

Geotechnical Investigation 1st Street Wall Replacement Union Street to Washington Street

The Dalles, Oregon

October 20, 2025

Prepared for

KPFF Consulting Engineers
111 SW 5th Avenue, Suite 2500
Portland, OR 97204

Prepared by



16520 SW Upper Boones Ferry Road, Suite 100
Tigard, OR 97224-7661
(503) 641-3478 | www.gri.com

TABLE OF CONTENTS

1	INTRODUCTION.....	1
2	PROJECT DESCRIPTION	1
3	SITE DESCRIPTION	1
	3.1 Site Conditions.....	1
	3.2 Geology.....	2
4	SUBSURFACE CONDITIONS	3
	4.1 General.....	3
	4.2 Pavement Thickness	3
	4.3 Machine-Drilled Borings.....	3
	4.4 Groundwater.....	5
5	CONCLUSIONS AND RECOMMENDATIONS	6
	5.1 General.....	6
	5.2 Seismic Considerations	7
	5.3 Earthwork	8
	5.4 Structural Fill	9
	5.5 Retaining Wall Excavation.....	10
	5.6 Retaining Wall Foundation Support.....	12
	5.7 Retaining Wall Lateral Design.....	14
	5.8 Retaining Wall Drainage	15
	5.9 Retaining Wall Global Stability	16
	5.10 Micropile Foundation Support of Elevated Sidewalks	17
	5.11 Construction-Induced Vibrations.....	19
6	DESIGN REVIEW AND CONSTRUCTION SERVICES	20
7	LIMITATIONS.....	20
8	REFERENCES.....	22

TABLES

Table 5-1:	Recommended Seismic Design Parameters (2020 AASHTO SBD).....	7
Table 5-2:	Preliminary Grout-to-Ground Bond Strengths at Bent 3	18
Table 5-3:	Soil Properties for LPILE Analysis	19
Table 5-4:	Recommended P-Modifiers for Group Effects.....	19

FIGURES

Figure 1:	Vicinity Map
Figure 2:	Site Plan
Figure 3:	Surcharge-Induced Lateral Pressure

FIGURES (Continued)

- Figure 4: Global Stability Section A-A' Static Case
- Figure 5: Global Stability Section A-A' Seismic Case
- Figure 6: Global Stability Section A-A' Post-Seismic Case

APPENDICES

- Appendix A: Field Explorations and Laboratory Testing
- Appendix B: Geoprofessional Business Association Guidance Document

1 INTRODUCTION

As requested, GRI completed a geotechnical investigation for the proposed retaining wall replacement and sidewalk improvements along E 1st Street in The Dalles, Oregon. The general location of the project is shown on the Vicinity Map, Figure 1. The purpose of the geotechnical investigation was to evaluate subsurface conditions and existing wall backfill materials at the site and develop recommendations for the design and construction of the proposed retaining wall replacement and sidewalk improvements. The investigation consisted of subsurface explorations and laboratory testing. Based on the results of the investigation, we have prepared geotechnical recommendations to support the design and construction of the new retaining walls. This report describes the work accomplished and provides our conclusions and recommendations.

2 PROJECT DESCRIPTION

Based on the information provided by KPFF Consulting Engineers, Inc. (KPFF), we understand improvements to the site will include the replacement of sidewalks supported by several existing rockery masonry walls or post-and-beam concrete structures. Current plans include supporting the new sidewalks with wire basket gabion or mechanically stabilized earth (MSE) walls and micropiles for support of elevated sidewalks. We understand the City of The Dalles desires to incorporate the stone material from the existing rockery masonry walls into the proposed gabion walls to the extent possible. We anticipate the exposed height of the wall will effectively match the height of the existing rockery walls plus subgrade embedment. The improvements will include areas between existing buildings along E 1st Street between Union Street and Washington Street. We anticipate that retaining wall construction will be performed using open-cut excavation methods.

3 SITE DESCRIPTION

3.1 Site Conditions

Our understanding of existing conditions at the project site is generally based on information provided by KPFF in its "Structural Assessment and Recommendations for Existing Stone Masonry Walls and Related Elements Along 1st Street Between Union Street and Washington Street," dated January 26, 2022. The project area is on the south side of E 1st Street between Union Street and Washington Street. E 1st Street is built up on fill varying from about 2 feet to 9 feet above existing grades south of the sidewalk. We understand the road embankment was originally retained by a drystack stone masonry wall located along the south side of the road. The sidewalks along E 1st Street were constructed after road construction and are supported by exterior masonry rockery walls with variable backfill. We understand the height of the rockery walls varies from around 8 feet to 9 feet between Union Street and Court Street and from around 4 feet to 6 feet between Court Street and Washington Street. A portion of the sidewalk is also supported

by concrete post-and-beam structures without backfill below the sidewalk. The approximate extent of the elevated sidewalk area is shown on the Site Plan, Figure 2.

Three masonry buildings of historic significance are located immediately adjacent to the sidewalk and south of E 1st Street. The locations and designations of the buildings are shown on Figure 2. The storefronts have at-grade access along the elevated grades of E 1st Street. The buildings have lower-level or basement-level floors accessed by the lower level and likely original site grades to the south. We understand the backfill materials were placed immediately against some of the historic buildings. Additionally, we understand that a portion of the elevated sidewalk adjacent to the Baldwin Saloon is in contact with or may be structurally supported by the exterior masonry walls of the building.

A railroad is present on the north side of E 1st Street, and the Lewis and Clark Festival Park is on the north side of the railroad. The Columbia River is approximately 500 feet north of the site.

3.2 Geology

The project area is in the northwest section of the Deschutes–Columbia Plateau physiographic province, within the northern extension of The Dalles Basin. The Deschutes–Columbia Plateau is dominated by numerous episodes of volcanic flows of the Columbia River Basalt Group (CRBG). The CRBG is a series of extensive basalt flows that originated from fissures in southeastern Washington and northeastern Oregon. The volcanic flows filled a progressively subsiding basin located near Yakima, Washington, and followed the ancient Columbia River drainage valley out to the Pacific Ocean. For the purposes of our report, the Columbia River Basalt Group is considered the basement material at this site.

Within the northwestern Deschutes-Columbia Plateau province lies The Dalles Basin, which is bordered by the Columbia Hills anticline on the north in Washington, the Cascade Range to the west, the Tygh Ridge anticline to the south, and the basalt upland to the east of the Deschutes River (Orr and Orr, 2000; Farooqui et al., 1981). The Dalles Basin was subsequently filled in during the late Miocene to early Pliocene as an eastward-spreading volcanoclastic debris fan, referred to as the Chenoweth Formation. The Chenoweth Formation overlies the Priest Rapids and Frenchman Springs Members of the Columbia River Basalt Group. Lahars and interbedded conglomerate and sandstone rich in andesite clast dominate the western part of the formation near the foothills of the Cascade Range. Moving northward and eastward, the proportion of lahars decreases while fluvial deposits of conglomerate, sandstone, and siltstone become more common, transitioning from a volcanic-sedimentary debris fan to a sedimentary deposit. Deposits of the Chenoweth Formation commonly have abrupt lithofacies changes and generally thin to the north in Washington (Farooqui et al., 1981).

The surficial geologic units mapped at the site consist of quaternary alluvium deposits with Wanapum Basalt and Priest Rapids Member to the south, west, and east (Korosec, 1987). Basalt was observed outcropped in the near vicinity of our drilling.

4 SUBSURFACE CONDITIONS

4.1 General

Subsurface materials and characteristics at the site were investigated on May 25 and 26, 2023, by GRI with four machine-drilled borings, designated B-1 through B-4. Boring B-1 was advanced to a depth of 31.5 feet, boring B-2 to a depth of 29 feet, boring B-3 to 27.9 feet, and boring B-4 to a depth of 14 feet. The approximate locations of the explorations completed for this investigation are shown on Figure 2. Logs of the borings and descriptions of the field and laboratory programs conducted to evaluate the engineering properties of the materials encountered in the borings are provided in Appendix A. The terms and symbols used to describe the materials encountered in the borings are defined in Table 1A and in the attached legend. A discussion of the subsurface materials disclosed by the borings is presented in the following sections.

4.2 Pavement Thickness

The pavement thickness at the boring locations disclosed a variable thickness of asphalt concrete (AC) pavement at the project site. Pavement thickness ranged from 3.5 inches to 4 inches in borings B-1 through B-3 and was 1.25 inches in boring B-4. Aggregate base thickness ranged between 7.5 inches and 10.75 inches; however, due to the mud rotary drilling method and underlying fill, these thicknesses should be considered approximate.

4.3 Machine-Drilled Borings

For the purpose of discussion, the materials disclosed by the drilled borings completed along E 1st Street have been grouped into the following units based on their physical characteristics and engineering properties; they are listed as they were encountered below the aforementioned pavement section:

- a. FILL
- b. SILT, SAND, CLAY, and GRAVEL (Alluvium)
- c. Clayey SILT (Chenoweth Formation)

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

a. Fill

The pavement sections at borings B-1 through B-4 are underlain by fill extending to a depth of about 12.5 feet in borings B-1 through B-4, respectively. The fill in boring B-1 consists of subrounded to subangular silty gravel to gravelly silt with variable

concentrations of fine- to coarse-grained sand and contains cobbles. The material in boring B-1 was typically brown and gray to dark gray. Based on Standard Penetration Test (SPT) blow counts, the relative density of the gravel was very loose to loose. Fill encountered in boring B-2 contained silty, fine- to coarse-grained sand to sandy silt with a subrounded gravelly zone at 5 feet below ground surface (bgs). The material in boring B-2 was typically brown, and, according to SPT blow counts, the relative density was loose. In boring B-3, the material encountered included silty, subrounded to subangular gravel to gravelly silt with variable concentrations of fine- to coarse-grained sand and silty fine- to medium-grained sand to sandy silt lenses up to approximately 2.5 feet thick. The material was typically brown and gray to dark gray. SPT blow counts indicate the relative density of the sand and gravel is loose. Fill encountered in boring B-4 contained silty, gravelly, fine- to coarse-grained sand to silty gravel with variable concentrations of sand and lenses of silty, fine-grained sand. The material was typically light brown and brown to gray. Based on SPT blow counts, the relative density of fill encountered in boring B-4 typically ranges from very loose to medium dense. Within boring B-4 at 10 feet bgs, thick woody debris and occasional porcelain fragments were observed in the drilling fluid cuttings, accompanied by a strong inorganic odor. The boring was terminated at 14 feet bgs, and the project team was notified of the potential contamination.

Drilling conditions within the fill material in all borings were generally difficult. Very loose to loose fill soils were encountered in all borings, with the exception of boring B-4 at 2.5 feet, where loose to medium dense SPT blow counts were observed. Drilling fluid circulation was lost at multiple depth intervals in borings B-1 and B-3, with total drilling fluid loss ranging from about 150 gallons (B-3) to 350 gallons (B-1). Drill casing was used in boring B-1 to a depth of 13.5 feet bgs to maintain drilling operations. Borings B-2 and B-4 maintained drilling fluid circulation but still lost approximately 80 gallons of drilling fluid each. Caving within the fill unit was common. Sample recovery in the fill material was poor and, due to the loose nature, it was likely the case that samplers pushed material down rather than obtaining a collective sample. Also, very loose to loose materials allow for drill bits to move around larger obstacles rather than drill through them. Boulders, although not directly encountered in our drilling explorations, should be anticipated.

b. SILT, SAND, CLAY, and GRAVEL (Alluvium)

Below the fill are alluvium deposits that consist of fine-grained silt, sandy silt, and clay and include coarse-grained deposits of fine- to medium-grained silty sand to sand with some silt. The base of the alluvium unit is marked by a layer of silty gravel to gravel with variable concentrations of sand and silt. The alluvium was encountered at a depth of 12.5 feet in borings B-1 through B-4. The alluvial deposits extended to depths of 23 feet in boring B-1, 23.5 feet in boring B-2, and 24 feet in boring B-3. Boring B-4 was terminated at a depth of 14 feet, just below the fill due to potential contamination encountered. The fine-grained

deposits of silt, sandy silt, and clay are light brown and gray to dark gray and contain occasional fine woody debris. Stratified bedding (alternating layers at least ¼-inch thick) is common and typically observed at 3 inches to 6 inches thick. The natural moisture content of the fine-grained deposits in borings B-1 through B-3 typically ranges from 37% to 45%. Atterberg limits testing of a silt sample recovered at a depth of 16 feet in boring B-3 yielded a liquid limit of 35% and plasticity index of 7%. Organic content testing of sandy silt recovered at a depth of 12.5 feet in boring B-1 yielded a result of 4% organics. SPT blow counts indicate that the consistency of the sandy silt material ranged from very soft to soft.

