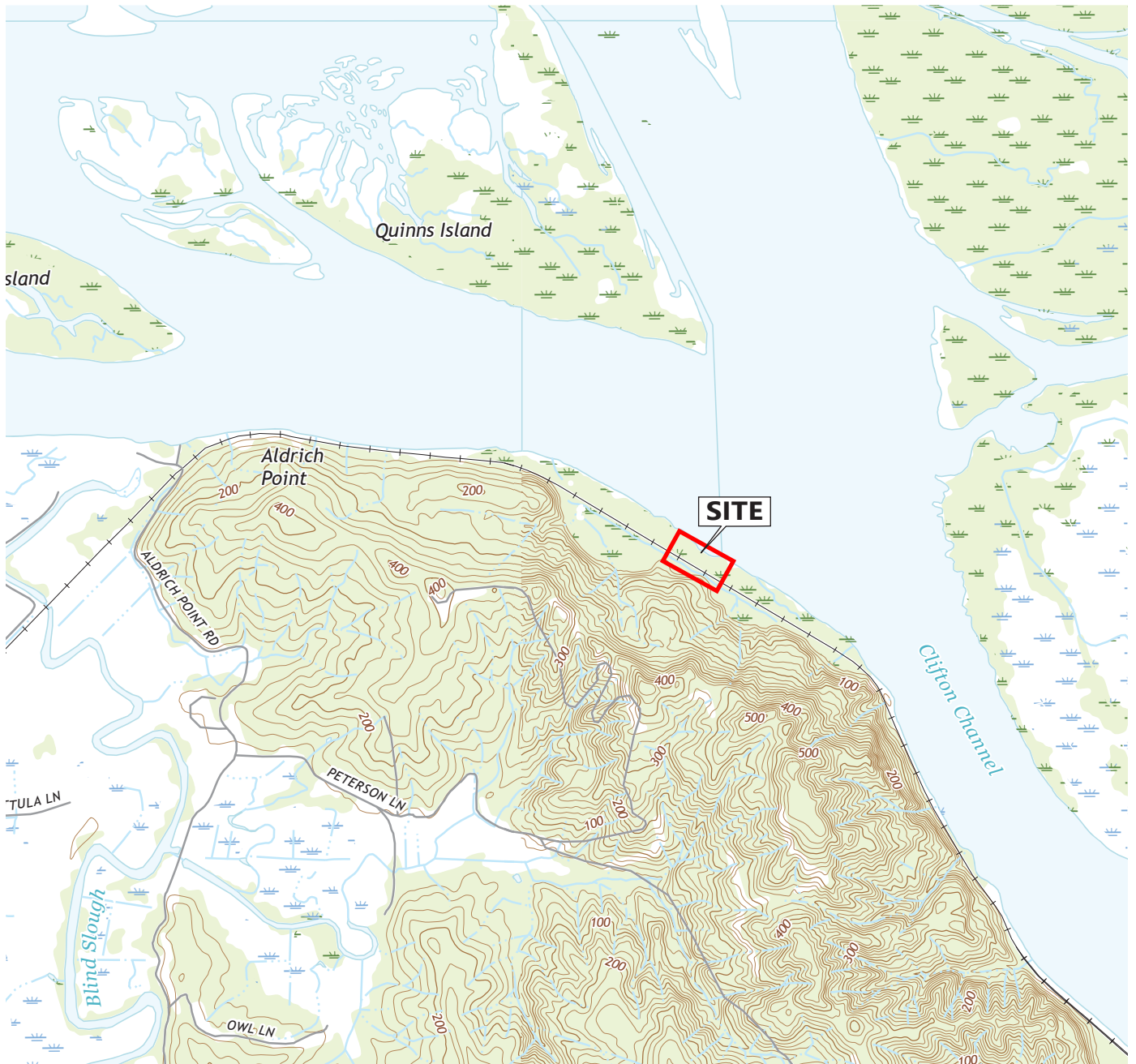
INTER-FLUVE, INC.
WOLF BAY AND ALDRICH POINT

VICINITY MAP WOLF BAY

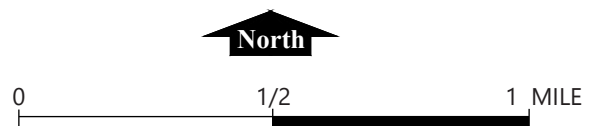
SEP. 2020

JOB NO. 6319

FIG. 1



USGS TOPOGRAPHIC MAPS
KNAPPA, OREG. (2017)
CATHLAMET, WASH. (2017)



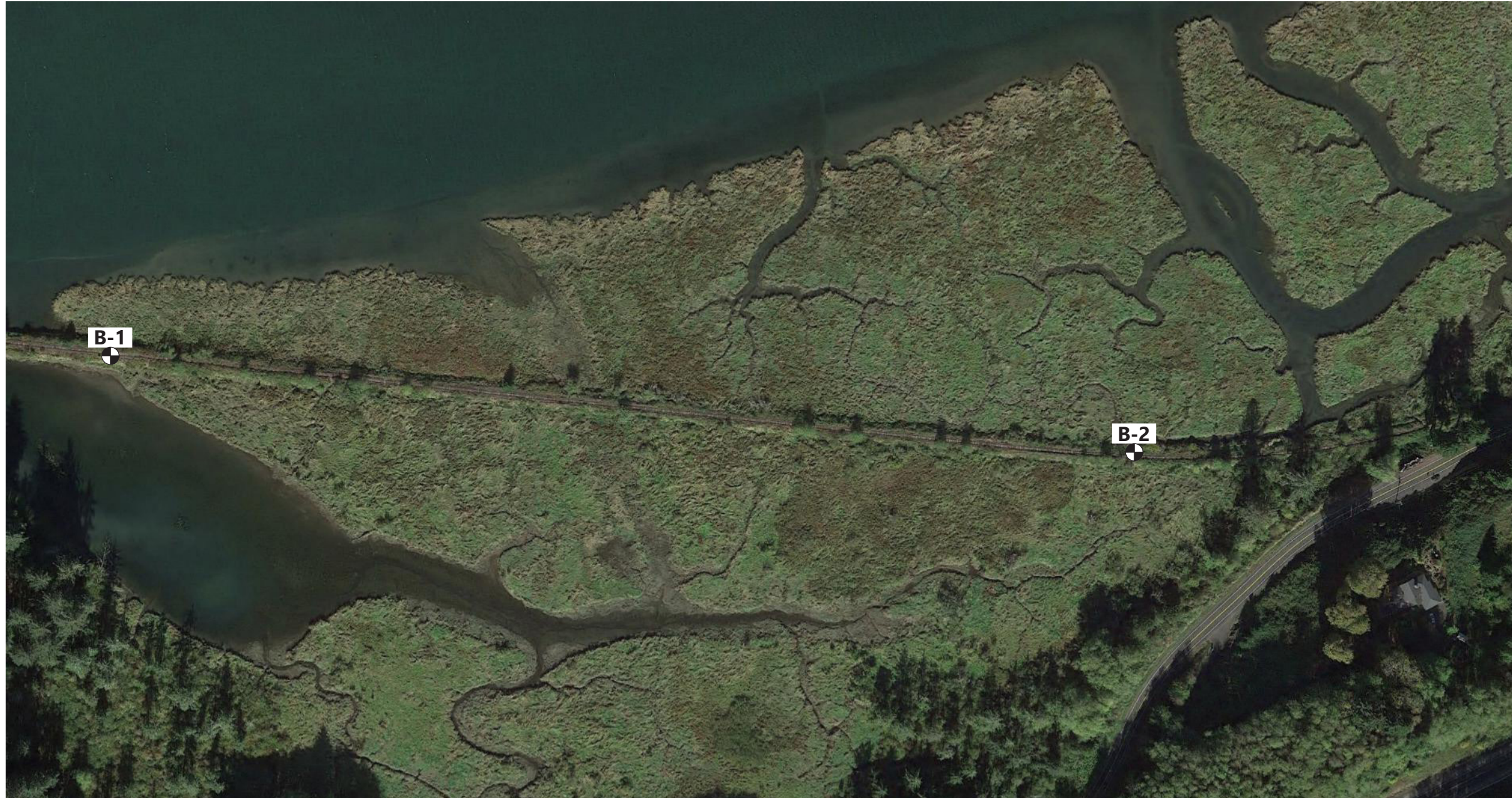
GRI INTER-FLUVE, INC.
WOLF BAY AND ALDRICH POINT

VICINITY MAP ALDRICH POINT

SEP. 2020

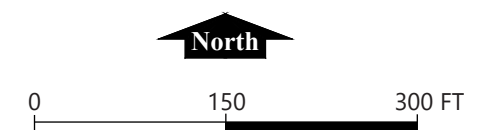
JOB NO. 6319

FIG. 2



● BORING COMPLETED BY GRI
(FEBRUARY 24-27, 2020)