Coarse-grained soils of silty sand and sand with some silt encountered in boring B-2 contained a natural moisture content ranging from 27% to 28%. SPT blow counts indicate that the relative density of the silty sand and sand with some silt ranged from very loose to medium dense. The base of the alluvium deposits is marked by silty gravel to gravel with variable concentrations of fine- to coarse-grained sand, as encountered in borings B-1 through B-3. Gravel thickness ranged from 2.5 feet to 4.5 feet. The natural moisture content of the gravel ranged between 14% to 15%. SPT blow counts indicate a relative density of dense to very dense.

c. Clayey SILT (Chenoweth Formation)

Clayey silt with a trace of sand was encountered below the alluvial deposit in boring B-1 at 23 feet, boring B-2 at 23.5 feet, and boring B-3 at 24 feet. The sand constituent is fine grained, and the clayey silt is light gray to light brown-orange that transitions to green with depth. The natural moisture content of the clayey silt soils in borings B-1 through B-3 typically ranges from 25% to 38%. One natural moisture content of 65% was recorded in boring B-1 at 30 feet. Atterberg limits testing in boring B-2 at 25 feet yielded a liquid limit of 60 and a plasticity index of 25%. SPT blow counts indicate that the consistency of the clayey silt ranges from very stiff to hard.

4.4 Groundwater

The explorations completed for this project were advanced with mud rotary drilling techniques, which do not allow for the direct observation of groundwater within the borings at the time of drilling. Wet soil samples were observed in borings B-1 through B-4 at the fill-alluvium interface at a depth of 12.5 feet bgs. Wet soils were also observed in borings B-3 and B-4 within the fill unit at a depth of 5 feet bgs. It is our interpretation that groundwater levels were at a depth of approximately 12.5 feet at the interface between the fill and alluvial deposits at the time of our explorations. Perched groundwater should be anticipated within the variable fill material. Groundwater levels vary seasonally and are typically highest during the winter and spring months and lowest during the late summer and fall.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

The existing retaining wall backfill encountered in our explorations generally consists of silt, sand, gravel, and cobble materials. The existing wall and backfill are underlain by alluvium consisting of silt and silty sand, grading to dense gravel. Based on free water observed in recovered samples, the depth to groundwater was estimated at approximately 12.5 feet bgs at the time of our explorations. Zones of perched groundwater were encountered within the retaining wall backfill at a depth of approximately 5 feet bgs.

The primary geotechnical considerations associated with the construction of the proposed improvements include the following:

- The presence of soft or loose sediments in the vicinity of temporary cut slopes planned for construction of new retaining walls and soft compressible materials below the retaining wall subgrade.
- The presence of gravels, cobbles, and potentially boulders, which can cause difficulty with excavation.
- The loss of drilling fluid during the explorations suggests the materials below the existing sidewalk and roadway have significant voids, and the potential exists for raveling or sloughing during excavation.
- The new loads imposed on the underlying compressible material have the potential to influence settlement below retaining wall foundations and adjacent existing buildings.
- The groundwater levels may be in the range of anticipated subgrade elevations and excavations.
- There are potentially shallow perched-groundwater conditions within the existing embankment fill, and the fine-grained soils are moisture sensitive. Completing construction during the dry season will likely improve foundation conditions in retaining wall subgrades and the cut slope stability of temporary excavation slopes during construction.

Recommendations for the design and construction of the proposed retaining walls and micropiles for support of elevated sidewalks are provided in the following sections.

5.2 Seismic Considerations

5.2.1 General

We understand the retaining walls will be seismically designed in accordance with the 2020 American Association of State Highway and Transportation Officials (AASHTO) *Load and Resistance Factor Design Bridge Design Specifications* (LRFD BDS) requirements.

The AASHTO seismic bridge design (SBD) uses the 2014 USGS seismic hazard maps to evaluate earthquake-induced accelerations associated with a probabilistic 1,033-year (7% probability of occurrence in 75 years) return period of ground motion. In addition, the AASHTO SBD methodology develops the design-level spectra for seismic analysis using two points at spectral periods of 0.2 seconds and 1.0 seconds (S_s and S_1). Site factors F_a and F_v are then used with a site class designation to adjust the bedrock spectral accelerations to ground-surface spectral accelerations. The parameters for the 1,033-year-interval hazard level were obtained for the site coordinates of 45.6024° N latitude and 121.1820° W longitude using the Oregon Department of Transportation (ODOT) design spectrum spreadsheet tool, which is consistent with the AASHTO SBD seismic design methodology. The peak ground acceleration (PGA), S_s , and S_1 spectral values corresponding to the 1,033-year hazard level for the project site are provided in Table 5-1, below. Based on site class definitions provided in the AASHTO SBD, the bridge and retaining wall structures can be designed using Site Class D. The following table includes site coefficients for Site Class D recommended for the project site.

Table 5-1: RECOMMENDED SEISMIC DESIGN PARAMETERS (2020 AASHTO SBD)

Seismic Parameter	Recommended Value*
Site Class	D
Peak Ground Acceleration	0.14 g
Site-Adjusted Peak Ground Acceleration, A_s	0.21 g
Spectral Response Acceleration Parameter at Short Periods, S_s	0.30 g
Spectral Response Acceleration Parameter at 1.0-Second Period, S_1	0.11 g
Design Spectral Response Acceleration Parameter at Short Periods, S_{DS}	0.47 g
Design Spectral Response Acceleration Parameter at 1.0-Second Period, S_{D1}	0.26 g

Abbreviations: AASHTO = American Association of State Highway and Transportation Officials; SBD = seismic bridge design

5.2.2 Liquefaction / Cyclic Softening

Liquefaction is the process by which loose, saturated granular materials, such as clean sand and, to a somewhat lesser degree, nonplastic and low plasticity silts, temporarily lose stiffness and strength during and immediately after a seismic event. This degradation in soil properties may be substantial and abrupt, particularly in loose sands. Liquefaction

occurs as seismic shear stresses propagate through saturated soil and distort the soil structure, causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure causes the pore-water pressure to increase between the soil grains. If the pore-water pressure becomes sufficiently large, the intergranular stresses become small, and the granular layer temporarily behaves as a viscous liquid rather than a solid. After liquefaction is triggered, there is an increased risk of settlement, loss of bearing capacity, lateral spreading, and/or slope instability, particularly along waterfront areas. Liquefaction-induced settlement occurs as the elevated pore-water pressures dissipate and the soil consolidates after the earthquake.

“Cyclic softening” is a term that describes a relatively gradual and progressive increase in shear strain with load cycles and is more common in fine-grained soils. Excess pore pressures may increase due to cyclic loading but will generally not approach the total overburden stress. Shear strains accumulate with additional loading cycles, but an abrupt or sudden decrease in shear stiffness is not typically expected. Settlement due to post-seismic consolidation can occur, particularly in lower-plasticity silts. Large shear strains can develop, and strength loss related to soil sensitivity may be a concern.

The potential for liquefaction and/or cyclic softening at the site was evaluated using the simplified method based on procedures recommended by Idriss and Boulanger (2008) with subsequent revisions (Boulanger and Idriss, 2014). This method uses PGA at the ground surface to predict the cyclic shear stresses induced by the earthquake. The U.S. Geological Survey (USGS) National Seismic Hazard Mapping Project was used to determine the contributing earthquake magnitude and PGA representing the seismic exposure of the site for the 1,033-year hazard level. For our evaluation, we used a PGA (A_s) of 0.21 g and a magnitude (M_w) of 6.9, corresponding primarily to crustal earthquake sources. We assumed a groundwater depth of 12.5 feet below the surface of E 1st Street.

The results of our evaluation indicate that a potential zone of interbedded sand and low plasticity silt underlying the existing fill at the site exists that would be susceptible to liquefaction / cyclic softening. Our analysis indicates the potential for up to about 3 inches of seismically induced settlement during the design earthquake and after earthquake shaking has ceased. Discussion of seismically induced retaining wall settlement is presented in Section 5.5.2 of this report.

5.3 Earthwork

The fine-grained soils that mantle the site are moisture sensitive, and groundwater may approach retaining wall subgrade elevations during the wet winter and spring months and periods of heavy or prolonged precipitation. Therefore, it is our opinion that earthwork

can be completed most economically during the dry summer months, typically extending from June to mid-October. To minimize disturbance of the moisture-sensitive, fine-grained soils, site grading can be completed using track-mounted hydraulic excavators. The excavation should be finished using a smooth-edged bucket to produce a firm, undisturbed surface. It may also be necessary to construct granular haul roads and work pads concurrently with excavation to minimize subgrade disturbance. If the subgrade is disturbed during construction, soft, disturbed soils should be overexcavated to firm native soil and backfilled with structural fill. We recommend the contract documents provide unit costs for overexcavation and structural backfill.

If construction occurs in wet ground conditions, granular work pads will be required to protect the underlying subgrade and provide a firm working surface for construction activities. In our opinion, a 12- to 18-inch-thick granular work pad should be sufficient to prevent disturbance of the subgrade by lighter construction equipment and limited traffic by dump trucks. Haul roads and other high-density traffic areas, including the use of Gradalls and forklifts, will require a minimum of 18 inches to 24 inches of fragmental rock, up to 6-inch nominal size, to reduce the risk of subgrade deterioration. The use of a woven geotextile fabric or geogrid over the subgrade may reduce the need for maintenance during construction.

5.4 Structural Fill

5.4.1 General

All fill placed within the roadway embankment should be placed as structural fill. It is the contractor's responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet the project specifications. Qualified geotechnical engineering staff should evaluate the compaction of each lift of fill. Recommendations for structural fill are provided in the following sections.

5.4.2 On-Site Soils

Based on our explorations and information provided by KPFF, the existing fill materials at the site are highly variable and contain clay and cobbles. In our opinion, the existing embankment fill and wall backfill materials at the site are not suitable for use as structural fill within the limits of the roadway.

5.4.3 Imported Fill

Imported structural fill should consist of 1- or ¾-inch-minus crushed rock meeting the gradation requirement of the Oregon Standard Specifications for Construction (OSSC) Section 02630.10. Wall backfill material should meet the requirements of OSSC Section 00596B.11(d). In our experience, granular material that has less than 5% passing the No. 200 sieve (washed analysis) can usually be placed during periods of wet weather.

Granular backfill should be placed in lifts and compacted with vibratory equipment to at least 95% of the maximum dry density, as determined in accordance with ASTM International (ASTM) D698. The relative compaction of structural fill should be evaluated using nuclear field-density testing.

5.4.4 Gabion Basket Fill

Material used to fill the gabion baskets should typically consist of durable, well-graded 4- to 8-inch rocks meeting the requirements of OSSC Section 00390.11(b). Cobbles recovered from the existing stone masonry wall that meet the specified grading and durability requirements may be reused for gabion walls on the project.

5.5 Retaining Wall Excavation

5.5.1 General

We understand the new retaining wall will be constructed using temporary excavation slopes. Based on a maximum exposed wall height of approximately 9 feet and an embedment depth of 2 feet, we anticipate that maximum temporary cut slope excavations up to around 11 feet will be required to accommodate construction of the new retaining wall. The method of excavation and the design of temporary excavation support are the responsibilities of the contractor and are subject to applicable local, state, and federal regulations, including the current Occupational Safety and Health Administration (OSHA) excavation and trench safety standards. The means, methods, and sequencing of construction operations and site safety are also the responsibilities of the contractor. The information provided below is for the use of our client and should not be interpreted to imply that we are assuming responsibility for the contractor's actions or site safety.

5.5.2 Temporary Excavation Slopes

Excavation for the new retaining wall will require the removal of the existing fill present between the older interior stone embankment slope and the exposed exterior rockery wall below the sidewalks. The geotechnical investigation encountered relatively high variability in subsurface materials near the excavation and wall backfill zone, and the actual conditions of the materials in the backslopes of temporary excavations will not likely be known until construction occurs. Based on conceptual drawings provided by KPFF, temporary excavation slopes will likely encroach upon the roadway with the use of shoring or very steep temporary backslopes. Due to the materials encountered in the excavations, steep backslopes greater than 1H:1V are not likely feasible without the use of shoring. Consequently, temporary shoring may be required to limit excessive encroachment into the roadway. Where excavation slopes are constructed in close proximity to the edge of the roadway and can be maintained without temporary shoring, the temporary excavations should be constructed in short segments of about 8 feet to 10 feet maximum along the base of the new retaining wall. Backfill should be placed immediately behind the

first row of gabion baskets or MSE walls to provide toe support for the remaining temporary excavation slope. Open cuts along the full length of the retaining wall should not be performed unless wall heights are less than about 4 feet and backslopes do not encroach on the roadway.

The stability of temporary excavation slopes will depend, in part, on the localized composition and relative consistency/density of the embankment materials, perched groundwater conditions encountered at the time of construction, and the contractor's ability to control these conditions. We anticipate temporary construction slopes in the range of 1H:1V (horizontal to vertical) to 1.5H:1V will remain stable within the range of anticipated temporary slope heights, assuming groundwater levels are at least 2 feet below the bottom of the excavation. Flatter slopes may be necessary if significant seepage conditions are encountered, or if gravels, cobbles, or boulders are encountered that are prone to sloughing. Some minor amounts of sloughing, slumping, or running of temporary slopes should be anticipated shortly after excavation and if perched groundwater seepage occurs. A blanket of relatively clean, well-graded, crushed rock placed against the slopes may be required to reduce the risk of these conditions if temporary excavation slopes encounter perched groundwater. We recommend the use of relatively clean, free-draining material, such as 2- to 4-inch-minus crushed rock, for this purpose. The thickness of the granular blanket should be evaluated based on actual conditions but is typically in the range of 12 inches to 24 inches.