SITE PLAN FROM FILE BY GOOGLE EARTH PRO, 2020



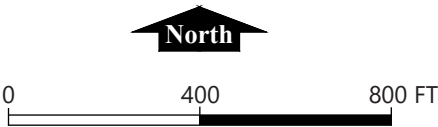
GRI INTER-FLUVE, INC.
WOLF BAY AND ALDRICH POINT

SITE PLAN WOLF BAY



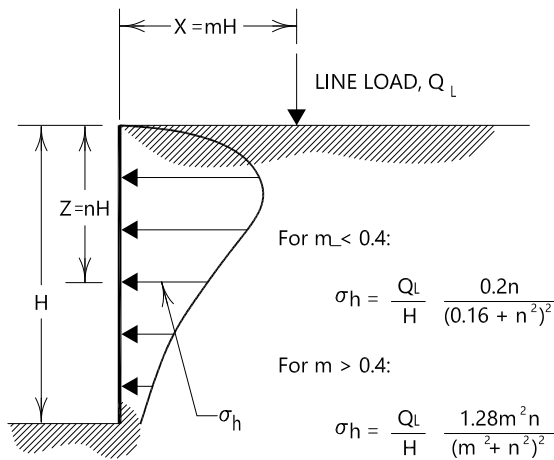
- HAND-AUGERED BORING COMPLETED BY GRI
(MARCH 12, 2020)
- DYNAMIC CONE PENETRATION TEST COMPLETED BY GRI
(MARCH 12, 2020)

SITE PLAN FROM FILE BY GOOGLE EARTH PRO, 2020

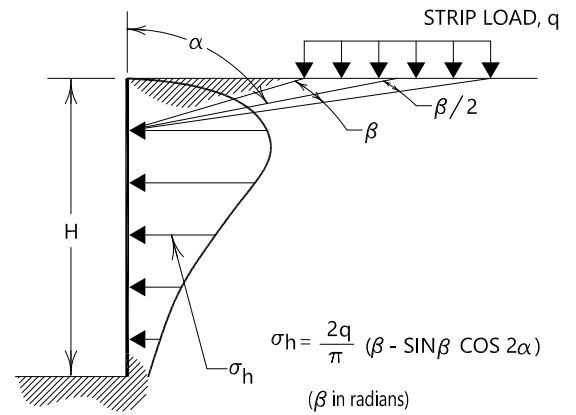


GRI INTER-FLUVE, INC.
WOLF BAY AND ALDRICH POINT

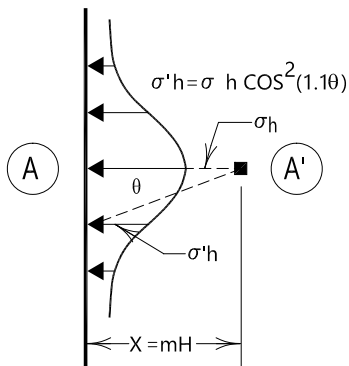
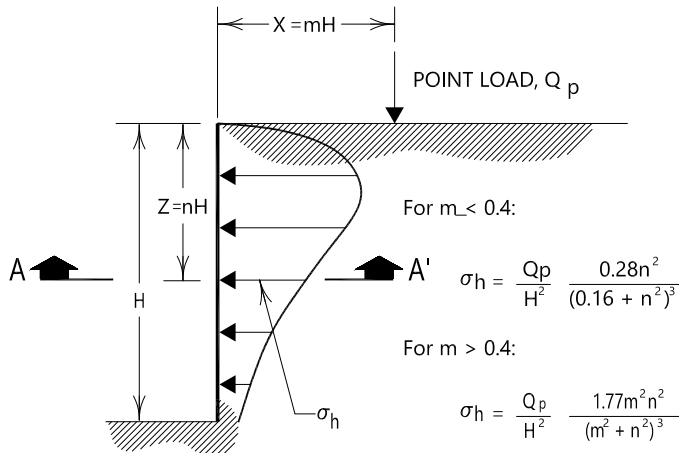
SITE PLAN ALDRICH POINT



LINE LOAD PARALLEL TO WALL



STRIP LOAD PARALLEL TO WALL



DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

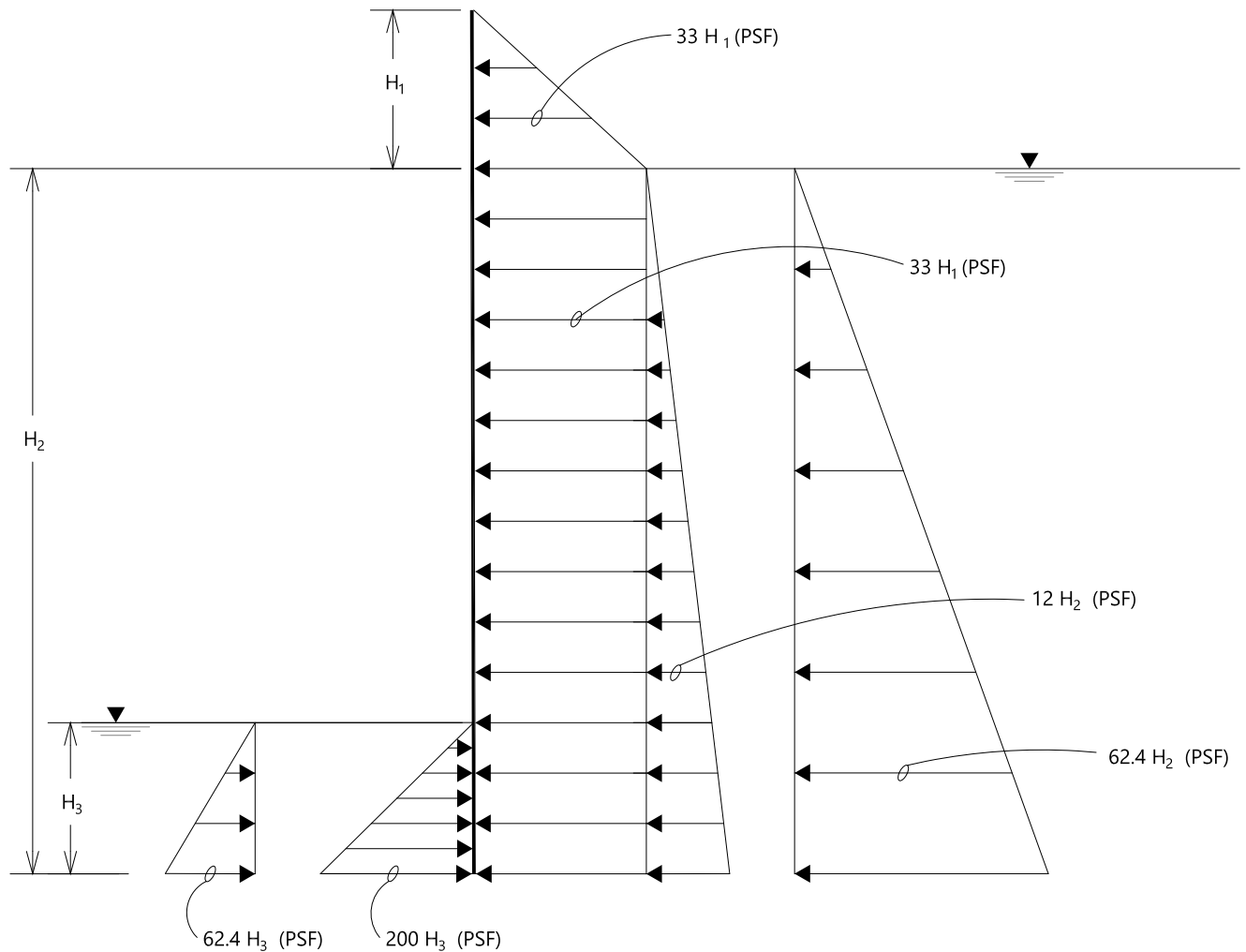
NOTES:

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.



INTER-FLUVE, INC.
WOLF BAY AND ALDRICH POINT

SURCHARGE-INDUCED LATERAL PRESSURE



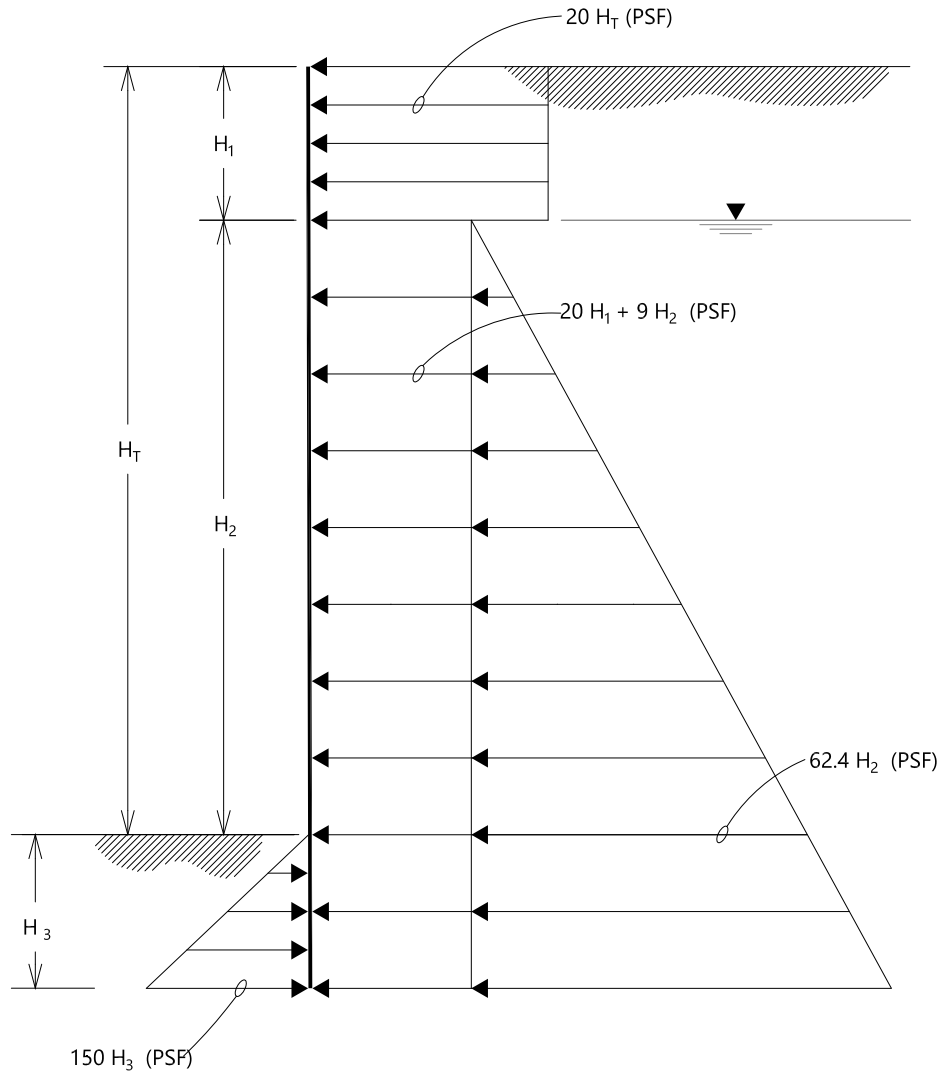
NOTES: 1 . SURCHARGE EFFECTS FROM TRAFFIC, CONSTRUCTION EQUIPMENT, ETC., SHOULD BE ADDED TO THE ABOVE DESIGN PRESSURES. THE ACTUAL AMOUNT OF THIS SURCHARGE WILL DEPEND ON THE CONTRACTOR'S APPROACH TO THE WORK; HOWEVER, WE RECOMMEND USING A MINIMUM UNIFORM PRESSURE OF 200 PSF.

2 . THE PRESSURES ACT OVER THE SURFACE AREA OF THE SHEETS.



INTER-FLUVE, INC.
WOLF BAY AND ALDRICH POINT

EARTH PRESSURES FOR CANTILEVER SHORING



- NOTES: 1 . SURCHARGE EFFECTS FROM TRAFFIC, CONSTRUCTION EQUIPMENT, ETC., SHOULD BE ADDED TO THE ABOVE DESIGN PRESSURES. THE ACTUAL AMOUNT OF THIS SURCHARGE WILL DEPEND ON THE CONTRACTOR'S APPROACH TO THE WORK; HOWEVER, WE RECOMMEND USING A MINIMUM UNIFORM PRESSURE OF 200 PSF.
- 2 . THE PRESSURES ACT OVER THE SURFACE AREA OF THE SHEETS.
- 3 . PASSIVE PRESSURE CAN BE INCREASED BY A FACTOR OF 2 FOR SOLDIER PILES TO ACCOUNT FOR ARCHING EFFECTS.



INTER-FLUVE, INC.
WOLF BAY AND ALDRICH POINT

EARTH PRESSURES FOR TEMPORARY BRACED SHORING

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 Wolf Bay site were investigated between February 24 and 27, 2020, with two borings, designated B-1 and B-2. Subsurface materials and conditions at the Aldrich Point site were investigated on March 12, 2020, with a hand-augered boring, designated HA-1, and a Kessler dynamic cone penetration (DCP) exploration, designated DCP-1. The approximate locations of the borings, hand-augered boring, and DCP are shown on the Site Plans, Figures 3 and 4. All explorations were observed by an experienced member of GRI's engineering staff.

A.1.1 Borings

Borings B-1 and B-2 were advanced to depths of about 100.3 feet and 100.4 ft, respectively. The borings were completed with mud-rotary drilling techniques using a GeoProbe 7822DT track-mounted drill rig provided and operated by Western States Soil Conservation of Hubbard, Oregon. Disturbed and undisturbed samples were typically obtained at 2.5-foot intervals of depth in the upper 15 feet and at 5-foot intervals below this depth. Disturbed samples were obtained using a standard split-spoon sampler. At the time of sampling, the standard penetration test (SPT) was conducted. This test consists of driving a standard split-spoon 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 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 representative portions were saved in airtight jars for further examination and physical testing in our laboratory. In addition, 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.

Subsurface materials and conditions at the Aldrich Point site were investigated on March 12, 2020, with one hand-augered boring, designated HA-1. Disturbed grab samples were typically obtained at 2-foot intervals of depth. The grab samples were carefully examined in the field, and representative portions were saved in airtight jars for further examination and physical testing in our laboratory.

Logs of the borings are provided on Figures 1A and 2A, and a log of the hand-augered boring is provided on Figure 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, percentage of material passing the No. 200 sieve, Torvane shear-strength values, and Atterberg-limits indices. The terms used to describe the soils are defined in Tables 1A and 2A.

A.1.2 Dynamic Cone Penetrometer Testing

One DCP test, designated DCP-1, was advanced to a depth of about 16.5 feet below the ground surface on March 12, 2020, using a Wildcat cone penetrometer manufactured by Triggs Technologies, Inc. The Wildcat cone penetrometer sounding consists of driving a 1.4-inch-diameter cone with a 35-pound weight falling 15 inches. The number of blows required to drive the cone 10 centimeters (approximately 4 inches) is recorded to assess the density or stiffness characteristics of the underlying soils. The DCP test results are provided on Figure 4A. The results display the soil density/consistency and the blows required to drive the cone tip in 10-centimeter increments. Soil samples are not collected during DCP testing.

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. 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 and 2A 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 hand-held 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 Figure 2A.

A.2.6 Atterberg Limits

Atterberg limits were determined for a selected soil sample in conformance with ASTM D4318. The test results are summarized on Figures 1A, 2A and 5A, and in Table 3A.

A.2.7 One-Dimensional Consolidation

One-dimensional consolidation testing was performed in accordance with ASTM D2435 on a relatively undisturbed soil sample obtained at a depth of about 15.3 feet in boring B-2. The test provides data on the compressibility of underlying fine-grained soils. Test results are shown on Figure 6A in the form of a curve showing effective stress versus percent strain. The initial dry unit weight and moisture content of the samples are also shown on the figure.