In our opinion, the short-term stability of temporary slopes will be adequate if surcharge loads due to construction traffic, vehicle parking, material laydown, etc. are maintained at a horizontal distance equal to the height of the slope away from the top of the open cut. Other measures that should be implemented to reduce the risk of localized failures of temporary slopes include the following: 1) using nonwoven geotextile fabric to protect the exposed cut slopes from surface erosion; 2) providing positive drainage away from the tops and bottoms of the cut slopes; 3) constructing and backfilling embedded structures as soon as practical after completing the excavation; 4) backfilling overexcavated areas as soon as practical after completing the excavation; and 5) periodically monitoring the area around the top of the excavation for evidence of ground cracking. It must be emphasized that following these recommendations will not guarantee sloughing or movement of the temporary cut slopes will not occur; however, the measures should serve to reduce the risk of a major slope failure. It should be realized, however, that blocks of ground and/or localized slumps may tend to move into the excavation during construction.

5.5.3 Groundwater Management

Groundwater was encountered in our explorations at a depth of approximately 12.5 feet bgs and may occur at higher levels during the wet winter months. Additionally, perched

groundwater may be encountered at shallower depths within the existing roadway embankment. Groundwater seepage, running-soil conditions, and unstable excavation side slopes or excavation subgrades, if encountered during construction, will require dewatering of the excavation and side slope support. The impact of these conditions can be reduced by completing excavations during the summer months, when perched groundwater levels are lowest, and by limiting the depths of the excavations.

We anticipate perched groundwater seepage, if encountered, can be controlled by pumping from temporary sumps in the bottom of the excavation. To facilitate dewatering, it will be necessary to overexcavate the base of the excavation to permit installation of a granular working blanket. We estimate the required thickness of the granular working blanket will be on the order of 1 foot or as required to maintain a stable excavation base. The actual required depth of overexcavation will depend on the conditions exposed in the excavations and the effectiveness of the contractor's dewatering efforts. The thickness of the granular blanket must be evaluated based on field observations made during construction. We recommend the use of relatively clean, free-draining material, such as 2- to 4-inch-minus crushed rock, for this purpose. The use of geotextile fabric over the excavation base will assist in stability and dewatering.

5.5.4 Excavation Adjacent to Historic Buildings

The project will include construction activities near existing historic buildings for the E 1st Street retaining wall improvements. The structural condition of the existing buildings is not well understood; therefore, the buildings should be considered sensitive to construction vibrations and changes in vertical and horizontal earth pressures in the vicinity of existing basement walls and foundations. We recommend existing buildings be shored where excavations occur along existing basement walls unless a structural integrity evaluation of the buildings is conducted by others and determines that soil, rock, concrete sidewalks, or other materials in contact with the buildings can be temporarily removed without adversely affecting the structures.

5.6 Retaining Wall Foundation Support

5.6.1 Subgrade Preparation

The retaining wall should be established in firm, native subgrade that underlies the wall area. Excavations for the retaining wall should be made with a smooth-edged bucket, and all retaining wall subgrades should be evaluated by a member of GRI's geotechnical engineering staff. Soft or otherwise unsuitable soil or organic material, if encountered at subgrade elevation, should be overexcavated and backfilled with compacted structural fill. We recommend embedding the toe of the retaining wall at least 24 inches bgs in front of the wall. To provide more uniform support, the retaining wall should be founded on a minimum 6-inch-thick layer of well-compacted ¾-inch-minus crushed rock that extends

at least 6 inches laterally beyond the edges of the gabion baskets. Since the proposed building site is in a developed area, it should be anticipated that some overexcavation may be required during construction. We recommend that overexcavation activities be evaluated by a member of GRI's geotechnical engineering staff and that contract documents provide unit costs for subgrade overexcavation and structural backfill.

5.6.2 Bearing Resistance and Settlement

Our bearing resistance and settlement estimates are based on the results of our subsurface explorations and consolidation testing performed on select undisturbed samples. We recommend the minimum width of MSE or gabion walls be equal to $0.7H$, where H is the total height of the wall (including the embedment depth). Assuming the wall subgrade is prepared as described above, the wall can be designed based on a factored soil bearing resistance of 2,000 pounds per square foot (psf) and 4,500 psf for the Strength and Extreme Event limit states, respectively. The factored bearing resistances are based on resistance factors of 0.45 and 1.0, respectively.

For portions of the new wall that are replacing material within the footprint of the existing masonry wall and backfill, we anticipate that total settlement will typically be less than $\frac{1}{2}$ inch since the net load increase will be minimal. However, our settlement models indicate that the tallest portions of the new wall located below the existing elevated sidewalk areas will add up to approximately 9 feet of new or additional backfill material over very soft and moderate to highly compressible soils. The new loads are estimated to induce up to approximately 3 inches to 5 inches of total consolidation settlement, with the highest settlement occurring near the corner of the new wall along E 1st Street and Court Street. Differential settlements of up to approximately 3 inches over 20 feet are anticipated where the transition from the existing masonry wall to the post-and-beam structure occurs along E 1st Street.

As discussed in the Seismic Considerations section of this report, our analysis indicates up to 3 inches of dynamic settlement could occur due to cyclic strength loss in the very soft silts or liquefaction in the very loose sands beneath the proposed retaining wall following a code-based seismic event. For wall design purposes, we recommend assuming differential seismic settlements of around 1.5 inches per 100 feet of wall.

5.6.3 Settlement Impacts to Existing Historic Buildings

As described above, our settlement analysis indicates approximately 3 inches to 5 inches of settlement may occur where the proposed wall will replace the existing elevated sidewalk areas immediately adjacent to the north and east sides of the historic Baldwin Saloon building. Consequently, construction of the proposed retaining wall in this area would likely induce settlement below the foundations of the existing building. We anticipate that the existing buildings will not tolerate the anticipated settlements imposed

by the adjacent retaining walls. For planning purposes, new walls imposing surcharge pressures on the soft, compressible soils below the site should not be constructed within 15 feet of existing buildings.

It is our opinion that, when constructing new walls near existing buildings, micropiles or lightweight fill material such as expanded polystyrene (EPS) geofoam blocks can be used to support elevated sidewalks instead of crushed rock backfill. A geofoam block wall could be designed to produce a negligible increase in the effective stresses experienced by the subgrade soils below the wall, resulting in a near net-zero increase in surcharge pressures and predicted settlements. Because the unit weight of EPS geofoam blocks is typically on the order of 2 pounds per cubic foot (pcf) to 3 pcf, the dead and live loads from the new sidewalk can effectively be balanced by overexcavation of the wall subgrade. We recommend assuming a unit weight of 115 pcf for the in-situ subgrade soils to determine the amount of overexcavation required to produce a zero-net-load condition below the proposed wall (i.e., every 1 foot of excavated subgrade soil will balance 115 psf of new sidewalk load).

The eastern end of the project likely contains hazardous materials in the soils below and possibly behind the proposed new retaining walls. Lightweight EPS geofoam blocks are not recommended near soils containing hydrocarbons or other substances that have the potential to dissolve geofoam. At the time of this report, we understand micropiles are the preferred alternative for support of elevated sidewalks in settlement sensitive areas, as discussed in the following sections.

5.7 Retaining Wall Lateral Design

5.7.1 Lateral Earth Pressures

The magnitude of lateral earth pressures acting on a retaining wall depends on whether the wall is designed as a yielding wall or a non-yielding wall. The at-rest earth pressure case is applicable to a wall that is relatively rigid and laterally supported at the top and bottom and therefore is unable to yield. The active earth pressure case is applicable to a wall that is capable of yielding slightly away from the backfill by either sliding or rotating about its base. We anticipate the gabion walls will be designed as yielding walls, assuming up to ½ inch of lateral displacement is acceptable.

Assuming the top of the backfill will be horizontal and the backfill completely drained, the gabion or MSE walls can be designed on the basis of an equivalent fluid unit weight of 35 pcf. We recommend a minimum vertical traffic surcharge load of 250 psf be assumed for all permanent walls where pavements are located within a horizontal distance less than the wall height. Lateral forces generated by the traffic surcharge pressures and additional

lateral forces due to construction equipment can be estimated using the guidelines provided on Surcharge-Induced Lateral Pressure, Figure 3.

To account for seismic loading, the Agusti and Sitar (2013) method was used to develop lateral earth pressures on permanent embedded structures. Using this method, the static lateral earth pressures should be increased by an equivalent fluid unit weight of 4 pcf for yielding walls with a level back slope. This results in a triangular distribution with the resultant acting at $\frac{1}{3}H$ up from the base of the wall, where H is the height of the wall in feet. The lateral force induced by an earthquake is in addition to the lateral earth pressures acting on the wall during static conditions.

Wall backfill should be compacted to about 95% of the maximum dry density, as determined by ASTM D698. Overcompaction of the backfill behind walls should be avoided. Heavy compactors and large pieces of construction equipment should not operate within 5 feet of any embedded wall to avoid the buildup of excessive lateral pressures, unless the walls have been designed to accommodate these pressures. Otherwise, compaction close to the walls may need to be accomplished using hand-operated vibratory plate compactors.

5.7.2 Lateral Resistance

Lateral loads (seismic, soil, etc.) should be resisted by frictional forces developed between the base of the wall and the underlying soil. The total frictional resistance between the wall and subgrade soils is the normal force times the coefficient of friction between the layers. We recommend a nominal value of 0.35 for the coefficient of friction for a gabion or MSE walls founded on approved subgrade. A resistance factor of 0.9 should be used to calculate the factored sliding resistance of the wall. The normal force is the sum of the vertical forces (dead load plus real live load). Lateral resistance from passive earth pressures in front of the wall should be neglected.

5.8 Retaining Wall Drainage

The lateral earth pressure criteria presented above are appropriate if the retaining wall is fully drained. We recommend the installation of a permanent drainage system behind the retaining wall. The drainage system for a gabion basket wall typically consists of a drainage blanket located immediately behind the gabion blocks. The drainage blanket should have a minimum width of 12 inches and consist of crushed drain rock that contains less than 2% fines content (washed analysis). The drainage blanket should extend to the base of the wall, where water should be collected in a perforated pipe and discharged to a suitable outlet, such as an approved storm drain. In addition, the wall design should include positive drainage measures to prevent ponding of surface water behind the top of the wall.

5.9 Retaining Wall Global Stability

The global stability of the retaining wall was evaluated for static, seismic, and post-seismic conditions with the aid of the software program Slide2, developed by Rocscience, Inc., of Toronto, Canada. The global stability of the retaining wall was evaluated using Spencer's and Morgenstern–Price's method of slices, which satisfies both force and moment equilibrium. The output of the analysis is a factor of safety, which is defined as the ratio of the forces and moments resisting movement to the forces and moments driving movement of the soil mass. This calculated result is used as a relative measure of the overall stability of the slope. As the factor of safety decreases and approaches 1.0, the relative stability of the slope is considered to decrease. A factor of safety less than 1.0 infers the soil mass is not in equilibrium and movement is likely to occur.

The model was developed based on concept typical sections and wall geometry information provided by KPFF, publicly available topographic data, and our subsurface explorations. We developed a cross-section for the analysis based on the location where the wall is expected to be tallest. The selected cross-section (A-A') is identified on Figure 2. The soil parameters used in the analyses were determined based on the laboratory test results, our experience with similar soils, and our liquefaction/cyclic softening analysis. The retaining wall was modeled using a unit weight of 135 pcf, an embedment depth of 2 feet, a width of approximately 8 feet, and infinite strength to drive the failure surface below the base of the wall. For our analysis, we assumed a horizontal pseudo-static coefficient of 0.10, which is equal to about one-half of the site-adjusted PGA (A_s) and a groundwater depth of 3.5 feet below assumed finish site grades in front of the wall. A uniform, distributed load of 250 psf was used to model live traffic loads behind the wall for the static case. The assumed model geometry, subsurface stratigraphy, and material parameters used in our analysis are shown on the Global Stability Section A-A' Static Case, Figure 4; Global Stability Section A-A' Seismic Case, Figure 5; and Global Stability Section A-A' Post-Seismic Case, Figure 6.

In accordance with the AASHTO LRFD and the ODOT *Geotechnical Design Manual*, maximum global static and seismic/post-seismic resistance factors of 0.67 and 0.90 (equivalent to factors of safety of 1.5 and 1.1), respectively, are required for new retaining walls. The static factor of safety required depends on the certainty of the geotechnical parameters and the improvements being supported by the retaining wall. Our analyses indicate the static and seismic global factors of safety for the new retaining wall are at least 1.5 and 1.1, respectively.