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	Modifier for Subclassification		
<i>Boulders:</i> > 12 in. <i>Cobbles:</i> 3-12 in. <i>Gravel:</i> 1/4 - 3/4 in. (fine) 3/4 - 3 in. (coarse) <i>Sand:</i> No. 200 - No. 40 sieve (fine) No. 40 - No. 10 sieve (medium) No. 10 - No. 4 sieve (coarse) <i>Silt/Clay:</i> Pass No. 200 sieve	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		Terminology for Planar Surface		
RQD (Rock Quality Designation), %	Description of Rock Quality	Bedding	Joints and Fractures	Spacing
0 - 25	Very Poor	Laminated	Very Close	< 2 in.
25 - 50	Poor	Thin	Close	2 in. – 12 in.
50 - 75	Fair	Medium	Moderately Close	12 in. – 36 in.
75 - 90	Good	Thick	Wide	36 in. – 10 ft
90 - 100	Excellent	Massive	Very Wide	> 10 ft

Table 3A
SUMMARY OF LABORATORY RESULTS

Sample Information				Atterberg Limits				Fines Content, %	Soil Type
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %		
B-1	S-2	5.0	10.0	44	--	--	--	71	SILT
	S-3	7.5	7.5	44	--	--	--	--	SILT
	S-4	10.0	5.0	43	--	--	--	81	SILT
	S-4	10.5	4.5	43	79	--	--	--	SILT
	S-4	11.5	3.5	45	--	--	--	82	SILT
	S-5	12.0	3.0	34	--	--	--	48	Silty GRAVEL
	S-6	15.0	0.0	57	--	--	--	91	SILT
	S-7	20.0	-5.0	23	--	--	--	96	SILT
	S-8	25.0	-10.0	20	--	--	--	--	SILT
	S-9	30.0	-15.0	34	--	51	21	--	Clayey SILT
B-2	S-10	35.0	-20.0	21	--	--	--	--	SILTSTONE
	S-1	2.5	12.5	25	--	--	--	5	SAND
	S-2	5.0	10.0	45	--	--	--	61	Gravelly SILT
	S-3	7.5	7.5	49	--	--	--	76	Clayey SILT
	S-3	7.8	7.2	50	72	--	--	--	Clayey SILT
	S-3	8.6	6.4	41	--	--	--	48	Silty SAND
	S-5	12.5	2.5	69	--	--	--	--	SILT
	S-6	15.0	0.0	56	--	--	--	95	SILT
	S-6	15.5	-0.5	54	69	--	--	--	SILT
	S-7	17.0	-2.0	56	--	50	12	74	SILT
	S-9	25.0	-10.0	74	--	--	--	65	Sandy SILT
	S-10	30.0	-15.0	38	--	--	--	--	SILT
	S-10	30.5	-15.5	45	--	63	16	--	SILT
	S-12	40.0	-25.0	35	--	--	--	98	Silty CLAY
	S-13	45.0	-30.0	27	--	--	--	--	Silty CLAY
	S-14	50.0	-35.0	26	--	--	--	--	Silty CLAY
	S-15	55.0	-40.0	19	--	--	--	--	Silty CLAY
	S-16	60.0	-45.0	21	--	--	--	--	Clayey SILT
	S-17	65.0	-50.0	22	--	--	--	--	SILT
	S-18	70.0	-55.0	22	--	--	--	--	SILT
	S-19	75.0	-60.0	21	--	--	--	--	SILT
	S-20	80.0	-65.0	19	--	--	--	--	SILT
HA-1	S-21	85.0	-70.0	24	--	--	--	--	SILT
	S-1	2.0	1.0	119	--	--	--	91	SILT
	S-2	4.0	-1.0	115	--	--	--	--	SILT
	S-3	6.0	-3.0	107	--	--	--	--	SILT
	S-4	8.0	-5.0	92	--	--	--	--	SILT
	S-5	9.0	-6.0	62	--	--	--	69	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

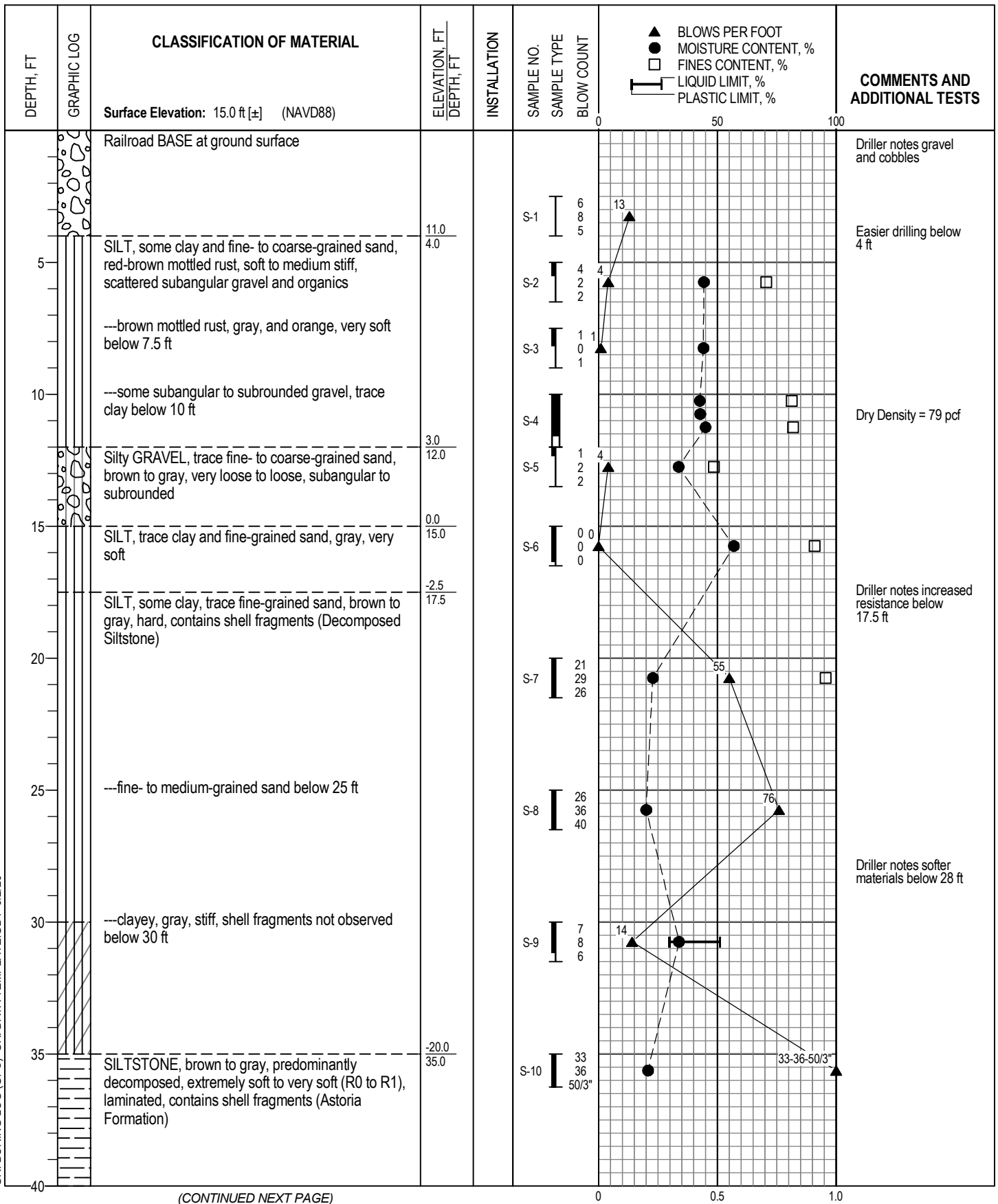
Symbol	Sampler Description
	2.0 in. O.D. split-spoon sampler and Standard Penetration Test with recovery (ASTM D1586)
	Shelby tube sampler with recovery (ASTM D1587)
	3.0 in. O.D. split-spoon sampler with recovery (ASTM D3550)
	Grab Sample
	Rock core sample interval
	Sonic core sample interval
	Push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