5.10 Micropile Foundation Support of Elevated Sidewalks

5.10.1 General Micropile Design Recommendations

Micropiles consist of small-diameter (commonly 4 inches to 10 inches), drilled-and-grouted, steel-reinforced elements. Micropiles are typically constructed by drilling a cased hole to the desired depth into a bearing layer, placing a steel bar or steel strands in the cased hole, filling the cased hole with grout from the bottom using tremie methods, and then withdrawing the casing from the hole to form a bond zone. A permanent section of steel casing remains in the drilled hole to provide structural rigidity within the upper portion of the pile. Provisions for pressure grouting and/or secondary grouting can be incorporated into the design to enhance the geotechnical capacity of the bond zone, and it should be anticipated that at least one episode of secondary grouting or pressure grouting may be required to achieve the desired bond-zone capacities, depending on the specific design and subsurface conditions.

The specific design, installation, and testing procedures for micropiles are typically developed by a specialty foundation contractor based on subsurface information and performance criteria provided by the owner's geotechnical and structural engineers. We recommend micropiles be designed, installed, and tested in accordance with the current Federal Highway Administration (FHWA) publication FHWA-NHI-05-039, *Micropile Design and Construction*. All micropiles should be equipped with appropriate corrosion protection measures.

Compressive and uplift resistances of the micropiles are developed by the grout-to-ground bond strength, α_b , in the soil layers below the depth of permanent casing. The pile resistance and associated bond-zone lengths depend on the types of soil and the specialty contractor's construction methods for pile installation. Therefore, final design and grouting techniques will be developed by the contractor based on performance requirements established by the design team. Micropiles should typically be installed with a minimum center-to-center spacing of 3 feet to reduce the potential for group effects under axial loading.

For preliminary planning of pile lengths and cost estimating, we recommend assuming the nominal (i.e., ultimate) grout-to-ground bond strengths provided in Table 2, below.

Table 5-2: PRELIMINARY GROUT-TO-GROUND BOND STRENGTHS AT BENT 3

Depth Below Roadway Surface, feet	Nominal Bond Strength, psi
0 to 12	N/A (unsupported)
12 to 21	6
21 to 24	15
> 24	10

Abbreviations: psi = pounds per square foot; N/A = not applicable

The recommended bond strengths are based on Type A (gravity-grouted) micropiles. We anticipate a minimum permanent casing depth of at least 15 feet below lowest adjacent grade will be included to extend the casing through existing fill and for lateral stiffness. The bond zone should be assumed to start below the depth of permanent casing and shall also be a minimum of 15 feet below the permanent casing. Greater grout-to-ground bond strength will likely be possible for piles that include secondary grouting or pressure grouting. Therefore, actual bond strengths and geotechnical resistance and overall micropile lengths should be confirmed with testing. Compressive and uplift resistances may also be limited by the structural capacity of the micropiles, which we assume will be checked by the structural designer.

Prior to installation of production micropiles, a sacrificial verification test pile should be installed near the production pile locations. The verification load test should be performed to confirm the design capacities of the micropiles and the optimum length of the bond zone. Successful verification test piles may be incorporated into the foundation; however, unsuccessful tests will require installation of an additional micropile. The verification test pile should be tested to a minimum of 200% of the maximum allowable design capacity (in uplift or compression) in accordance with ASTM D3689. We also recommend proof testing about 10% of the production piles to 150% of the design load using allowable stress design. The proof test for the compression loads may be performed in tension to facilitate the testing. A successful verification or proof test will typically sustain the test load for at least 10 minutes with less than 0.04 inches of creep movement. If this creep limit is exceeded, the test shall be extended to 60 minutes and creep limited to 0.08 inches. In addition, the measured deflection at the design load (i.e., allowable or ultimate load, depending on loading condition) needs to be less than the maximum allowable deflection specified by the structural engineer.

5.10.2 Micropile Lateral Capacity

Lateral resistance for plumb (i.e., vertical) micropiles may be evaluated using the computer software LPILE developed by Ensoft, Inc., of Austin, Texas. However, due to the relatively small diameter of micropiles, lateral capacity is often limited. For additional lateral

resistance, battered micropiles may be evaluated, which can provide lateral resistance developed from axial compression or tension.

If lateral resistance is evaluated using LPILE or similar software, we recommend pile lateral resistance be evaluated assuming a groundwater depth of approximately 10 feet. For preliminary analysis, the recommended LPILE parameters for lateral pile design are summarized in Table 5-3, below.

Table 5-3: SOIL PROPERTIES FOR LPILE ANALYSIS

Soil Unit	Depth, feet	LPILE Soil Type	Soil Properties				
			K, pci	γ' , pcf	ϕ'	c, psf	ϵ_{50}
Unsupported	0 to 12	--	--	--	--	--	--
Soft silt and silty sand	12 to 21	API Sand	20	48	26	--	--
Dense gravel and silt	21 to 24	API Sand	125	68	38	--	--
Stiff to hard clay	>24	Stiff clay without free water	--	63	--	3,500	0.005

Abbreviations: pci = pounds per cubic inch; pcf = pounds per cubic foot; psf = pounds per square foot; -- = no value

Note:

Groundwater with submerged conditions should be assumed below a depth of 10 feet.

LPILE provides isolated single-pile capacities. Depending on the direction of the loading and layout of the micropiles, group effects under lateral loading may need to be considered. Lateral group effects can be modeled in LPILE by applying an appropriate P-modifier. The P-modifier is a function of the center-to-center spacing, D, of the micropiles and is provided in Table 5-4, below.

Table 5-4: RECOMMENDED P-MODIFIERS FOR GROUP EFFECTS

Center-to-Center Pile Spacing	P-Modifiers for Rows 1, 2, and 3+
3D	0.8, 0.4, 0.3
4D	0.9, 0.6, 0.5
5D	1.0, 0.85, 0.7
6D	1.0, 1.0, 0.9

Abbreviation: D = diameter of pile

5.11 Construction-Induced Vibrations

Construction equipment, such as vibratory rollers, hydraulic breakers, and bulldozers, is capable of producing vibration amplitudes that could cause cosmetic or structural damage to the existing historic buildings at the site. Vibration levels should be controlled below

established threshold levels to mitigate the risk of vibration-induced damage to the structures. Section 6.2 of the 2020 Caltrans Vibrations Manual provides a summary of peak particle velocity (PPV) thresholds for different building types that have been developed by various sources. The 2004 AASHTO *Standard Recommended Practice for Evaluation of Transportation-Related Earthborne Vibrations* recommends a maximum allowable PPV of 0.12 inches per second to prevent damage to “historic sites or other critical locations” from intermittent construction or maintenance activities. Estimations of PPV from various construction sources can be made using typical source levels and a simple prediction equation provided in the 2006 Federal Transit Administration *Transit Noise and Vibration Impact Assessment* manual.

If construction equipment capable of producing vibrations in the range of the maximum PPV provided above is used on the project, we recommend that a vibration consultant be present to perform vibration monitoring during construction near the historic buildings at the site. Additionally, we recommend that pre- and post-construction damage survey assessments of each building be performed to document the existing and post-construction conditions of the structures.

6 DESIGN REVIEW AND CONSTRUCTION SERVICES

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. To observe compliance with the intent of our recommendations, the design concepts, and the plans and specifications, it is our opinion all construction operations dealing with earthwork, retaining walls, foundations, and pile installations should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in our 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 different from those described in this report.

7 LIMITATIONS

This report has been prepared to aid the project team in the design of this project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to earthwork and design and construction of the proposed improvements along E 1st Street. In the event that any changes in the design and location of the project elements as outlined in this report are planned, we should be given the opportunity to

review the changes and modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations in this report are based on the data obtained from the subsurface explorations at the locations shown on Figure 2 and 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 variations in subsurface conditions may exist between exploration locations. This report does not reflect variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions differ from those encountered in the explorations, we should be advised at once so we can observe and review these conditions and reconsider our recommendations where necessary.

We have included the Geoprofessional Business Association guidance document "Important Information about This Geotechnical-Engineering Report" to assist you and others in understanding the use and limitations of this report, included as Appendix B. We recommend you read this document.

Submitted for GRI,



RENEWS: 12/2025
Christopher K. Ell, PE, GE
Principal

Per T. Onsager, PE
Project Engineer

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 Association of State Transportation Officials (AASHTO), 2004, Standard recommended practice for evaluation of transportation-related earthborne vibrations: AASHTO Designation: R 8-96, Washington, D.C.

AASHTO, 2011, Guide specification for LRFD seismic bridge design, Second Edition.

AASHTO, 2014, LRFD bridge design specifications, 7th Edition.

ASTM International (ASTM), 2021, ASTM D698: Standard test methods for laboratory compaction characteristics of soil using standard effort.

ASTM, 2022, ASTM D3689: Standard test methods for deep foundation elements under static axial tensile load.

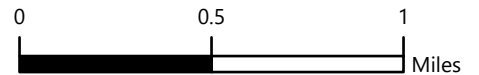
Federal Transit Administration, 2006, Transit noise and vibration impact assessment: Washington, D.C.

Oregon Department of Transportation (ODOT), 2018, Geotechnical design manual:

U.S. Department of Transportation Federal Highway Administration, 2005, NHI Course No. 132078: Micropile Design and Construction: Publication No. NHWA NHI-05-039,



BASEMAP PROVIDED BY ESRI, 2023





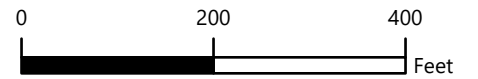
KPFF CONSULTING ENGINEERS
 1ST STREET - UNION STREET TO
 WASHINGTON STREET THE DALLES, OREGON

VICINITY MAP



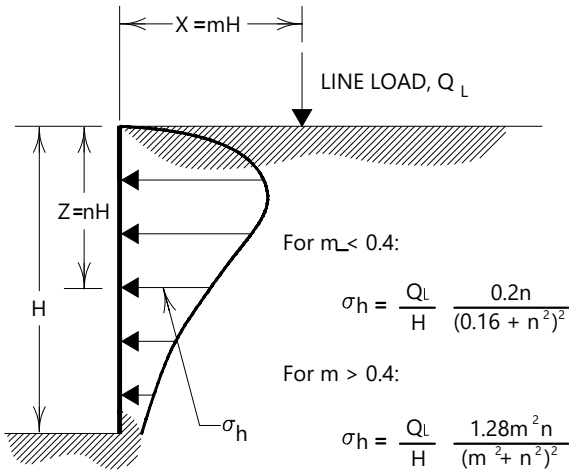
BASEMAP PROVIDED BY GOOGLE EARTH, 2020

-  BORING COMPLETED BY GRI
-  APPROXIMATE EXTENTS OF EXISTING ELEVATED SIDEWALK

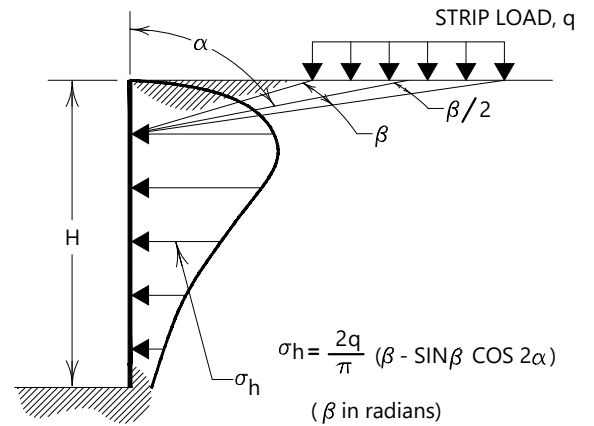


KPFF CONSULTING ENGINEERS
 1ST STREET - UNION TO WASHINGTON
 STREET THE DALLES, OREGON

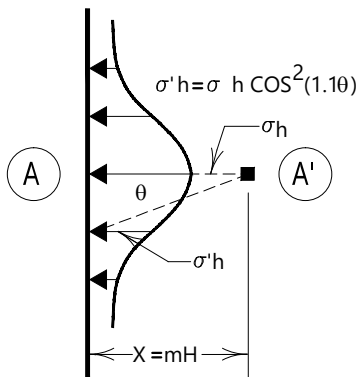
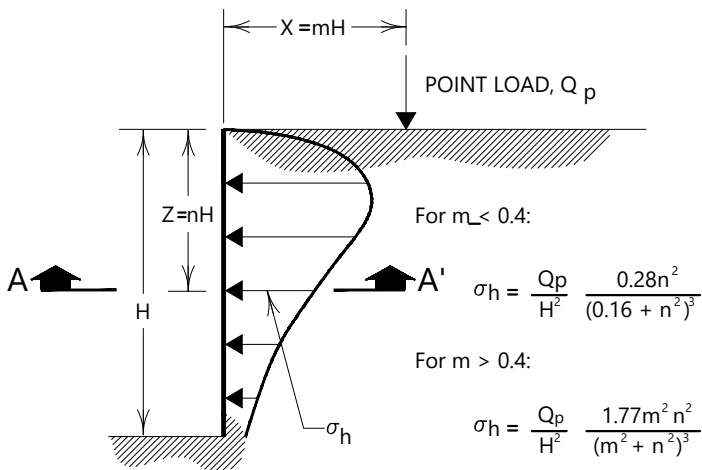
SITE PLAN