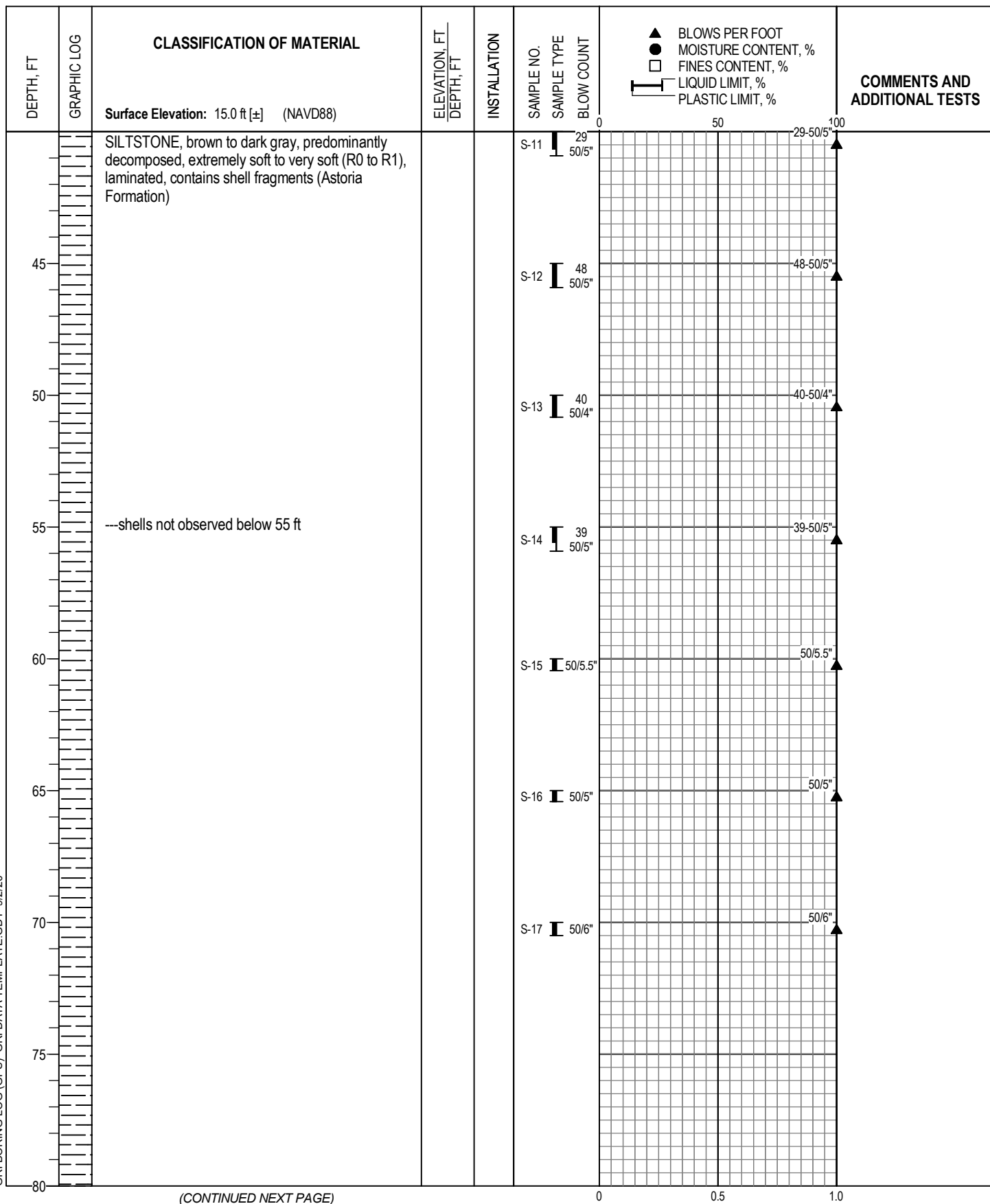
(CONTINUED NEXT PAGE)

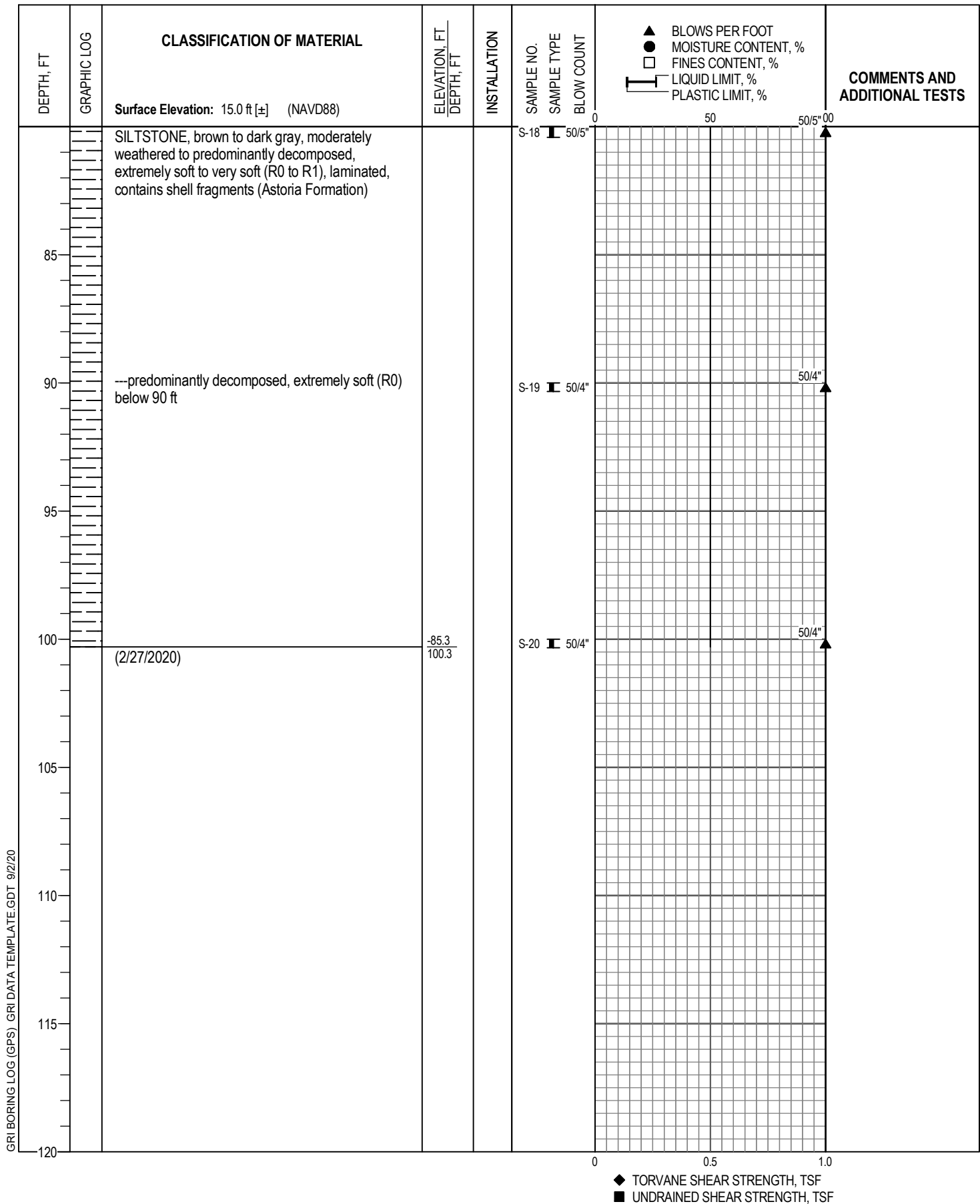
Logged By: N. Utevsy	Drilled by: Western States Soil Conservation, Inc.
Date Started: 2/26/20	GPS Coordinates: 46.17032° N -123.6989° W (WGS 84)
Drilling Method: Mud Rotary	Hammer Type: Auto Hammer
Equipment: Geoprobe 7822DT	Weight: 140 lb
Hole Diameter: 5 in.	Drop: 30 in.
Note: See Legend for Explanation of Symbols	Energy Ratio: Not Available