LINE LOAD PARALLEL TO WALL



STRIP LOAD PARALLEL TO WALL

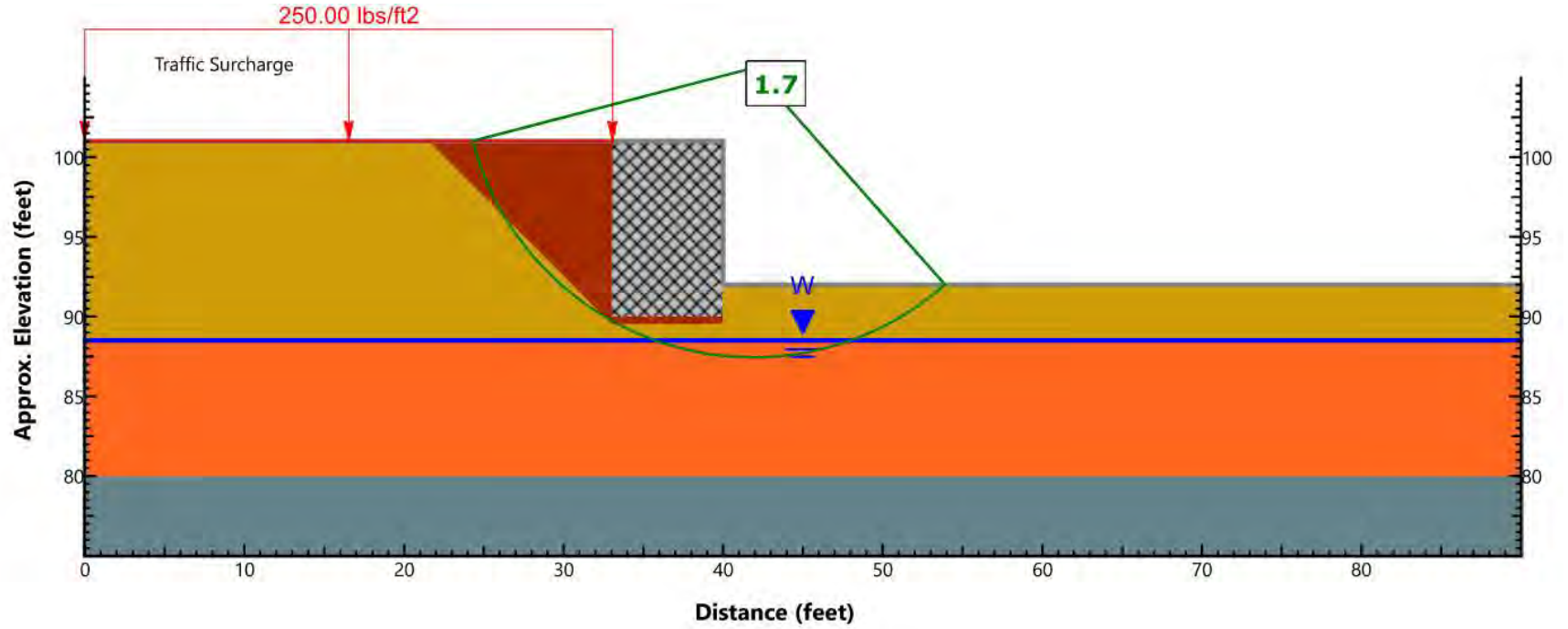


DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

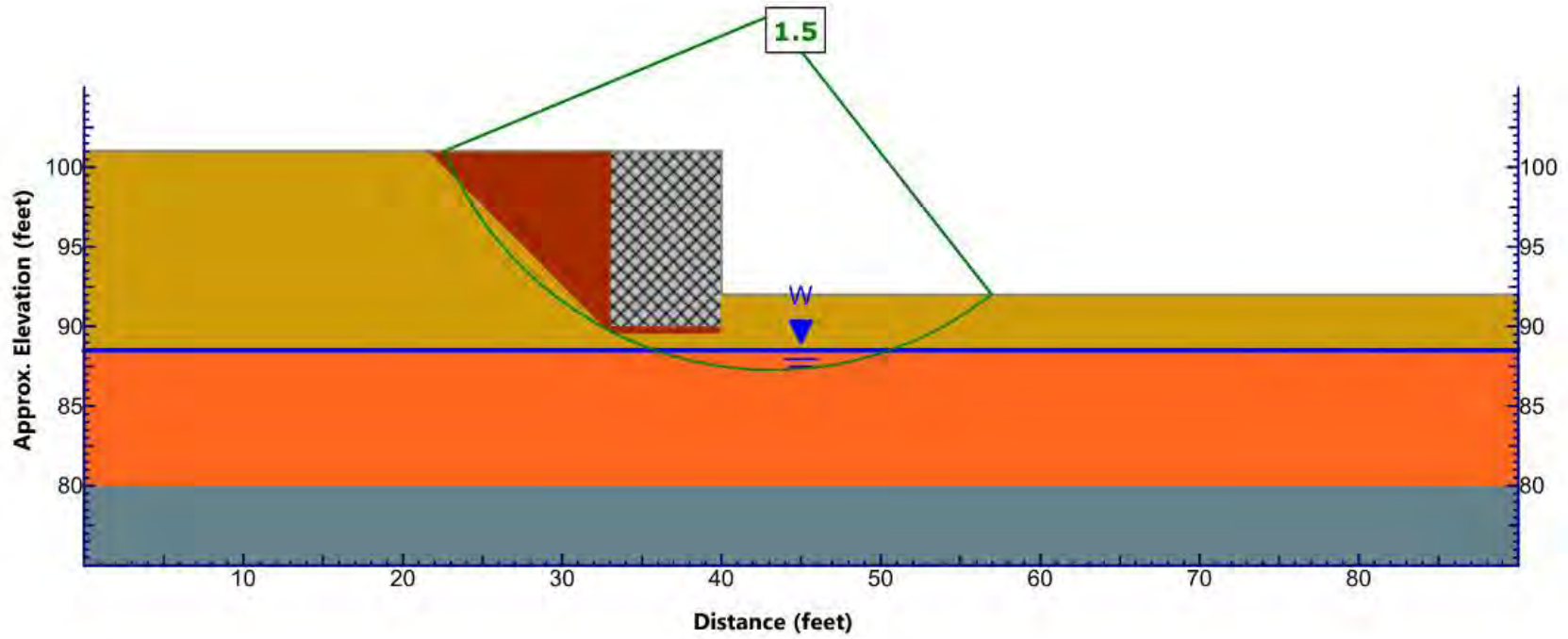
1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.



Material Name	Color	Unit Weight (lbs/ft ³)	Strength Type	Cohesion (psf)	Phi (deg)
Gabion Wall		135	Infinite strength		
Existing Fill		125	Mohr-Coulomb	0	30
SILT and Silty SAND		115	Mohr-Coulomb	0	28
Gravel		125	Mohr-Coulomb	0	35
Proposed Fill		130	Mohr-Coulomb	0	34

GRI KPFF CONSULTING ENGINEERS
 1ST STREET – UNION TO WASHINGTON STREET
 THE DALLES, OREGON

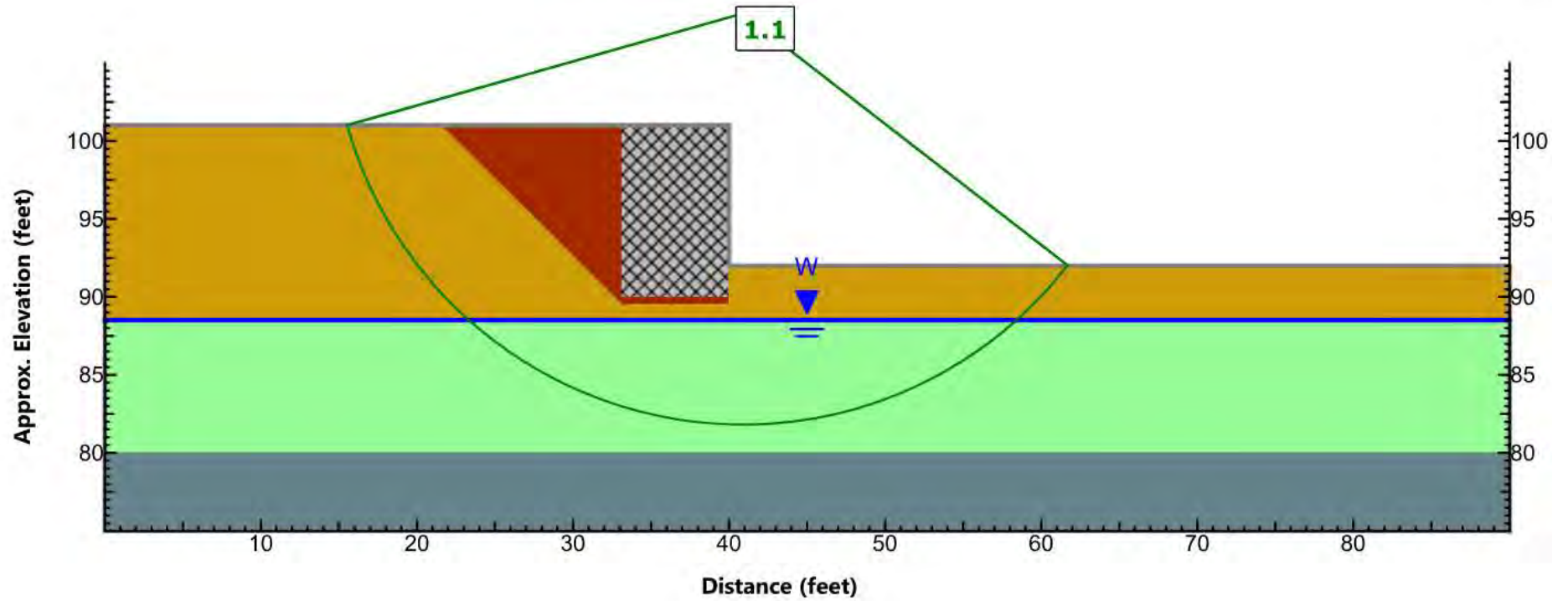
GLOBAL STABILITY
 SECTION A-A'
 STATIC CASE



Material Name	Color	Unit Weight (lbs/ft ³)	Strength Type	Cohesion (psf)	Phi (deg)
Gabion Wall		135	Infinite strength		
Existing Fill		125	Mohr-Coulomb	0	30
SILT and Silty SAND		115	Mohr-Coulomb	0	28
Gravel		125	Mohr-Coulomb	0	35
Proposed Fill		130	Mohr-Coulomb	0	34

GRI KPFF CONSULTING ENGINEERS
 1ST STREET – UNION TO WASHINGTON STREET
 THE DALLES, OREGON

GLOBAL STABILITY
 SECTION A-A'
 SEISMIC CASE



Material Name	Color	Unit Weight (lbs/ft ³)	Strength Type	Cohesion (psf)	Phi (deg)
Gabion Wall		135	Infinite strength		
Existing Fill		125	Mohr-Coulomb	0	30
Gravel		125	Mohr-Coulomb	0	35
Proposed Fill		130	Mohr-Coulomb	0	34
SILT and Silty SAND (Liq.)		115	Undrained	200	

GRI KPFF CONSULTING ENGINEERS
 1ST STREET – UNION TO WASHINGTON STREET
 THE DALLES, OREGON

GLOBAL STABILITY
 SECTION A-A'
 POST-SEISMIC CASE

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

A.1 FIELD EXPLORATIONS

A.1.1 General

Subsurface materials and characteristics at the site were investigated by GRI on May 25 and 26, 2023. The approximate locations of the explorations completed for this investigation are shown on the Site Plan, Figure 2. An experienced member of the GRI engineering staff directed the explorations and maintained a log of the materials and conditions disclosed during the course of the work. The explorations completed are further discussed in the following sections.

A.1.2 Machine-Drilled Borings

Four machine-drilled borings, designated B-1 through B-4, were drilled using mud rotary drilling methods and a truck-mounted CME 75 drill rig operated by Western States Soil Conservation, Inc. of Hubbard, Oregon. Boring B-1 was advanced to a depth of 31.5 feet, boring B-2 to a depth of 29.0 feet, boring B-3 to 27.9 feet, and boring B-4 to a depth of 14.0 feet.

Disturbed soil samples were obtained from the boring at about 2.5-foot intervals. Disturbed soil samples were obtained using a 2-inch outside-diameter (O.D.) split-spoon sampler. Standard Penetration Tests (SPT) were 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 split-spoon sampler the last 12 inches is known as the Standard Penetration Resistance (SPT N-value). The SPT N-value provides a measure of relative density of granular soils and the relative consistency of cohesive soils. The split-spoon samples were examined in the field, and portions were saved in airtight jars or plastic bags. All samples were returned to our laboratory for further examination and physical testing.

Relatively undisturbed samples of fine-grained soil were obtained by pushing 3-inch O.D. Shelby tubes into the undisturbed soil a maximum distance of 24 inches using the drill rig. The soils exposed in the ends of the Shelby tube were examined and classified in the field. After classification, the tube was sealed with rubber caps and tape to preserve the natural moisture contents of the soils. The sample was returned to our laboratory for further examination and testing.