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

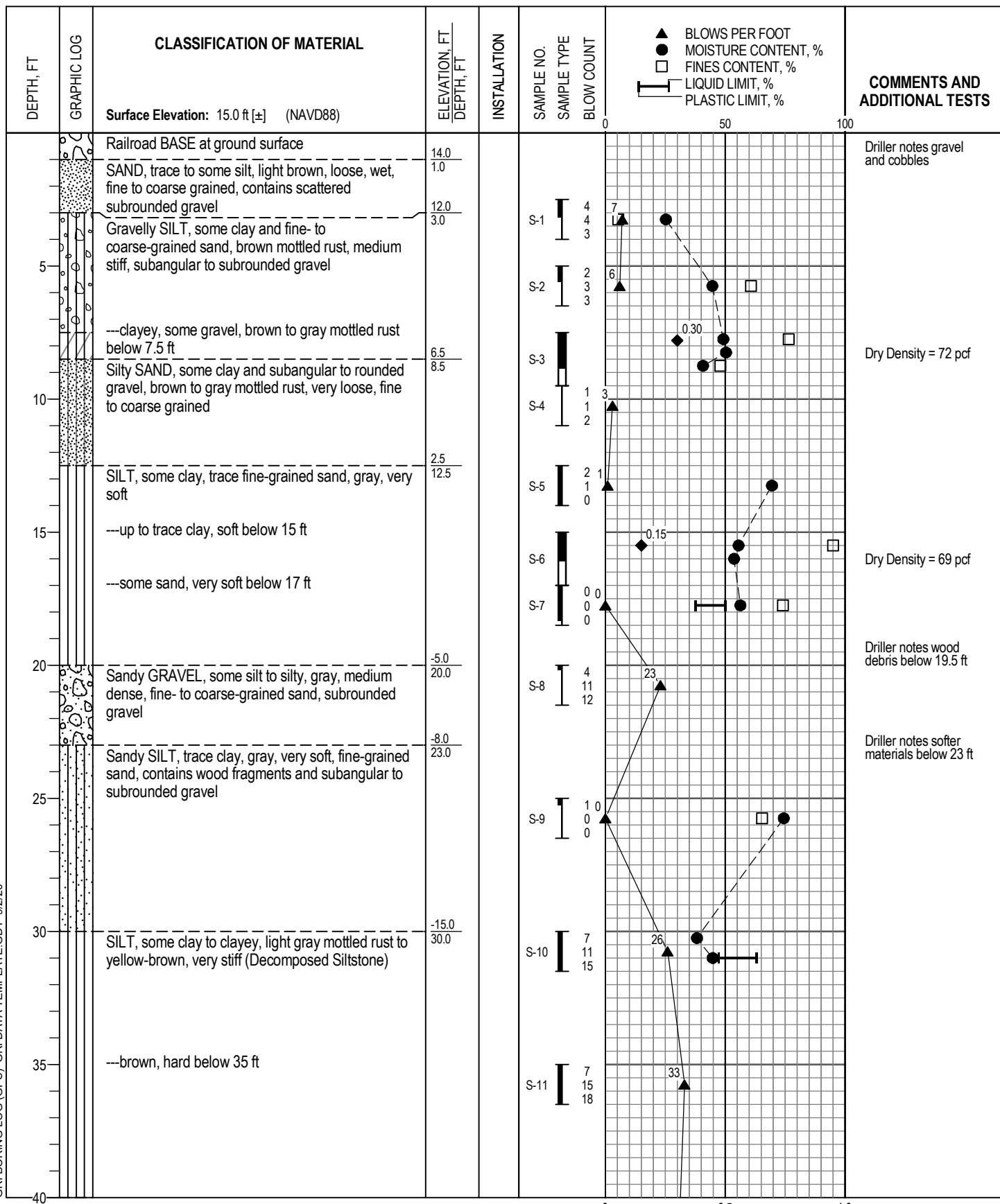


BORING B-1





BORING B-1



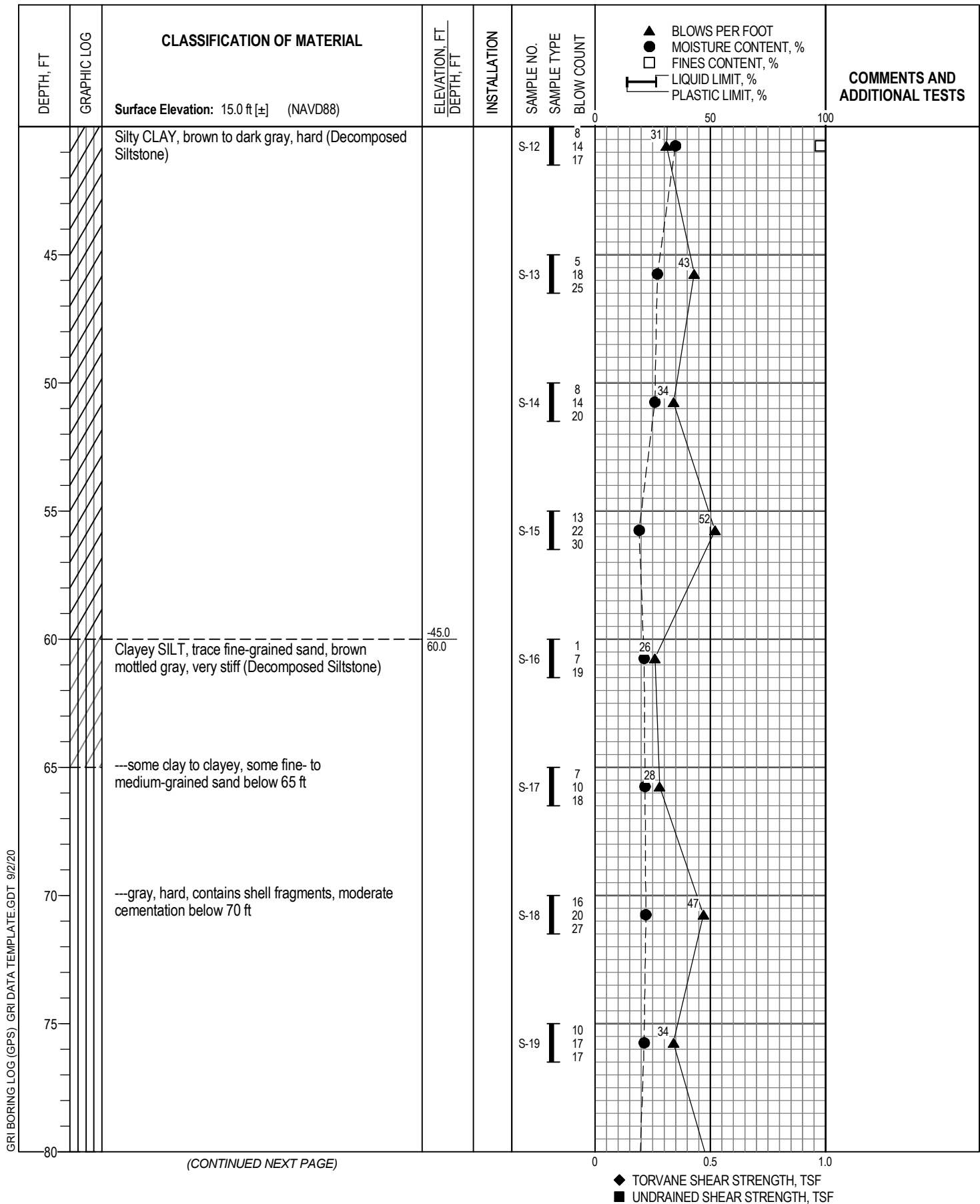
(CONTINUED NEXT PAGE)

Logged By: N. Utefsky	Drilled by: Western States Soil Conservation, Inc.
Date Started: 2/24/20	GPS Coordinates: 46.16975° N -123.6909° W (WGS 84)
Drilling Method: Mud Rotary	Hammer Type: Auto Hammer
Equipment: Geoprobe 7822DT	Weight: 140 lb
Hole Diameter: 5 in.	Drop: 30 in.
Note: See Legend for Explanation of Symbols	Energy Ratio: Not Available

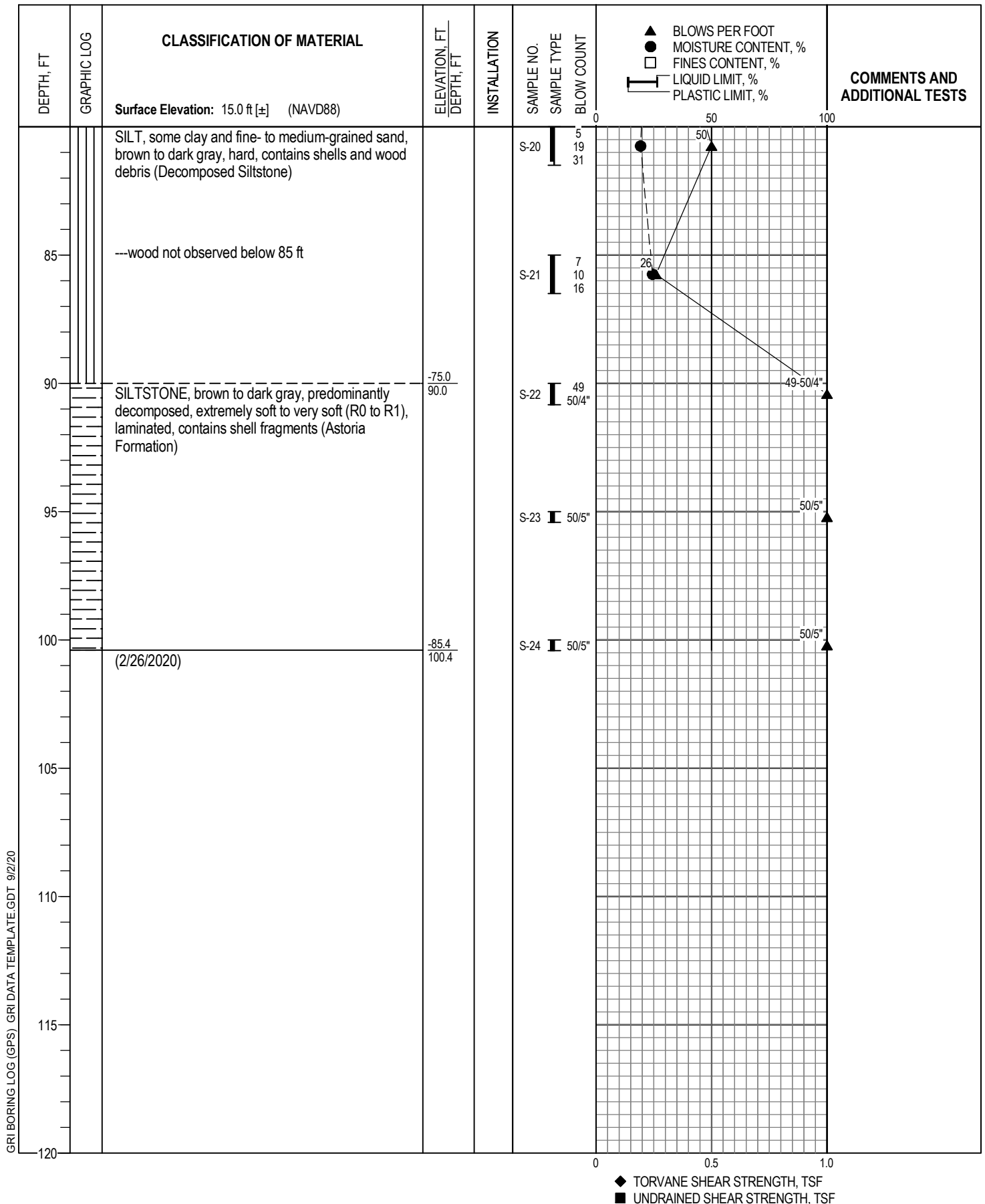
◆ TORVANE SHEAR STRENGTH, TSF
 ■ UNDRAINED SHEAR STRENGTH, TSF



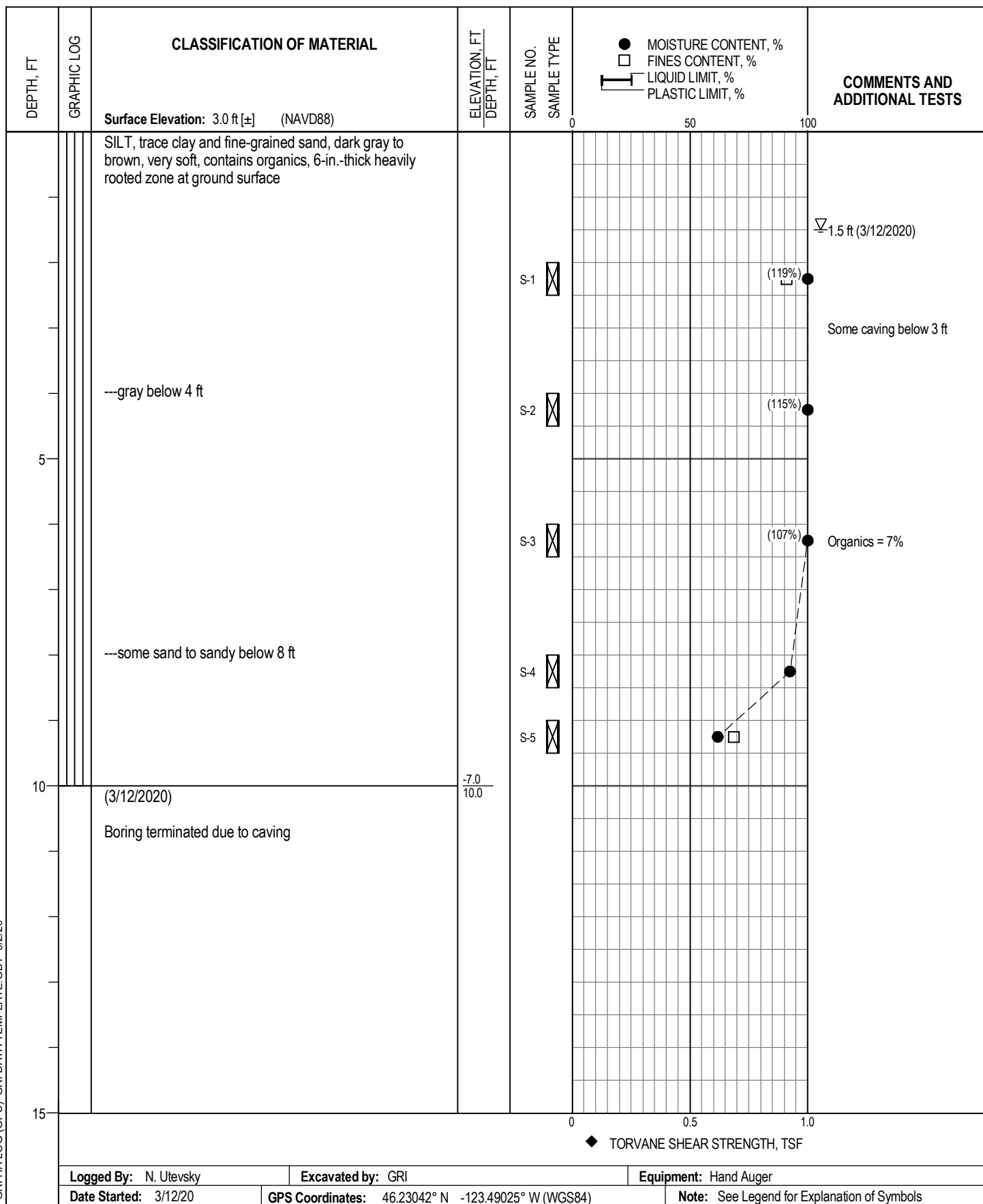
BORING B-2



BORING B-2

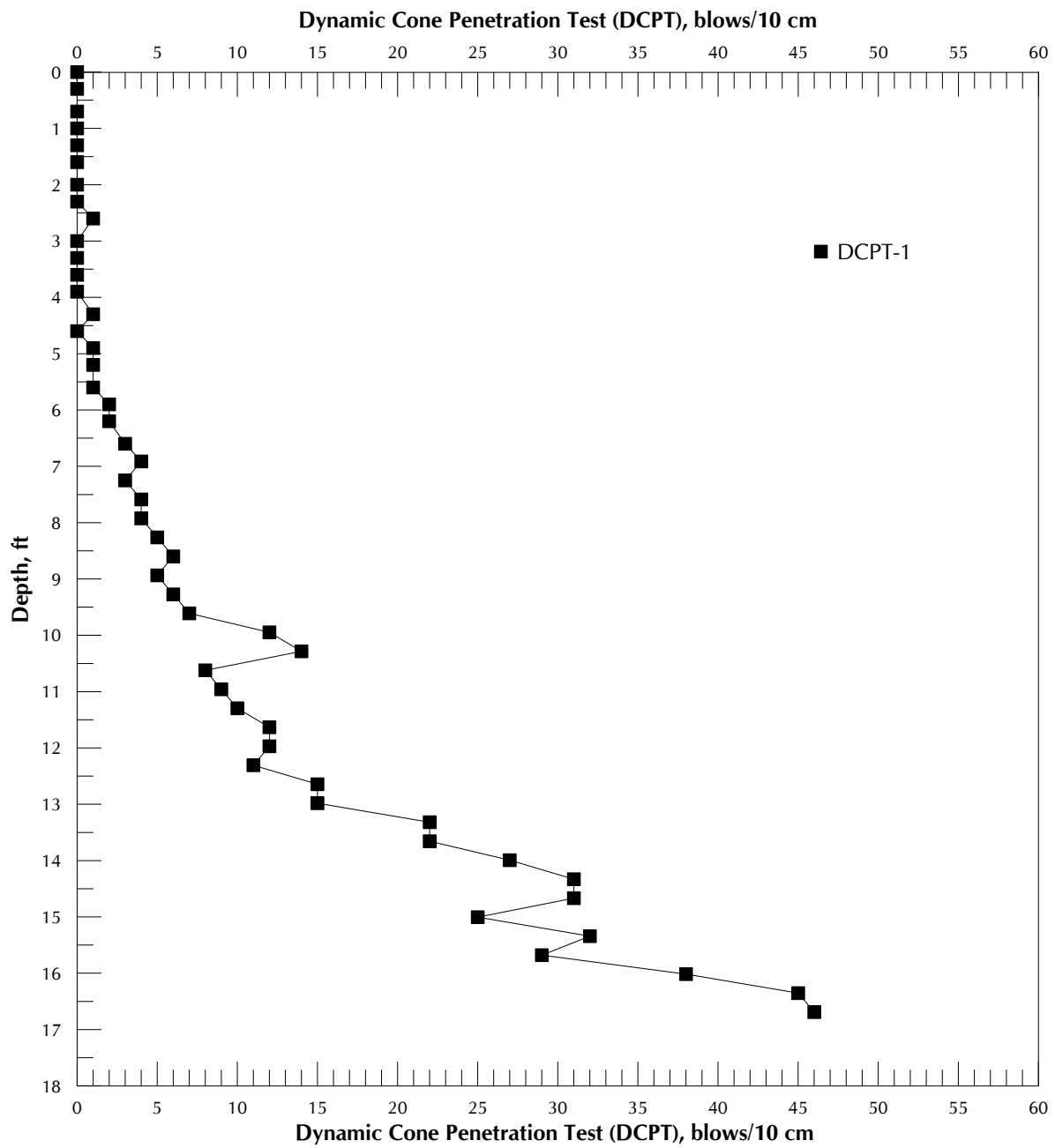


BORING B-2



GRI

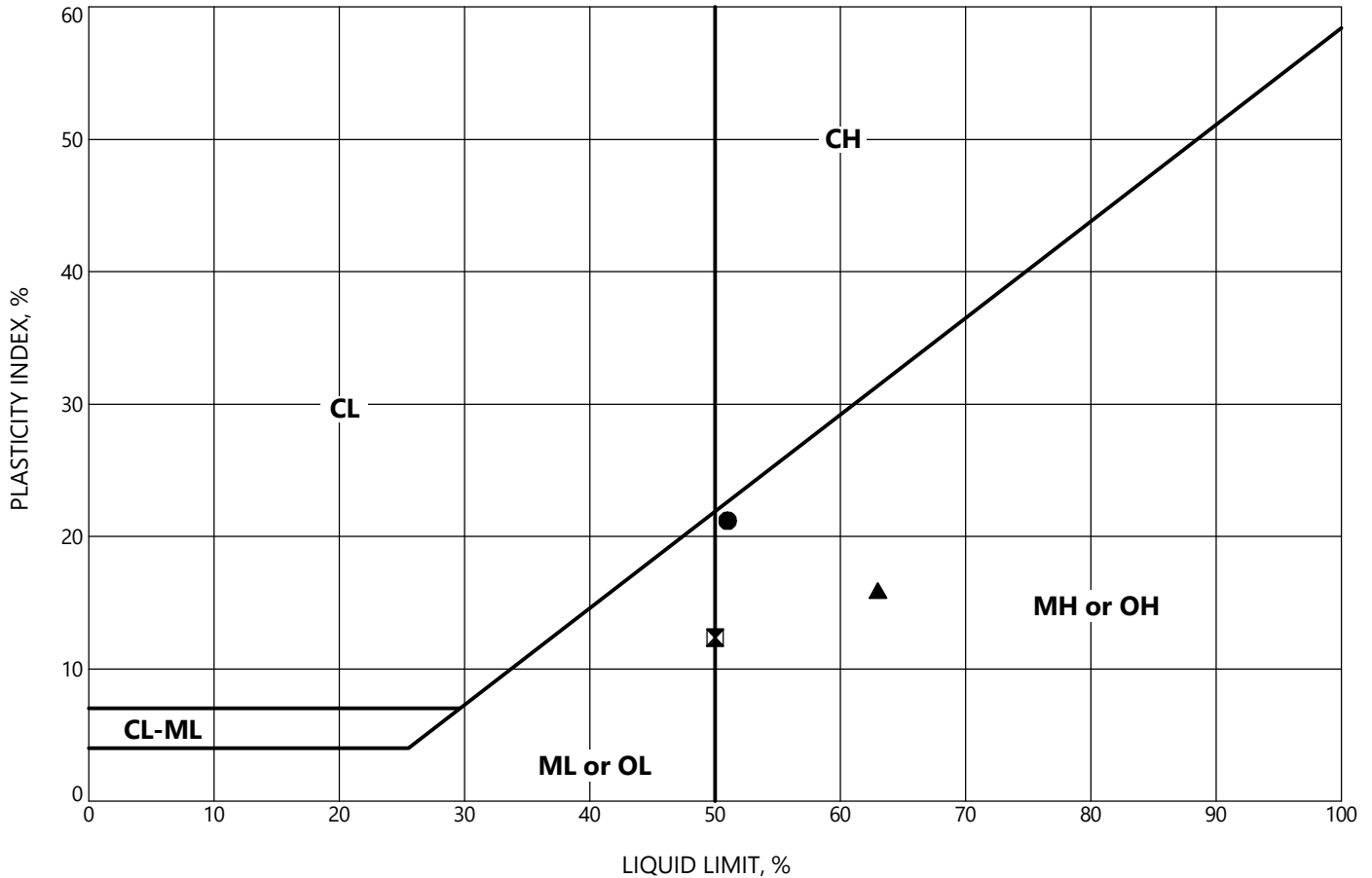
BORING HA-1



DYNAMIC CONE PENETRATION

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

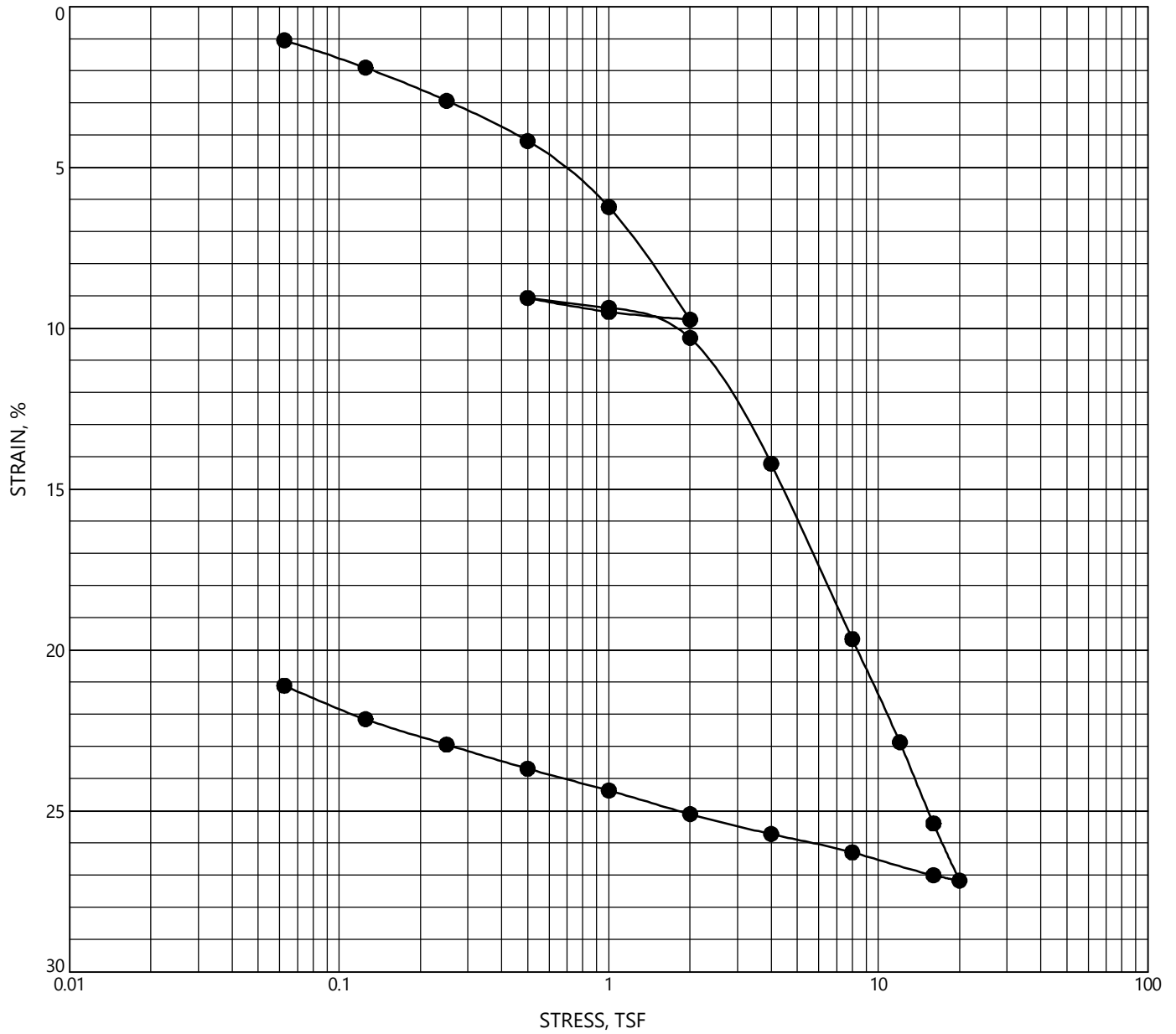
GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-1	S-9	30.0	Clayey SILT, trace fine- to medium-grained sand, gray (Decomposed Siltstone)	51	30	21	34
⊗	B-2	S-7	17.0	SILT, some fine-grained sand, trace clay, gray	50	38	12	56
▲	B-2	S-10	30.5	SILT, some clay to clayey, light gray mottled rust to yellow-brown (Decomposed Siltstone)	63	47	16	45

GRI

PLASTICITY CHART



					Initial	
Location	Sample	Depth, ft	Classification		γ_d , pcf	MC, %
● B-2	S-6	15.3	SILT, trace fine-grained sand, gray, soft		64	58



CONSOLIDATION TEST