Logs of the borings are provided on Figures 1A through 4A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depths at which the materials and/or characteristics of the materials change. To the right of the

descriptive summary, the numbers and types of samples are indicated. Farther to the right, SPT N-values are shown graphically, along with the natural moisture contents, Atterberg limits, and percents passing the No. 200 sieve, where applicable. The terms and symbols used to describe the materials encountered in the borings are defined in Table 1A and the attached legend.

A.2 LABORATORY TESTING

A.2.1 General

The samples obtained from the borings were examined in our laboratory, where the physical characteristics of the samples were noted and the field classifications modified where necessary. At the time of classification, the natural moisture content of each sample was determined. Additional testing included consolidation testing, grain-size analyses, organic content tests, and Atterberg limits tests. The following sections 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 included on the borings logs, Figures 1A through 4A and in the Summary of Laboratory Results, Table 2A.

A.2.3 Field Vane Shear Strength

The approximate undrained shear strength of some fine-grained soil samples was estimated using a field vane shear device on the Shelby tube samples. The field vane 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 field vane shear-strength tests are summarized on Figures 1A through 3A.

A.2.4 Atterberg Limits

Atterberg limits testing was conducted on select soil samples in conformance with ASTM D4318. The test results are summarized on the boring logs, Figures 1A through 4A, on the Plasticity Chart, Figures 5A, and in Table 2A.

A.2.5 Washed-Sieve Grain-Size Analysis

To assist in classification of the soils, samples of known dry weight were washed over a No. 200 sieve. The material retained on the sieve was oven-dried and weighed. The percentage of material passing the No. 200 sieve was then calculated. The results are summarized on the boring logs, Figures 1A through 4A, and in Table 2A.

A.2.6 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 4A, where applicable, and in Table 2A.

A.2.7 One-Dimensional Consolidation

One-dimensional consolidation testing was performed in accordance with ASTM D2435 on relatively undisturbed soil samples obtained from borings B-2 and B-3 at a depth of about 16 feet. The test provides data on the compressibility of the underlying fine-grained soils. Test results are summarized on Figure 6A in the form of a curve showing effective stress versus percent strain. The initial dry unit weights and moisture contents of the samples are also shown on the figures.

A.3 REFERENCES

ASTM International (ASTM), 2011, ASTM D2435: Standard test methods of one-dimensional consolidation properties of soils using incremental loading.

ASTM, 2017, ASTM D2216: Standard test methods for laboratory determination of water (moisture) content of soil and rock by mass.

ASTM, 2018, ASTM D4318: Standard test methods of liquid limit, plastic limit, and plasticity index of soils.

ASTM, 2024, ASTM D2937: Standard test method for density of soil in place by the drive-cylinder method.

Table 1A

GUIDELINES FOR DESCRIPTION OF SOIL¹

Description of Relative Density for Cohesionless (Coarse-Grained) Soils

Relative Density	Standard Penetration Resistance (N-values) blows/foot (ft)	3-inch Sampler, 140-lb hammer approx. N-Value (blows/ft) ²	3-inch Sampler, 300-lb hammer approx. N-Value (blows/ft) ¹
Very Loose	0 - 4	0 – 10	0 – 5
Loose	4 - 10	10 – 24	5 – 11
Medium Dense	10 - 30	24 – 73	11 – 34
Dense	30 - 50	73 – 122	34 – 57
Very Dense	over 50	over 122	over 57

Description of Relative Consistency for Cohesive (Fine-Grained) Soils

Relative Consistency	Standard Penetration Resistance (N-values) blows/ft	3-inch Sampler, 140 lb hammer approx. N-Value (blows/ft) ¹	3-inch Sampler, 300 lb hammer approx. N-Value (blows/ft) ²	Torvane or Undrained Shear Strength, tsf
Very Soft	0 - 2	0 – 3	0 – 1	less than 0.125
Soft	2 - 4	3 – 6	1 – 3	0.125 - 0.25
Medium Stiff	4 - 8	6 – 12	3 – 6	0.25 - 0.50
Stiff	8 - 15	12 – 23	6 – 11	0.50 - 1.0
Very Stiff	15 - 30	23 – 46	11 – 22	1.0 - 2.0
Hard	30 – 60	46 – 92	22 – 43	over 2.0
Very Hard	over 60	over 92	over 43	

Grain-Size Classification	Modifier for Subclassification		
<i>Boulders:</i> > 12 inches		Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
<i>Cobbles:</i> 3 inches - 12 inches	Adjective	Percentage of Other Material (By Weight)	
<i>Gravel:</i> ¼ inch - ¾ inch (fine) ¾ inch - 3 inches (coarse)	trace: some: sandy, gravelly:	<15 (sand, gravel) 15 - 30 (sand, gravel) 30 - 50 (sand, gravel)	<15 (sand, gravel) 15 - 30 (sand, gravel) 30 - 50 (sand, gravel)
<i>Sand:</i> No. 200 - No. 40 sieve (fine) No. 40 - No. 10 sieve (medium) No. 10 - No. 4 sieve (coarse)	trace: some: silty, clayey:	<5 (silt, clay) 5 - 12 (silt, clay) 12 - 50 (silt, clay)	<i>Relationship of clay and silt determined by plasticity index test</i>
<i>Silt/Clay:</i> Pass No. 200 sieve			

1. Soil descriptions are developed using visual-manual procedures (ASTM D2488) and generally follow ODOT Geotechnical Design Manual (Chapter 5) guidelines.
2. Oversized sampler (OD = 3 inches, ID = 2.4 inches) blow counts converted to SPT N-Value using equations provided by Burmister, D.M., 1948, The importance and practical use of relative density in soil mechanics: Proceedings of ASTM, v. 48:1249.

Table 2A
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-1	2.5	--	25	--	--	--	--	FILL	
	S-2	5.0	--	16	--	--	--	--	FILL	
	S-3	7.5	--	32	--	--	--	--	FILL	
	S-4	10.0	--	4	--	--	--	--	FILL	
	S-5	12.5	--	37	--	--	--	68	SILT	
	S-6	15.0	--	40	--	--	--	--	SILT	
	S-7	16.5	--	38	82	--	--	85	SILT	
	S-8	18.5	--	14	--	--	--	--	Silty GRAVEL	
	S-9	20.0	--	14	--	--	--	--	Silty GRAVEL	
	S-11	25.0	--	31	--	--	--	--	Clayey SILT	
	S-12	27.5	--	35	--	--	--	--	Clayey SILT	
	S-13	30.0	--	65	--	--	--	--	Clayey SILT	
	B-2	S-1	2.5	--	15	--	--	--	--	FILL
S-2		5.0	--	9	--	--	--	--	FILL	
S-4		10.0	--	33	--	--	--	--	FILL	
S-5		12.5	--	42	--	--	--	--	SILT	
S-7		17.0	--	28	--	--	--	31	Silty SAND	
S-8		20.0	--	27	--	--	--	--	Silty SAND	
S-9		22.5	--	15	--	--	--	--	GRAVEL	
S-10		25.0	--	31	--	60	25	--	Clayey SILT	
S-11		27.5	--	25	--	--	--	--	Clayey SILT	
B-3		S-2	5.0	--	25	--	--	--	--	FILL
		S-3	7.5	--	3	--	--	--	--	FILL
	S-4	10.0	--	24	--	--	--	--	FILL	
	S-5	12.5	--	42	--	--	--	--	SILT	
	S-6	16.0	--	40	--	35	7	--	SILT	
	S-7	17.0	--	43	--	--	--	84	SILT	
	S-8	20.0	--	45	--	--	--	--	CLAY	
	S-11	27.5	--	38	--	--	--	--	Clayey SILT	
B-4	S-2	5.0	--	25	--	--	--	--	Silty GRAVEL	

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

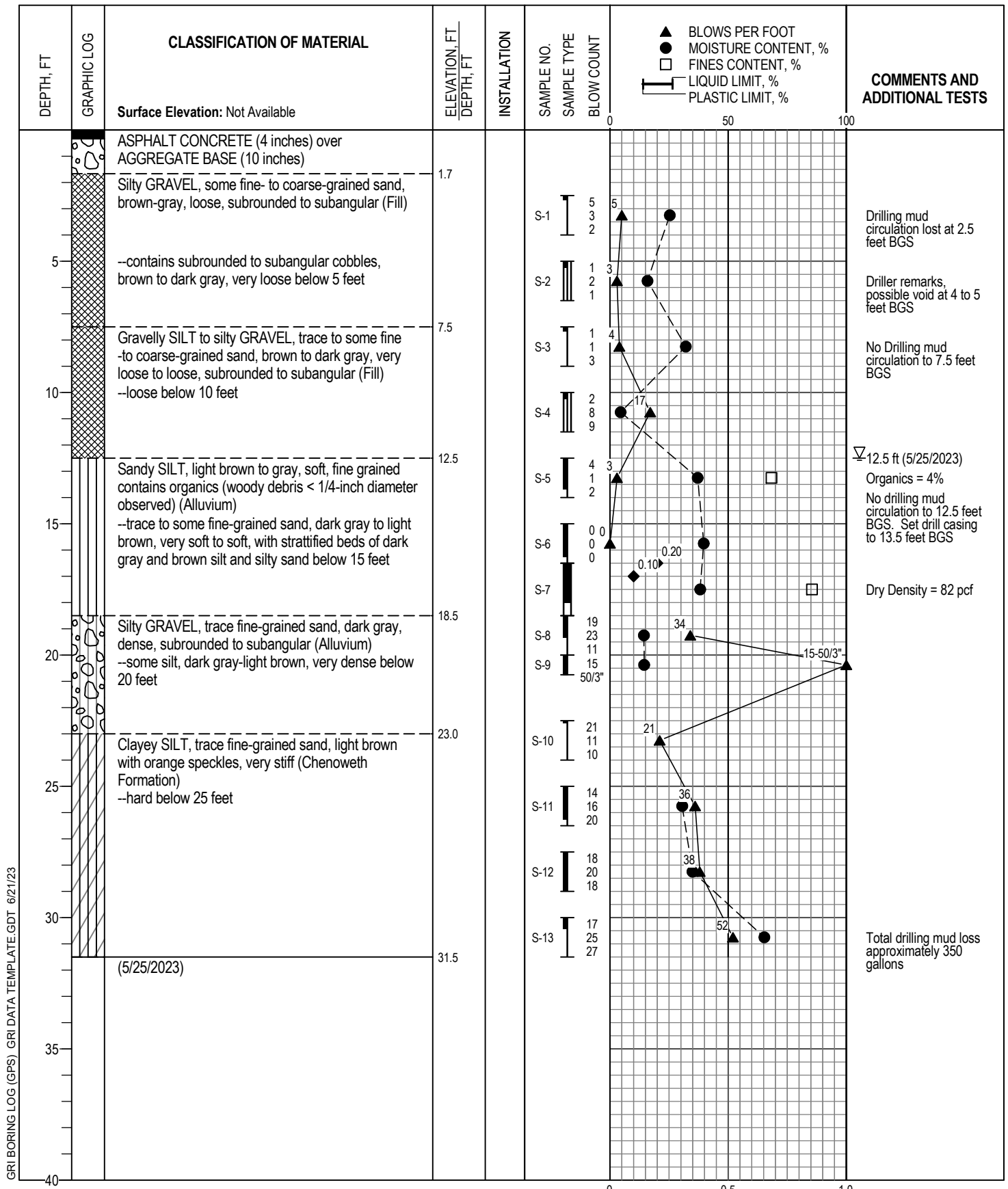
Symbol	Sampler Description
	2.0 in. O.D. split-spoon sampler and Standard Penetration Test with recovery (ASTM D1586)
	Shelby tube sampler with recovery (ASTM D1587)
	3.0 in. O.D. split-spoon sampler with recovery (ASTM D3550)
	Grab Sample
	Rock core sample interval
	Sonic core sample interval
	Push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

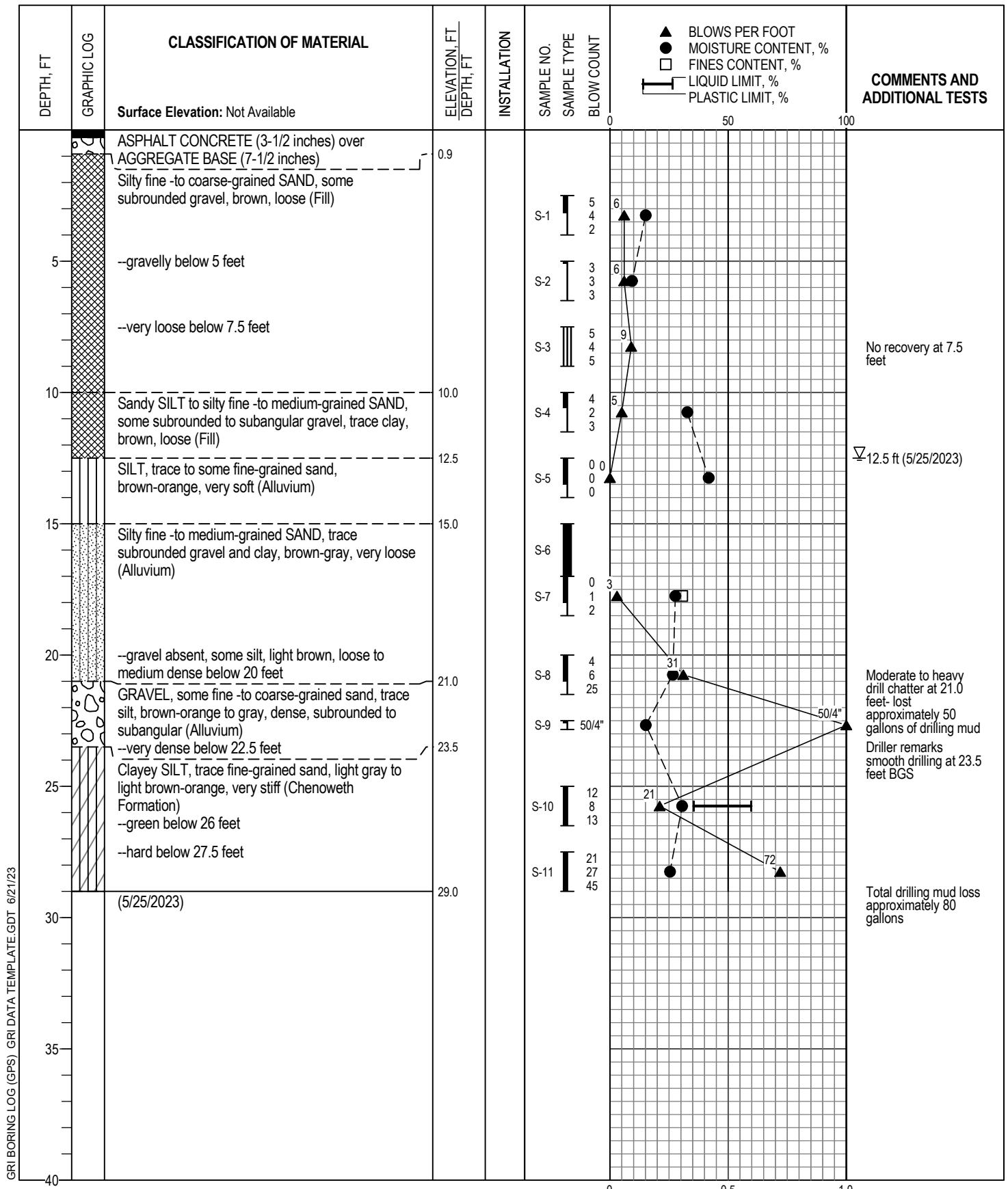
Symbol	Typical Description
	Groundwater level during drilling and date measured
	Groundwater level after drilling and date measured
	Rock/sonic core or push probe recovery (%)
	Rock quality designation (RQD, %)



GRI BORING LOG (GPS), GRI DATA TEMPLATE.GDT 6/21/23

Logged By: J. Heidgerken		Drilled by: Western States Soil Conservation, Inc.	
Date Started: 5/25/23	GPS Coordinates: 45.602841° N 121.183112° W (WGS 84)		
Drilling Method: Mud Rotary	Equipment: CME 75 HT Truck-Mounted Drill Rig		Hammer Type: Auto Hammer
Hole Diameter: 5 in.			Weight: 140 lb Drop: 30 in.
Note: See Legend for Explanation of Symbols	Energy Ratio: 0.86		

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



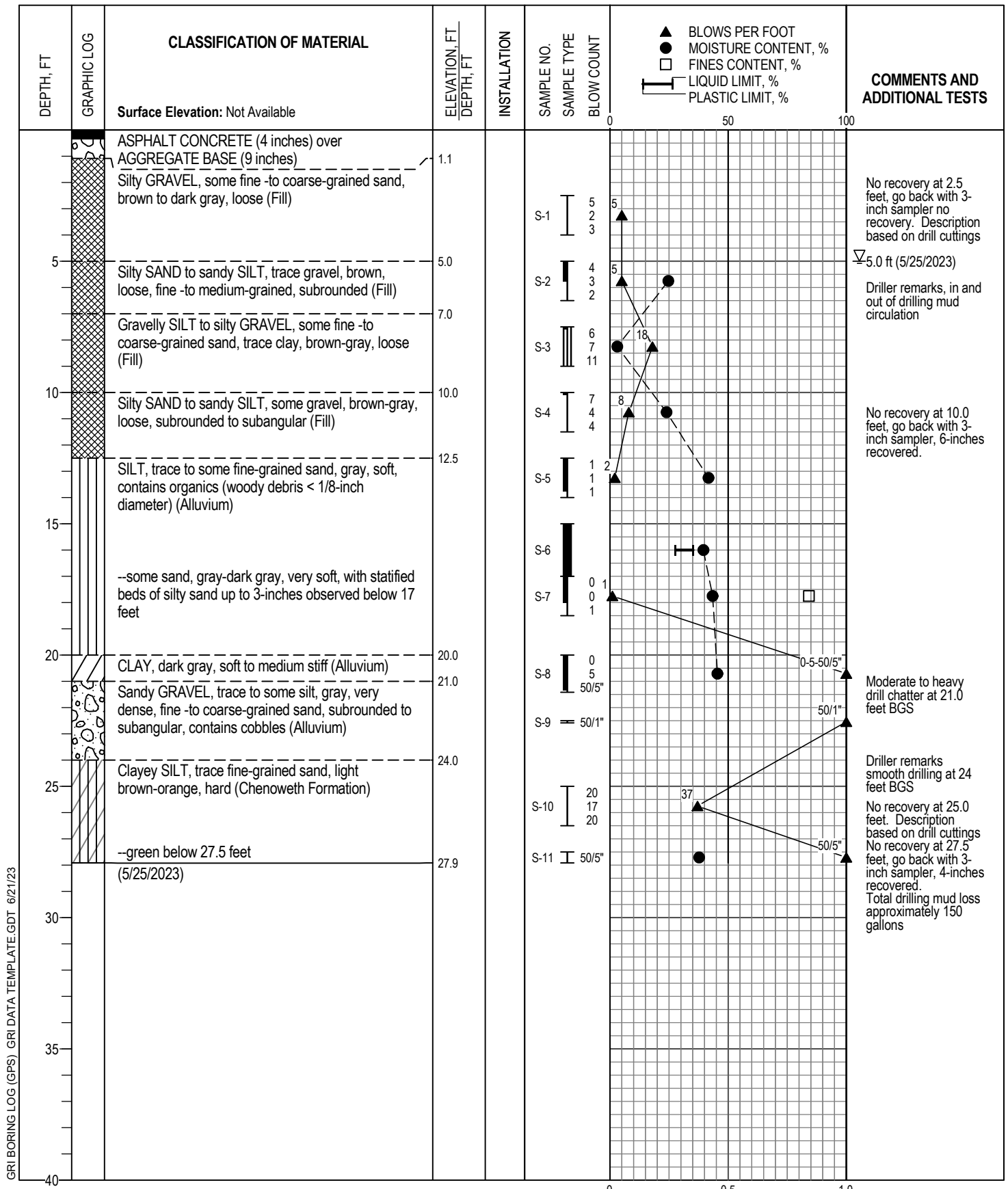
GRI BORING LOG (GPS), GRI DATA TEMPLATE.GDT 6/21/23

Logged By: J. Heidgerken	Drilled by: Western States Soil Conservation, Inc.
Date Started: 5/25/23	GPS Coordinates: 45.602693° N 121.182762° W (WGS 84)
Drilling Method: Mud Rotary	Hammer Type: Auto Hammer
Equipment: CME 75 HT Truck-Mounted Drill Rig	Weight: 140 lb
Hole Diameter: 5 in.	Drop: 30 in.
Note: See Legend for Explanation of Symbols	Energy Ratio: 0.86

- ◆ TORVANE SHEAR STRENGTH, TSF
- UNDRAINED SHEAR STRENGTH, TSF



BORING B-2

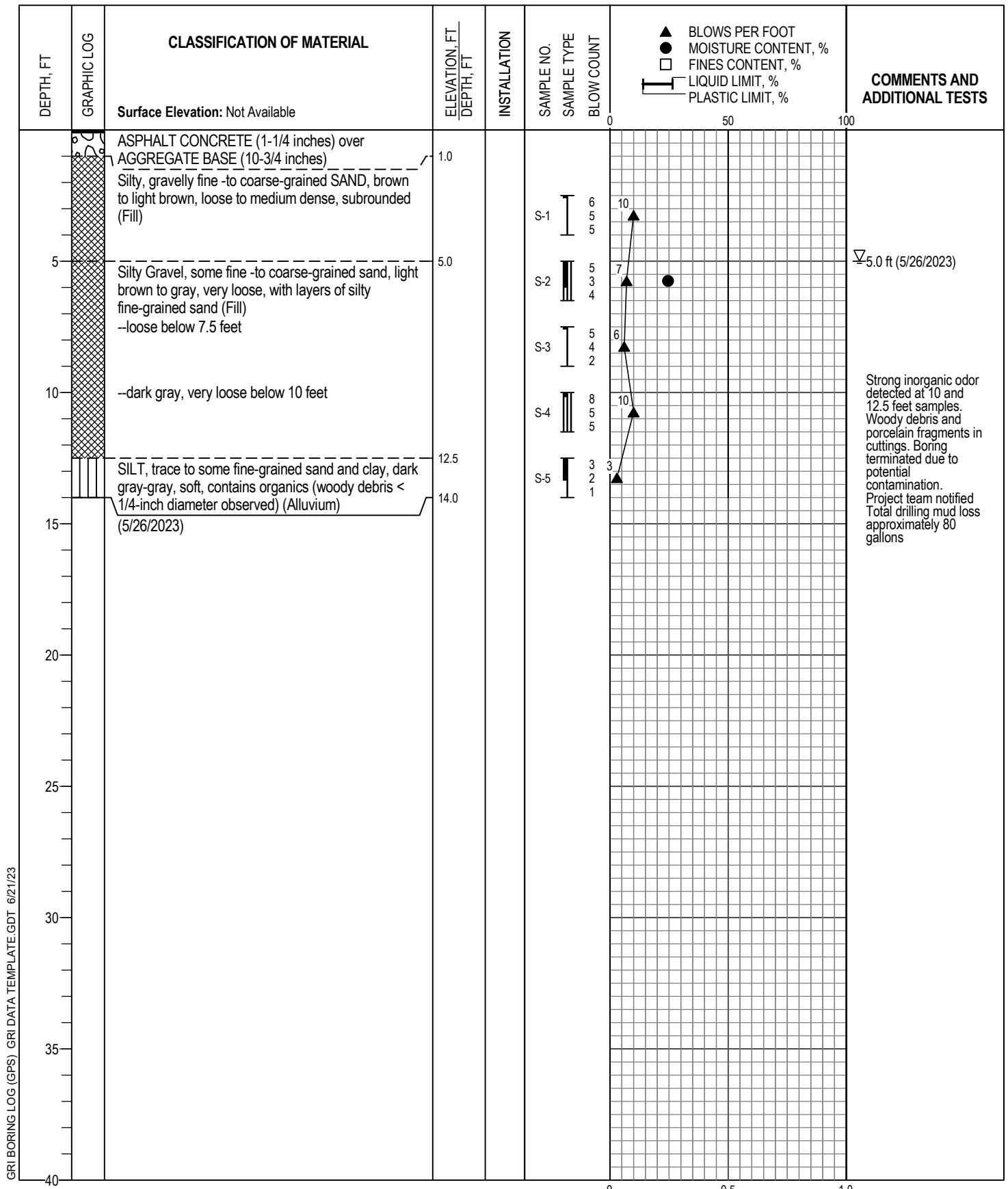


GRI BORING LOG (GPS), GRI DATA TEMPLATE.GDT 6/21/23

Logged By: J. Heidgerken	Drilled by: Western States Soil Conservation, Inc.
Date Started: 5/25/23	GPS Coordinates: 45.602314° N 121.181954° W (WGS 84)
Drilling Method: Mud Rotary	Hammer Type: Auto Hammer
Equipment: CME 75 HT Truck-Mounted Drill Rig	Weight: 140 lb
Hole Diameter: 5 in.	Drop: 30 in.
Note: See Legend for Explanation of Symbols	Energy Ratio: 0.86

- ◆ TORVANE SHEAR STRENGTH, TSF
- UNDRAINED SHEAR STRENGTH, TSF

GRI BORING B-3



GRI BORING LOG (GPS), GRI DATA TEMPLATE.GDT 6/21/23

Logged By: J. Heidgerken		Drilled by: Western States Soil Conservation, Inc.	
Date Started: 5/26/23	GPS Coordinates: 45.602118° N 121.181547° W (WGS 84)		
Drilling Method: Mud Rotary	Equipment: CME 75 HT Truck-Mounted Drill Rig		Hammer Type: Auto Hammer
Hole Diameter: 5 in.			Weight: 140 lb Drop: 30 in.
Note: See Legend for Explanation of Symbols	Energy Ratio: 0.86		

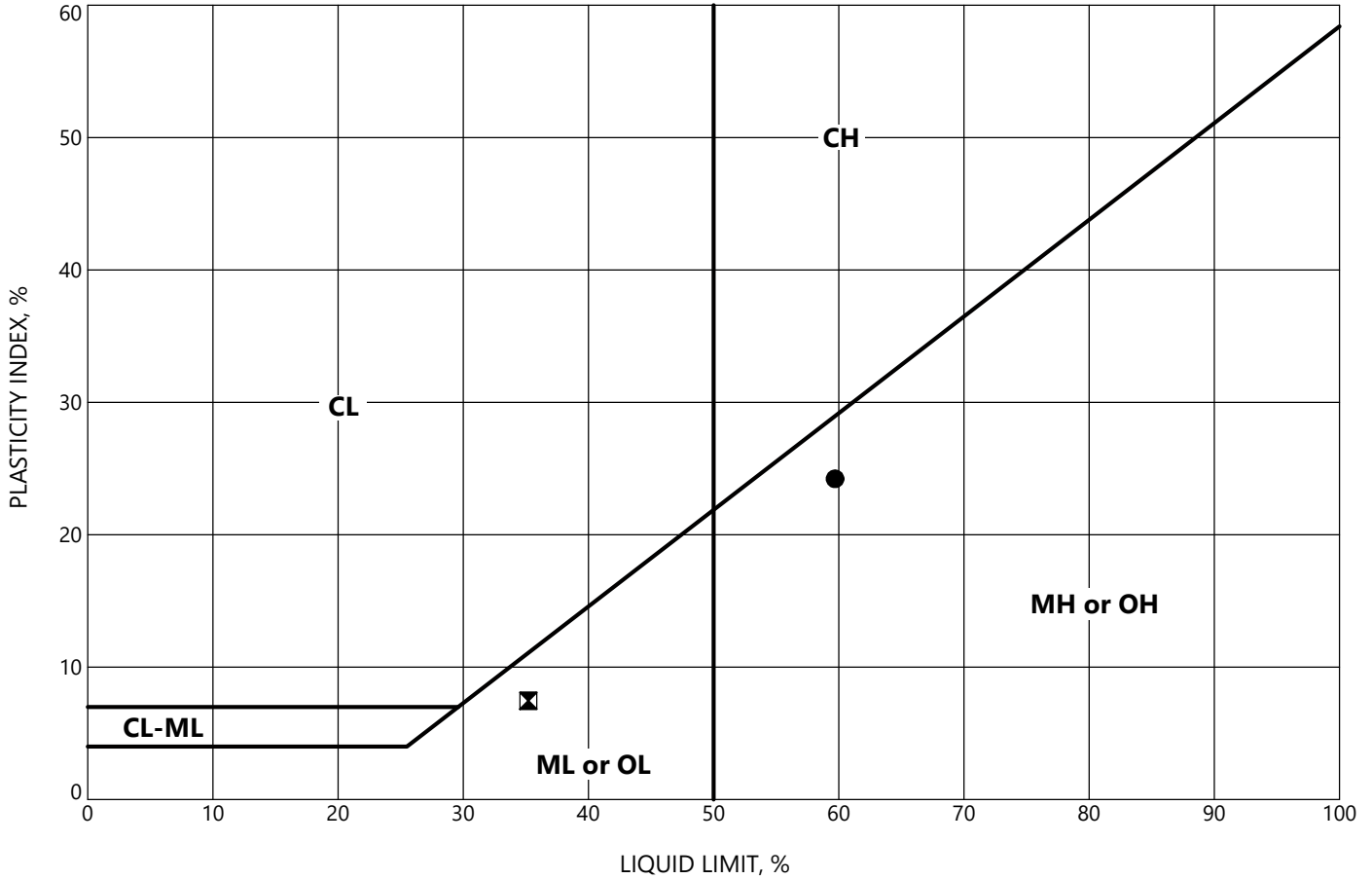
- ◆ TORVANE SHEAR STRENGTH, TSF
- UNDRAINED SHEAR STRENGTH, TSF



BORING B-4

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

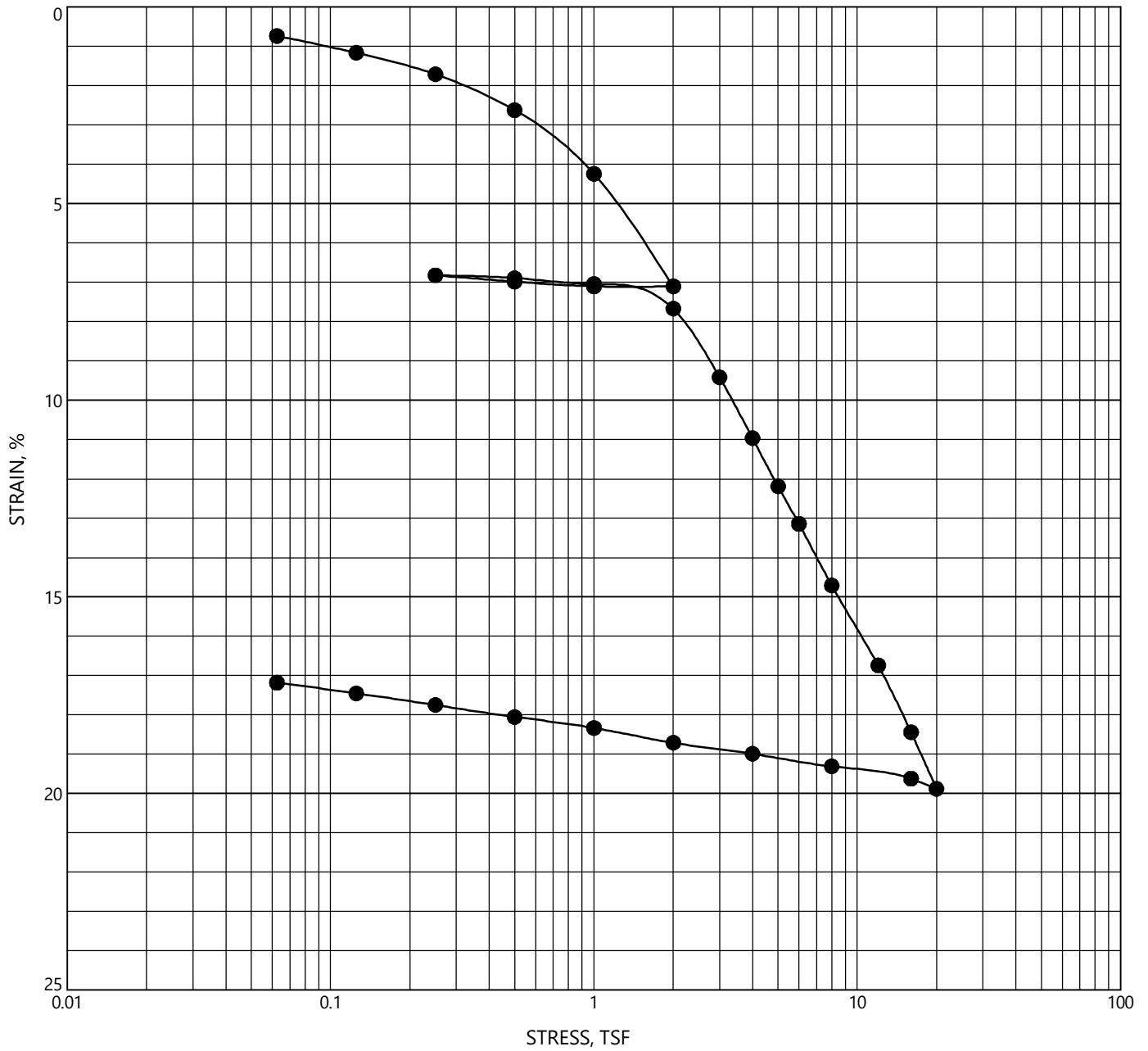
GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	B-2	S-10	25.0	clayey SILT, trace fine-grained sand, light gray to light brown-orange	60	35	25	31
■	B-3	S-6	16.0	SILT, trace clay and fine-grained sand, gray	35	28	7	40



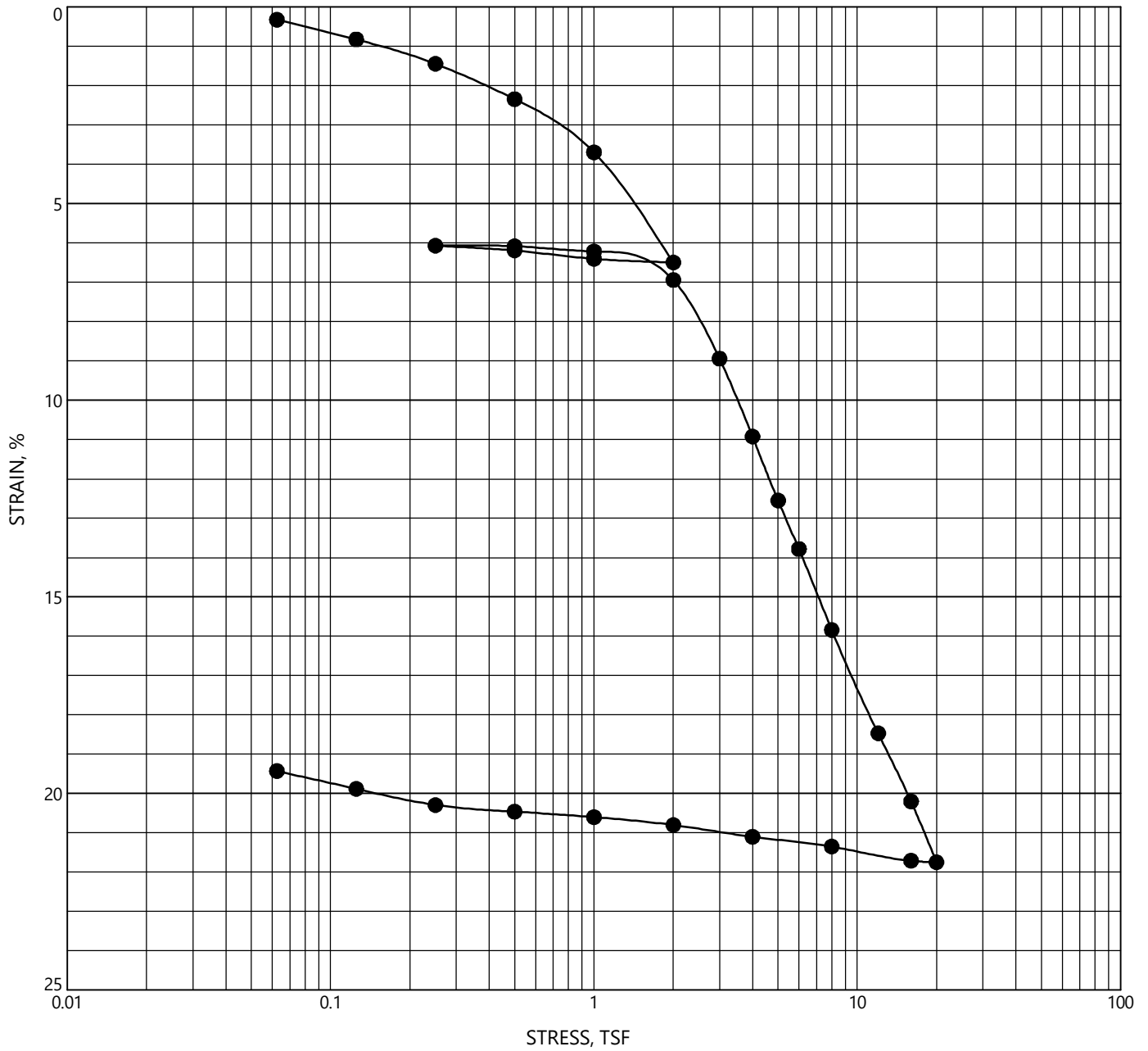
PLASTICITY CHART



●	Location	Sample	Depth, ft	Classification	Initial	
					γ_d , pcf	MC, %
●	B-2	S-6	16.5	Silty SAND, brown, fine to medium grained	87	34



CONSOLIDATION TEST



●	Location	Sample	Depth, ft	Classification	Initial	
					γ_d , pcf	MC, %
●	B-3	S-6	15.6	SILT, trace to some fine-grained sand	77	41



CONSOLIDATION TEST

APPENDIX B

Geoprofessional Business Association Guidance Document

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer

will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept*

responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

Most of the “Findings” Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site’s subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report’s Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals’ misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals’ plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note*

conspicuously that you’ve included the material for information purposes only. To avoid misunderstanding, you may also want to note that “informational purposes” means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled “limitations,” many of these provisions indicate where geotechnical engineers’ responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a “phase-one” or “phase-two” environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer’s services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, *proper implementation of the geotechnical engineer’s recommendations will not of itself be sufficient to prevent moisture infiltration.* **Confront the risk of moisture infiltration** by including building-envelope or mold specialists on the design team. **Geotechnical engineers are not building-envelope or mold specialists.**



Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org